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REPORT ON

Foundation Investigation and Design Replacement of Highway 401 Underpass at Wales Road, Site No. 31-162 Highway 401, 5 km East of Ingleside, ON W.P. 4055-08-01 G.W.P. 4064-12-00

REPORT

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PART A

**FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF HIGHWAY 401 UNDERPASS
AT WALES ROAD, SITE 31-162
HIGHWAY 401, 5 KM EAST OF INGLESIDE, ON
TOWNSHIP OF OSNABRUCK
W.P. 4055-08-01
G.W.P. 4064-12-00**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group Ltd. (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the Design-Build of bridge and culvert replacements at various locations in the Eastern Region of Ontario as part of the 22 Structures MEGA 2 project. Foundation investigation and input for the detailed design of several bridge replacements was added to the overall scope of work following award of the project. This report presents the results of the detailed foundation investigation conducted for the replacement of the Wales Road (County Road 12) underpass, Site No. 31-162 located on Highway 401 about 5 km east 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 scope change proposal to MMM, dated July 10, 2014.



2.0 SITE DESCRIPTION

The Wales Road underpass is located on Highway 401 at Wales Road, approximately 5 km east of the Ingleside interchange within the Township of Osnabruck in the United Counties of Stormont, Dundas, and Glengarry. Wales Road is also known as County Road 12.

In this area, Highway 401 is a four-lane, divided highway and Wales Road is a two-lane county road. In the area of the bridge, Wales Road has been constructed on embankments that are about 6 m in height above Highway 401 and the natural ground level, with the Wales Road pavement surface located at about Elevation 84.5 m in the vicinity of the bridge. The Wales Road embankment has side slopes that are inclined between about 2 horizontal to 1 vertical and 3 horizontal to 1 vertical (i.e., 2H:1V and 3H:1V). Based on visual observation at the time of the site investigation, the existing embankment slopes appear to be performing satisfactorily.

The existing bridge consists of a four-span continuous concrete T-beam structure overlain with an asphalt wearing surface. It is understood that the structure was built circa 1962 and was previously rehabilitated in 1986. The existing structure is aligned in an approximate north-south orientation, and is about 64.6 m long and 10.4 m wide. The superstructure is founded on three piers supported on spread footing foundations and two abutments supported by steel H-piles. The pile caps for the abutments are located within the embankments at about Elevation 81 m.

2.1 Regional Geological Conditions

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

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

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



3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the proposed bridge replacement was carried out between October 14 and 29, 2014 during which time three boreholes (designated as 14-711 to 14-713, inclusive) were advanced at the locations shown on Drawing 1.

The boreholes were advanced with 200 mm diameter continuous-flight hollow-stem augers and/or rotary drilling methods (during rock coring) using a truck-mounted drill rig, supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of about 20.5 to 27.4 m below the ground surface. Following penetration of the overburden soil, the boreholes were cored between about 1.3 to 2.8 m into the bedrock using NQ-size coring equipment. Soil samples in the boreholes were obtained at regular intervals using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

A standpipe piezometer was installed in Borehole 14-711 to monitor the groundwater level at the site. The standpipe consists of a 51 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section located within a till deposit at an elevation of about 73.5 to 75 m, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. The boreholes were backfilled with bentonite pellets, mixed with native soils in the overburden and bentonite pellets in the bedrock. The site conditions were restored following completion of work.

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

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

Borehole Number	Borehole Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)
14-711	North Abutment	4989341.8	191434.8	84.5
14-712	South Abutment	4989271.2	191475.1	84.4
14-713	Central Pier (Within the median of Highway 401)	4989301.0	191445.7	78.5

Notes: 1) Northing and Easting coordinates shown are relative to the MTM NAD83 (Zone 8) coordinate system.
2) Ground surface elevations shown are relative to Geodetic Datum.



4.0 SITE STRATIGRAPHY

4.1 General

The detailed subsurface soil, bedrock and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are provided 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 B6 contained in Appendix B.

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

In general, the subsurface conditions at the location of the proposed bridge replacement consist of granular embankment fill at the abutments, and grade fill at the central pier, overlying glacial till deposits and then limestone bedrock.

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

4.2 Pavement Structure and Embankment Fill

The Wales Road pavement structure was penetrated in the northbound lane at Boreholes 14-711 and 14-712. At these borehole locations, the pavement structure consists of about 100 mm of asphaltic concrete overlying about 200 to 300 mm of gravelly sand. The granular base layer is underlain by about 2.6 m to 2.7 m of sand and gravel fill.

The remainder of the embankment fill below this level generally consists of silty sand to sandy silt containing varying amounts of gravel. Cobbles were encountered below about 2.9 m depth at Borehole 14-712. Asphalt pieces were encountered in the embankment fill at Borehole 14-711 (between about 5.3 m and 5.6 metres depth) and Borehole 14-712 (between about 6.1 m and 6.9 m depth). Organic matter was present within the embankment fill in Borehole 14-712 at depths of between about 6.9 and 8.5 m. The Wales Road embankment fill was fully penetrated at depths of about 7.6 and 8.5 m (corresponding to Elevations of 76.9 and 75.9 m) at Boreholes 14-711 and 14-712, respectively.

At Borehole 14-713, the fill within the median of Highway 401 consisted of about 1.4 m of sand and gravel with trace silt.

Standard Penetration Test (SPT) “N” values measured in the Wales Road embankment and Highway 401 median fill generally ranged from 8 to greater than 50 blows per 0.3 m of penetration, indicating these materials are loose to very dense. Refusal to advancement of the split-spoon sampler was encountered at discrete locations within the fill in Boreholes 14-711 and 14-712. The split spoon refusal is inferred to be a result of the sampler encountering deleterious fill (asphalt pieces) and/or cobbles within the fill.

The results of grain size distribution testing carried out on five samples of the fill materials are displayed on Figures B1 and B2. The results of this testing suggest that the fill materials encountered within about 3 m of ground surface (see Figure B1) consist of sand and gravel containing trace silt while samples collected from greater depth (See Figure B2) were comprised of silty, gravelly sand. The measured water contents of selected samples of the fill vary from approximately 4 to 8 percent.



4.3 Clayey Silt

A thin layer of clayey silt was encountered below the grade fill in the Highway 401 median at the location of Borehole 14-713. The deposit is about 0.1 m thick and was fully penetrated in the borehole to a depth of about 1.5 m below the existing grade.

4.4 Glacial Till

The fill and the clayey silt deposit, where encountered, are underlain by an extensive deposit of glacial till. In general, the upper portion of the glacial till is composed of a heterogeneous mixture of gravel and cobbles in a matrix of silty sand to sandy silt. With depth, the till contains boulders.

The surface of the glacial till deposit was encountered between Elevation 75.9 and 77.0 m, and it was fully penetrated in the boreholes at depths ranging from about 18.2 and 24.6 m (corresponding to Elevations ranging from 59.9 m to 60.3 m). At the borehole locations, the glacial till was about 16 m to 17 m thick. In Borehole 14-711, a layer of sandy gravel was encountered within the glacial till deposit between 14.5 m and 15.2 m depth. Similarly, a deposit of sandy silt containing sand seams was encountered within the till strata in Borehole 14-711 at depths between about 23.0 m and 23.6 m.

The SPT “N” values measured in the overall glacial till deposit ranged widely varying from 3 to 160 blows per 0.3 m of penetration. However, based on the distribution of the recorded blow counts, the overall glacial till deposit consists of an upper layer that is typically loose to compact with SPT “N” values generally ranging from about 7 to 20 blows per 0.3 m of penetration, and a lower dense to very dense layer, where the SPT “N” values were generally greater than 50 blows per 0.3 m or where the SPT sampler typically met effective refusal. Diamond drilling techniques were required to penetrate the lower till deposit at several locations.

The results of grain size distribution testing carried out on 12 samples of the glacial till are provided on Figure B3 in Appendix B. The results of grain size distribution testing carried out on a sample from the sandy gravel layer and the sandy silt layer within the glacial till deposit are provided on Figure B4 and B5, respectively. 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 were measured to range from about 7 to 15 percent.

4.5 Bedrock

At all borehole locations, drilling was terminated within limestone bedrock that underlies the glacial till. The bedrock was cored for lengths between 1.3 to 2.8 m at each borehole location. The following table summarizes the bedrock surface depths and elevations as encountered at the three borehole locations.

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
14-711	84.5	24.6	59.9
14-712	84.4	24.5	59.9
14-713	78.5	18.2	60.3



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The bedrock encountered in the boreholes typically consists of fresh, grey, thinly to medium bedded, nodular limestone that contains thin black shale interbeds and partings.

The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples range from about 90 to 100 percent, indicating excellent quality rock. The discontinuities observed in the rock core are generally associated with the joints and bedding of the bedrock.

Laboratory unconfined compressive strength testing was carried out on two selected specimens of the bedrock core. The results of the testing, which are summarized on Figure B6 in Appendix B, measured unconfined compressive strengths ranging from 121 to 138 MPa indicating the limestone bedrock is typically very strong.

4.6 Groundwater Conditions

A monitoring well was installed in Borehole 14-711, and the groundwater level measured in the monitoring well is given in the table below. The measured water level taken one month after drilling was complete is at about the natural ground surface at the site.

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
14-711	84.5	6.2	78.3	November 26, 2014

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



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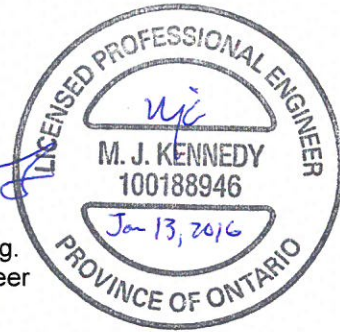
5.0 CLOSURE

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

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PART B

FOUNDATION DESIGN REPORT
REPLACEMENT OF HIGHWAY 401 UNDERPASS
AT WALES ROAD, SITE 31-162
HIGHWAY 401, 5 KM EAST OF INGLESIDE, ON
TOWNSHIP OF OSNABRUCK
W.P. 4055-08-01
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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 Wales Road (County Road 12) 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 consists of a two-lane, four-span, concrete T-beam structure that was originally constructed circa 1962. The two middle spans are about 20.1 m long, and the two outer spans are about 12.2 m long. It is understood that the preferred alternative for the proposed replacement consists of a two-span structure on the same alignment as the existing bridge with no significant change in width. The new underpass will be founded on abutments located within or near the existing abutment foundation footprints. The proposed Wales Road pavement grades at the new structure will be up to about 0.9 m higher than the existing pavement grades at the abutments.

6.2 Existing Foundations

Based on the original design drawings (Drawings TWP #30-162-1-A and #30-1162-2-A, revised in 1962), the existing abutment foundations are understood to consist of steel H-piles (BP10x42) at least 11.6 m long, driven in to the compact to very dense glacial till at about Elevation 70.1 metres. There are two rows of seven piles each. The piles in each row are spaced at about 1.5 metre intervals, transverse to the bridge alignment. The front row piles (closest to Highway 401) are battered at about 1H:4V. The back row piles are vertical. At the pile cap, the two rows of piles are offset from each other 1.2 metres, along/parallel to the bridge alignment, with the exception of the two outer-most piles in the back row which are offset about 1.8 metres from the front row piles. Based on the original design drawings, the design load on each pile was about 300 kN (30 tons).

Both abutment pile caps are perched within the existing Wales Road approach embankments with the top of each pile cap at about Elevation 81.5 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 H-piles at the abutments of the existing four-span bridge are not expected to be sufficient for support of the new abutments for the proposed two-span structure. The existing piles would have been driven through the embankment fill which consists of a generally compact to very dense silty gravelly sand. Cobbles were encountered in the borehole put down through the south embankment fill. The underlying till deposits also contains cobbles and boulders, which may have deflected the piles from their alignment.

The pier foundations are understood to consist of spread footings, with plan dimensions of about 2.3 m by 10.4 m, that are founded within the glacial till at Elevation 75.9 m, which is about 2.6 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 Wales 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 advanced through the upper, loose to compact portions of the glacial till and driven to refusal on/within either the very dense portions of the till or the limestone bedrock are feasible for support of the replacement bridge structure. This option would allow the pile caps to be maintained at a higher elevation than shallow foundations 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. Diamond drilling methods were required to advance the boreholes through the very dense, lower portion of the till deposits. In this regard, it is anticipated that driven piles may “hang up”/encounter refusal within the very dense till particularly at the north abutment where there is approximately 9 m of dense to very dense till present above the bedrock surface. Higher design capacities could be achieved for piles founded on bedrock in comparison to piles driven to refusal within the till; however, the installation of pile to bedrock is expected to require pre-drilling through the very dense till. The use of pile points is recommended for driven piles to minimize the potential for pile damage while penetrating the existing embankment fill and glacial till deposit (which contains cobbles as well as boulders at depth). Consideration must also 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.
- **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” due to the presence of cobbles and/or boulders within the existing embankment fill and underlying glacial till. As described for steel H-piles, pre-augering through the dense to very dense portions of the glacial till may be required.
- **Drilled concrete caissons:** Caissons deriving their support from bearing within the limestone bedrock are also feasible for this site. Caissons would require the use of temporary or permanent liners to mitigate the potential risks of ground loss from the water-bearing cohesionless till soils during construction. In addition, the caissons must be socketed into the bedrock a sufficient length to provide the required bearing resistance. The presence of cobbles and boulders 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 very strong limestone 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 (i.e. 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).



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- **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 dense native till, below the embankment fill and Highway 401 grade fill. An integral abutment configuration would not be achievable with this foundation type at the 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 at or above the top of the glacial till deposit and therefore groundwater control would be required during excavation and construction. As the replacement structure is to include new abutments located near the existing abutments, removal of the existing abutments and piles must also be considered. This option has not been considered further in this report for the abutment foundations due to the increased excavation depths (e.g., excavations to depths of up to 9 m below ground surface would be required for a spread footing at the south abutment), groundwater control requirements and protection system requirements that would be required for a shallow foundation system in comparison to a deep foundation option at the abutment locations.
- **Spread footings “perched” on a compacted granular pad in the approach embankment:** Footings “perched” in the Wales 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. Construction of the new abutments behind the existing abutments could minimize potential interference, but excavation to remove the existing abutments (together with associated protection systems) would still be required, as would subexcavation and replacement of any existing loose fill or fill containing organics and/or deleterious material 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 refusal within very dense glacial till or to the bedrock, in an integral abutment configuration, and to support the central pier on spread footings founded on the glacial till.

6.4 Shallow Foundations

6.4.1 Founding Elevations

As identified above, a spread footing founded within the glacial till is the preferred option for supporting the central pier from a foundations engineering perspective. If adopted for the replacement structure, spread footings should be founded on the typically compact to dense glacial till below any existing fill, clayey silt or compressible organic soil.

The following table provides the maximum (highest) founding elevations recommended for design of the spread foundation for the central pier to be founded on the compact to dense glacial till deposit.

Foundation Element	Borehole Number	Founding Stratum	Footing Founding Elevation (m)
Central Pier	13-713	Compact to very dense till	Below 76.8



The existing grade fill should be removed within the footprint of the proposed pier foundation. The original design drawings indicate that the underside of the existing central pier footing is at about Elevation 75.9 m. Assuming that the proposed footing is within the footprint of the existing foundation, the excavation for the pier foundation would extend to up to about 2.6 m below the existing grade at the central pier location. In November 2014, the groundwater level was measured at Elevation 78.3 m in the well installed at the site. Therefore, excavation to the founding elevation presented above would require dewatering. A Non-Standard Special Provision has been provided in Appendix C to address this requirement.

The spread footing at the central pier 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 excavation to remove the existing fill below the design footing elevation is required, compacted Granular “A” fill could then be used to raise the foundation level to the design elevation to minimize the concrete requirements while still maintaining the required foundation depth of 1.7 m for frost protection purposes.

Deleterious materials including asphalt pieces as well as some organic matter were encountered in the fill materials at the site and a thin layer of clayey silt was present between the fill and till bearing stratum at Borehole 14-713. 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.

6.4.2 Geotechnical Resistance

Spread footings placed on the properly prepared glacial till deposit, at or below the design elevation 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 320 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 2006 Canadian Highway Bridge Design Code (CHBDC) 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 2006 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, $\tan \delta$ or $\tan \phi'$, may be taken as follows:

- Cast-in-place footing to concrete working slab: $\tan \delta = 0.6$
- Cast-in-place concrete working slab to glacial till: $\tan \phi' = 0.62$

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



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

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

6.5.1 Founding Elevations

The pier and abutments for the replacement structure may be supported on steel H-piles or closed-ended steel pipe (tube) piles driven to refusal within the lower, very dense portion of the till deposits or 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 table provides estimated pile tip elevations for steel H-piles or pipe piles founded on the bedrock.

Foundation Element	Borehole Number	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)
North Abutment	13-711	59.9	59.8
Central Pier	13-713	60.3	60.2
South Abutment	13-712	59.9	59.8

However, the results of the field investigation also indicate that auger refusal was met at several locations within the lower portion of the glacial till at the site, and diamond drilling techniques were required to penetrate through this deposit. This lower dense to very dense layer was encountered at about Elevation 69.3 m at the borehole put down near the north abutment, and about Elevation 62.2 m at the borehole put down near the south abutment. Based on these conditions, predrilling through the lower, very dense portions of the till deposits is expected to be required to advance the piles to the bedrock.

If end bearing on the limestone bedrock is preferred, and full penetration of the dense to very dense glacial till is required, it is recommended that a contingency item be provided to pre-auger through the embankment fill and deep glacial till at both abutments. The auger size should be chosen to loosen the soil within a diameter smaller than the size of the pile. For example, pre-augering for a 310 x 110 H-pile should be carried out using an auger with a cutting diameter no larger than about 300 mm. The loosened soil is to be left in place following augering. However, due to the additional efforts, costs, and time required for predrilling, it is recommended that that piles for the abutments should be designed based on the capacities provided for piles end-bearing within the glacial till as discussed in the Section 6.5.2.

The depth of pile penetration into the lower, very dense portions of glacial will vary, and will depend on the pile driving equipment and the subsurface conditions at the pile tip. However, at the north abutment it is anticipated that the pile tips could meet effective refusal at Elevations of 66.2 to 64.3 m. At the south abutment, the pile tips could meet effective refusal at Elevations 61.2 to 59.9 m (at the bedrock surface).

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 piles following removal of the existing pile cap and exposure of the existing H-piles. If driving adjacent to the existing H-piles is not considered to be feasible, consideration may be given to extraction of the existing piles.

The original design drawings indicate that the approximately 11.6 m long abutment H-piles were driven in to the loose to dense till with tip elevations of about 70.1 m. The existing piles likely derive axial capacity from both frictional shaft resistance as well as end bearing on the dense glacial till. Therefore, significant force may be required to break the soil adhesion bond and frictional resistance of the glacial till.

A removal method preferred by piling contractors uses a vibratory hammer during pulling of the piles. This method is not considered to be suitable at the abutments due to the potential for loosening of the embankment fill which would result in settlement. Alternatively, the soil/pile bond could be reduced by first driving the pipe piles deeper, and then pulled using conventional piling equipment or using a jack system on the pile.

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

Cobbles were encountered in the existing embankment fill at the borehole put down near the south abutment. Though no cobbles were encountered in the fill at the borehole put down at the north abutment, cobbles may also be present elsewhere in the north embankment fill. Diamond drill coring was required to penetrate portions of the lower glacial till deposit. 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 being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with a pile point (e.g., Titus Standard H Point, or similar) to facilitate penetration into/through the dense overburden, improve seating of the piles on the bedrock, and reduce the potential for damage to the piles during driving in accordance with OPSS 903 (Construction Specification for Deep Foundations). If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II (Foundation, Piles, Steel Tube Pile Driving Shoe).

6.5.2 Axial Geotechnical Resistance

For design of HP 310x110 piles driven to practical refusal within the very dense glacial till at the estimated tip elevations provided for refusal in the glacial till in Section 6.5.1, the factored axial geotechnical resistance at ULS may be taken as 1,600 kN. The axial geotechnical resistance at Serviceability Limit States (SLS, for less than 25 mm of settlement) for such piles may be taken as 1,300 kN. Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm diameter steel pipe piles having a minimum wall thickness of 9.5 mm.

For design of HP 310x110 piles driven to found on the limestone bedrock at the estimated tip elevations provided for bearing on the bedrock in Section 6.5.1, the factored axial geotechnical resistance at ULS may be taken as 2,000 kN. Serviceability Limit States (SLS) resistances do not apply to piles founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. As noted above, pre-drilling is expected to be required to advance the piles through the lower, very dense portions of the till.



Pile installation should be in accordance with OPSS 903 (*Construction Specification for Deep Foundations*). The drawings should incorporate the appropriate note stating that the piles should be equipped with pile points (e.g. Titus Standard H Point, or similar) 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.

Based on the incompressible nature of the glacial till that underlies the site, the placement of 0.9 m of additional embankment fill at the new abutments is not anticipated to produce ground settlements large enough to generate downdrag loads on the abutment piles.

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 2006 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: s_u is the undrained shear strength of the soil (kPa); and,
 B is the pile diameter/width (m).

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

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



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Location	Elevation (m)	Soil Type	n_h (MN/m ³)	s_u (kPa)
North Abutment	78.3 – PCL ¹	Compact to Very Dense Sand and Gravel (Fill)	6 to 15	-
	76.9 – 78.3	Compact Gravelly, Silty Sand (Fill)	2 to 4	-
	69.3 – 76.9	Loose to Compact Glacial Till	2 to 4	-
	59.9 – 69.3	Very Dense Glacial Till	8 to 11	-
	59.9	Bedrock	-	-
Central Pier	65.6 – PCL ¹	Compact Glacial Till	2 to 4	-
	60.3 – 65.6	Very Dense Glacial Till	8 to 11	-
	60.3	Bedrock	-	-
South Abutment	78.3 – PCL ¹	Compact to Dense Sand and Gravel (Fill)	6 to 15	-
	75.9 – 78.3	Loose to Very Dense Sand and Silt (Fill)	2 to 11	-
	62.3 – 75.9	Loose to Compact Glacial Till	2 to 6	-
	59.9 – 62.3	Very Dense Glacial Till	8 to 11	-
	59.9	Bedrock	-	-

Note: ¹ PCL = Pile Cap Level

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

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

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

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.8.7 of the *Commentary* to the 2006 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 2006 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 2006 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. The use of liners or casings will be required in order to advance the caissons through the overburden with minimal loss of ground. The casings should be extended so that they are “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

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

Rock-socketed caissons should be designed based on the side-wall (shaft) resistance of the rock socket and a factored geotechnical resistance at ULS of 2,000 kPa, provided that the caisson socket is within competent bedrock (i.e., RQD greater than 75 percent). This value assumes that the side wall of the socket will be cleaned of any cuttings or smeared material.

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

For a 0.9 m diameter caisson socketed 2 m in to the competent bedrock, this would equate to a factored axial geotechnical resistance at ULS of about 11,000 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 35°; and abutment wall heights less than 6 m without a retained soil system.

An integral abutment arrangement is considered feasible at this site since the flexible pile-supported abutment foundations discussed in Section 6.5 meet MTO's foundation criteria for integral abutments.

6.8 Seismic Considerations

The site is located near Cornwall, Ontario and according to Table A.3.1.1 of the 2006 CHBDC, the zonal acceleration ratio, A , applicable to this site is 0.2. The corresponding acceleration related seismic zone, Z_a , is 4.

Below the groundwater table, the soil at the site generally consists of an extensive deposit of loose to very dense glacial till consisting of sand, gravel, silt, cobbles, and boulders overlying limestone bedrock. The measured SPT "N" values obtained in the glacial till ranged widely varying from 3 to 160 blows per 0.3 m of penetration, but were typically between about 7 and 20 blows per 0.3 m of penetration in the upper, loose to dense deposit (above about Elevation 69.3 m at the borehole put down near the north abutment, and about Elevation 22.2 m at the borehole put down near the south abutment). The results of grain size distribution testing carried out on samples of the glacial till measured fines contents between 25 and 50 percent.

A preliminary liquefaction assessment was carried out using the subsurface information collected at the site. The methodology used for the assessment is consistent with those outlined in Section C4.6.2 of the *Commentary* to the 2006 CHBDC and state-of-practice techniques. The assessment involved comparing the cyclic shear stresses applied to the soil by the design ground motions outlined in the 2006 CHBDC, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

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

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

6.9 Lateral Earth Pressures for Design

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

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

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



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- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular*), 3190.101 (*Walls, Retaining and Abutment, Wall Drain*), and 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with 2006 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 2006 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 2006 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) on Figure C6.20 of the *Commentary* to the 2006 CHBDC, the pressures are based on the existing embankment fill and the following parameters (unfactored) may be used:

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

- For Case (b) on Figure C6.20 of the *Commentary* to the 2006 CHBDC, the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:



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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 2006 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 2006 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 2006 CHBDC and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.3$). For structures which allow lateral yielding, (k_h) is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.1$).
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case (a) and Case (b)) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.



Seismic Active Pressure Coefficients, K_{AE}

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

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

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

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

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

6.10 Approach Embankments

It is understood that the overall grade of Wales Road will be raised up to about 0.9 m to accommodate the additional structure depth required for the proposed replacement bridge and clearance above Highway 401. In general, the existing width and alignment of Wales 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 Wales Road embankments, the road structure is generally underlain by embankment fill consisting of gravel and sand, overlying silty sand till (containing gravel, cobbles, and boulders), and limestone bedrock.

6.10.1 General Embankment Construction

It is recommended that all organic material or existing loose surficial fill present within the widening footprint be stripped prior to placement of new embankment fill. The existing sand and gravel embankment fill may be left in place.

The new embankment fill associated with the grade raise for the bridge replacement should be placed and compacted in accordance with OPSS.PROV 206 (*Construction Specifications for Grading*) and OPSS.PROV 501 (*Construction Specification for Compacting*). Benching of the existing Wales Road



embankment side slopes should be carried out to “key in” the new fill materials in areas where the embankment is widened or additional fill is placed to accommodate the grade raise, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil (OPSS 802 – *Construction Specification for Topsoil*) and seeding (OPSS.PROV 804 – *Construction Specification for Seed and Cover*) or pegged sod (OPSS 803 – *Construction Specification for Sodding*) is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS.PROV 804 (*Seed and Cover*).

6.10.2 Global Stability

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

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

Provided that the approach embankment side slopes are maintained 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 Wales Road grade is to be increased more than 0.9 m above the existing grades, the embankment side-slope stability will have to be re-assessed.

6.10.3 Settlement

The additional loading imposed by the proposed 0.9 m grade raise will result in an increase in effective stress within the glacial till that underlies the site. The elastic compression of the existing glacial till 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 sand and gravel 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

The excavations for a spread footing or pile cap are expected to extend through the grade fill at the central pier, and into the typically compact to dense silty sand till. This excavation would extend below the ground water table up to about 2 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 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). It is anticipated that excavations in the compact to dense glacial till below the water table could be maintained at 1H:1V, but may need to be flattened if localized layers of fine sand or silt are encountered. However, with appropriate groundwater control, it is anticipated that temporary excavation slopes through the glacial till can be maintained at 1H:1V.

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

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

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

6.11.2 Groundwater and Surface Water Control

Based on readings taken at the monitoring well installed in the northern embankment and groundwater conditions observed in the boreholes immediately following drilling, the groundwater level is expected to be about 6 m below the existing Wales Road grade at the abutment locations and within about 1 m of the existing Highway 401 grade at the central pier location.



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

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

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

6.11.3 Subgrade Protection

If the pier is 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.

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



FOUNDATION REPORT REPLACEMENT OF HIGHWAY 401 UNDERPASS AT WALES ROAD

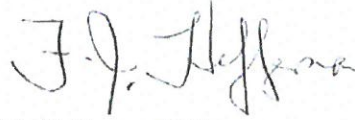
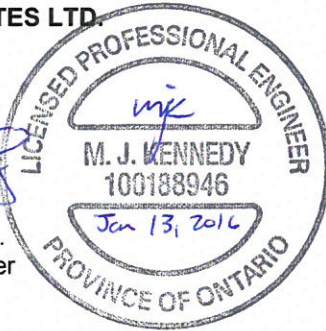
7.0 CLOSURE

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

GOLDER ASSOCIATES LTD.



Matt Kennedy, P.Eng.
Geotechnical Engineer



Fin Heffernan, P.Eng.
Designated MTO Foundations Contact



SG/MJK/KN/FJH/ob

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FOUNDATION REPORT REPLACEMENT OF HIGHWAY 401 UNDERPASS AT WALES ROAD

Table 1 – Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-piles driven to refusal in glacial till or to found on bedrock	<ul style="list-style-type: none"> Feasible for support of bridge replacement Preferred option from a foundations perspective 	<ul style="list-style-type: none"> 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 	<ul style="list-style-type: none"> Potential for encountering obstructions (cobbles and/or boulders) in the lower glacial till stratum 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 	<ul style="list-style-type: none"> Moderate cost 	<ul style="list-style-type: none"> High risk of driven H-piles “hanging up” in glacial till; recommend piles be designed assuming they meet refusal in the glacial till unless predrilling is specified to allow installation to the bedrock surface.
Steel pipe (tube) piles, driven to refusal in glacial till or found on bedrock	<ul style="list-style-type: none"> Feasible for support of bridge replacement 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings, reducing depth of excavation and temporary protection system Higher geotechnical resistances and negligible settlement 	<ul style="list-style-type: none"> Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; this could result in more piles “hanging up”, 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 	<ul style="list-style-type: none"> Moderate cost 	<ul style="list-style-type: none"> High risk of pipe piles “hanging up” in glacial till



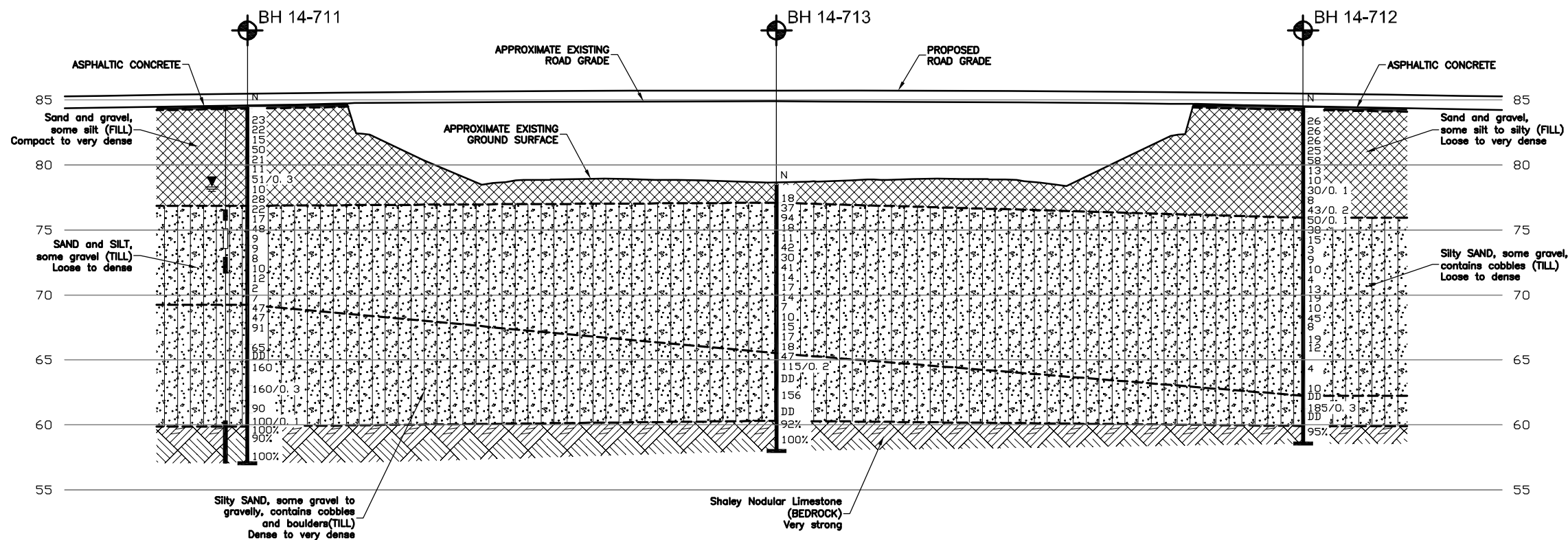
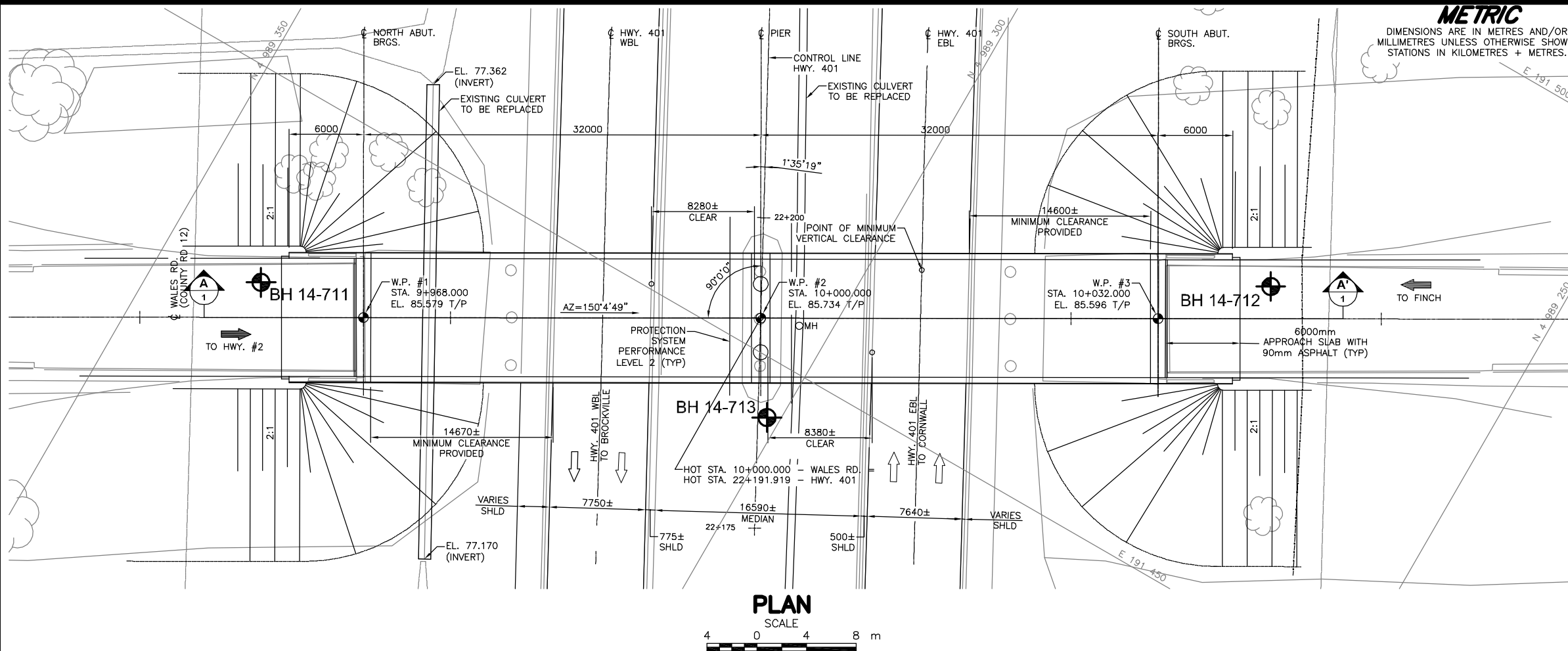
FOUNDATION REPORT REPLACEMENT OF HIGHWAY 401 UNDERPASS AT WALES ROAD

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons founded on bedrock	<ul style="list-style-type: none">Feasible	<ul style="list-style-type: none">Could eliminate the need for deep foundation cap at the central pier and allow for structural continuity between caissons and piersConstruction from existing grade would reduce excavation and groundwater control requirements (reduced impact on Highway 401)	<ul style="list-style-type: none">Significant caisson length required (at least 18 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 depositRock coring, churn drilling or chisel drilling required to form rock sockets in strong to very strong bedrockConflict with existing abutment piles likely, requiring removal of existing piles	<ul style="list-style-type: none">Construction of deep caissons more expensive than alternative foundation options	<ul style="list-style-type: none">Ground conditions make caisson installation difficultSignificant length required would result in high foundation construction costSome risk of difficulty in removing existing abutment piles to avoid conflict with new caissons



FOUNDATION REPORT REPLACEMENT OF HIGHWAY 401 UNDERPASS AT WALES ROAD

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread/strip footings on silty sand glacial till	<ul style="list-style-type: none"> Feasible at central pier Feasible but likely not practical at abutment locations 	<ul style="list-style-type: none"> Existing structure supported on shallow foundations at piers, and foundations have performed reasonably 	<ul style="list-style-type: none"> Significant excavations to depths of greater than 9 m would be required through the existing embankments at the abutment locations Excavation at the central 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 more settlement than deep foundations Precludes use of integral abutments; potentially greater maintenance required 	<ul style="list-style-type: none"> Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration 	<ul style="list-style-type: none"> Risk of instability of existing embankment slopes without appropriate temporary protection measures during excavation at abutments to significant depth



CONT No.
WP No. 4055-08-01
GWP No. 4064-12-00

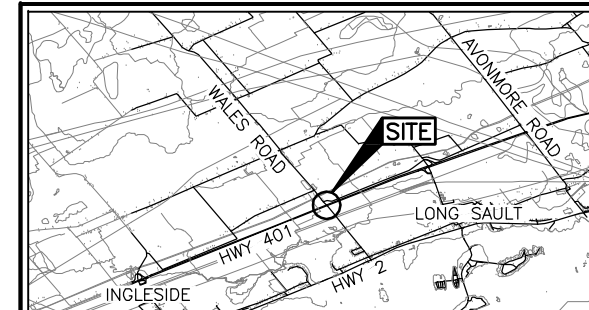
HIGHWAY 401 UNDERPASS
AT WALES ROAD
BOREHOLE LOCATIONS AND
SOIL STRATA



SHEET



Golder Associates Ltd.
OTTAWA ONTARIO, CANADA



KEY PLAN

SCALE 0 2 4 KM

LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- DD Rotary Diamond Drill Coring
- 100% Total Core Recovery (REC)
- WL in piezometer
- Seal
- Piezometer

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
14-711	84.5	4989341.8	191434.8
14-712	84.4	4989271.2	191475.1
14-713	78.5	4989301.0	191445.7



NOTES

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

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

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

REFERENCE

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

NO.	DATE	BY	REVISION
Geocres No. 31H-253			
HWY. 401	PROJECT NO. 12-1121-0099		DIST. Eastern
SUBM'D. MJK	CHKD. MJK	DATE: 01/12/2016	SITE: 31-162
DRAWN: JM	CHKD. MJK	APPD. FJH	DWG. 1



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		III. SOIL DESCRIPTION		
AS	Auger sample	(a) Cohesionless Soils		
BS	Block sample	Density Index (Relative Density)	N	
CS	Chunk sample		Blows/300 mm	
DO or DP	Seamless open-ended, driven or pushed tube samplers		Or Blows/ft.	
DS	Denison type sample		0 to 4	
FS	Foil sample		4 to 10	
RC	Rock core		10 to 30	
SC	Soil core		30 to 50	
SS	Split spoon sampler		over 50	
ST	Slotted tube	(b) Cohesive Soils C _u or S _u	Consistency	
TO	Thin-walled, open		kPa	
TP	Thin-walled, piston		Psf	
WS	Wash sample		0 to 12	
DT	Dual tube sample		12 to 25	
DD	Diamond drilling		25 to 50	
			50 to 100	
			100 to 200	
		Over 200		
		0 to 250		
		250 to 500		
		500 to 1,000		
		1,000 to 2,000		
		2,000 to 4,000		
		Over 4,000		
II. PENETRATION RESISTANCE		IV. SOIL TESTS		
Standard Penetration Resistance (SPT), N:		w	Water content	
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split spoon sampler for a distance of 300 mm (12 in.).		w _p or PL	Plastic limited	
Dynamic Cone Penetration Resistance (DCPT); N_d:		w _l or LL	Liquid limit	
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive an uncased 50 mm (2 in.) diameter, 60 ⁰ cone attached to “A” size drill rods for a distance of 300 mm (12 in.).		C	Consolidaiton (oedometer) test	
PH: Sampler advanced by hydraulic pressure		CHEM	Chemical analysis (refer to text)	
PM: Sampler advanced by manual pressure		CID	Consolidated isotropically drained triaxial test ¹	
WH: Sampler advanced by static weight of hammer		CIU	Consolidated isotropically undrained triaxial test with porewater pressure measurement ¹	
WR: Sampler advanced by weight of sampler and rod		D _R	Relative density	
Cone Penetration Test (CPT):		DS	Direct shear test	
An electronic cone penetrometer with a 60 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.		G _s	Specific gravity	
		M	Sieve analysis for particle size	
		MH	Combined sieve and hydrometer (H) analysis	
		MPC	Modified Proctor compaction test	
		SPC	Standard Proctor compaction test	
		OC	Organic content test	
		SO ₄	Concentration of water-soluble sulphates	
		UC	Unconfined compression test	
		UU	Unconsolidated undrained triaxial test	
		V	Field vane test (LV-laboratory vane test)	
		γ	Unit weight	

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes:

$$^1 \tau = c' + \sigma' \tan \phi'$$

$$^2 \text{ shear strength} = (\text{compressive strength}) / 2$$

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of rock material weathering

Faintly Weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to (W.R.T.) Core Axis

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

Description and Notes

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

Abbreviations

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

PROJECT 12-1121-0099-1710		RECORD OF BOREHOLE No 14-711		SHEET 1 OF 4	METRIC
G.W.P. 4055-08-01		LOCATION N 4989341.8 ; E 191434.8		ORIGINATED BY DWM	
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core		COMPILED BY JM	
DATUM Geodetic		DATE October 17-22, 2014		CHECKED BY MJK	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W _p	W	W _L					
84.5	GROUND SURFACE																			
0.0	ASPHALTIC CONCRETE																			
84.1	Gravelly sand (FILL) Brown																			
0.4	Sand and gravel, trace silt (FILL) Compact Grey-brown Dry		1	SS	23															
			2	SS	22															
			3	SS	15															
81.5	Silty sand (FILL) Very dense to dense Grey-brown Moist		4	SS	50															
80.7	Gravel, some sand, contains cobbles (FILL) Compact Brown Dry		5	SS	21															
79.9	Sandy silt to silty sand, some gravel, trace to some clay (FILL) Compact Brown Moist		6	SS	11															
	- Contains asphalt pieces from 5.3 m to 5.6 m depth		7	SS	51/0.3															
78.9	Gravelly silty sand, trace clay (FILL) Loose to compact Brown Moist		8	SS	10															
			9	SS	28															
76.9	SAND and SILT, trace to some gravel, contains cobbles (TILL) Compact Brown Moist to wet		10	SS	22															
7.6			11	SS	17															
75.4	Silty SAND, trace gravel (TILL) Dense to loose Grey Wet		12	SS	48															
9.1																				

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IMGINT\1211210099.GPJ GAL-GTA.GDT 10/29/15 JM

PROJECT <u>12-1121-0099-1710</u>		RECORD OF BOREHOLE No 14-711		SHEET 2 OF 4		METRIC	
G.W.P. <u>4055-08-01</u>		LOCATION <u>N 4989341.8 ; E 191434.8</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>October 17-22, 2014</u>		CHECKED BY <u>MJK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIQUID MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
--- CONTINUED FROM PREVIOUS PAGE ---								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L				
								20 40 60 80 100					25 50 75				
73.8	Silty SAND, trace gravel (TILL) Dense to loose Grey Wet		13	SS	9		74									16 47 29 8	
10.7	Silty SAND, some gravel, trace clay (TILL) Compact to very loose Grey Wet		14	SS	9		73										
			15	SS	8		72										
			16	SS	10		71										
			17	SS	12		70										
			18	SS	2		71										
70.0							70										
14.5	Sandy GRAVEL, some silt to silty, trace clay (TILL) Loose Grey Wet		19	SS	7		69									50 25 19 6	
69.3							68										
15.2	Gravelly Silty SAND, trace clay, contains cobbles and boulders (TILL) Dense to very dense Grey Wet		20	SS	47		67										
			21	SS	47		66										
			22	SS	91		65										
66.2																	
18.3	SAND and SILT, trace gravel, contains cobbles and boulders (TILL) Very dense Grey Wet		23	SS	65		66									28 41 27 4	
	- Coring required to penetrate from 18.9 m to 19.7 m depth		24	RC	DD												

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IMGINT\1211210099.GPJ GAL-GTA.GDT 10/29/15 JM

PROJECT		12-1121-0099-1710		RECORD OF BOREHOLE No 14-711		SHEET 3 OF 4		METRIC												
G.W.P.		4055-08-01		LOCATION		N 4989341.8 ; E 191434.8		ORIGINATED BY DWM												
DIST		Eastern HWY 401		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core		COMPILED BY JM												
DATUM		Geodetic		DATE		October 17-22, 2014		CHECKED BY MJK												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20	40	60	80	100						20	40	60
--- CONTINUED FROM PREVIOUS PAGE ---																				
63.0	SAND and SILT, trace gravel, contains cobbles and boulders (TILL) Very dense Grey Wet - Coring required to penetrate from 18.9 m to 19.7 m depth		25	SS	160															
21.5	Silty SAND, trace gravel (TILL) Very dense Grey Wet		26	SS	160/0.3															
61.5	Sandy SILT, some sand seams Very dense Grey Wet		27	SS	90															
60.9	Silty SAND, trace gravel, contains cobbles and boulders (TILL) Very dense Grey Wet																			
59.9	Shaley Nodular Limestone (BEDROCK) Bedrock cored from depths of 24.6 m to 27.4 m For bedrock coring details refer to Record of Drillhole 14-711		28	SS	100/0.1															
24.6			1	RC	REC 100%															
			2	RC	REC 100%															
			3	RC	REC 100%															
57.1	END OF BOREHOLE																			
27.4	NOTES: 1. Water level in well screen at a depth of 6.2 m below ground surface (Elev. 78.3 m), measured on November 26, 2014.																			

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IMG\1211210099.GPJ GAL-GTA.GDT 10/29/15 JM

PROJECT: 12-1121-0099-1710

RECORD OF DRILLHOLE: 14-711

SHEET 4 OF 4

LOCATION: N 4989341.8 ; E 191434.8

DRILLING DATE: October 17-22, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC Drilling

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

1 : 50



LOGGED: DWM

CHECKED: MJK

GTA-RCK 031 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IM\GINT\1211210099.GPJ GAL-MISS.GDT 10/29/15 JM

PROJECT <u>12-1121-0099-1710</u>		RECORD OF BOREHOLE No 14-712		SHEET 1 OF 4		METRIC	
G.W.P. <u>4055-08-01</u>		LOCATION <u>N 4989271.2; E 191475.1</u>		ORIGINATED BY <u>DWM</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>October 14-16, 2014</u>		CHECKED BY <u>MJK</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20	40	60	80		100	w _p	w	w _L			
84.4	GROUND SURFACE																			
0.0	ASPHALTIC CONCRETE																			
0.3	Gravelly sand (FILL) Brown Sand and gravel, trace silt (FILL) Compact Grey-brown Dry		1	SS	26															
			2	SS	26															
			3	SS	26															
81.5	Gravelly silty sand, contains cobbles (FILL) Compact Grey-brown Moist		4	SS	25															
2.9			5	SS	58															
			6	SS	13															
			7	SS	10															
78.3	Silty sand, some gravel, contains asphalt pieces (FILL) Brown to black Moist		8	SS	30/0.1															
6.1																				
77.5	Layered silty sand/sandy silt, trace gravel and organic matter, contains cobbles (FILL) Loose Grey-brown Moist		9	SS	8															
6.9			10	SS	43/0.2															
75.9	Silty SAND, some gravel to gravelly, contains cobbles (TILL) Dense Brown Moist to wet		11	SS	50/0.1															
8.5																				
			12	SS	38															

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IMGINT\1211210099.GPJ GAL-GTA.GDT 10/29/15 JM

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

PROJECT		12-1121-0099-1710		RECORD OF BOREHOLE No 14-712		SHEET 3 OF 4		METRIC											
G.W.P.		4055-08-01		LOCATION		N 4989271.2 ; E 191475.1		ORIGINATED BY											
DIST		Eastern HWY 401		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core		COMPILED BY											
DATUM		Geodetic		DATE		October 14-16, 2014		CHECKED BY											
JMK																			
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W			W _L	GR	SA
62.3	Gravelly Silty SAND, trace clay (TILL) Loose to dense Grey Wet		25	SS	4														
22.2	SAND and SILT, some gravel, trace clay, contains cobbles and boulders (TILL) Very dense Grey Wet - Coring required to penetrate from 22.2 m to 24.5 m depth																		
			26	SS	10														
			27	SS	185/0.3														
59.9	Nodular Limestone (BEDROCK)		1	RC	DD														
24.5	Bedrock cored from depths of 24.5 m to 25.8 m For bedrock coring details refer to Record of Drillhole 14-712		1	RC	REC 100%														
58.6	END OF BOREHOLE																		
25.8																			

PROJECT: 12-1121-0099-1710

RECORD OF DRILLHOLE: 14-712

SHEET 4 OF 4

LOCATION: N 4989271.2 ;E 191475.1

DRILLING DATE: October 14-16, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY												FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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DEPTH SCALE

1 : 50



LOGGED: DWM

CHECKED: MJK

GTA-RCK 031 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IMG\INT\1211210099.GPJ GAL-MISS.GDT 10/29/15 JM

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

PROJECT 12-1121-0099-1710		RECORD OF BOREHOLE No 14-713		SHEET 2 OF 4		METRIC														
G.W.P. 4055-08-01		LOCATION N 4989301.0; E 191445.7		ORIGINATED BY DWM																
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core		COMPILED BY JM																
DATUM Geodetic		DATE October 28-29, 2014		CHECKED BY MJK																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m³	GR SA SI CL			
								20 40 60 80 100	20 40 60 80 100	25 50 75										
--- CONTINUED FROM PREVIOUS PAGE ---																				
66.3	Silty SAND, some gravel, trace clay, contains cobbles (TILL) Loose to compact Grey Wet		13	SS	10		68													
12.2	Silty SAND, some gravel, trace clay, contains cobbles and boulders (TILL) Compact Grey Wet		14	SS	15		67													
65.6	Sandy SILT, some clay and gravel (TILL) Dense Grey Wet		15	SS	17		66													
13.0	Gravelly Silty SAND, contains cobbles and boulders (TILL) Very dense Grey Moist		16	SS	18		65													
64.8	- Coring required to penetrate from 14.6 m to 16.1 m depth		17	SS	47		64													
13.7	- Coring required to penetrate from 17.2 m to 17.8 m depth		18	SS	115/0.2		63													
60.7	GRAVEL and COBBLES (TILL)		19	RC	DD		62													
17.8	Nodular Limestone (BEDROCK)		20	SS	156		61													
60.3	Bedrock cored from depths of 18.2 m to 20.5 m		21	RC	DD		60													
18.2	For bedrock coring details refer to Record of Drillhole 14-713		1	RC	REC 100%		59													
			2	RC	REC 100%															

Continued Next Page

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

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

SHEET 4 OF 4

DATUM: Geodetic

DRILLING CONTRACTOR: CCC Drilling

[illegible]

DEPTH SCALE

1 : 50



LOGGED: DWM

CHECKED: MJK



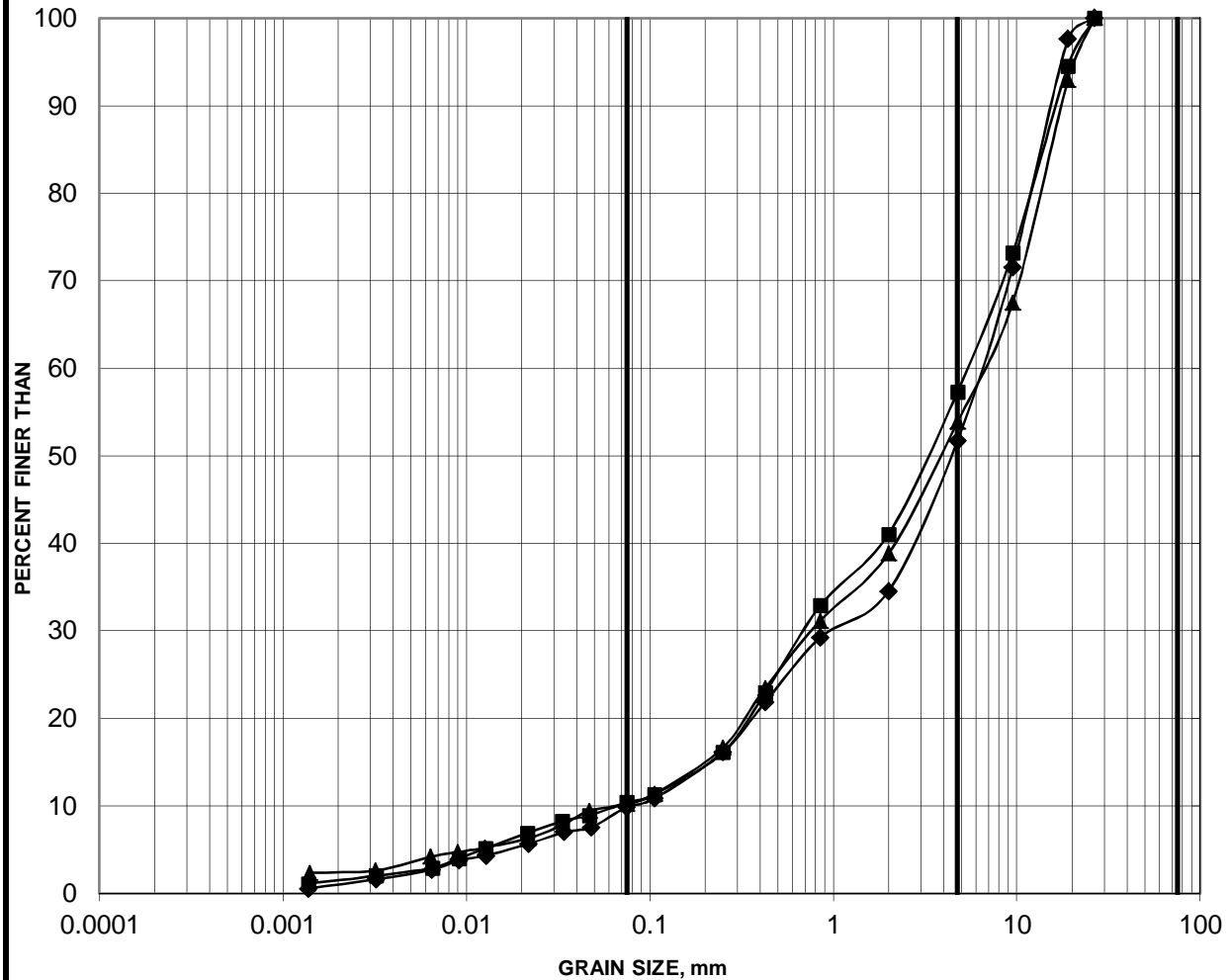
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND AND GRAVEL (FILL)



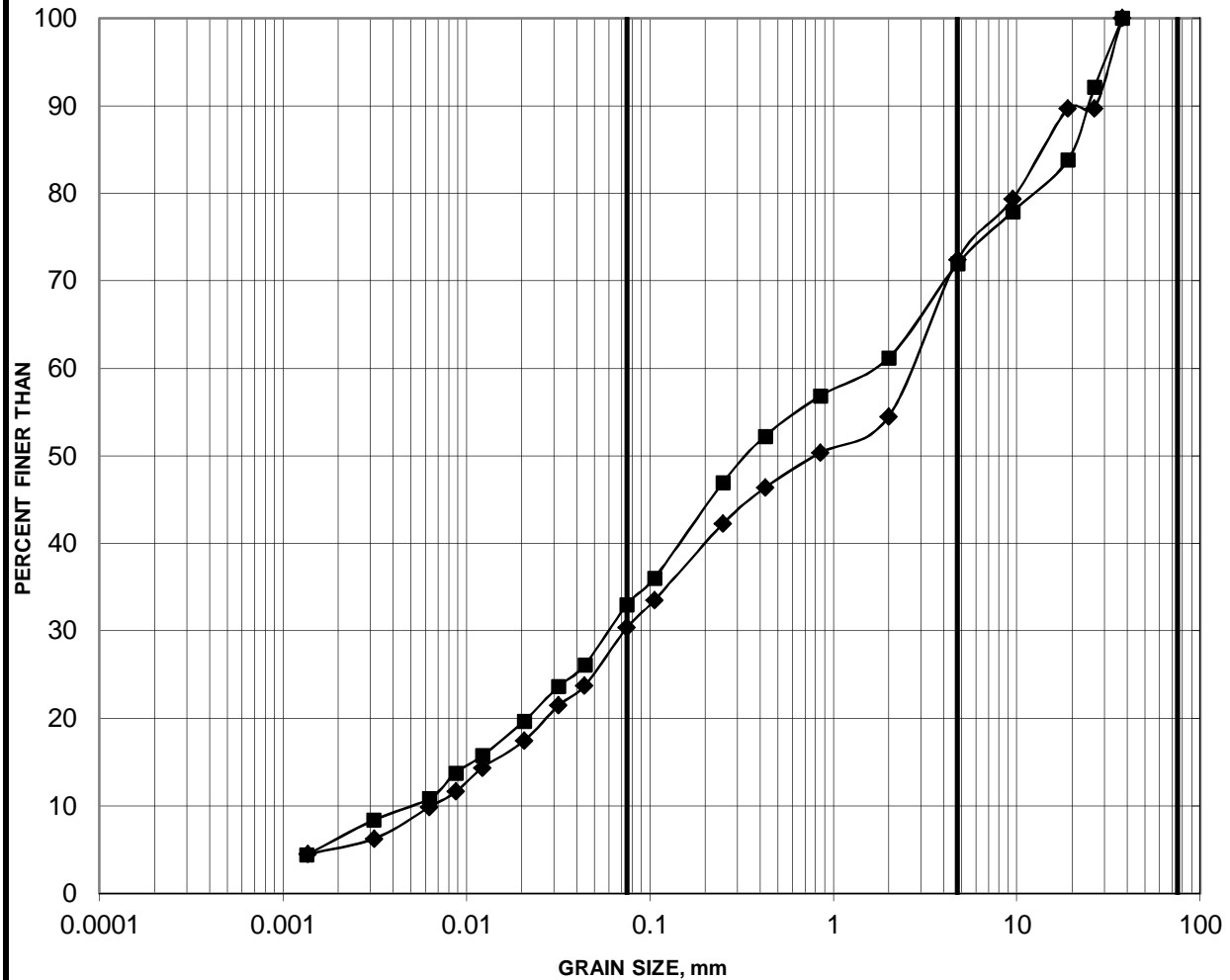
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
■ 14-711	2	1.52-2.13
◆ 14-712	3	2.29-2.90
▲ 14-713	1	0.76-1.37

GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty, Gravelly SAND (EMBANKMENT FILL)



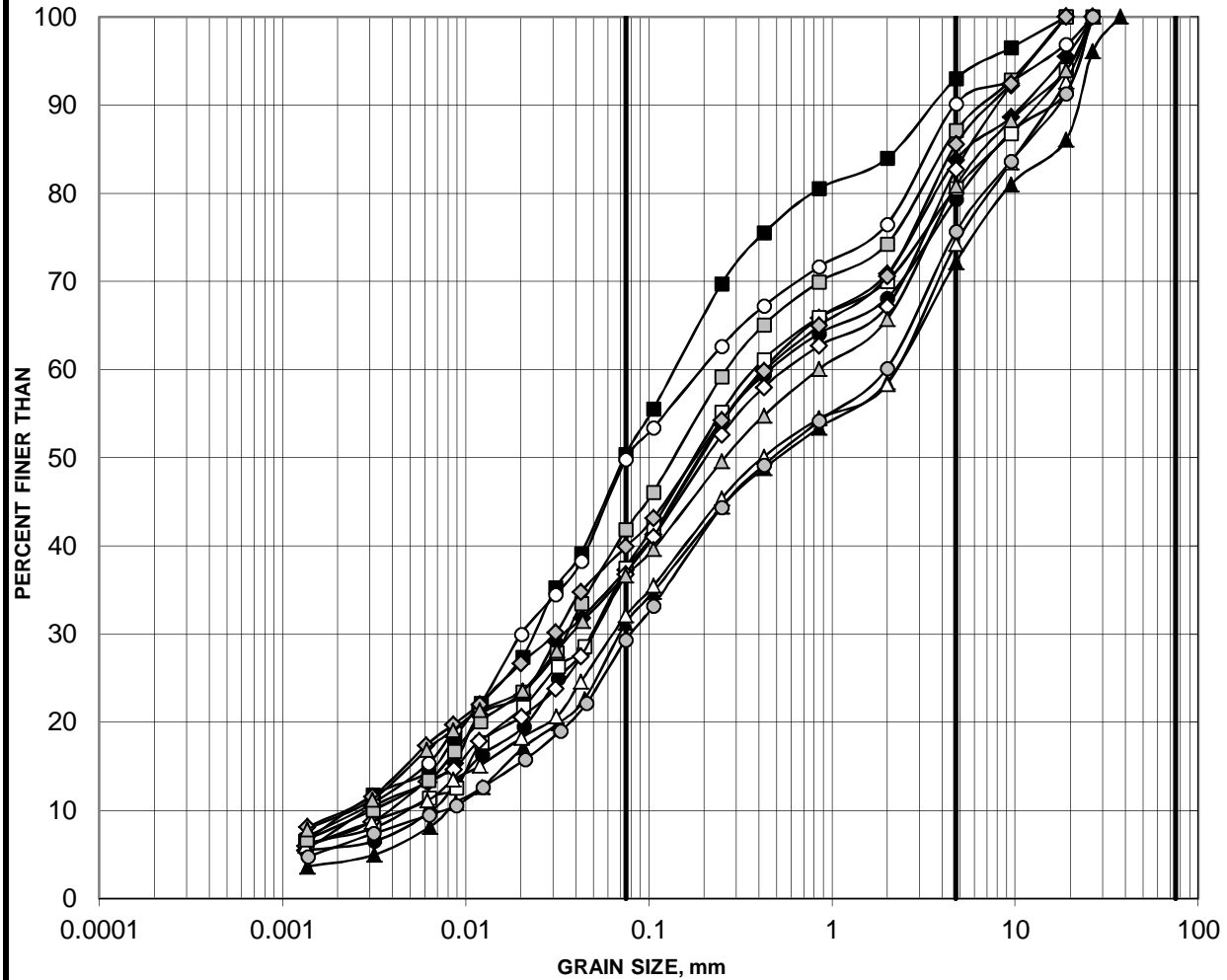
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
14-711	9	6.86-7.47
14-712	4	3.05-3.66

GRAIN SIZE DISTRIBUTION

FIGURE B3

SILT AND SAND, SOME GRAVEL TO GRAVELLY (GLACIAL TILL)



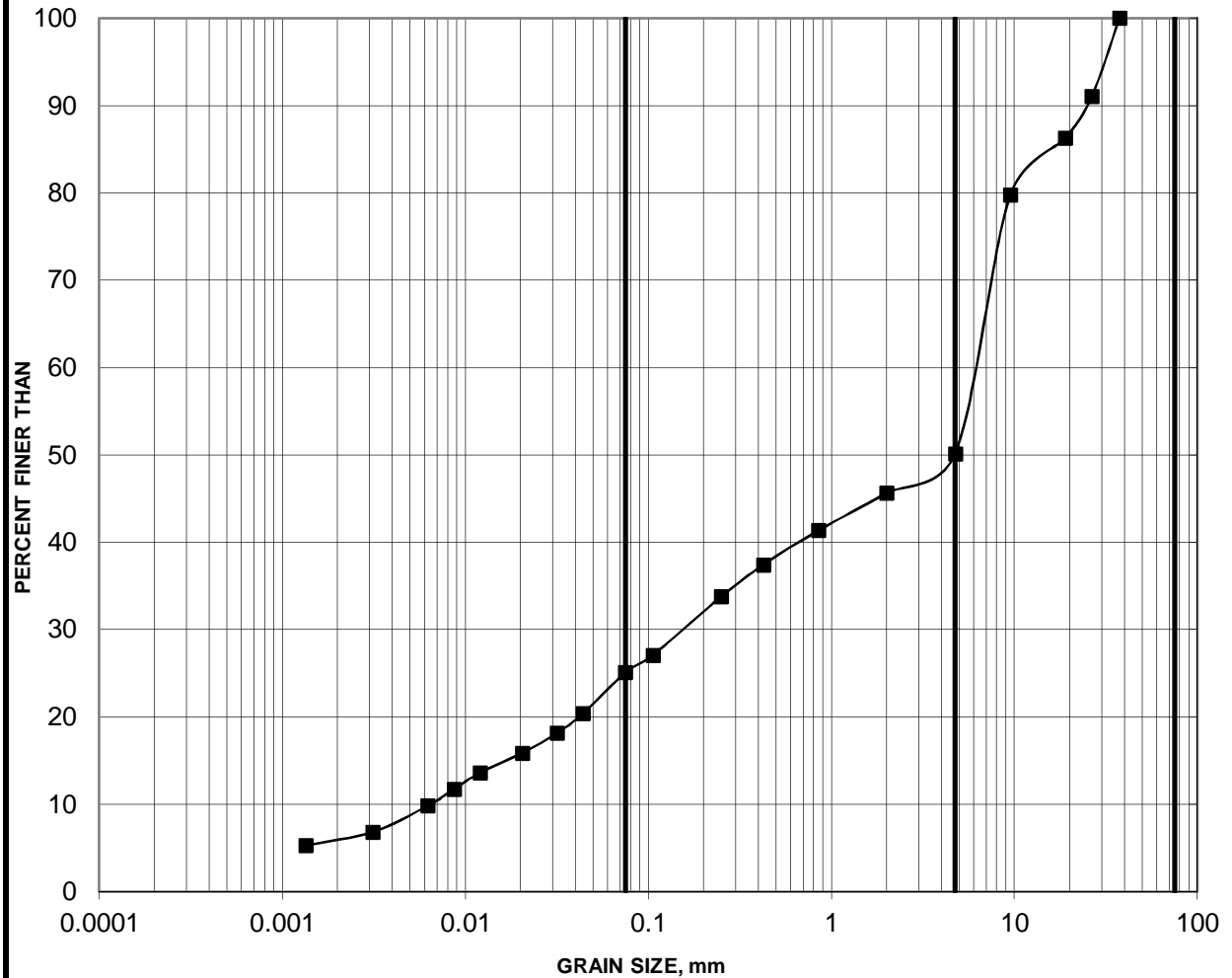
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
14-711	11	8.38-8.99
14-711	15	11.43-12.04
14-711	21	16.01-16.62
14-712	12	9.15-9.76
14-712	14	10.67-11.28
14-712	17	12.96-13.57
14-712	22	16.58-17.19
14-712	27	23.17-23.48
14-713	3	2.29-2.90
14-713	11	8.38-8.99
14-713	15	11.43-12.04
14-713	20	15.94-16.40

GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY, SANDY GRAVEL (GLACIAL TILL)

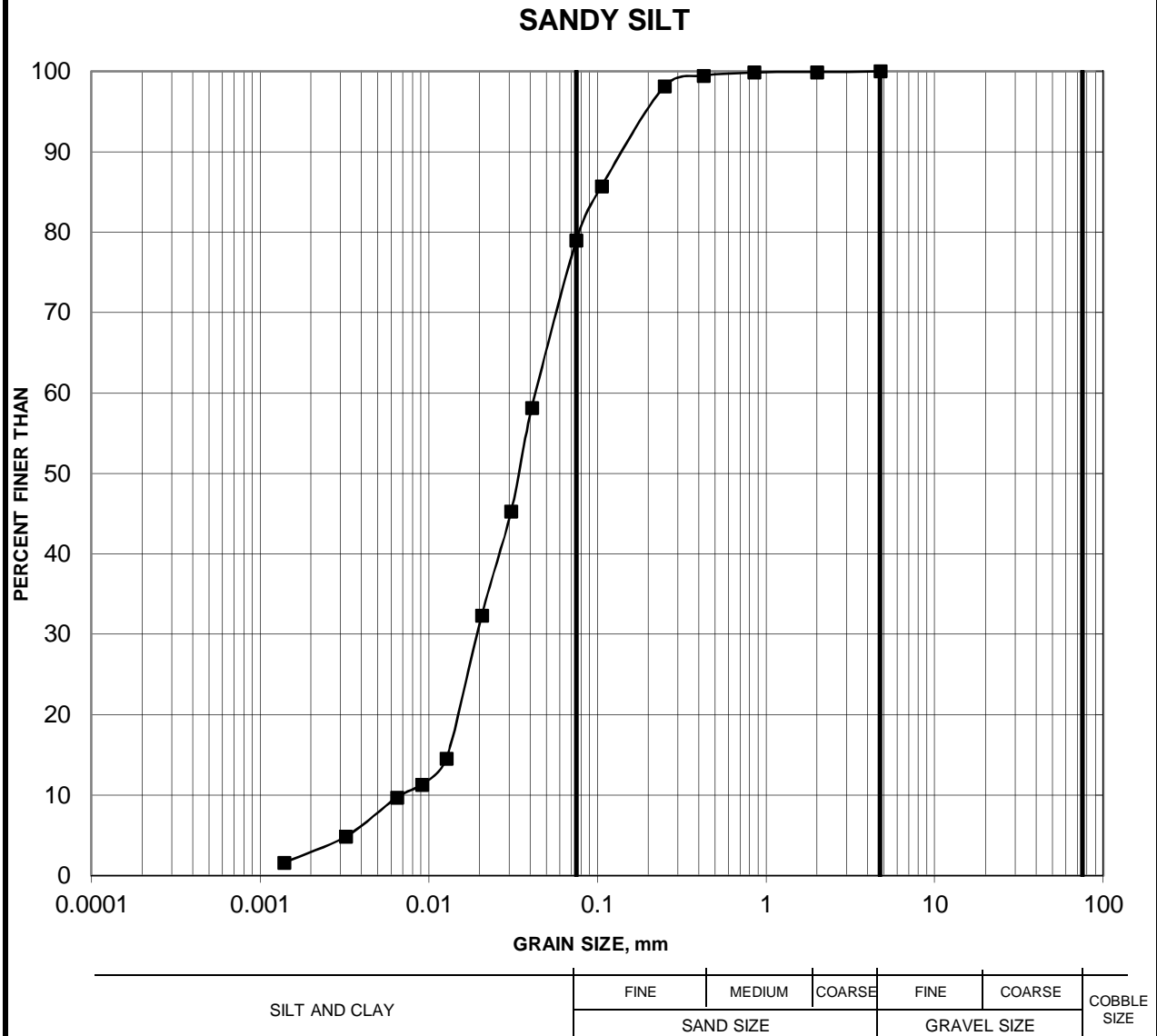


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
—■— 14-711	19	14.48-15.09

GRAIN SIZE DISTRIBUTION

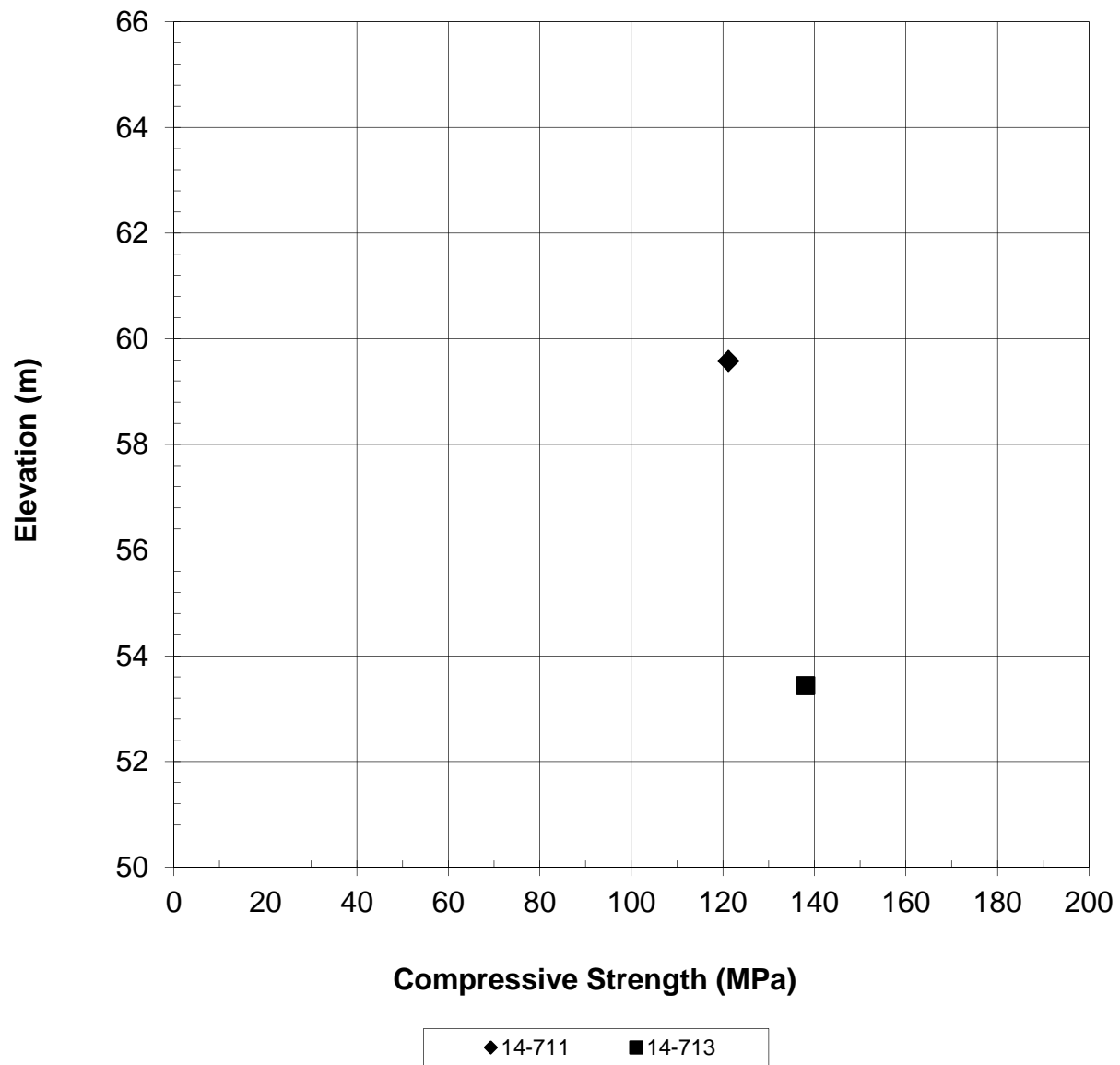
FIGURE B5



Borehole	Sample	Depth (m)
—■ 14-711	27	22.95-23.56

**SUMMARY OF LABORATORY COMPRESSIVE STRENGTH
UNCONFINED COMPRESSION TESTS**

FIGURE B6





APPENDIX C

Non-Standard Special Provisions



FOUNDATION REPORT REPLACEMENT OF HIGHWAY 401 UNDERPASS AT WALES ROAD

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.



FOUNDATION REPORT REPLACEMENT OF HIGHWAY 401 UNDERPASS AT WALES ROAD

WORKING SLAB – Item No.

Special Provision

1.0 SCOPE

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

2.0 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling – Structures

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – Not Used

5.0 MATERIALS

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

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

7.01 Excavation

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

7.02 Protection of Founding Soil

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

7.03 Dewatering

Dewatering shall be carried out in accordance with OPSS 902.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

10.01 Working Slab – Item

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

END OF SECTION



FOUNDATION REPORT REPLACEMENT OF HIGHWAY 401 UNDERPASS AT WALES ROAD

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 Wales 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 Wales 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 Wales 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



FOUNDATION REPORT REPLACEMENT OF HIGHWAY 401 UNDERPASS AT WALES ROAD

DEEP FOUNDATIONS – Item No.

Special Provision

1.0 SCOPE

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

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

2.0 BASIS OF PAYMENT

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

END OF SECTION



APPENDIX D

Bedrock Core Photographs



Bedrock Core Photographs Borehole 14-711 (24.6m to 27.4m)

Figure D1

24.61m



27.43m EOH



Bedrock Core Photographs Borehole 14-712 (24.5m to 25.8m)

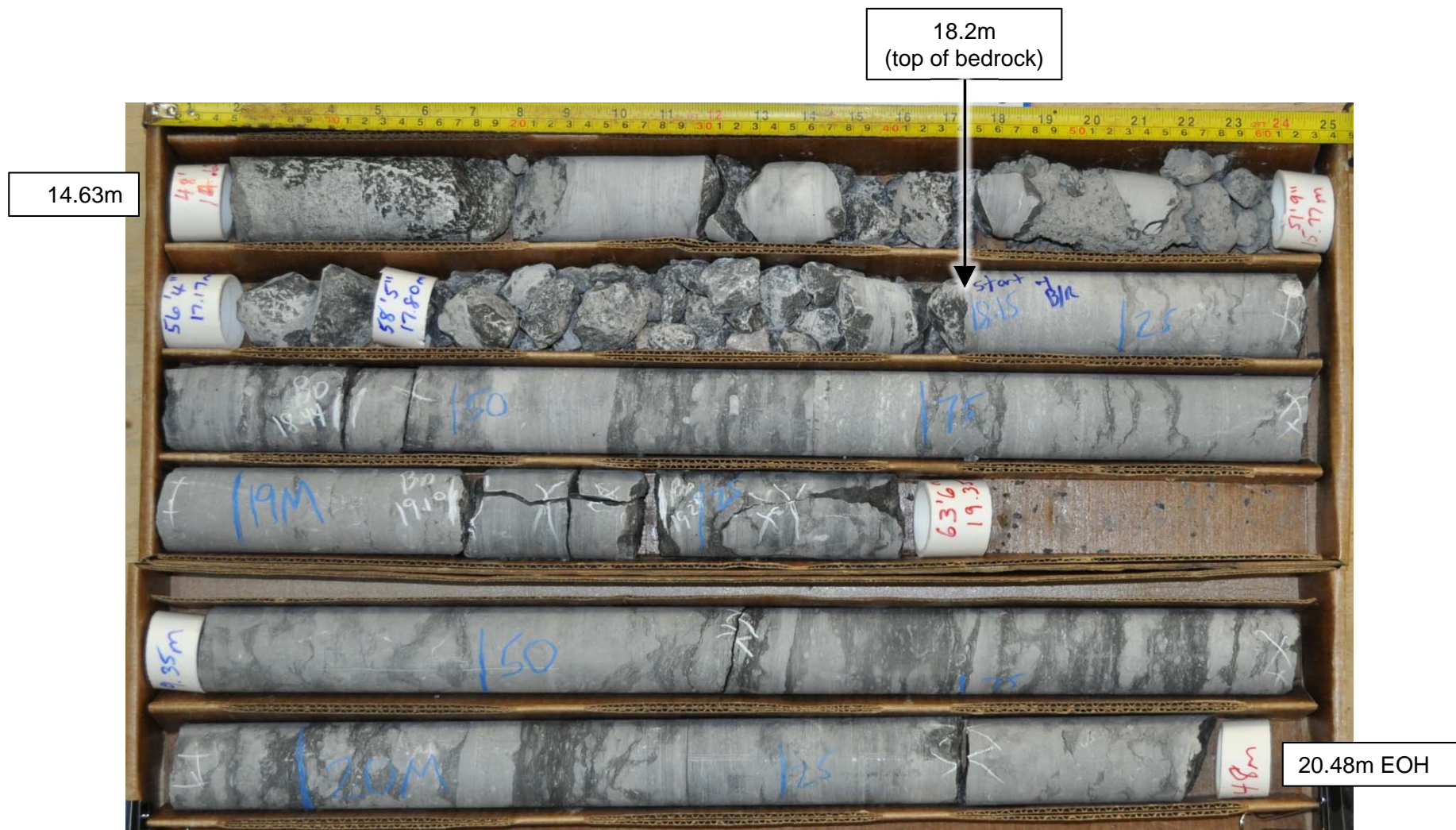
Figure D2





Bedrock Core Photographs Borehole 14-713 (18.2m to 20.4m)

Figure D3



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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