December 2014

REPORT ON

Foundation Investigation and Design Aultsville Road Underpass Replacement Site No. 31-159 Highway 401, 6 km West of County Road 14 Ingleside, Ontario W.P. 4143-10-01

Submitted to:
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REPORT

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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. Two additional bridge replacements were added as part of Scope Change 4. This report presents the results of the detailed foundation investigation conducted for the replacement of the Aultsville Road underpass, Site No. 31-159 (WP 4143-10-01) located on Highway 401 about 6 km west of Ingleside, 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. 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 October 18, 2013.

2.0 SITE DESCRIPTION

The Aultsville Road underpass is located on Highway 401, about 6 km west of Ingleside, Ontario. The existing structure (Site No. 31-159) is located at about Station 11+525 on Highway 401.

The existing bridge consists of a four-span concrete deck with abutments founded on steel pipe piles and piers supported on spread footings. The existing structure is aligned approximately north-south, and is about 63 m long and 10 m wide. It is understood that the structure was built in 1963 and has sustained damage to several girders from impacts incurred on both travelled lanes of Highway 401.

The natural ground surface within the area of the Aultsville Road overpass is near the existing Highway 401 grade (about Elevation 86 m).

Highway 401 in this area is a four-lane, divided highway. Aultsville Road is a two-lane roadway with a rural cross-section. In the area of the bridge, Aultsville Road has been constructed on embankments that are on the order of about 6 to 7 m in height above Highway 401 and the natural ground level, with the Aultsville Road pavement surface at about Elevation 92 m in the vicinity of the bridge. The Aultsville 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.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the proposed bridge replacement was carried out from December 11 to 19, 2013 and on January 14, 2014 during which time three boreholes (numbered 13-411 to 13-413, inclusive) were advanced at the locations shown on Drawing 1.

The boreholes were advanced with 108 mm inside diameter continuous-flight hollow-stem augers and/or wash boring using NW casing with a truck-mounted drill rig, supplied and operated by George Downing Estate Drilling of Grenville-sur-la-rouge, Quebec. The boreholes were advanced to depths of about 22.1 to 27.3 m below the existing pavement/ground surface in the overburden. The boreholes were then cored between about 2.9 to 4.3 m into the bedrock using NQ-size coring equipment. Soil samples in the boreholes were obtained at intervals of about 0.6 to 3.1 m, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

A standpipe piezometer was installed in Borehole 13-412 to monitor the groundwater level at the site. The standpipe consists of a 32 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. The boreholes were backfilled with bentonite pellets, mixed with native soils in the overburden and bentonite pellets in the bedrock. The site conditions were restored following completion of work.

The field work was supervised by members of Golder's technical and engineering staff, who located the boreholes, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil 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, organic content, Atterberg limits, 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 borehole locations were measured relative to existing site features by Golder personnel. The elevations and horizontal coordinates of the boreholes were established based on site survey data received from MMM (survey dated December 2, 2013). The boreholes and locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)
13-411	North Abutment	4984284.3	182050.8	91.7
13-412	South Abutment	4984204.1	182075.7	91.9
13-413	Central Pier (Within the median of Highway 401)	4984239.6	182052.5	85.9

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 Regional Geological Conditions

The site is located in the physiographic region known as the Glengarry Till Plain, just east of the Winchester Clay Plain, as delineated in The Physiography of Southern Ontario.[1]

The Glengarry Till Plain is characterized by the undulating to rolling ground surface where the depth to bedrock is typically less than 30 m and glacial till is typically less than 7 m deep.[1]

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole and Drillhole sheets contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B7 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 embankment fill and organic silt at the abutments, and grade fill at the central pier, overlying glacial till and dolostone bedrock.

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

4.2.1 Pavement Structure and Embankment Fill

The Aultsville Road pavement structure was penetrated in the northbound lane at Boreholes 13-411 and 13-412. At the borehole locations, the pavement structure consists of about 100 mm of asphalt/concrete overlying about 200 mm of gravelly sand base course. The granular base is underlain by about 6.1 to 6.6 m of subbase/embankment fill. The subbase/embankment fill generally consists of sand to silty sand, containing trace to some gravel. The Aultsville Road embankment fill was fully penetrated to depths of about 6.4 and 6.9 m (Elevations 85.3 and 85.0 m) at Boreholes 13-411 and 13-412, respectively.

The grade fill within the median of Highway 401 was penetrated at Borehole 13-413. At the borehole location, the grade fill consisted of about 3.1 m of silty sand and gravel. Cobbles and boulders were also encountered within the median grade fill.

Standard Penetration Test (SPT) “N” values measured in the fill generally range from 1 to 34 blows per 0.3 m of penetration, indicating a very loose to dense state of packing. Refusal to advancement of the split-spoon sampler was encountered in the grade fill in Borehole 13-413, with SPT “N” values of 14 blows and 50 blows per 0.15 m of penetration at which point the sampler was observed to be “bouncing” on inferred cobbles and/or boulders.

[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.

The results of grain size distribution testing carried out on four samples of the Aultsville Road approach embankment fill are provided on Figures B1 and B2 in Appendix B. The result of grain size distribution testing carried out on one sample of the grade fill within the Highway 401 median is shown on Figure B3 in Appendix B. The measured water contents of selected samples of the fill vary from approximately 4 to 12 percent.

4.2.2 Organic Silt

About 0.1 and 0.5 m of organic silt was encountered below the Aultsville Road embankment fill at Boreholes 13-411 and 13-412, respectively.

The results of Atterberg limit testing carried out on one sample of the organic silt from Borehole 13-412 indicate a plasticity index of about 12 percent and liquid limit of about 43 percent, as shown on Figure B6 in Appendix B. This result plots below the A-line on the plasticity chart, as is typical for an organic material. The measured natural water contents of two samples of this material are about 30 and 31 percent. The measured organic contents of two samples of this material are about 8 and 10 percent.

4.2.3 Glacial Till

The fill and organic silt deposit, where encountered, are underlain by a deposit of glacial till. In general, the glacial till is a heterogeneous mixture of gravel and cobbles in a matrix of silty sand. The surface of the till deposit was encountered between Elevation 82.9 and 84.9 m, and the deposit was fully penetrated in the boreholes to depths between 22.1 and 27.3 m (Elevations 63.9 to 64.9 m). At the borehole locations, the glacial till had a thickness between about 19.0 and 20.3 m.

The results of grain size distribution testing carried out on nine selected samples of the glacial till are provided on Figure B4 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 eighteen selected samples of the till ranged from about 5 to 10 percent.

The results of Atterberg limit testing carried out on one sample of the glacial till indicates a plasticity index of about 3 percent and liquid limit of about 13 percent, as shown on Figure B6 in Appendix B, confirming that the till is non-plastic.

The SPT “N” values measured in the glacial till range from 5 to 130 blows per 0.3 m of penetration, indicating a variable, loose to very dense state of packing; it is noted, however, that some of the lower SPT “N” values (below about 15 blows per 0.3 m of penetration) may have been due to disturbance from groundwater inflow to the borehole during sampling. Effective refusal of the split-spoon sampler was encountered in the upper portion of the till in all three boreholes, between about Elevations 81 and 84 m, and this is inferred to have occurred on cobbles and/or boulders in the till deposit. At Borehole 13-412, rotary diamond drilling techniques were required to advance through the glacial till deposit between about Elevation 71.5 and 69.0 m, due to refusal to auger advance over that interval.

4.2.4 Silty Clay

At Borehole 13-411, a 1.8 m thick deposit of silty clay was encountered beneath the glacial till, with its surface at about Elevation 64.9 m and its base at about Elevation 63.1 m. The presence of cobbles has been inferred in this deposit from the behaviour of the rig and drill string during augering.

One measured SPT "N" value within this deposit was 8 blows per 0.3 m of penetration. The results of the in-situ testing indicate a stiff consistency of the silty clay deposit.

The results of grain size distribution testing carried out on one sample of the deposit are provided on Figure B5 in Appendix B. The results of Atterberg limit testing carried out on one sample of the silty clay indicate a plasticity index of about 23 percent and a liquid limit of about 44 percent, as shown on Figure B6, confirming that the deposit is a silty clay of intermediate plasticity. The measured natural water content on one sample of the deposit was about 45 percent, near the liquid limit for the material.

4.2.5 Bedrock

At all borehole locations, bedrock was encountered beneath the till and silty clay. The bedrock was cored for lengths between 2.9 and 4.3 m. 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)
13-411	91.7	28.6	63.1
13-412	91.9	27.3	64.6
13-413	85.9	22.1	63.8

The bedrock encountered in the boreholes typically consists of grey to grey-green dolostone with interbeds of grey shaley limestone and black shale partings. The bedrock is slightly weathered to fresh and typically very strong.

The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples range from about 82 to 100 percent, indicating 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 B7 in Appendix B. The results of the unconfined compressive strength testing on three sample of the bedrock indicate values ranging from 155 to 221 MPa.

4.2.6 Groundwater Conditions

A monitoring well was installed in Borehole 13-412, and the groundwater level measured in the monitoring well is summarized in the table below. The measured level is slightly below the natural ground surface at the site.

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
13-412	91.9	6.7	85.2	March 21, 2014
		6.9	85.0	August 21, 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 Mr. Matt Kennedy, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

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PART B

FOUNDATION DESIGN REPORT
AULTSVILLE ROAD UNDERPASS REPLACEMENT
SITE 31-159
HIGHWAY 401, 6 KM WEST OF COUNTY ROAD 14
INGLESIDE, ONTARIO
W.P. 4143-10-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the existing Aultsville Road 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 detail 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 detail 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 is shown on Drawing 1 and consists of a two-lane, four-span, concrete I-beam structure that was originally constructed in 1963. The two middle spans are about 20.4 m long, and the two outer spans are about 11.3 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 no significant change in width. The new underpass will be founded on abutments located within or near the existing abutment foundation footprints. The proposed Aultsville Road pavement grades at the new structure will be up to about 1.0 m higher than the existing pavement grades.

6.2 Existing Foundations

The existing Aultsville Road underpass is a four-span structure with a reinforced concrete deck and non-integral abutments. The existing bridge is understood to be in fair condition. Based on the 1962 design drawings (Drawings TWP #30-159-1A and #28-202-10A), the existing abutment foundations are understood to consist of 324 mm (12.75") diameter steel pipe piles approximately 6.4 m long, driven to the compact to very dense till at about Elevation 82.2 m. There are two rows of five piles each: one battered at about 1H:4V and one vertical. The design load on each pile was about 400 kN (40 tons). Both abutment pile caps are perched within the existing Aultsville Road approach embankments with the top of each pile cap at about Elevation 89.0 m. The existing abutments are supported on piles deriving resistance from within the upper portion of the glacial till only. The limited capacity of the pipe piles at the abutments of the existing four-span bridge are not considered to be sufficient for support of the new abutments for the proposed two-span structure.

The piles would have been driven through the embankment fill which consists of a generally loose to compact silty sand containing some gravel; although no cobbles or boulders were encountered during advancement of the boreholes put down through the embankment fill during the current investigation, such obstructions may be present in the existing fill. The pipe piles would have been driven on line through the embankment fill and, based on assessment of the borehole data, would have reached their design capacity after penetrating about 1 m into the very dense till. The very dense till contains cobbles and boulders, which may have deflected the piles from their alignment. The position of the existing pile heads may be verified when the pile caps are removed during construction.

The pier foundations are understood to consist of spread footings that measure about 2.1 m by 9.8 m, and which are founded on the very dense glacial till at about Elevation 83.2 m, which is on the order of 3 m below the Highway 401 grade.

6.3 Foundation Options

Based on the subsurface conditions, both shallow and deep foundation options have been considered for the replacement of the existing Aultsville 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 dolostone 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 driving shoes is recommended to minimize damage while penetrating the glacial till deposit (which contains cobbles and boulders) and seating onto the dolostone bedrock. Consideration must be given to removal of or avoiding interference with the existing abutment piles, as the proposed new abutment is to be located at approximately the same location as the existing abutment; however, it is understood that based on MMM's initial assessment, there should be room to install new piles such that conflict with the existing piles can be avoided.

- **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 till deposit.

- **Drilled concrete caissons:** Caissons deriving their support from bearing within the dolostone 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 may require churn drilling and possibly rock coring techniques to penetrate obstructions where encountered in the glacial till. The caisson sockets will also have to be advanced by rock coring and/or chisel drilling into the strong to very strong dolostone bedrock. For this deep foundation option, consideration must be given to removal of the existing abutment piles, as the proposed new abutment is to be located at approximately the same location as the existing abutment; 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 fill and organic silt encountered at the abutments, and below the existing Highway 401 grade fill at the central pier location. This foundation type would not permit the use of integral abutments. Some minimal 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 is anticipated to be within about 1 m of the top of the glacial till deposit and therefore some groundwater control would be required during excavation and construction. As the replacement structure is to be constructed on approximately the same alignment, with the new abutments located near the existing abutments, removal of the existing abutments and piles must also be taken into account.

- **Spread footings "perched" on a compacted granular pad in the approach embankment:** Footings "perched" in the Aultsville Road approach embankments have been considered for support of the new abutments. A longer, and therefore more expensive, replacement structure would be required to permit the open configuration and abutment foreslopes in front of the footing. This could minimize interference between the existing and new abutment foundations. However, excavation to remove the existing abutments (together with associated protection systems and dewatering) would still be required, as would subexcavation and replacement of the existing loose fill and organic silt layer beneath the footprint of the new abutments. Given the higher costs for the longer span structure and the fact that the excavation/subexcavation requirements for this option are more significant than for the other options, this option has not been considered further in this report.

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 support the central pier on spread footings founded on the glacial till deposit. Spread footings at the central pier will require slightly deeper excavation to reach the glacial till as compared with a pile cap for deep foundations, with greater groundwater control and protection system implications.

6.4 Shallow Foundations

6.4.1 Founding Elevations

If adopted for the replacement structure, spread footings should be founded on the very dense glacial till below any existing fill or compressible organic soil.

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 7.5 to 8.0 m below the existing Aultsville Road grade at the abutments, and up to about 3.1 m below the existing grade at the central pier location. The groundwater level was measured in the well installed at the site at Elevation 85.2 m in March 2014 and, therefore, dewatering of the lower portions of the excavations to the founding elevations presented below may 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.

In the boreholes put down through the north and south approach embankments, a thin layer of compressible organic silt was encountered beneath the embankment fill and above the underlying glacial till. The thickness of the deposit encountered ranged from 0.1 m to 0.5 m as encountered in the boreholes, but it may be more variable depending on whether or not it was stripped in some areas prior to construction of the existing embankments. 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, 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.

Foundation Element	Borehole Number	Founding Stratum	Footing Founding Elevation (m)
North Abutment	13-411	Compact to very dense till	Below 84.2
Central Pier	13-413	Very dense till	Below 82.9*
South Abutment	13-412	Very dense till	Below 84.2

Note: * Compacted Granular "A" fill could be used to raise the foundation level to Elevation 83.2 m, to minimize the concrete requirements while still maintaining the required foundation depth of 1.7 m for frost protection purposes.

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 350 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 abutments for the replacement structure may be supported on steel H-piles driven to found on the dolostone bedrock or closed-ended steel pipe (tube) piles founded on the bedrock. Based on the borehole results from the investigation, and assuming about 0.1 m of penetration into the bedrock to allow for some weathering in the upper portion of the rock, 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	13-411	63.1	63.0
Central Pier	13-413	63.8	63.7
South Abutment	13-412	64.6	64.5

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 pipe piles following removal of the existing pile cap and exposure of the existing pipe piles. It is understood that to minimize the length of the proposed bridge (and associated construction costs) and to provide an integral abutment configuration, one row of H-piles is proposed to be driven on a line offset about 400 mm from the existing vertical piles.

The borehole logs at the abutments indicate that there is a very dense layer within the till from about Elevation 81 to 84 m that contains cobbles and boulders. The boreholes did penetrate this layer by augering without the need for diamond drill coring. Steel H-piles reinforced at the tip with a driving shoe should penetrate this layer and continue through the underlying compact to dense till to the surface of the bedrock. However, it is recommended that a contingency item be provided to pre-auger to about elevation 81 m, through the very dense till and below the tips of the existing piles. 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 310x110 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 driving shoe 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 *(Deep Foundations).* If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II *(Steel Tube Pile Driving Shoe)*.

6.5.2 Axial Geotechnical Resistance

For design of HP 310x110 piles driven to 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 dolostone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm diameter steel pipe piles having a minimum wall thickness of 9.5 mm. However, it is noted that based on the presence of cobbles and boulders within the till deposit, and the very dense nature of portions of the till deposit, some steel H-piles or pipe piles may not reach the bedrock. Provided that these piles meet practical refusal in the very dense glacial till at depth, the factored axial geotechnical resistance at ULS for such piles may be taken as 1,600 kN and the axial geotechnical resistance at SLS may be taken as 1,300 kN.

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

6.5.3 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	85.3 – PCL[1]	Loose to Compact Sandy Silt (Fill)	2 to 5	-
	84.8 – 85.3	Organic Silt	-	-
	64.9 – 84.8	Compact to Very Dense Glacial Till	5 to 15	-
	63.1 – 64.9	Very Stiff Silty Clay	-	75 to 125
	63.1	Bedrock	-	-
Central Pier	82.9 – PCL[1]	Compact to Very Dense Silty Sand (Fill)	4 to 15	-
	63.9 – 82.9	Loose to Very Dense Glacial Till	3 to 15	-
	63.9	Bedrock	-	-
South Abutment	85.0 – PCL[1]	Compact to Dense Silty Sand (Fill)	3 to 7	-
	84.9 – 85.0	Organic Silt	-	-
	64.6 – 84.9	Loose to Very Dense Glacial Till	3 to 15	-
	63.1	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 six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

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

Alternatively, support of the abutments or central pier may be provided by caisson foundations. Due to the relatively high water table and the difficulty in socketting a liner into the strong to 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 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 “seated” a minimum of 300 mm 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

The *unfactored* geotechnical side wall (shaft) resistance at ULS can be taken as 1,500 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 smeared material. End-bearing resistance may also be considered in design provided that the base of each caisson is thoroughly cleaned of any cuttings or other material. The *unfactored* geotechnical end-bearing resistance at ULS can be taken as 5,000 kPa. 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 4,600 kN. SLS resistances do not apply to caissons founded within the dolostone bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

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.

The flexible pile-supported abutment foundations discussed in Section 6.5 meet MTO's foundation criteria for integral abutments.

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 loose 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.

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 but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in accordance with OPSS 501 *(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, 3190.101, and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. 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 proposed embankment fill and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM):

Material	SSM
Soil Unit Weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_P	3.0

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

Material	Granular A	Granular B Type II
Soil Unit Weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	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, K_p 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)	
	SSM	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/m³), 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 Aultsville Road will be raised up to about 1.0 m to accommodate an increase in the soffit elevation of the bridge required for clearance above Highway 401. In general, the existing width and alignment of Aultsville 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 Aultsville Road embankments, the road structure is generally underlain by embankment fill consisting of gravelly sand, overlying silty sand fill containing some gravel that is underlain by silty sand till (containing gravel, cobbles, and boulders), and dolostone bedrock. A layer of organic silt that ranged on thickness from 0.1 m to 0.5 m was encountered beneath the embankment fill, overlying the glacial till deposit.

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 of organic silt and the significant embankment height and duration of time since the original construction, removal of the topsoil/organic material 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 *(Earth Excavation and Grading)* and OPSS 501 *(Compacting)*. Benching of the existing Aultsville 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 – *Topsoil*) and seeding (OPSS 804 – *Seed and Cover*) or pegged sod (OPSS 803 – *Sodding*) is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS 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 1.0 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 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 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 Aultsville Road grade is to be increased more than 1.0 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 1.0 m grade raise would result in further consolidation settlement of the organic silt layer present beneath the embankments. For an increase in grade of 1.0 m, the consolidation settlement of the organic silt 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 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

If spread footings are adopted for support of the replacement structure, the foundation excavations are expected to extend through the existing embankment fill (consisting of gravelly sand and silty sand fill) and organic silt at the abutments or silty sand and gravel fill at the central pier, and into the very dense silty sand till. The excavations would extend up to about 8 m below the existing Aultsville Road grade at the abutments and up to about 3 m below the existing Highway 401 median grade. At the abutments, if deep foundations are adopted, 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, organic silt, and glacial till 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). Excavations in organic silt below the water table would be classified as Type 4 soil, based on OSHA and excavations in these materials should be sloped no steeper than 3H:1V. However, with appropriate groundwater control, it is anticipated that temporary excavation slopes through the relatively thin organic silt layer can also 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. Where shoring is required, the protection system should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539.

The selection and design of the protection system will be the responsibility of the Contractor. However, the following comments are provided to aid in the costing and assessment of temporary 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 sheetpile 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 glacial till may impact the depth that sheetpiling 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 sheetpiling would have to be socketted to sufficient depth to provide the necessary passive resistance for the retained soil height of up to about 8 m at the abutments, and 3 m at the central pier. 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 southern embankment and groundwater conditions observed in the boreholes immediately following drilling, the groundwater level is expected to be about 7 m below the existing Aultsville 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 abutments and central pier, or pile/caisson caps at the central pier are anticipated to extend up to about 1 to 2 m below the groundwater level. Dewatering is recommended to lower the groundwater level to approximately 0.5 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 lower to moderate permeability.

The groundwater level is expected to be encountered within less than 2 m of the excavations, 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 abutments and 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 abutments or pier are to be founded on shallow spread footings, all embankment fill, topsoil, organics, and soft or loose soils should be removed from below the proposed founding elevations and wasted or reused as landscaping fill, as required. Subgrade preparation should be performed and monitored in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*).

The 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 to about Elevation 81 m.

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., and reviewed by Ms. Lisa Coyne, P.Eng. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

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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 footings, reducing depth of excavation and temporary excavation support requirements ▪ Higher geotechnical resistances and negligible settlement ▪ Less potential for interference with existing piles (vs. pipe piles) ▪ 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 ▪ Temporary protection systems may be required at the central pier ▪ Some groundwater control would still be required at the central pier	▪ Moderate cost	▪ Low risk of driven H-piles “hanging up” in glacial till ▪ Contingency for pre-augering to Elevation 81 m
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	▪ Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; this could result in more piles “hanging up”, 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 for pre-augering to Elevation 81 m

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)	▪ Significant caisson length required (at least 20 m at central pier, and greater at abutments if caisson cap is perched within embankments) ▪ Temporary or permanent liners required to control ground and groundwater in water-bearing till deposit ▪ Rock coring, churn drilling or chisel drilling required to form rock sockets in strong to very strong bedrock ▪ Conflict with existing abutment piles likely, requiring removal of existing piles	▪ Construction of deep caissons more expensive than alternative foundation options	▪ Significant length required would result in high foundation construction cost ▪ Some risk of difficulty in removing existing abutment piles to avoid conflict with new caissons
Spread/strip footings on very dense silty sand glacial till	▪ Feasible at central pier ▪ Not practical at abutments due to requirement for significant excavation through existing embankments	▪ Existing structure supported on shallow foundations at piers, and foundations have performed reasonably	▪ Significant excavations to depths of greater than 7 m at the abutment locations through the existing embankments ▪ Excavation to a depth of about 3 m at the pier location, between the travelled lanes of Highway 401, will require temporary protection systems ▪ Groundwater control requirements during construction ▪ Lower geotechnical resistances as compared with deep foundations; potential for about 25 mm of settlement ▪ 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

MINISTRY OF TRANSPORTATION, ONTARIO

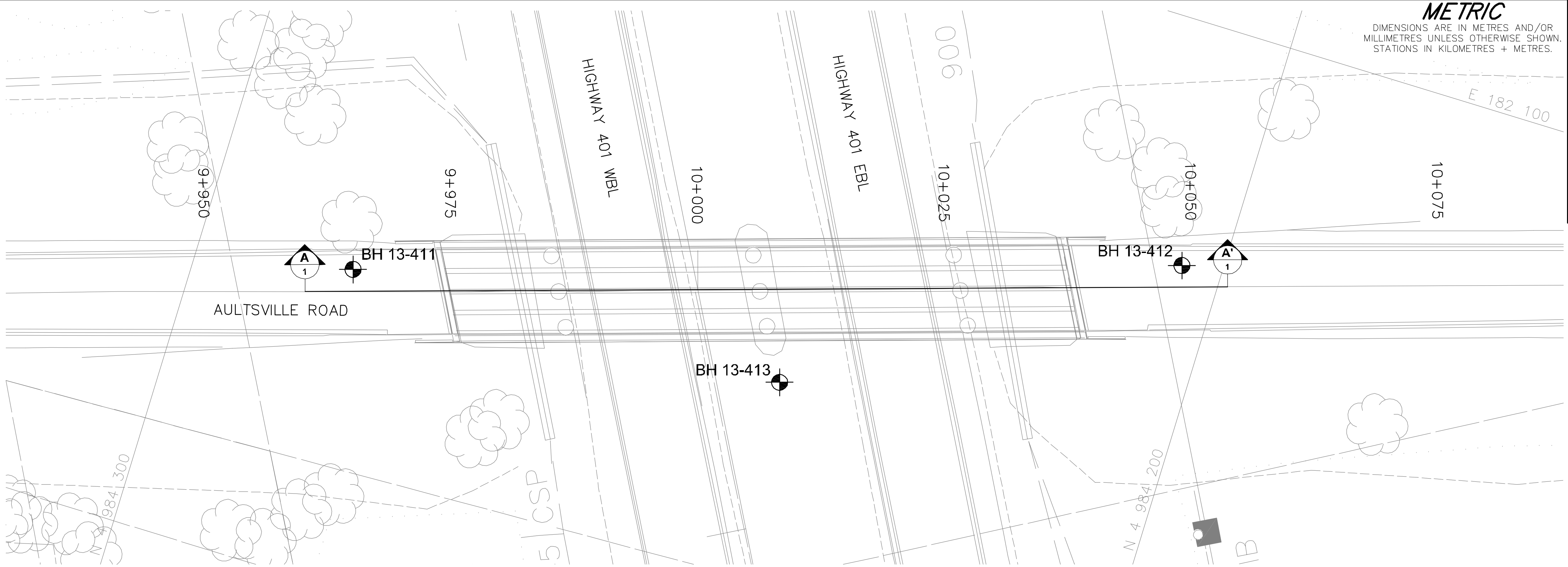
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2014-4037
GWP No. 4143-10-01

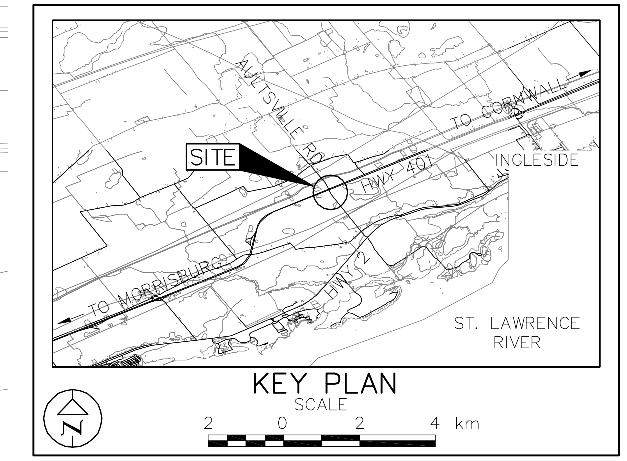
HWY 401 – BRIDGE REPLACEMENT
AULTSVILLE ROAD UNDERPASS
SITE No. 31-159
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



PLAN
SCALE
5 0 5 10 m



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation
- WL in piezometer

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
13-411	91.7	4984284.3	182050.8
13-412	91.9	4984204.1	182075.7
13-413	85.9	4984239.6	182052.5

NOTES

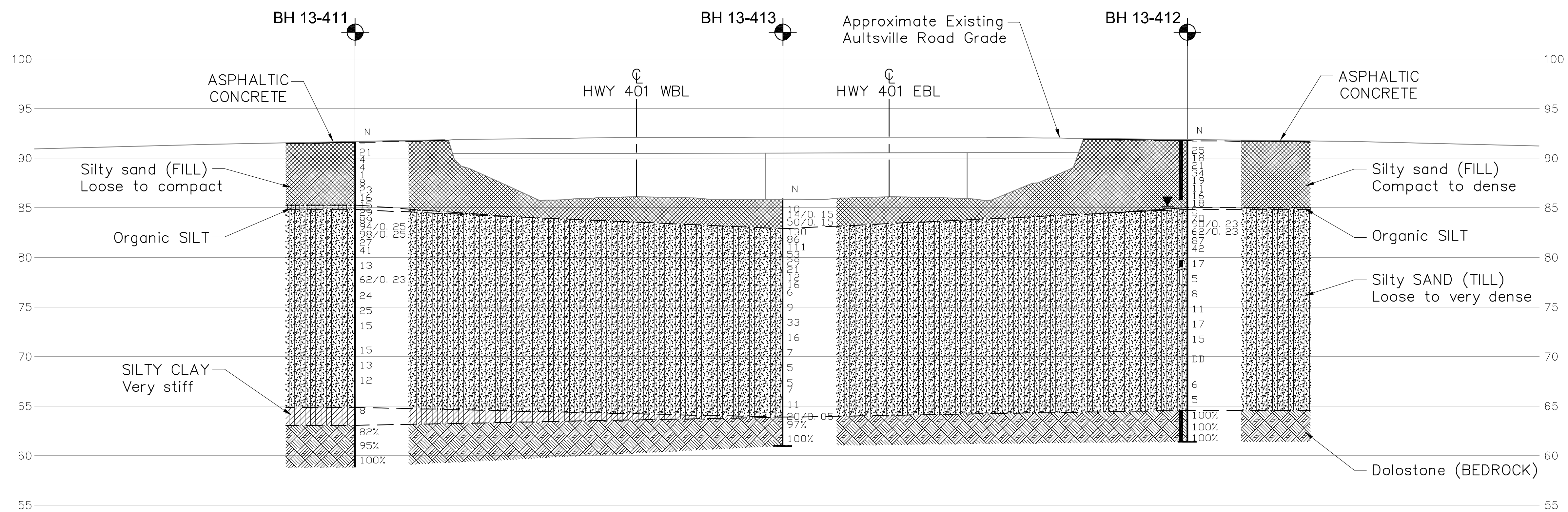
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the design configuration as shown elsewhere in the Contract Documents.

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. BC00300401001-2013 01 23.dwg, received February 11, 2014.



A-A' PROFILE ALONG AULTSVILLE ROAD

HORIZONTAL SCALE
5 0 5 10 m
5 0 5 10 m
VERTICAL SCALE

LICENSED PROFESSIONAL ENGINEER M. J. KENNEDY 100168946 Dec 3, 2014 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN Dec 3, 2014 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31B-86			
HWY. 401	PROJECT NO. 12-1121-0099		DIST. EASTERN
SUBM'D. MJK	CHKD. FJH	DATE: March 2014	SITE: 31-202
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 1

PLOT DATE: December 10, 2014

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 = $\tan\delta$
c'	effective cohesion
c_u or s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3) / 2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3) / 2$
q	$(\sigma_1 - \sigma_3) / 2$ or $(\sigma'_1 - \sigma'_3) / 2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes:
[1] $\tau = c' + \sigma' \tan\phi'$
[2] 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		

RECORD OF BOREHOLE No 13-411

METRIC

PROJECT 12-1121-0099-1410 | LOCATION N 4984284.3 ;E 182050.8 | ORIGINATED BY DG/KE

G.W.P. 4143-10-01 | BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core | COMPILED BY JM

DIST Eastern HWY 401 | DATE December 13-19, 2013 | CHECKED BY MJK

DATUM Geodetic

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	W (%)	UNIT WEIGHT / REMARKS	GR	SA	SI	CL
91.7	GROUND SURFACE									
0.0	ASPHALTIC CONCRETE	1	GRAB	-						
0.3	Gravelly sand (FILL) Grey Dry									
	Sand, trace to some gravel (FILL) Very loose to compact Brown Dry	2	SS	21	○		18	71	9	2
		3	SS	4						
		4	SS	4	○					
88.9 2.8	Silty sand, some gravel, trace clay (FILL) Loose to compact Grey-brown Moist	5	SS	1						
		6	SS	8	○		17	40	32	11
		7	SS	23						
		8	SS	16						
85.3 6.4	Organic SILT Moist	9	SS	15	○	OC = 10.1%				
84.8 6.9	Silty SAND, some gravel, trace to some clay, containing cobbles and boulders (TILL) Compact to very dense Grey Moist to wet	10	SS	29						
		11	SS	89	○					
		12	SS	94/0.25						
		13	SS	98/0.25	○		24	40	30	10
		14	SS	27						
		15	SS	41						
		16	SS	13	○					
		17	SS	62/0.23						

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) 25, 50, 75; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL. STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE 91 to 77.

Continued Next Page

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

RECORD OF BOREHOLE No 13-411 SHEET 2 OF 4 METRIC

PROJECT 12-1121-0099-1410
G.W.P. 4143-10-01 LOCATION N 4984284.3 ;E 182050.8 ORIGINATED BY DG/KE
DIST Eastern HWY 401 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core COMPILED BY JM
DATUM Geodetic DATE December 13-19, 2013 CHECKED BY MJK

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE 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, trace to some clay, containing cobbles and boulders (TILL) Compact to very dense Grey Moist to wet	18	SS	24	76	○		12 41 40 7
		19	SS	25	75, 74			
		20	SS	15	73, 72, 71	○		
		21	SS	15	70			
		22	SS	13	69, 68	○		
		23	SS	12	67, 66, 65			
64.9 26.8	SILTY CLAY, trace sand, containing cobbles Very stiff Grey Wet	24	SS	8	64	Wp–W–WL		0 3 37 60
63.1 28.6	Dolostone (BEDROCK) Bedrock cored from depths of 28.6 m to 32.9 m For bedrock coring details refer to Record of Drillhole 13-411	1	RC	REC 100%	63, 62		UCS = 221.0 MPa	RQD = 82%

Continued Next Page

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

GTA-MTO 001 1211210099.GPJ GAL-GTA.GDT 08/18/14 JM

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 13-411

SHEET 3 OF 4

METRIC

PROJECT 12-1121-0099-1410			
G.W.P. 4143-10-01	LOCATION N 4984284.3 ;E 182050.8		ORIGINATED BY DG/KE
DIST Eastern	HWY 401	BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core	COMPILED BY JM
DATUM Geodetic	DATE December 13-19, 2013		CHECKED BY MJK

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Dolostone (BEDROCK) Bedrock cored from depths of 28.6 m to 32.9 m For bedrock coring details refer to Record of Drillhole 13-411		2	RC	REC 100%		61				RQD = 95%
			3	RC	REC 100%		60				RQD = 100%
58.8							59				
32.9	END OF BOREHOLE										

GTA-MTO 001 1211210099.GPJ GAL-GTA.GDT 08/18/14 JM

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

PROJECT: 12-1121-0099-1410

RECORD OF DRILLHOLE: 13-411

LOCATION: N 4984284.3 ;E 182050.8

DRILLING DATE: December 13-19, 2013

DATUM: Geodetic

DRILL RIG: CME 55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Downing Drilling

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN
		Continued from Record of Borehole 13-411	63.06		
29–32	Rotary Drill NQ Core	Dolostone (BEDROCK), with thin interbeds of grey limestone, occasional black shale partings Fresh Medium to thickly bedded Greenish to dark grey Very fine to fine grained Non-porous Strong to very strong	28.63	1	0
				2	0
				3	0
33		END OF DRILLHOLE	58.82 32.87		

Discontinuity data:

TYPE AND SURFACE DESCRIPTION	Jr	Ja
JN,PL,SM	1	1
JN,PL,RO	1.5	1
BD,PL,SM	1	1
JN,UN,SM	2	1
BD,PL,RO	1.5	1
JN,PL,SM	1	1

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

Depth scale: 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: DG/KE

CHECKED: MJK

Foundation Design

RECORD OF BOREHOLE No 13-412

SHEET 1 OF 4

METRIC

PROJECT 12-1121-0099-1410

G.W.P. 4143-10-01

LOCATION N 4984204.1 ;E 182075.7

ORIGINATED BY DG/HEC

DIST Eastern HWY 401

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

COMPILED BY JM

DATUM Geodetic

DATE December 11-13, 2013

CHECKED BY MJK

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
91.9	GROUND SURFACE									
0.0	ASPHALTIC CONCRETE	1	GRAB	-						
0.3	Gravelly sand (FILL) Grey Dry									
	Sand, trace to some gravel, trace clay (FILL) Compact Brown Dry	2	SS	25						
		3	SS	18	○		15	76	7	2
89.5 / 2.4	Silty SAND, some gravel, trace to some clay Compact to dense Grey-brown Moist	4	SS	21						
		5	SS	34	○					
		6	SS	19						
		7	SS	11	○					
		8	SS	16	○		14	42	32	12
		9	SS	18						
85.0	Organic SILT Moist									
7.0	Silty SAND, some gravel, trace clay, containing cobbles and boulders (TILL) Very dense Grey Moist	10	SS	5	⊢○⊣	OC = 8.2%				
		11	SS	70						
		12	SS	90/0.23						
		13	SS	62/0.23						
		14	SS	87	○		26	39	27	8
81.2 / 10.7	Silty SAND, some gravel, trace clay, containing cobbles and boulders (TILL) Loose to compact Grey Wet	15	SS	42						
		16	SS	17	○					
		17	SS	5						

Elevation scale: 91, 90, 89, 88, 87, 86, 85, 84, 83, 82, 81, 80, 79, 78, 77

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 13-412

METRIC

PROJECT 12-1121-0099-1410
G.W.P. 4143-10-01
LOCATION N 4984204.1 ;E 182075.7
ORIGINATED BY DG/HEC
DIST Eastern HWY 401
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core
COMPILED BY JM
DATUM Geodetic
DATE December 11-13, 2013
CHECKED BY MJK

--- CONTINUED FROM PREVIOUS PAGE ---

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
	Silty SAND, some gravel, trace clay, containing cobbles and boulders (TILL) Loose to compact Grey Wet	18	SS	8			○		14 42 36 8
						76			
		19	SS	11		75			
						74			
		20	SS	17		73	○		
						72			
		21	SS	15		71			
		22	RC	DD		70, 69			
						68			
		23	SS	6		67	○		15 45 32 8
						66			
		24	SS	5		65			
64.6 / 27.3	Dolostone (BEDROCK) Bedrock cored from depths of 27.3m to 30.5 m For bedrock coring details refer to Record of Drillhole 13-412	1	RC	REC 100%		64		UCS = 220.6 MPa	RQD = 100%
		2	RC	REC 100%		63			RQD = 100%
		3	RC	REC 100%		62			RQD = 100%

Dynamic cone penetration resistance plot: 20 40 60 80 100
Shear strength kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)
Plastic limit W_P, Natural moisture content W, Liquid limit W_L; Water content (%) 25 50 75

Continued Next Page

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

GTA-MTO 001 1211210099.GPJ GAL-GTA.GDT 08/22/14 JM

RECORD OF BOREHOLE No 13-412

SHEET 3 OF 4

METRIC

PROJECT 12-1121-0099-1410

G.W.P. 4143-10-01

LOCATION N 4984204.1 ;E 182075.7

ORIGINATED BY DG/HEC

DIST Eastern HWY 401

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

COMPILED BY JM

DATUM Geodetic

DATE December 11-13, 2013

CHECKED BY MJK

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
61.4			3	RC	REC 100%						RQD = 100%
30.5	END OF BOREHOLE NOTES: 1. Water level in well at a depth of 6.7 m (Elev. 85.2 m), measured on March 21, 2014. 2. Water level in well at a depth of 6.9 m (Elev. 85.0 m), measured on August 21, 2014.										

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

○ 3% STRAIN AT FAILURE

RECORD OF DRILLHOLE: 13-412

PROJECT: 12-1121-0099-1410
LOCATION: N 4984204.1 ;E 182075.7
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 11-13, 2013
DRILL RIG: CME 55
DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja
		Continued from Record of Borehole 13-412	64.57					
28	Rotary Drill NQ Core	Dolostone (BEDROCK), with thin interbeds of shaley limestone, occasional nodular textures Fresh Thinly to medium bedded Light greenish grey Fine to medium grained Non-porous Strong to very strong	27.30	1	100	BD,PL,SM BD,ST,SM BD,CU,SM	1 2 2	1 1 1
29				2	100			
30			61.41	3	100			
		END OF DRILLHOLE	30.46					

DEPTH SCALE
1 : 75

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LOGGED: DG/HEC
CHECKED: MJK

GTA-RCK 031 1211210099.GPJ GAL-MISS.GDT 08/18/14 JM

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 13-413

SHEET 1 OF 3

METRIC

PROJECT 12-1121-0099-1410
G.W.P. 4143-10-01
LOCATION N 4984239.6 ;E 182052.5
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 January 14, 2014
CHECKED BY MJK

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL
85.9	GROUND SURFACE								
0.0	Silty sand and gravel, containing cobbles and trace organic material (FILL) Compact to very dense Brown to grey Moist								
		1	SS	10	○	31	44	19	6
		2	SS	14/0.15					
		3	SS	50/0.15					
82.9 / 3.1	Silty SAND, some gravel, trace to some clay, containing cobbles (TILL) Very dense Grey Moist to wet	4	SS	130					
		5	SS	86	○	18	44	28	10
		6	SS	111					
		7	SS	53	○				
79.8 / 6.1	Silty SAND, trace to some gravel, trace clay (TILL) Loose to compact Grey Wet	8	SS	29					
		9	SS	21					
		10	SS	12	○	28	37	27	8
		11	SS	16					
		12	SS	6	○				
		13	SS	9					
74.6 / 11.3	Silty SAND, trace to some gravel, trace clay (TILL) Compact to dense Grey Wet								
		14	SS	33	○	16	51	25	8
		15	SS	16					

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (85, 84, 83, 82, 81, 80, 79, 78, 77, 76, 75, 74, 73, 72); 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

GTA-MTO 001 1211210099.GPJ GAL-GTA.GDT 08/18/14 JM

RECORD OF BOREHOLE No 13-413

SHEET 2 OF 3 **METRIC**

PROJECT 12-1121-0099-1410
G.W.P. 4143-10-01 LOCATION N 4984239.6 ;E 182052.5 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 January 14, 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
70.7 / 15.2	Silty SAND, some clay, trace to some gravel (TILL) Loose to compact Grey Wet	16	SS	7	70	○		
		17	SS	5	69			
		18	SS	5	68	W_P–W		15 46 28 11
		19	SS	7	67			
		20	SS	11	66, 65			
63.8		21	SS	20/0.05	64			
22.1	Dolostone (BEDROCK) Bedrock cored from depths 22.1 m to 25.0 m For bedrock coring details refert to Record of Drillhole 13-413	1	RC	REC 100%	63		UCS = 155.4 MPa	RQD = 97%
61.0		2	RC	REC 100%	62			RQD = 100%
25.0	END OF BOREHOLE				61			

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

GTA-MTO 001 1211210099.GPJ GAL-GTA.GDT 08/18/14 JM

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

RECORD OF DRILLHOLE: 13-413

PROJECT: 12-1121-0099-1410
LOCATION: N 4984239.6 ;E 182052.5
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: January 14, 2014
DRILL RIG: CME 55
DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

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

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja
	Continued from Record of Borehole 13-413	63.84					
23	Dolostone (BEDROCK), with thin to medium thick interbeds of grey shaley limestone, occasional shale partings Fresh Thinly to medium bedded Greenish grey Fine to medium grained Non-porous Medium strong to strong	22.10	1	100%	BD,CU,SM BD,CU,SM BD,CU,SM	2 2 2	1 1 1
24			2	100%	BD,CU,IR	2	1
25		60.99 24.95					
	END OF DRILLHOLE						

Depth scale marks: 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37

DEPTH SCALE
1 : 75

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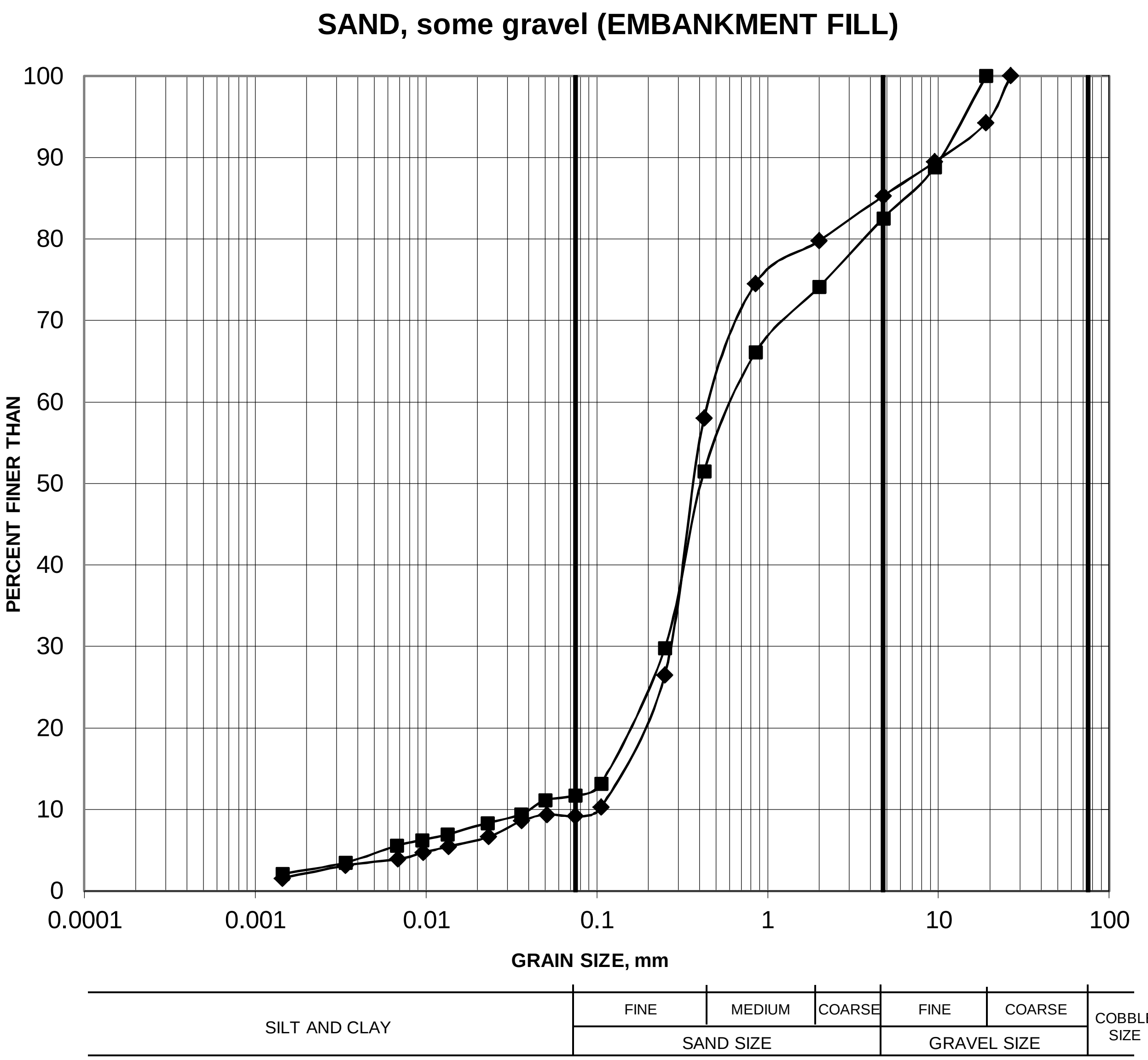
LOGGED: DWM
CHECKED: MJK

GTA-RCK 031 1211210099.GPJ GAL-MISS.GDT 08/18/14 JM

APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1



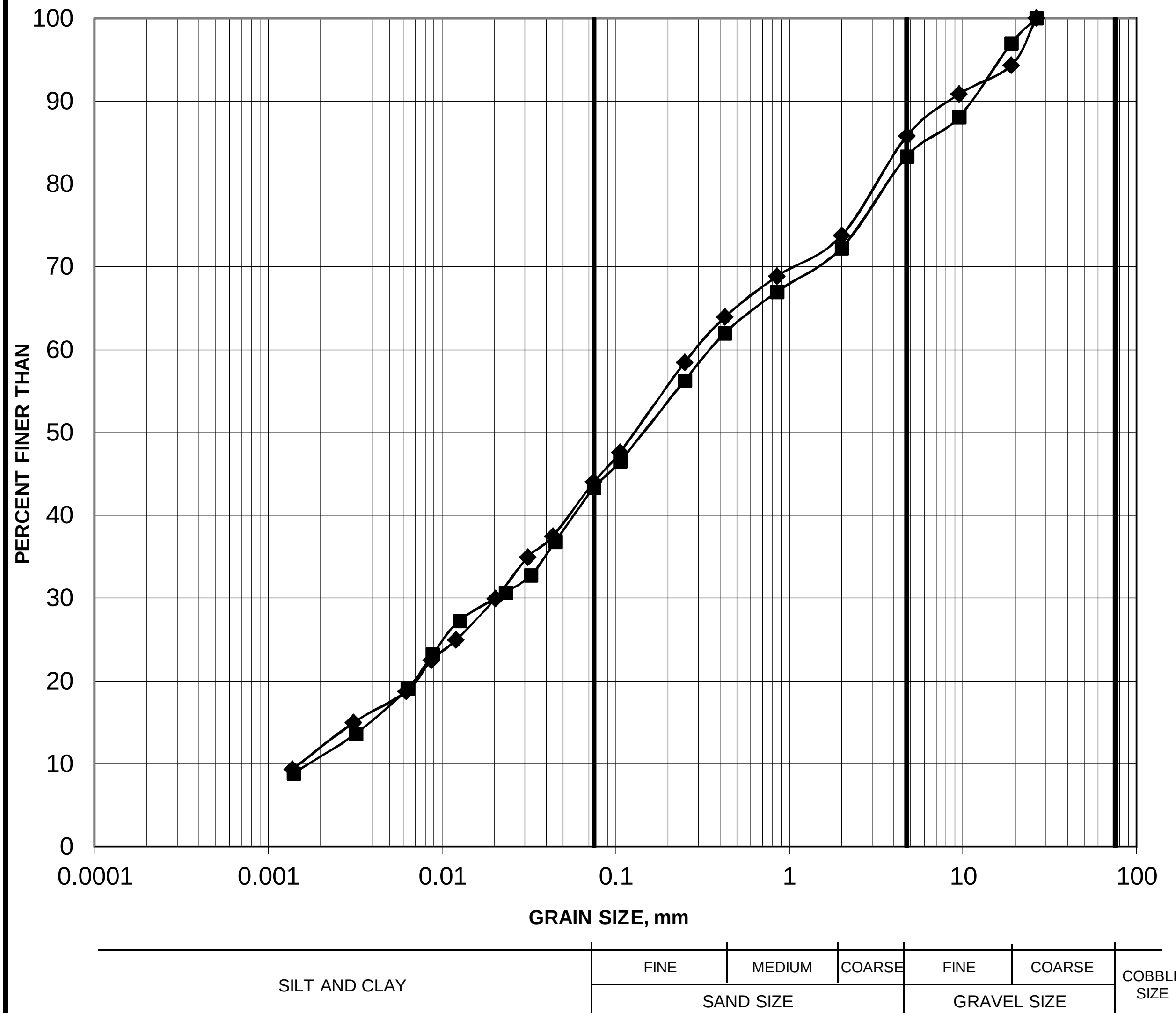
Borehole	Sample	Depth (m)
13-411	2	0.76-1.37
13-412	3	1.52-2.13

Project: 12-1121-0099/1410 **Golder Associates**

Created by: CW
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B2

Silty SAND, some gravel (EMBANKMENT FILL)



Borehole	Sample	Depth (m)
13-411	6	3.81-4.42
13-412	8	5.33-5.94

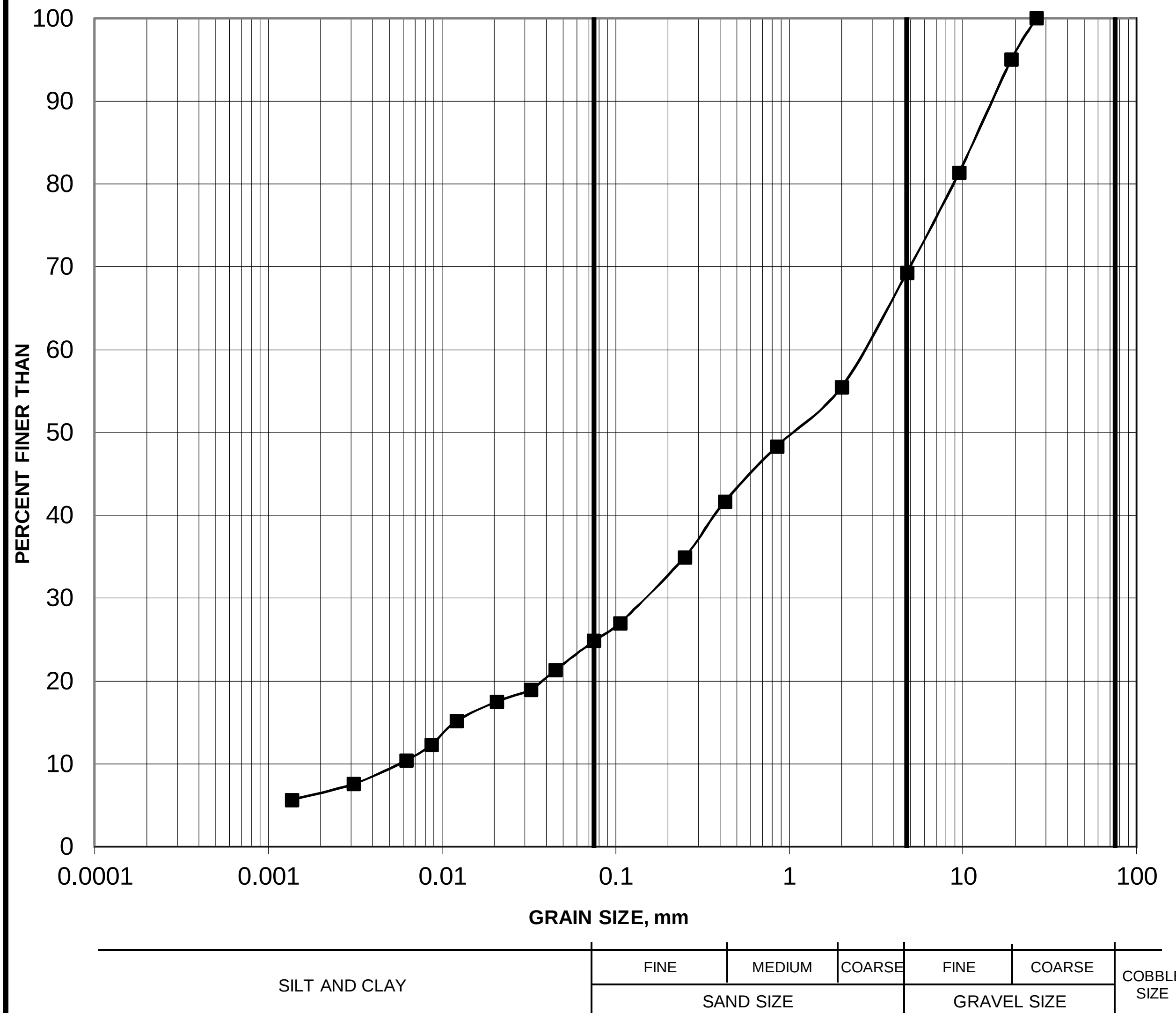
Project: 12-1121-0099/1410

Golder Associates

Created by: CW
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B3

Silty SAND and GRAVEL (MEDIAN FILL)



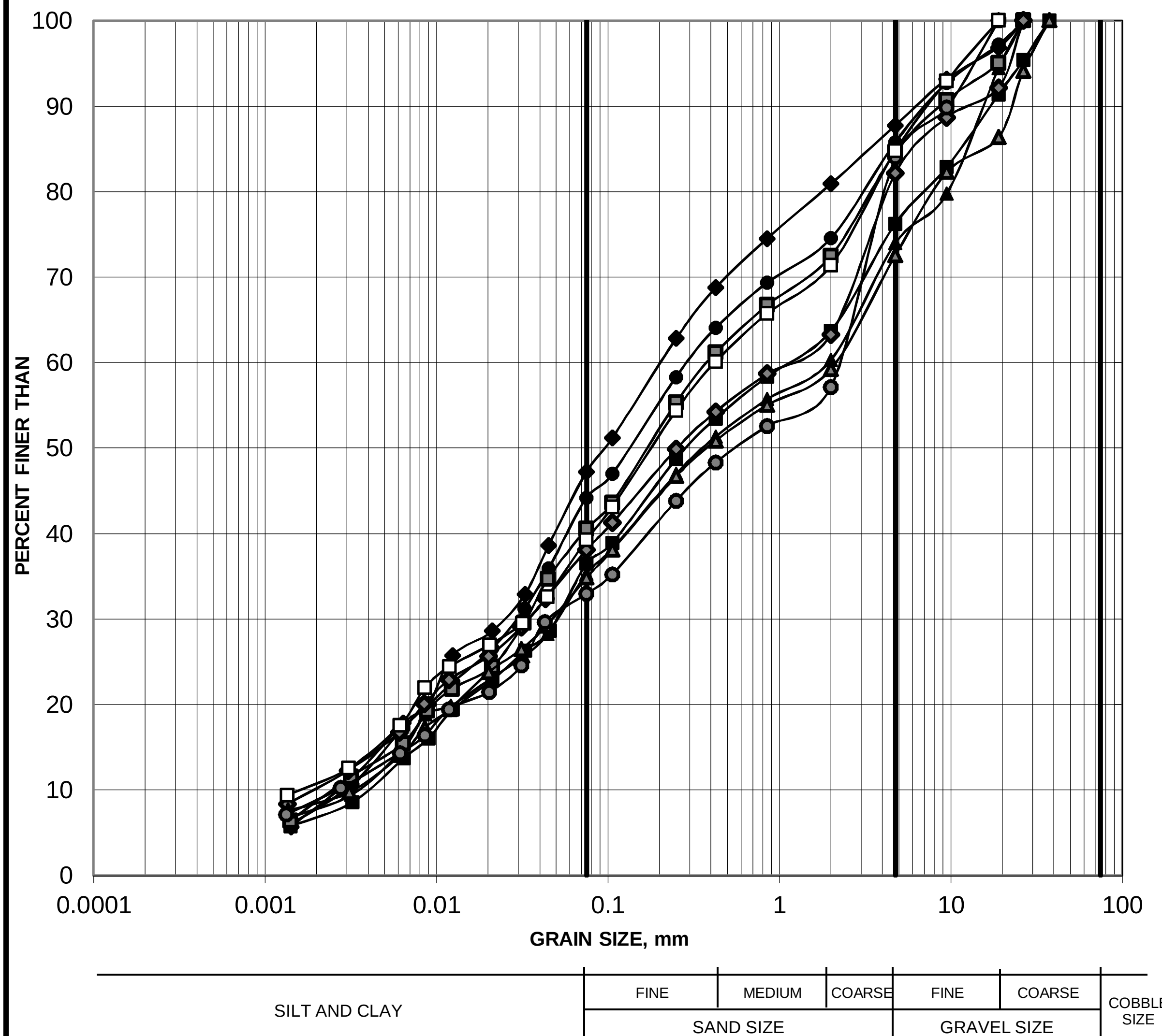
Borehole	Sample	Depth (m)
13-413	1	0.76-1.37

Project: 12-1121-0099/1410 **Golder Associates**

Created by: CW
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B4

Silty SAND, some gravel (GLACIAL TILL)



Borehole	Sample	Depth (m)
13-411	13	9.15-9.55
13-411	18	15.24-15.85
13-412	14	9.91-10.44
13-412	18	15.24-15.85
13-412	23	24.38-24.99
13-413	5	3.81-4.27
13-413	10	7.62-8.23
13-413	14	12.19-12.80
13-413	18	18.29-18.90

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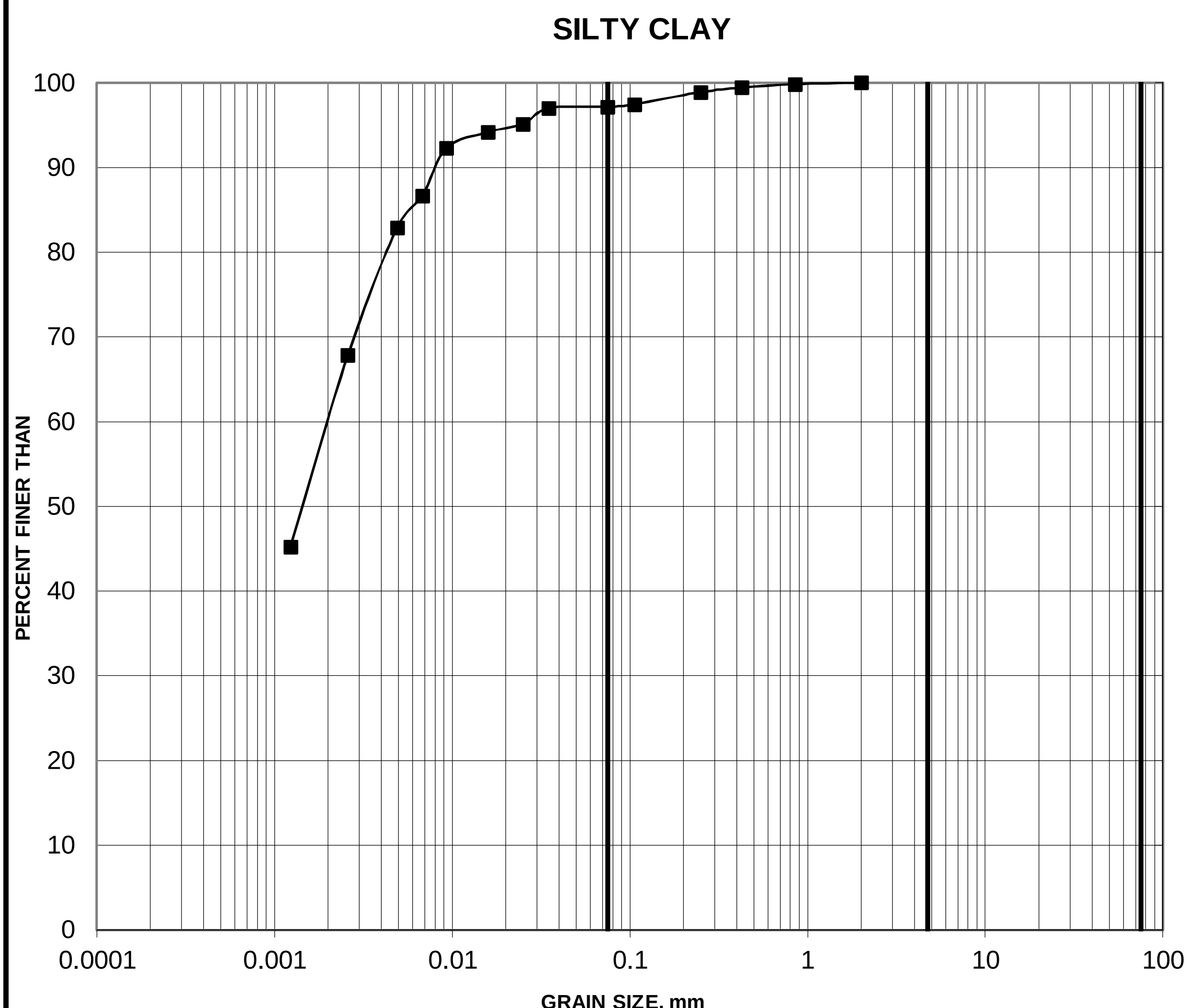
Checked by: CNM

Project: 12-1121-0099/1410 **Golder Associates**

GRAIN SIZE DISTRIBUTION	FIGURE B5



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

Borehole	Sample	Depth (m)
13-411	24	26.82-27.43

Project: 12-1121-0099/1410

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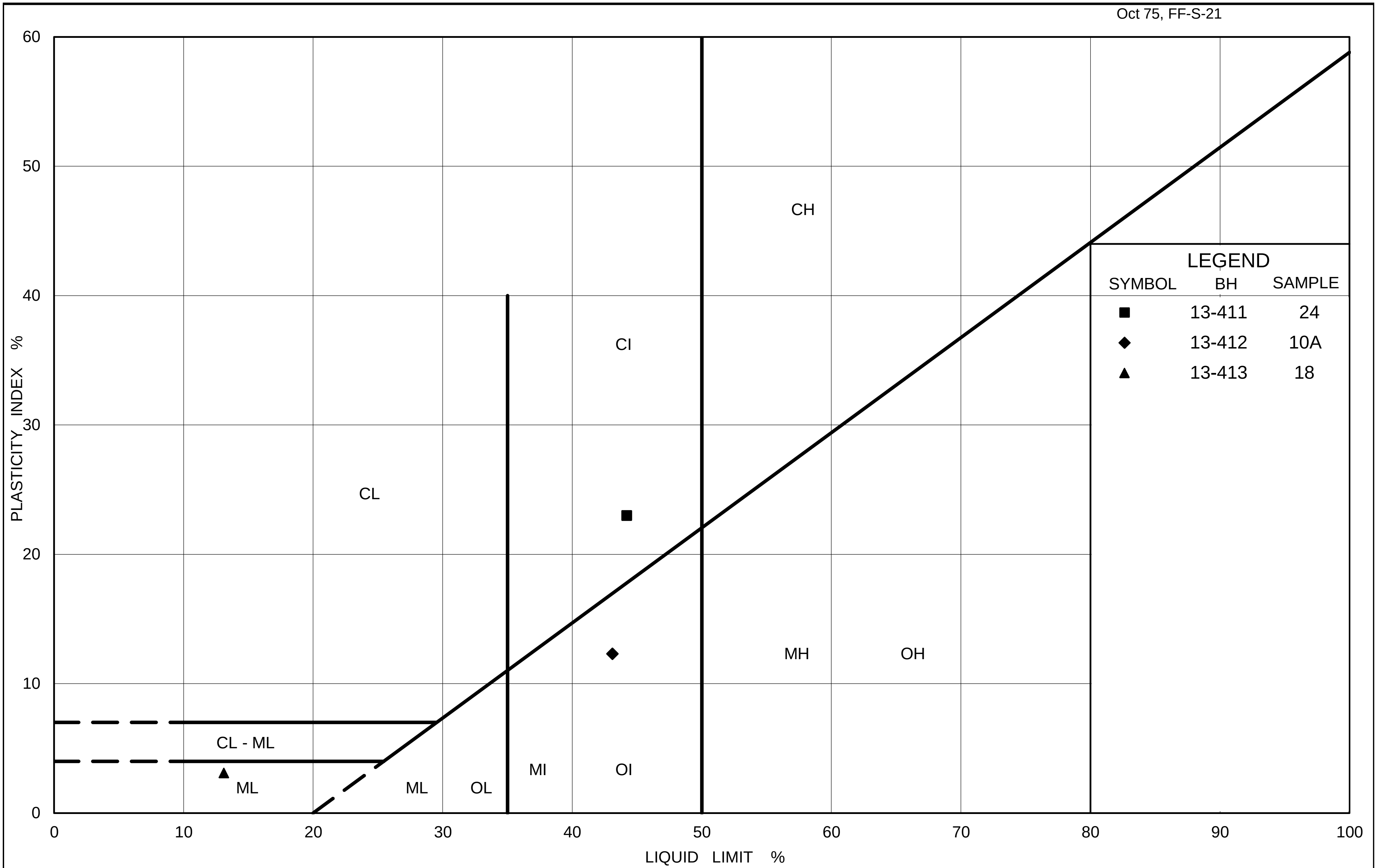


Ministry of Transportation

Ontario

PLASTICITY CHART

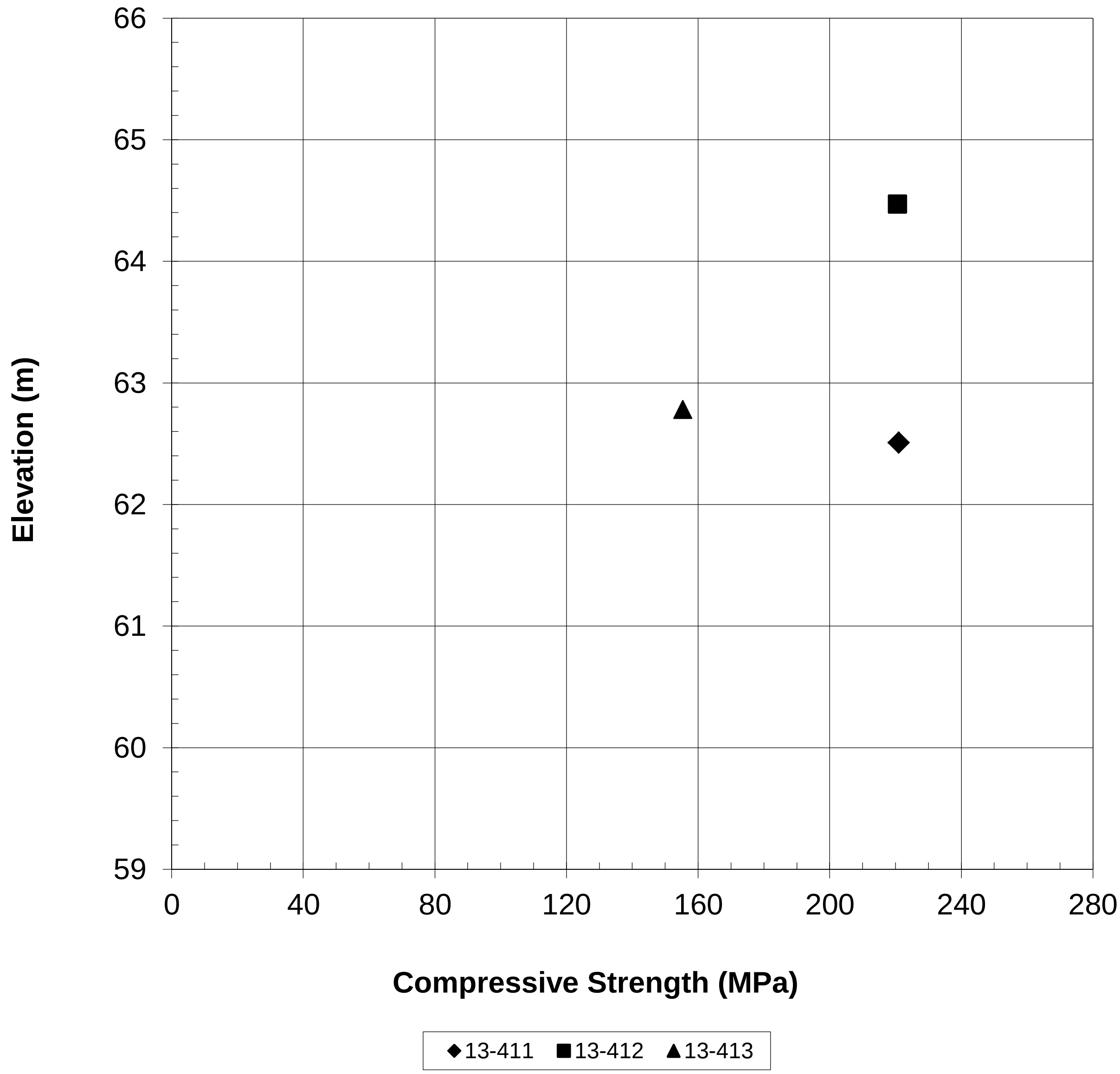
FIG No. B6

Project No. 12-1121-0099/1410

Compiled By : CW Checked By : CNM

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH UNCONFINED COMPRESSION TESTS

FIGURE B7



Project: 12-1121-0099-1410

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Created by:	MJK
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 0.3 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 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 Aultsville 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 Aultsville 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 Aultsville 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 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 dolostone bedrock, which is strong to very strong. Appropriate construction procedures and equipment will be required to penetrate 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

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

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