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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN REPORT
CLYDE AVENUE OVERPASS BRIDGE WIDENING
STRUCTURE SITE 3-43
HIGHWAY 417
W.P. 4058-01-00**

Submitted to:

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July 2007

Geocres 31G5-211
05-1120-210-2000-1



PART A

**FOUNDATION INVESTIGATION REPORT
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STRUCTURE SITE 3-43
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the rehabilitation of five bridges on Highway 417 in the City of Ottawa. The section of Highway 417 included in this assignment (W.P. 4058-01-00) extends from Maitland Avenue to Island Park Drive.

Foundation investigation services are required for the following components under W.P. 4058-01-00:

- Bridge widenings at Clyde Avenue, Carling Avenue Eastbound (EB), Kirkwood Avenue, Carling Avenue Westbound (WB), and Merivale Road.
- Eighteen retaining walls, including both new walls as well as replacement of some existing walls.

This report addresses the proposed widening of the bridge over Clyde Avenue including the wing (retaining) walls and approach embankment widening. A separate report addresses the retaining walls located outside the bridge approaches.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated January 2005. The work was carried out in accordance with Golder's Quality Control Plan for this project dated December 7, 2005.

2.0 SITE DESCRIPTION

The Clyde Avenue bridge is a slab on steel girder structure for Highway 417 and is located within a commercial area of Ottawa.

Clyde Avenue is a two lane road (with wide lanes to accommodate on-street parking) with an urban cross-section and sidewalks on both sides. The surrounding land on either side of Highway 417 is relatively flat and level and is generally occupied by light commercial and retail buildings.

The existing bridge has concrete abutments supported on footings founded on bedrock. The superstructure consists of a concrete deck supported on steel girders. The overpass consists of two separate bridges (one for each of the eastbound and westbound lanes).

It is understood that the abutment stem walls vary from poor to fair condition with spalls and delaminations covering approximately 20% of the exposed face. From a foundation perspective, the bridge is understood to be performing adequately.

The existing approach embankments are 5 to 6 m high relative to the surrounding ground surface with 2H:1V side slopes. The highway profile at the approaches does not seem to have experienced significant differential settlement of the roadway, although the maintenance history at this location is not currently known.

A previous investigation was conducted for the design of the existing bridge by McRostie & Associates for MTO in 1958. The results of that investigation are contained in the report titled "Report on Foundation Investigation at Ottawa Queensway and Clyde Avenue, Bridge No. 4 to Deleuw, Cather and Company of Canada Limited" (Geocres No. 58-F-224-C).

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out on May 8 and 9, 2006. On those days, four boreholes (Boreholes 06-1 to 06-4, inclusive) were put down at the locations shown on Drawing 1. The boreholes were drilled at the approximate location of the ends of the proposed abutment widening. The boreholes were advanced using a truck mounted drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. The boreholes were advanced to depths which vary from 4.8 m to 6.0 m below present ground surface.

Samples of the overburden were obtained at 0.6 m to 1.2 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. The bedrock was cored for depths ranging from 3.0 to 3.5 metres, after practical refusal to augering had been reached. The water levels in the open boreholes were observed throughout the drilling operations, and one standpipe was installed to monitor the groundwater level at the site. The standpipe consists of 20 mm outside diameter HDPE tubing with a 0.6 m long slotted tip. The boreholes were backfilled with bentonite mixed with soil cuttings. The site conditions were restored following completion of the field work.

The field work was supervised on a full-time basis by members of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Ottawa for further examination, and to Golder Associates' laboratory in Mississauga for testing. Index and classification tests consisting of water content determinations, Atterberg Limit testing and grain size distribution analyses were carried out on selected soil samples.

The groundwater level was measured in the standpipe in Borehole 06-2 on June 12, 2006, about one month after completion of drilling.

The borehole locations were determined by Golder relative to existing site features. The borehole elevations were determined by MRC from a digital terrain model based on the locations provided by Golder. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-1	North-west abutment	5026530.9	363653.6	77.7
06-2	North-east abutment	5026548.7	363666.6	78.1
06-3	South-east abutment	5026503.3	363683.4	77.8
06-4	South-west abutment	5026487.2	363672.2	78.4

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within the minor physiographic region known as the Ottawa Valley Clay Plain, as delineated in *The Physiography of Southern Ontario*¹, that lies within the major physiographic region of the Ottawa-St. Lawrence Lowland.

The Ottawa Valley Clay Plain region is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock.² This region is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain by igneous and metamorphic bedrock of the Precambrian Shield.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, four boreholes were advanced within the limits of the foundation elements for the proposed widening of the Clyde Avenue bridge. The borehole locations and ground surface elevations are shown on Drawing 1 in Appendix A. Soil stratigraphy sections projected along the highway centreline and across the abutment foundation areas are shown on Drawing 2 in Appendix A.

The detailed subsurface soil, bedrock, and groundwater conditions encountered in the boreholes and the results of the in-situ and laboratory testing are given on the Record of Borehole sheets in Appendix B and on Figures 1 to 3 in Appendix C. 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. Subsoil conditions will vary between and beyond the borehole locations.

Four boreholes had been previously advanced at the present bridge abutment locations on behalf of the Ministry in 1958, as previously noted, (Geocres No. 58-F-224-C) and the Record of Borehole sheets from that investigation are attached in Appendix D.

In summary, the soils encountered during the current investigation within the limits of the widening consist of sand and silty clay fill materials to depths of about 0.9 m to 1.7 m, over thin

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.

² Belanger, J.R. "Urban Geology of Canada's National Capital Area", in *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.

deposits of native sand, weathered silty clay, and glacial till extending to depths of about 1.8 to 2.8 m. These overburden materials are underlain by limestone and dolomitic limestone bedrock.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections. In the following discussion, emphasis is placed on the subsurface conditions indicated in the boreholes from the present investigation. The Geocres information, which reflects conditions prior to construction of the existing bridge, is referenced only in regard to the bedrock surface elevation.

4.2.1 Topsoil

Topsoil, about 0.2 m in thickness, was encountered at all the boreholes.

4.2.2 Sand and Silty Clay Fill

Fill material associated with previous uses of the site and roadway construction underlies the topsoil at all of the boreholes. In Boreholes 06-1 and 06-4, drilled at the west abutment widening, about 1.7 and 0.9 m of fill were encountered, respectively, with the base of the fill at Elevation 76.0 m and 77.5 m in the two boreholes. In Boreholes 06-2 and 06-3, advanced at the east abutment widening, 1.5 and 1.1 m of fill were encountered, respectively, with the base of the fill at Elevations 76.5 m and 76.7 m.

The fill materials range in composition from sand to silty sand and silty clay. The silty clay fill underlies the sandy fill at boreholes 06-1 and 06-2. Grain size distribution test results obtained from one sample of the sand fill at Borehole 06-3 are shown on Figure 1.

Two standard penetration test N values in the silty clay fill of 7 and 8 blows per 0.3 m of penetration indicate this material to be stiff.

4.2.3 Sand

A layer of fine to coarse sand with trace amounts of gravel underlies the fill materials at Borehole 06-2. This layer is about 0.8 m thick. A measured SPT "N" value of 45 blows per 0.3 m of penetration indicates the deposit to have a dense relative density.

4.2.4 Silty Clay

The fill materials at Borehole 06-4 and the sand layer at Borehole 06-2 are underlain by a deposit of silty clay, which is about 0.6 m and 0.5 m thick, respectively. The silty clay at these locations has been weathered to a grey-brown colour. The measured SPT "N" values in this deposit were

both 5 blows per 0.3 m of penetration. These test results indicate that the weathered silty clay has a very stiff consistency.

The results of Atterberg limit testing on one selected sample of the weathered silty clay indicate a plasticity index of 26 percent and a liquid limit of 48 percent. These results, summarized on the plasticity chart on Figure 2, confirm that this material is a silty clay of medium plasticity. The measured natural water content of one sample of the weathered silty clay was 33 percent.

4.2.5 Silty Sand to Sandy Silt Till

A 0.3 m to 0.7 m thick layer of glacial till was encountered below the silty clay deposit in Borehole 06-4 and beneath the fill materials in Boreholes 06-1 and 06-3; the deposit is absent from Borehole 06-2. The surface of this till deposit was encountered between about elevations 76.0 m and 76.9 m in the boreholes (at depths below ground surface ranging from 1.1 m to 1.7 m).

Based on local experience and observations of the drilling resistance, the glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand to sandy silt, with a trace of clay. Due to the limited thickness of this deposit, only limited standard penetration testing could be carried out before sampler refusal was encountered on the bedrock surface. However, one measured SPT "N" value of 8 blows per 0.3 m of penetration indicates the deposit to have a loose relative density. Grain size distribution test results obtained for one sample of the glacial till at Borehole 06-1 are shown on Figure 3. It should be noted however that this sample was retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposit. The measured natural water contents of two samples of the glacial till were 6 and 11 percent.

4.2.6 Dolomitic Limestone and Limestone Bedrock

Limestone bedrock was encountered at all of the abutment widenings. Dolomitic limestone bedrock overlies the limestone at Boreholes 06-1 and 06-2, on the north side of Highway 417, and extends to depths of about 3.8 and 4.4 m below ground surface.

The following table summarizes the bedrock surface depths and elevations as encountered at the locations of Boreholes 06-1 to 06-4, and as encountered at the previous boreholes 1 to 4; the bedrock was cored in all eight of these boreholes.

Borehole Location	Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
West Abutment	06-1	77.7	2.2	75.5
	1	78.7	2.8	75.9

Borehole Location	Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
	3	79.2	2.9	76.3
	06-4	78.4	1.8	76.6
East Abutment	06-2	78.1	2.8	75.3
	4	77.1	1.3	75.8
	2	79.0	2.7	76.3
	06-3	77.8	1.8	76.0

The dolomitic limestone and limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded and is generally unweathered. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 19 to 100 percent, indicating very poor to excellent quality rock, generally increasing with depth. The lowest RQD values were recorded for the upper metre of rock in Boreholes 06-3 and 06-4. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted in the upper bedrock at Boreholes 06-3 and 06-4. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.3 Groundwater Conditions

A piezometer was installed in Borehole 06-2, sealed within the bedrock. The water level measured in that piezometer is summarized in the following table:

Borehole No.	Borehole Location	Date	Depth (m)	Elevation (m)
06-2	East abutment	June 12, 2006	3.1	75.0

Boreholes 06-1, 06-3 and 06-4 remained dry during the short time that they remained open following completion of the overburden drilling and prior to commencing the bedrock coring operations.

It should be expected that the groundwater levels will fluctuate seasonally.

5.0 CLOSURE

The investigation was carried out using equipment supplied and operated by Marathon Drilling. The field portions were supervised by Mr. Doug Grylls under the direction of Mr. William Cavers, P.Eng. The testing was carried out in the Mississauga laboratory of Golder Associates. The report was prepared by Mr. William Cavers, P.Eng., under the direction of the Project Manager, Mr. Michael Cunningham, P.Eng. This report was reviewed by Mr. Fintan J. Heffernan, P.Eng., the designated MTO contact for this project.

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PART B

**FOUNDATION INVESTIGATION AND DESIGN REPORT
CLYDE AVENUE OVERPASS Bridge WIDENING
STRUCTURE SITE 3-43
HIGHWAY 417
W.P. 4058-01-00**

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed widening of the existing single-span Clyde Avenue overpass structure on Highway 417 in Ottawa, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

The existing Clyde Avenue overpass bridge is a single span structure with shallow foundations supported on the bedrock. The proposed work is to include complete replacement of the existing bridge deck and girders. Rapid replacement techniques will likely be used for this work. The existing abutments will also be extended to the north and south by about 5.7 metres and will be converted to semi-integral type abutments. In addition, the roadway profile of Clyde Avenue will be lowered by about 100 mm to maintain the existing vertical clearance between Clyde Avenue and the widened overpass structure.

Wing (retaining) walls extending 13 to 16 metres back from the abutments have been proposed. Embankments with 2H:1V side slopes may extend past the ends of the wing walls.

6.2 Bridge Foundation Options

The foundation system for the widening of this bridge should be compatible with the existing bridge foundations and the following options have been considered for the widening:

- Shallow foundations supported on the discontinuous native soils.
- Shallow foundations supported by the dolomitic limestone and limestone bedrock.
- Caisson or pile supported abutments.

The last option, using pile or caisson supported abutments, is not considered practical or appropriate for this site due to the shallow depth of bedrock. Assuming that conventional cast-in-place abutment walls would be used for the widening consistent with the existing abutments, then the pile cap level would be no more than 1 metre above the bedrock surface. Shallow foundations on bedrock is considered to be more cost effective than foundations on short piles. Piles less than

3 metres long are also problematic with regards to lateral stability, and should generally be avoided. The existing bridge is also supported on spread footings founded on the bedrock. From a foundations perspective this therefore indicates that fully integral abutments, which require piles, would not be suitable for the widening. Semi-integral abutments are however compatible with shallow foundations.

It is also not considered feasible to support the bridge structure on spread footings on or within the glacial till layer. Firstly, the earth cover requirements for frost protection of the foundations make this option not feasible at the south abutment widenings where the bedrock surface is at only 1.8 metres depth. For the northerly abutment widenings, the glacial till is absent at Borehole 06-2 (east abutment) and the clay deposit that overlies bedrock at that location is not suitable to support the significant abutment loads. On the west side, the till deposit is only 0.5 metres thick and the much higher resistance available from the underlying bedrock makes that strata much preferred for supporting the foundation loads. Larger settlements would also occur for footings on the till, versus footings on the rock, and those settlements would be entirely differential relative to the existing structure.

It is therefore considered that the most feasible and cost-effective foundations for this bridge structure is spread footings founded on the shallow bedrock. This option is also consistent with the existing bridge foundation construction.

Geotechnical recommendations for the design of foundations for the bridge abutments are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the bridge foundation options is presented in Table 1 following the text of this report.

6.2.1 Shallow Foundations

Shallow foundations on bedrock are recommended for the support of the bridge abutments. The borehole information indicates bedrock surface levels at the abutment locations as follows:

Foundation Element	Bedrock Surface Elevation (metres)
West Abutment – North Widening	75.5
West Abutment – South Widening	76.6
East Abutment – North Widening	75.3
East Abutment – South Widening	76.0

The above bedrock surface elevations are provided based on the borehole data available. It must be confirmed during construction by the Quality Verification Engineer that the bedrock surface elevations are consistent with those anticipated.

The founding level of the existing bridge is understood to be at elevation 75.4 metres. It is recommended that the new foundations be founded at this same elevation.

At the *south* widenings, the results of the boreholes indicate that the bedrock surface is above elevation 75.4 metres. Some bedrock removal will therefore be required to achieve this founding elevation, although the upper portion of the bedrock is indicated to be highly fractured (with RQD values less than 20%). However, at the south-east widening, the highly fractured zone is indicated to extend to as low as elevation 75.0 metres and therefore excavation below elevation 75.4 m could be required to reach competent bedrock. It is not recommended to step the footing down. Rather, provision should therefore be made in the Contract documents for additional excavation and placement of mass concrete wherever the competent bedrock surface is below the design founding level. A Non Standard Special Provision has been included in Appendix E of this report for the placement of mass concrete. Excavation of fractured rock immediately adjacent to the existing footing is considered acceptable provided the excavation is no deeper than about 0.5 m. For deeper excavations, the excavation should be stepped down at 1H:1V.

At the *north* widenings, the bedrock surface is indicated to be within 0.1 m of the anticipated founding elevation and the bedrock is indicated to be of better quality.

Based on the above, since the bedrock surface is at or near the founding elevation of the existing bridge and since bedrock excavation to near this elevation may be required to remove fractured rock, it is recommended that the widened abutments be founded at Elevation 75.4 metres, which is consistent with the existing bridge foundations.

6.2.1.1 Limit States Factored Geotechnical Resistance

The bridge abutments can be supported on spread footings placed on the limestone bedrock. The upper portion of the bedrock is, in local areas, of poor quality (e.g., RQD values of less than 40 percent, as encountered in Boreholes 06-3 and 06-4 for the south widenings), and subexcavation of any weathered or highly fractured bedrock will be required prior to construction of the footing. MTO's Special Provision SP902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to footing construction, to ensure that all weathered and/or fractured rock has been removed from the foundation areas prior to construction of the spread footings.

Spread footings placed on the surface of limestone bedrock or on mass concrete may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 3,000 kPa.

Serviceability Limit States (SLS) resistances do not apply to the design of footings on the limestone bedrock since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the *Canadian Highway Bridge Design Code (CHBDC)*.

6.2.1.2 Resistance to Lateral Loads

Resistance to lateral forces for the bridge foundations (i.e., sliding resistance between the concrete footings and bedrock) should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, may be taken as 0.70 for cast-in-place concrete footings constructed on bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a resistance factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, the sliding resistance of spread footings founded on the bedrock can be supplemented by dowelling into the bedrock or by excavating a shear key below the underside of the footing.

The horizontal resistance of dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or is stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. A Non Standard Special Provision for the installation and testing of dowels has been included in Appendix E of this report.

Shear keys in good quality rock (rock with an RQD of greater than 75%) can be proportioned using a ULS horizontal geotechnical resistance of 500 kPa. This represents an unfactored value; in accordance with the *CHBDC*, a resistance factor is to be applied in calculating the horizontal resistance. This resistance assumes that the shear key has a depth of at least 0.5 metres, extends at least 0.25 metres into good quality rock (e.g., below the surficial fractured rock encountered in Boreholes 06-3 and 06-4) and that the bearing surface is vertical and undisturbed. In the particular case of the south-east widening this requirement means that the shear key should extend to at least Elevation 74.8 m. It is assumed that the top of rock elevation around the footing is at or above the footing founding level (i.e. the footing is not next to a slope in the bedrock). The geotechnical resistance should be reduced if the surrounding rock is unavoidably disturbed, or excavation is planned in rock adjacent to the footing. Excavation for the shear key should be carried out using

mechanical excavation methods which should include closely spaced line drilling along the perimeter of the shear key prior to excavation of the key, such that a uniform vertical face will be created. Excavations immediately adjacent to the existing abutment footings may, with closely spaced line drilling, extend vertically to 0.5 m. lower than the underside of the existing footing; excavations extending deeper should be stepped down at 1H:1V below that level.

Dowelling is typically more labour intensive in comparison to the relatively limited rock excavation required for a shear key. However, dowels may be placed immediately adjacent to the existing footing without risk of disturbing the rock underlying the existing footing; excavation for a shear key immediately adjacent to the existing footing can only be carried out to a limited depth.

6.2.1.3 Frost Protection

For spread footings founded on the limestone bedrock at this site, frost susceptibility is not an issue.

It is understood that the existing abutment wing walls will be left in place within the widened embankment to provide additional resistance to the overturning forces on the abutments. The concrete wing walls will therefore remain in place below the widened pavement structure and this could potentially result in differential frost heaving. The potential for this frost heaving to occur would be reduced by removing the upper portion of the existing wingwalls to a depth of 1.8 metres below final pavement profile grade.

6.3 Wing (Retaining) Wall Options

Wing (retaining) walls extending 13 to 16 metres back from the abutments have been proposed and it is considered that both cantilevered reinforced concrete retaining walls supported on shallow foundations or RSS walls may be used at this location.

The choice of wing wall system will depend on the desired appearance, the anticipated costs, performance and on other considerations such as constructability. Considering the location of this wall and the surrounding area, a medium appearance criteria is recommended for the wing walls at this site.

Settlements at this location should be less than the maximum allowable limits for RSS walls and the founding conditions are suitable for support of reinforced concrete retaining walls on shallow footing. Therefore, from a foundation perspective, either option is feasible and practical at this location.

RSS retaining walls may require some subexcavation of the existing embankment fill in order to install the RSS reinforcing strips and granular fill; it is expected that temporary excavation

support measures would be required to ensure the stability of the existing embankment side slopes during this removal. However RSS walls would still typically be less than expensive than concrete walls.

The foundation system for the wing (retaining) walls should be consistent with the bridge widening and the following options have been considered:

- Shallow foundations supported on the discontinuous native soils.
- Shallow foundations supported by the dolomitic limestone and limestone bedrock.
- Caisson or pile supported wing (retaining) walls.

The last option, using pile or caisson supported wing (retaining) walls, is not considered practical or appropriate for this site as noted in Section 6.2.

It is considered feasible to support retaining walls on spread footings on or within the native soils at this site. Larger settlements would however occur for retaining wall footings on the native soils, versus the bridge abutment footings on the rock, and those settlements would be entirely differential relative to the bridge abutments. If bridge wing (retaining) walls supported on the native soils are considered, the retaining walls should be provided with an articulated joint with the bridge abutments.

The wing (retaining) walls can also be supported on spread footings founded on the shallow bedrock. This option is also consistent with the abutment widening construction and is considered to be the preferred option at this location.

Geotechnical recommendations for the design of foundations for the wing (retaining) walls are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the foundation options is presented in Table 2 following the text of this report.

6.3.1 Shallow Foundations

Shallow foundations on bedrock are recommended for the support of the wing walls.

The wing walls can also be founded on the native soils (i.e., weathered silty clay, sand or glacial till). The existing fill materials are however not suitable for support of the wall footings and need to be removed from the foundation areas.

6.3.1.1 Limit States Factored Geotechnical Resistance

The wing walls can be supported on spread footings placed on the limestone bedrock. The upper portion of the bedrock is, in local areas, of poor quality (e.g., RQD values of less than 40 percent, as encountered in Boreholes 06-3 and 06-4 for the south widenings), and subexcavation of any weathered or highly fractured bedrock will be required prior to construction of the footing. MTO's Special Provision SP902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to footing construction, to ensure that all weathered and/or fractured rock has been removed from the foundation areas prior to construction of the spread footings. The surface of the bedrock may be at a lower elevation than the design footing elevation at some locations due to removal of weathered/fractured rock or natural variability in the bedrock surface. The footings may be placed on mass concrete at those locations.

Spread footings placed on the surface of the limestone bedrock or on mass concrete may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 3,000 kPa taking into account the poor quality of the upper portion of the bedrock. Serviceability Limit States (SLS) resistances do not apply to the design of footings on the limestone bedrock since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Retaining walls can be supported on spread footings placed on undisturbed native soils (i.e., weathered silty clay, sand or glacial till) or on a pad of compacted engineered fill placed on the native soils. A factored geotechnical resistance at ULS of 250 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa may be used for the design of footings placed on undisturbed native soils.

RSS walls, designed with the geotechnical resistances given above, should be founded on a minimum 0.3 m thick compacted Granular A or Granular B Type II levelling pad constructed on the surface of the native soils.

Where the retaining walls are to be founded on a pad of compacted engineered fill, the existing fill material should be removed from the full zone of influence of the foundations, which is considered to extend down and out from the edge of the foundations at a slope of 1H:1V. The engineered fill should similarly be placed to fill the full zone of influence of the foundations. The engineered fill should consist of Granular B Type I or II placed in maximum 300 mm thick lifts and compacted to at least 95% of its standard Proctor dry density.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied

perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the *Canadian Highway Bridge Design Code (CHBDC)*.

6.3.1.2 Resistance to Lateral Loads

Resistance to lateral forces for the wing (retaining) wall foundations should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, may be taken as 0.70 for cast-in-place concrete footings constructed on bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a resistance factor is to be applied in calculating the horizontal resistance. If necessary, the sliding resistance of spread footings founded on the bedrock can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or is stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. A Non Standard Special Provision for the installation and testing of dowels has been included in Appendix E of this report.

Resistance to lateral forces for retaining wall footings founded on the native soils should similarly be calculated in accordance with Section 6.7.5 of the *CHBDC*. The parameter values in the following table may be used to calculate the lateral resistance at the footing-soil interface:

<i>Foundation</i>	<i>Founding Elevation (m)</i>	<i>Founding Material</i>	<i>Drained Conditions (Long Term Loading)</i>		<i>Undrained Conditions (Short Term Loading)</i>
			<i>$\tan \delta^*$</i>	<i>c' (kPa)</i>	<i>C_u (kPa)</i>
North-west retaining wall	76.0 – 75.5	Till	0.50	0	
North-east retaining wall	76.5 – 75.8	Sand	0.45	0	
	75.8 – 75.3	Weathered Silty Clay	0.43	0	50
South-east retaining wall	76.7 – 76.0	Till	0.50	0	
South-west retaining wall	77.5 – 76.9	Weathered Silty Clay	0.43	0	50
	76.9 – 76.6	Till	0.50	0	

Note: The $\tan \delta^*$ values are based on 2/3 of the soil friction angle.

For footings on the silty clay, the resistance to both long term and short term loading needs to be evaluated. Resistance values for both conditions are provided in the above table.

At locations where footings are founded on or within a soil layer underlain by the weathered silty clay the short term shear resistance within the silty clay shall also be checked in accordance with the CHBDC. For example, if the footings at the north-east retaining wall are founded on the sand layer, the resistance at the sand/footing interface as well as the resistance within the underlying weathered silty clay should be checked.

Where retaining walls will be supported on engineered fill, a $\tan \delta^*$ value of 0.5 may be used at the footing – engineered fill interface.

The resistances obtained using the above parameters represent unfactored values; in accordance with the CHBDC, a resistance factor of 0.8 is to be applied in calculating the horizontal resistance.

6.3.1.3 Frost Protection

For spread footings founded on the properly prepared limestone bedrock at this site, frost susceptibility is not an issue.

Spread footings founded on or within native soils should be provided with a minimum of 1.8 m of soil cover for frost protection.

6.4 Site Coefficient & Seismic Liquefaction

For seismic design purposes, the Site Coefficient, S , for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type I.

Seismic liquefaction occurs when earthquake vibrations cause an increase in the pore water pressure within the soil, which reduces the effective stress between the soil particles and the soil's frictional resistance to shearing. This phenomenon, which leads to a temporary reduction in the shear strength of the soil may cause:

- Large lateral movements of even gently sloping ground, referred to as “lateral spreading”, which could impact embankment stability;
- Reduced shear resistance (i.e., bearing capacity) of soils which support foundations, as well as reduced resistance to sliding; and,

- Reduced shaft resistance for deep foundations as well as reduced resistance to lateral loading.

In addition, 'seismic settlements' may occur once the vibrations and shear stresses have ceased. Seismic settlement is the process where the soils stabilize into a denser arrangement after an earthquake, causing potentially large surface settlements.

The following conditions are more prone to experiencing seismic liquefaction:

- Coarse grained soils (i.e., more probable for sands than for silts);
- Soils having a loose state of packing; and,
- Soils located below the groundwater level.

The silty clay and glacial till soils at this site are too fine-grained to be potentially liquefiable. Portions of the fill material are coarser in gradation, however the fill materials are located above the groundwater level and are therefore also not liquefiable. Only the sand deposit encountered in Borehole 06-2 is sufficiently coarse grained and could, at times of the year, be saturated, to therefore require further evaluation. However, with an SPT 'N' value of 45, the material is dense and, considering the seismicity of the Ottawa area, is also not considered to be liquefiable.

Based on the above, there is not considered to be a potential liquefaction hazard at this site, and therefore liquefaction need not be considered in the design of foundations or embankments.

6.5 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the abutment stems and retaining walls:

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in accordance with MTO's Special Provision 105S10. 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 taper should be in accordance with OPSD 3101.150 and 3121.150.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with MTO's Special Provision 105S10. 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 1.8 m behind the back of the abutment stem (Case I in Figure C6.9.1(I) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(I) of the *Commentary to the CHBDC*).
- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade material:

Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	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

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- Seismic loading will result in increased lateral earth pressures acting on the walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the *CHBDC*, this site is located in Seismic Performance Zone 3. The site-specific zonal acceleration ratio for Ottawa is 0.2. Based on experience, for the subsurface conditions at this site, no significant amplification of the ground motion will occur. The seismic lateral earth

pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.2$.

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which do not allow lateral yielding (i.e. the abutment walls for this structure), 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 (i.e. possibly the wing walls for this structure), 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 I and Case II) 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}

	Case I	Case II	
		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 0.20. 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:

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

where $\sigma_h(d)$ is the lateral earth pressure at depth, d, (kPa);
 K_a is the static active earth pressure coefficient;
 K_{AE} is the seismic active earth pressure coefficient;
 γ is the unit weight of the backfill soil (kN/m³),
as given previously;
d is the depth below the top of the wall (m); and
H is the total height of the wall (m).

6.6 Approach Embankment and Wing (Retaining) Wall Design and Construction

The proposed work will require widening the existing approach embankments laterally by 5 to 6 metres on both sides of the highway. The existing embankments are about 5 metres in height. Retaining walls for the widened embankments will also be required due to the space restraints imposed by adjacent properties. These retaining walls may be constructed as cantilevered reinforced concrete walls or as RSS walls and may be founded on the native soils, engineered fill or bedrock.

Based on the borehole results, the embankment subgrade soils for the embankment widening will consist of fill materials and/or dense native sands, very stiff weathered silty clay and/or loose to very dense silty sand to sandy silt till. The fill materials overlying the native soils vary in composition from fine sand to silty clay.

6.6.1 Subgrade Preparation and Embankment Construction

Any topsoil, organic matter and softened / loosened soils should be stripped from within the limits of the widening, including the existing embankment sideslope and the new footprint. The existing inorganic fill materials may remain in place, however all subgrade soils should be proof-rolled prior to fill placement.

Embankment fill should be placed in regular lifts with a loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the material's Standard Proctor maximum dry density in accordance with SP 105S10.

The final lift prior to placement of the granular subbase and base courses should be compacted to 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

6.6.2 Embankment Stability

With appropriate subgrade preparation and proper placement of earth or granular soils, the 5 to 6 m high approach embankments with side slopes maintained at 2 horizontal to 1 vertical, founded

on the existing fill materials, sand, stiff clay and glacial till over bedrock, will have a factor of safety greater than 1.3 against deep-seated slope instability.

Wing (retaining) walls, up to 6 m in height, founded on the native soils or engineered fill, after removal of the existing fill materials and surficial topsoil, will also have a factor of safety greater than 1.3 against deep seated slope instability.

The internal stability of mechanically-reinforced (RSS) walls should be checked by the supplier.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the retaining walls and embankment side slopes will have factors of safety of greater than 1.1 against deep-seated slope instability based on an acceleration of 0.1g. The results do however indicate that some shallow sloughing (with factors of safety less than 1.1) could occur of the embankment side slopes during seismic loading. That sloughing would not however impair the short term use of the structure and is mainly a maintenance/repair issue. The potential for sloughing could be reduced by providing well vegetated side slopes.

Static slope stability analyses for the above embankment and retaining wall configurations were carried out using the following parameters:

Material	Bulk Unit Weight (kN/m³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Earth or Granular Embankment Fill	21	32°	
Existing Sand Fill	19	29°	
Existing Silty Clay Fill	17.5		50
Weathered Silty Clay	17.5		80
Sand	19	29°	
Till	22	32°	

6.6.3 Embankment Settlement

Settlement of the approach embankment widenings adjacent to the abutments will occur due to compression of the new embankment fill itself, as well as compression of the existing fill materials and relatively thin native overburden soils.

Provided that the new embankment fill material consists of granular fill, Select Subgrade Material or clean earth fill, the settlement of the embankment fill itself is expected to be less than about 25 mm. The use of granular fill for the new embankment construction would reduce the magnitude of *post-construction* settlement (likely to less than half that value), since the majority of the settlement of granular fills will occur during construction.

The embankment fill materials will be underlain by existing fill materials. The existing sandy fill materials are underlain by silty clay fill (up to 0.9 metres in thickness) at some locations. The subgrade settlement due to compression of the existing sand and silty clay fill materials should be minor in magnitude provided the subgrade surface is proof-rolled and most of this settlement would likely occur entirely during construction.

The existing fill materials are underlain by weathered silty clay and/or sandy silt to silty sand glacial till. The coefficient of consolidation of the weathered silty clay crust, typically being a fissured soil and being stressed within its re-compression limits, is relatively high. Therefore the subgrade settlements resulting from compression of the weathered silty clay crust, which should be modest in magnitude, would be expected to occur quite rapidly, likely entirely during embankment construction. The subgrade settlements resulting from compression of the silty sand to sandy silt glacial till would also likely be expected to occur entirely during embankment construction. Overall, settlement due to compression of the native subgrade soils would be expected to not exceed 25 mm.

6.7 Design and Construction Considerations

6.7.1 Existing Utilities

Existing utilities at this bridge location include a 406 mm diameter watermain extending parallel to Highway 417, south of the bridge location, and a 300 mm diameter storm sewer extending along Clyde Avenue adjacent to the west abutment.

The 406 mm watermain and the 300 mm storm sewer are outside the zone of the influence of the abutment and retaining wall footings but temporary shoring may be required as described below in Section 6.7.3 to reduce the potential for ground loss and resulting loss of support to these services.

6.7.2 Excavation

Excavations to expose the bedrock surface (to allow for construction of spread footings) would extend to about 2 m to 3 m depth below the existing ground surface. The excavations will typically extend through between 1 m and 1.5 m of existing sand and silty clay fill materials, then through less than 1 m of sand, very stiff silty clay and/or loose to very dense silty sand to sandy silt till. The groundwater level at the site is typically at or below the bedrock surface.

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The native overburden materials are classified as Type 3 soils and the existing fill materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a

relatively short period) through these overburden soils above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

6.7.3 Temporary Shoring

It is anticipated that temporary roadway protection will be required along Highway 417 to permit construction of the abutment widenings, and may also be required along Clyde Avenue to permit construction of the new foundations and reduce the potential for loss of support for the 300 mm storm sewer. Temporary protection may also be required at the southerly retaining walls to reduce the potential for disturbance and loss of support to the 406 mm watermain.

It is understood that the design of the shoring will be entirely the responsibility of the contractor. The shoring will have to be designed to resist lateral earth pressures that are controlled by the flexibility of the shoring and its method of support. However, conceptually, the temporary protection could consist of either soldier piles and lagging or steel sheet piling.

For the excavation along Highway 417, it may be feasible to embed soldier piles or sheet steel piling sufficiently into the overburden without additional lateral support for excavations up to 3 metres in depth (i.e., it may be feasible to cantilever the shoring). For deeper excavations, where the overburden thickness is insufficient to embed the shoring, or where steel sheet piles cannot be driven to sufficient depth due the presence of cobbles and boulders within the glacial till, it may be necessary to provide lateral support using either rakers supported on footings within the excavation or using tie-backs grouted into the bedrock behind the shoring.

Soldier pile and lagging would likely be the preferred shoring type along Clyde Avenue due to the presence of cobbles and boulders within the glacial till and the relatively shallow excavation depth; steel sheet piles would have difficulty penetrating the glacial till. Soldier piles may be cantilevered by socketing and grouting into the bedrock or lateral support could alternatively be provided by internal struts, rakers braced to footings within the excavation, or tie-backs grouted into the bedrock behind the shoring.

To the expected depths of excavation, it is not expected that basal heaving or basal instability will be a concern.

The temporary excavation support should be in accordance with MTO Special Provision 902S01. The temporary system for the embankment and roadway protection at the abutment widenings should be designed to Performance Level 2 as defined in SP 902S01.

6.7.4 Construction Staging

It is understood that the four abutment widenings will be constructed sequentially and it is not anticipated that this staged construction will be a concern with respect to the foundations and embankment design.

6.7.5 Decommissioning of Boreholes

The standpipe in Borehole 06-2 will be decommissioned.

6.7.6 Groundwater and Surface Water Control

The groundwater level at the site is typically at or below the bedrock surface. Excavations to expose the bedrock surface for founding of spread footings will likely involve minimal groundwater and surface water control. It should be possible to handle ground and surface water inflows by pumping from well filtered sumps established in the bedrock.

7.0 CLOSURE

This report was prepared by Mr. William Cavers, P.Eng., under the direction of the Project Manager, Mr. Michael Cunningham, P.Eng. This report was reviewed by Mr. Fintan J. Heffernan, P.Eng., the designated MTO contact for this project.

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TABLE 1
COMPARISON OF BRIDGE FOUNDATION ALTERNATIVES
CLYDE AVENUE OVERPASS BRIDGE WIDENING
HIGHWAY 417
W.P. 4058-01-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread footings supported on bedrock	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> Compatible with existing bridge foundations High bearing resistance Negligible settlement Semi-integral design is feasible 	<ul style="list-style-type: none"> Minimal additional excavation (less than 1 m) versus footings on glacial till 	<ul style="list-style-type: none"> Expected to be least expensive option 	<ul style="list-style-type: none"> Low risk option
Spread footings supported on native soils	<ul style="list-style-type: none"> Not feasible due to low resistance and potential differential settlement relative to existing foundations 	<ul style="list-style-type: none"> Minimizes excavation depth. 	<ul style="list-style-type: none"> Much lower resistance than underlying bedrock Not compatible with existing bridge foundations 	<ul style="list-style-type: none"> Expected to be more expensive option than spread footings founded on bedrock due to larger footings 	<ul style="list-style-type: none"> Potential differential settlement relative to existing bridge
Piles socketed into bedrock.	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Allows for integral abutment design 	<ul style="list-style-type: none"> Drilling into bedrock required at each pile location Not consistent with existing bridge foundations 	<ul style="list-style-type: none"> Most expensive option 	<ul style="list-style-type: none"> Low risk option
Piles end-bearing on the bedrock surface.	<ul style="list-style-type: none"> Not feasible due to short pile length 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A

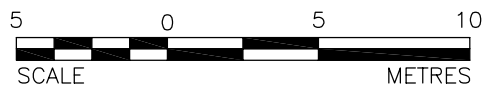
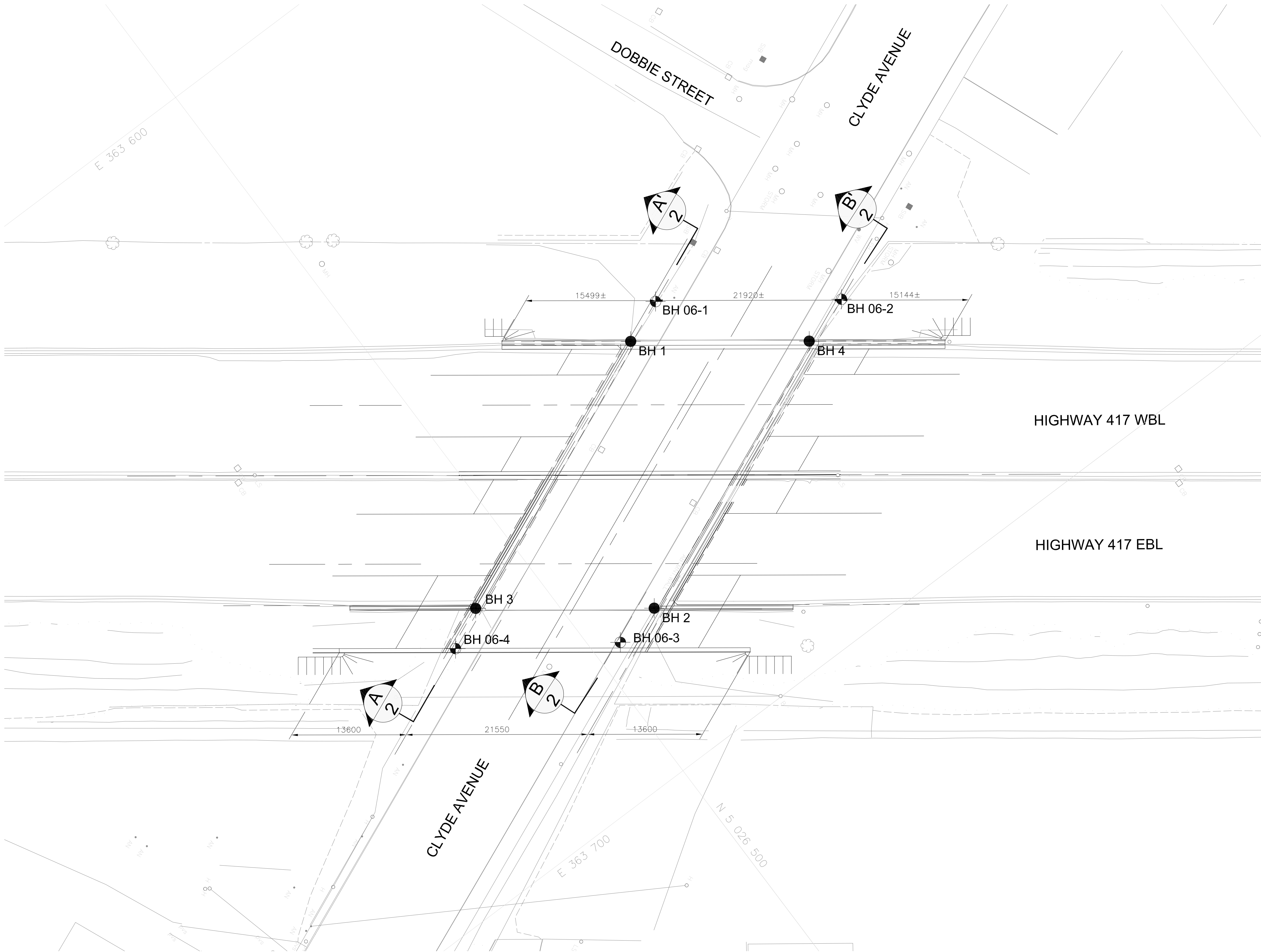
TABLE 2
COMPARISON OF APPROACH RETAINING WALL FOUNDATION ALTERNATIVES
CLYDE AVENUE OVERPASS BRIDGE WIDENING
HIGHWAY 417
W.P. 4058-01-00

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread footings supported on bedrock	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Compatible with abutment foundations High bearing resistance Negligible settlement 	<ul style="list-style-type: none"> Minimal additional excavation (less than 1 m) versus footings on glacial till 	<ul style="list-style-type: none"> Expected to be least expensive option 	<ul style="list-style-type: none"> Low risk option
Spread footings supported on native soils	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Minimizes excavation depth. 	<ul style="list-style-type: none"> Much lower resistance than underlying bedrock Not compatible with abutment widenings 	<ul style="list-style-type: none"> Expected to be more expensive option than spread footings founded on bedrock due to larger footings 	<ul style="list-style-type: none"> Potential differential settlement relative to abutments
Piles socketed into bedrock.	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> Drilling into bedrock required at each pile location 	<ul style="list-style-type: none"> Most expensive option 	<ul style="list-style-type: none"> Low risk option
Piles end-bearing on the bedrock surface.	<ul style="list-style-type: none"> Not feasible due to short pile length 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A

APPENDIX A

Drawing 1 - Clyde Avenue, Borehole Locations
Drawing 2 - Clyde Avenue, Soil Strata

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METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

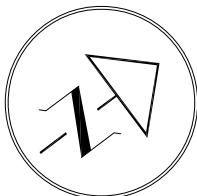
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WP No. WP 4058-01-00

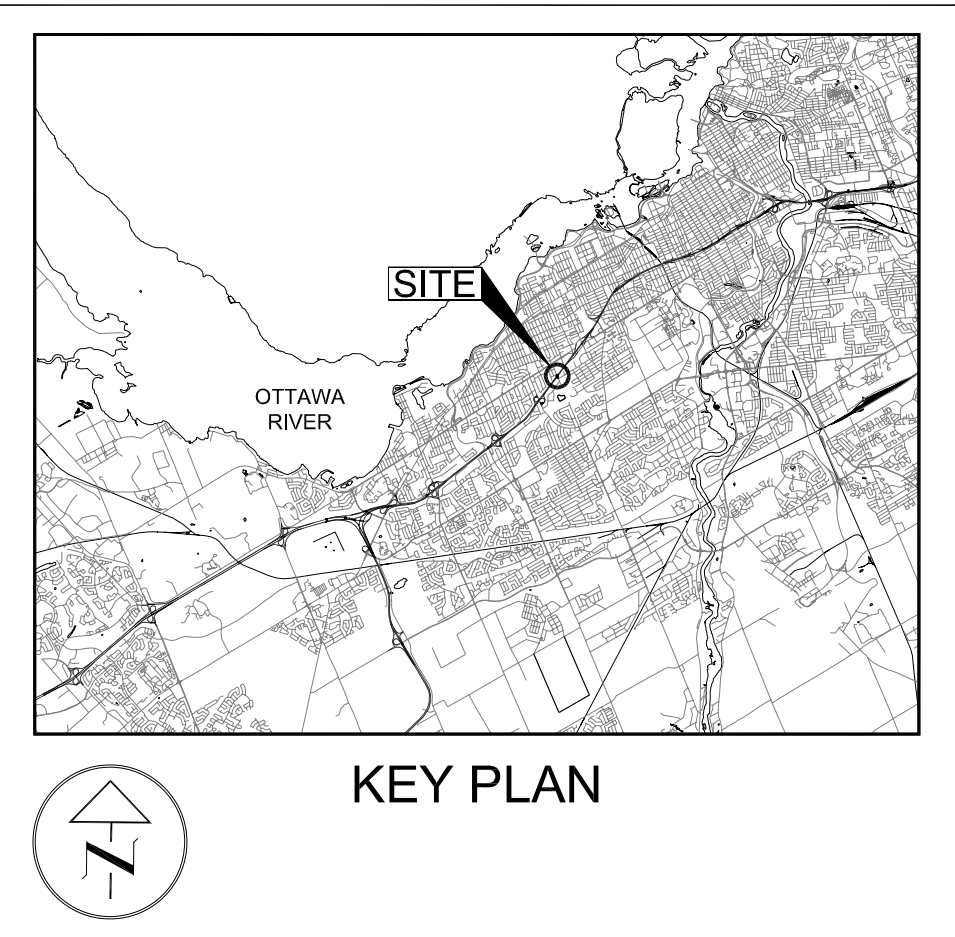
CLYDE AVENUE
BOREHOLE LOCATIONS



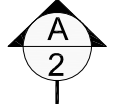
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SHEET



LEGEND			
	Borehole – Current Golder Associates Ltd. Investigation		
	Borehole – Previous MTO Investigation Geocres No. 58-F-224-C		
	Location of cross-section		
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-1	77.7	5026530.9	363653.6
06-2	78.1	5026548.7	363666.6
06-3	77.8	5026503.3	363683.4
06-4	78.4	5026487.2	363672.2
1	78.8	5026525.7	363655.6
2	79.1	5026508.9	363682.6
3	79.3	5026491.9	363669.9
4	77.1	5026542.6	363668.3

NOTES
This drawing is for subsurface information only. Any surface details are for conceptual illustration.
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION
Geocres No. 31G5-211			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: SEPTEMBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 1

HWY. 417

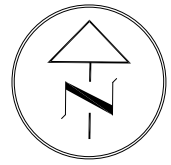
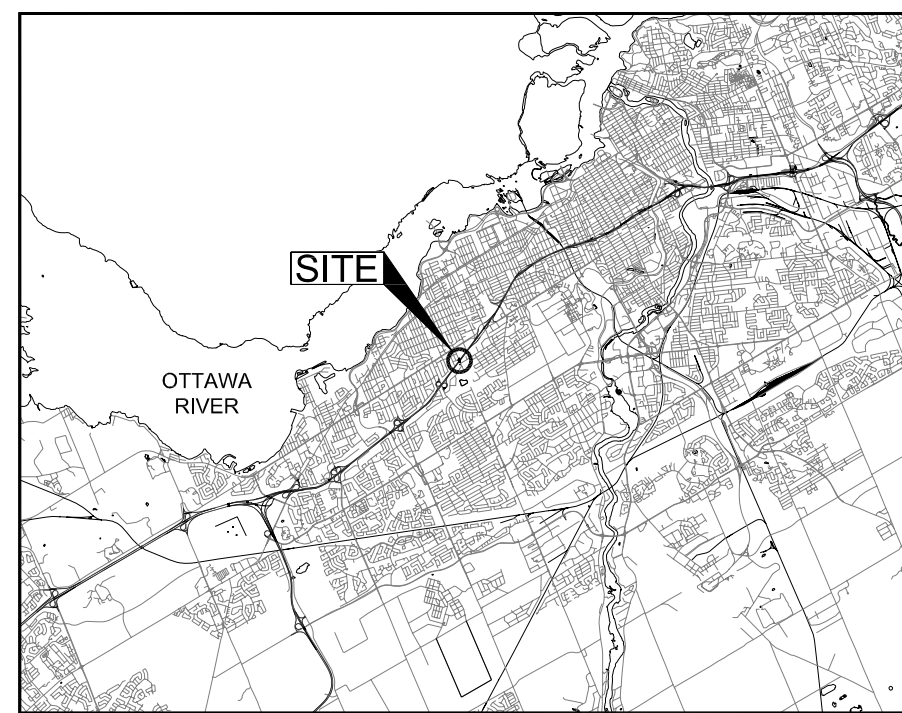
WP No. WP 4058-01-00

CLYDE AVENUE
SOIL STRATA

SHEET



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole – Current Golder Associates Ltd. Investigation
- Borehole – Previous MTO Investigation Goecres No. 58-F-224-C
- Seal
- Piezometer
- N Standard Penetration Test value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on June 12, 2006

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-1	77.7	5026530.9	363653.6
06-2	78.1	5026548.7	363666.6
06-3	77.8	5026503.3	363683.4
06-4	78.4	5026487.2	363672.2
1	78.8	5026525.7	363655.6
2	79.1	5026508.9	363682.6
3	79.3	5026491.9	363669.9
4	77.1	5026542.6	363668.3

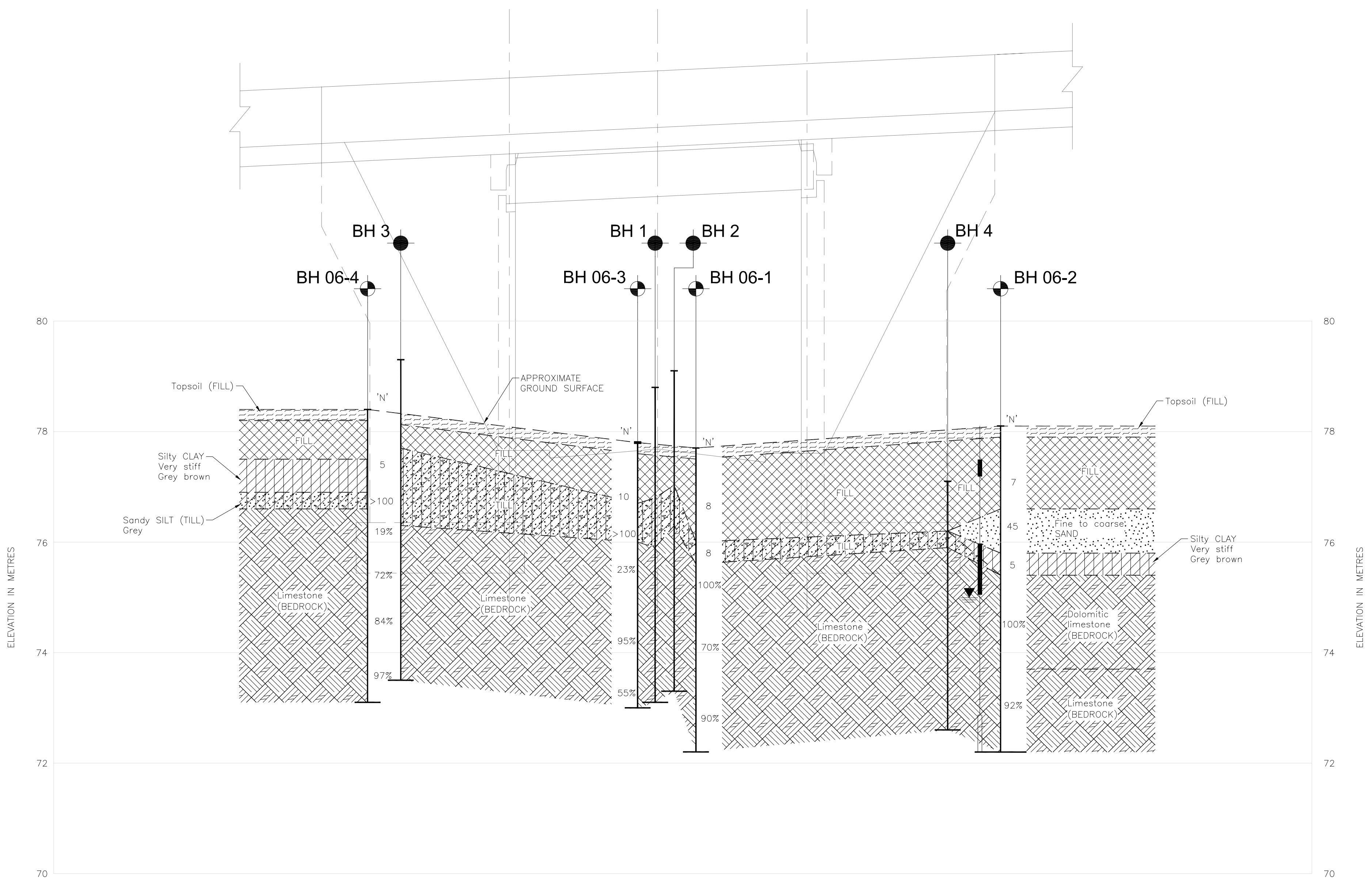
METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

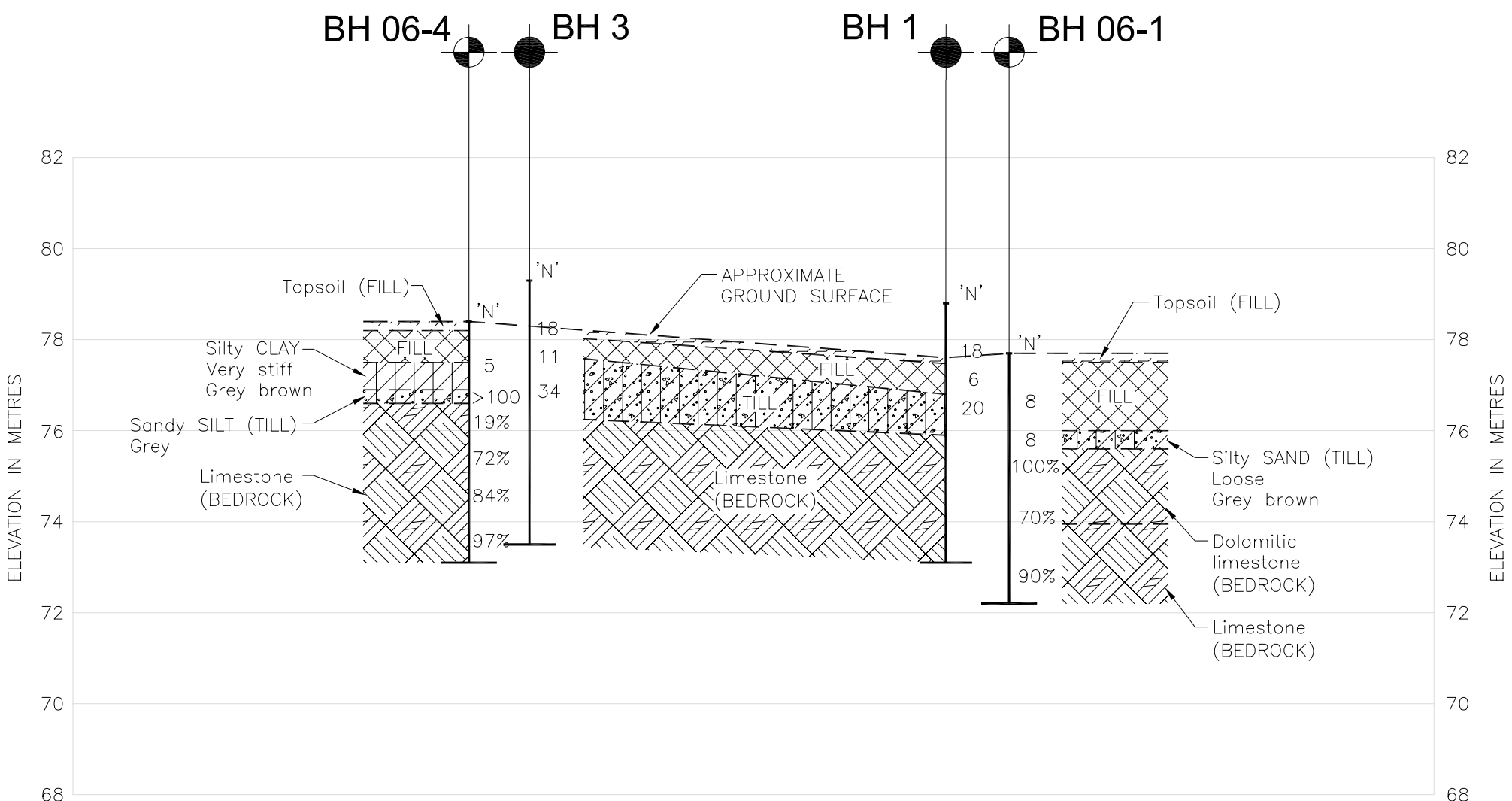
NOTES

This drawing is for subsurface information only. Any surface details are for conceptual illustration. The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence. Base plan provided in electronic format by McCormick Rankin Corporation

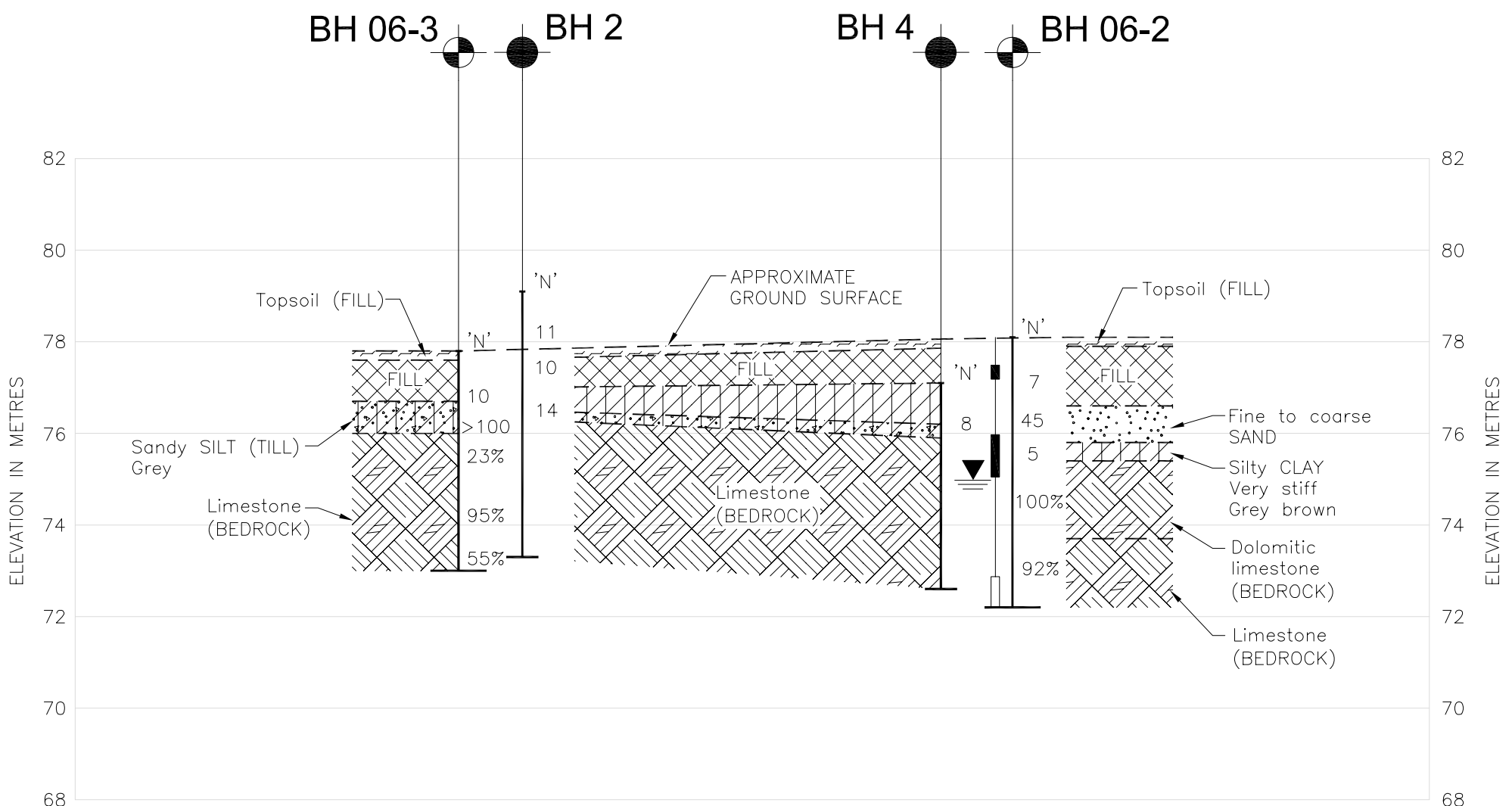
NO.	DATE	BY	REVISION
Geocres No. 31G5-211			
HWY. 417		PROJECT NO. 05-1120-210-2000	DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: SEPTEMBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 2



PROFILE ALONG CL HIGHWAY 417



SECTION A-A'



SECTION B-B'



APPENDIX B

**Lists of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Records of Boreholes 06-1 to 06-4**

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

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

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

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

Peizo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded Electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a)

Cohesionless Soils

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

(b)

Cohesive Soils

Consistency	C_{u2S_u}	C_{u2S_u}
	Kpa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limited
w_l	liquid limit
C	consolidaiton (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	modified Proctor compaction test
SPC	standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane 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
F	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 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 ($D_R = p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is p . Unit weight symbol is γ where $\gamma = pg$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio = σ'_p/σ'_{vo}

(d) Shear Strength

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

Notes: 1. $\tau = c' + \sigma' \tan \phi'$

2. Shear strength = (Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

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

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

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

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

Completely weathered: rock is wholly decomposed and in a friable condition but the rock 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.

O:\Templates\Rock Description Terminology

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

Abbreviations

B –	Bedding	Ca-	Calcite
FO-	Foliation/Schistosity	P-	Polished
CL -	Cleavage	S-	Slickensided
SH -	Shear Plane/Zone	SM-	Smooth
VN-	Vein	R-	Ridged/Rough
F -	Fault	ST-	Stepped
CO-	Contact	PL-	Planar
J -	Joint	FL-	Flexured
FR-	Fracture	UE-	Uneven
MF -	Mechanical	W-	Wavy
A-	Angular	C-	Curved
BP-	Bedding Plane	H-	Hackly
BL-	Blast Induced	SL-	Sludge Coated
	Parallel To	TCA-	To Core Axis
	Perpendicular To	STR-	Stress Induced

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-1		1 OF 1	METRIC
W.P. 4058-01-00	LOCATION N 5026530.9; E 363653.6	ORIGINATED BY D.G.			
DIST HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY J.M.			
DATUM Geodetic	DATE May 9, 2006	CHECKED BY M.I.C.			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
							20	40	60	80	100						
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED										
77.7	GROUND SURFACE						20	40	60	80	100						
0.0 77.5	Topsoil (FILL)																
0.2	Fine sand, trace silt (FILL) Brown Moist		1	A.S.													
76.9																	
0.8	Silty clay, some sand (FILL) Grey brown Moist		2	SS	8												
76.0																	
1.7	Silty SAND, trace gravel and clay (TILL) Loose Grey brown Moist		3	SS	8												
75.5																	
2.2	Dolomitic limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		4	NQ RC	DD												
74.0			5	NQ RC	DD												
3.8	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong Bedrock cored between 2.2m 5.5m depth. For bedrock coring details refer to Record of Drillhole 06-1.																
			6	NQ RC	DD												
72.2																	
5.5	End of Borehole Note: Borehole dry upon completion of drilling.																

MISS_MTO 05-1120-210-1000.GPJ ON_MOT.GDT 7/13/07

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-1

SHEET 1 OF 1

LOCATION: N 5026530.9; E 363653.5

DRILLING DATE: May 9, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR/FX-FRACTURE F-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION		
								CL-CLEAVAGE				J-JOINT				R-ROUGH				MB-MECH. BREAK							
								SH-SHEAR				P-POLISHED				ST-STEPPED				W-WAVY							
								VN-VEIN				S-SLICKENED				PL-PLANAR				C-CURVED							
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec											
								TOTAL CORE %	SOLID CORE %							10 ⁻⁶	10 ⁻⁵	10 ⁻⁴									
3	Rotary Drill NQ Core	ROCK SURFACE		75.26																							
		Dolomitic limestone (BEDROCK)		2.20																							
		Fresh Thinly to medium bedded Grey Medium strong			1																						
4	Rotary Drill NQ Core	Limestone (BEDROCK)		73.65																							
		Fresh Thinly to medium bedded Grey Medium strong		3.80																							
					2																						
5	Rotary Drill NQ Core																										
6	Rotary Drill NQ Core																										
7	Rotary Drill NQ Core																										
8	Rotary Drill NQ Core																										
9	Rotary Drill NQ Core																										
10	Rotary Drill NQ Core																										
11	Rotary Drill NQ Core																										
12	Rotary Drill NQ Core																										
		End of Drillhole		71.65																							
				5.50																							

DEPTH SCALE

1: 50



LOGGED: D.G.

CHECKED: W.C.

MIS-ROCK 001 05-1120-210-1000-ROCK.GPJ GLDR CAN.GDT 7/13/07

PROJECT 05-1120-210-2000			RECORD OF BOREHOLE No 06-2			1 OF 1			METRIC										
W.P. 4058-01-00			LOCATION N 5026548.7; E 363666.6			ORIGINATED BY D.G.													
DIST HWY 417			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY J.M.													
DATUM Geodetic			DATE May 9, 2006			CHECKED BY M.I.C.													
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 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100			WATER CONTENT (%) w _p — w — w _L 25 50 75			γ kN/m ³			GR SA SI CL		
78.1	GROUND SURFACE																		
0.0 77.9	Topsoil (FILL)		1	A.S.			78												
0.2	Fine sand, trace silt (FILL) Brown Moist																		
77.2																			
0.9	Silty clay, some sand, trace gravel (FILL) Grey brown Moist		2/3	SS	7		77												
76.5																			
1.5	Fine to coarse SAND, trace gravel Dense Brown Moist		4	SS	45		76												
75.8																			
2.3	Silty CLAY (Weathered Crust) Very stiff Grey brown Moist		5/6	SS	5		75												
75.3																			
2.8	Dolomitic limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		7	NQ RC	DD		74												
73.7																			
4.4	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong Bedrock cored between 2.8m 5.8m depth. For bedrock coring details refer to Record of Drillhole 06-2.		8	NQ RC	DD		73												
72.2																			
5.8	End of Borehole Note: Water level in standpipe at 3.1m depth below ground surface on June 12, 2006																		

MISS_MTO 05-1120-210-1000.GPJ ON MOT.GDT 7/13/07

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-2

SHEET 1 OF 1

LOCATION: N 5026548.7; E 363666.6

DRILLING DATE: May 9, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN NO.	PENETRATION RATE (mm/min)	FLUSH % RETURN	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									CL-CLEAVAGE		J-JOINT		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK		B-BEDDING							
									SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY											
									VN-VEIN		S-SLICKENSIDED		PL-PLANAR		C-CURVED											
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY												
								TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION		K _{cm/sec}												
3	Rotary Drill No Core	ROCK SURFACE		75.26	1																					
		Dolomitic limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		2.80																						
4																										
6		Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		73.68 4.40	2																					
6		End of Drillhole		72.26 5.80																						
7																										
8																										
9																										
10																										
11																										
12																										

DEPTH SCALE

1 : 50



LOGGED: D.G.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-1000-ROCK.GPJ GLDR. CAN.GDT 7/13/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-3		1 OF 1	METRIC
W.P. 4058-01-00	LOCATION N 5026503.3; E 363683.4	ORIGINATED BY D.G.			
DIST HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY J.M.			
DATUM Geodetic	DATE May 8, 2006	CHECKED BY M.I.C.			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
77.8	GROUND SURFACE							20	40	60	80	100					
0.0	Topsoil (FILL)																
0.2	Silty sand (FILL) Grey brown																
77.2																	
0.6	Fine sand, trace silt, gravel and clay (FILL) Brown Moist																
76.7			1/2	SS	10												
1.1	Sandy SILT, some gravel, trace clay (TILL) Grey Moist																
76.0			3	SS	>100												
1.8	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong Bedrock cored between 1.8m 4.8m depth. For bedrock coring details refer to Record of Drillhole 06-3.		4	NQ RC	DD												
			5	NQ RC	DD												
			6	NQ RC	DD												
73.0																	
4.8	End of Borehole Note: Borehole dry prior to commencing coring operations.																

MISS_MTO 05-1120-210-1000.GPJ ON_MOT.GDT 7/13/07

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-3

SHEET 1 OF 1

LOCATION: N 5026503.3; E 363683.4

DRILLING DATE: May 8, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/rev)	FLUSH % RETURN	FR/FX-FRACTURE F-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION				
								CL-CLEAVAGE				J-JOINT				R-ROUGH				UE-UNEVEN						MB-MECH. BREAK			
								SH-SHEAR				P-POLISHED				ST-STEPPED				W-WAVY						B-BEDDING			
								VN-VEIN				S-SLICKENSIDED				PL-PLANAR				C-CURVED									
								RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY											
								TOTAL CORE %	SOLID CORE %					DIP w.r.t CORE AXIS				K _{cm/sec}											
								888888	888888	888888		888888		888888				10 ⁻⁸ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³				2 4 6							
2	Rotary Drill NQ Core	ROCK SURFACE		75.99																									
		Limestone (BEDROCK)		1.80																									
		Fresh Thinly to medium bedded Grey Medium strong		1																									
3																													
4																													
5		End of Drillhole		72.99																									
				4.80																									
6																													
7																													
8																													
9																													
10																													
11																													

DEPTH SCALE

1:50



LOGGED: D.G.

CHECKED: W.C.

MIS-ROCK 001 05-1120-210-1000-ROCK.GPJ GLDR CAN.GDT 7/13/07

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-4		1 OF 1	METRIC
W.P. 4058-01-00	LOCATION N 5026487.2; E 363672.2		ORIGINATED BY D.G.		
DIST HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.		
DATUM Geodetic	DATE May 8, 2006		CHECKED BY M.I.C.		

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
78.4	GROUND SURFACE													
0.0	Topsoil (FILL)		1	A.S.										
0.2	Moist Fine sand, some silt (FILL) Brown Moist						78							
77.5														
0.9	Silty CLAY (Weathered Crust) Very stiff Grey brown Moist		2/3	SS	5		77							
76.9														
1.5	Sandy SILT, some gravel, trace clay (TILL.) Grey Moist		4	SS	>100									
76.6														
1.8	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong Bedrock cored between 1.8m 5.3m depth. For bedrock coring details refer to Record of Drillhole 06-4.		5	NQ RC	DD		76							
			6	NQ RC	DD									
			7	NQ RC	DD		75							
			8	NQ RC	DD		74							
73.1	End of Borehole													
5.3	Note: Borehole dry prior to commencing coring operations.													

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-4

SHEET 1 OF 1

LOCATION: N 5026487.2; E 363672.2

DRILLING DATE: May 8, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH %	RECOVERY TOTAL CORE % SOLID CORE %	R.O.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP W.L.L. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE		76.58											
2		Limestone (BEDROCK)		1.80											
		Fresh													
		Thinly to medium bedded													
		Grey													
		Medium strong													
3															
4															
5															
		End of Drillhole		73.08											
				5.30											
6															
7															
8															
9															
10															
11															

DEPTH SCALE

1 : 50



LOGGED: D.G.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-1000-ROCK GPJ GLDR CAN.GDT 7/13/07

APPENDIX C

Figure 1 Grain Size Distribution Test Result – Sand Fill

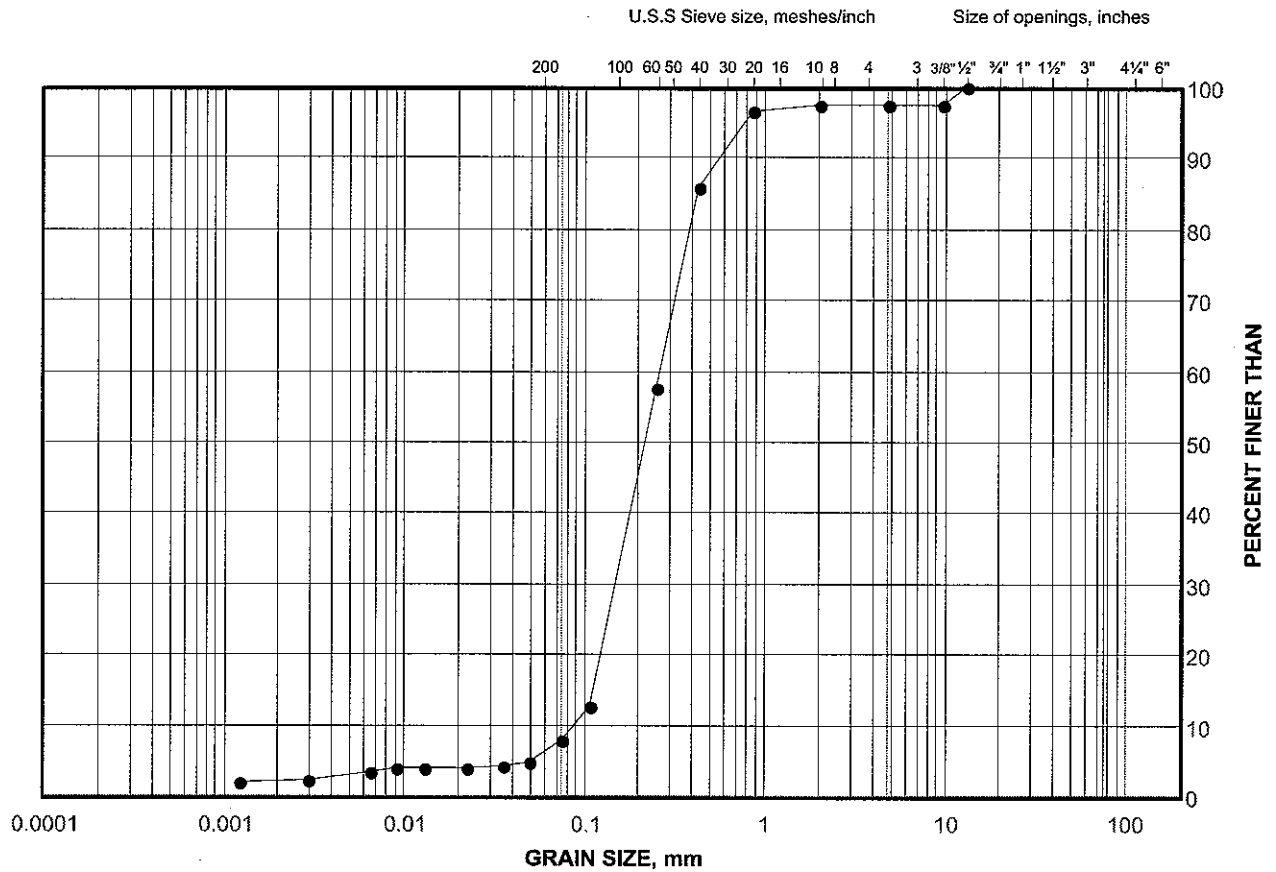
Figure 2 Plasticity Chart – Weathered Silty Clay

Figure 3 Grain Size Distribution Test Results – Glacial Till

GRAIN SIZE DISTRIBUTION

Sand Fill

FIGURE 1



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

LEGEND

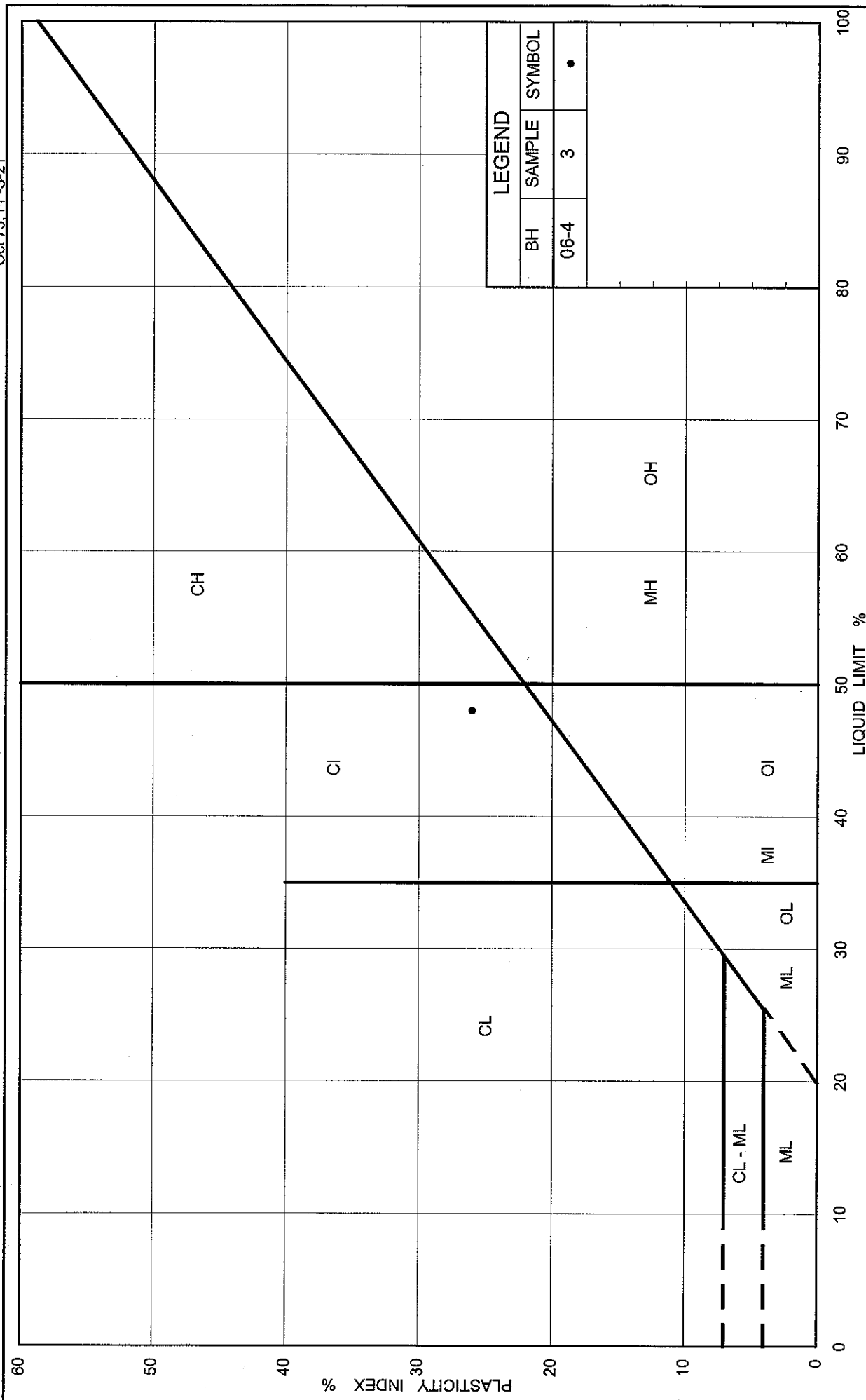
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-3	1	0.8-1.1

Project Number: 05-1120-210

Checked By: _____

Golder Associates

Date: 08-Jun-06



LEGEND		
BH	SAMPLE	SYMBOL
06-4	3	•

Ministry of Transportation
Ontario

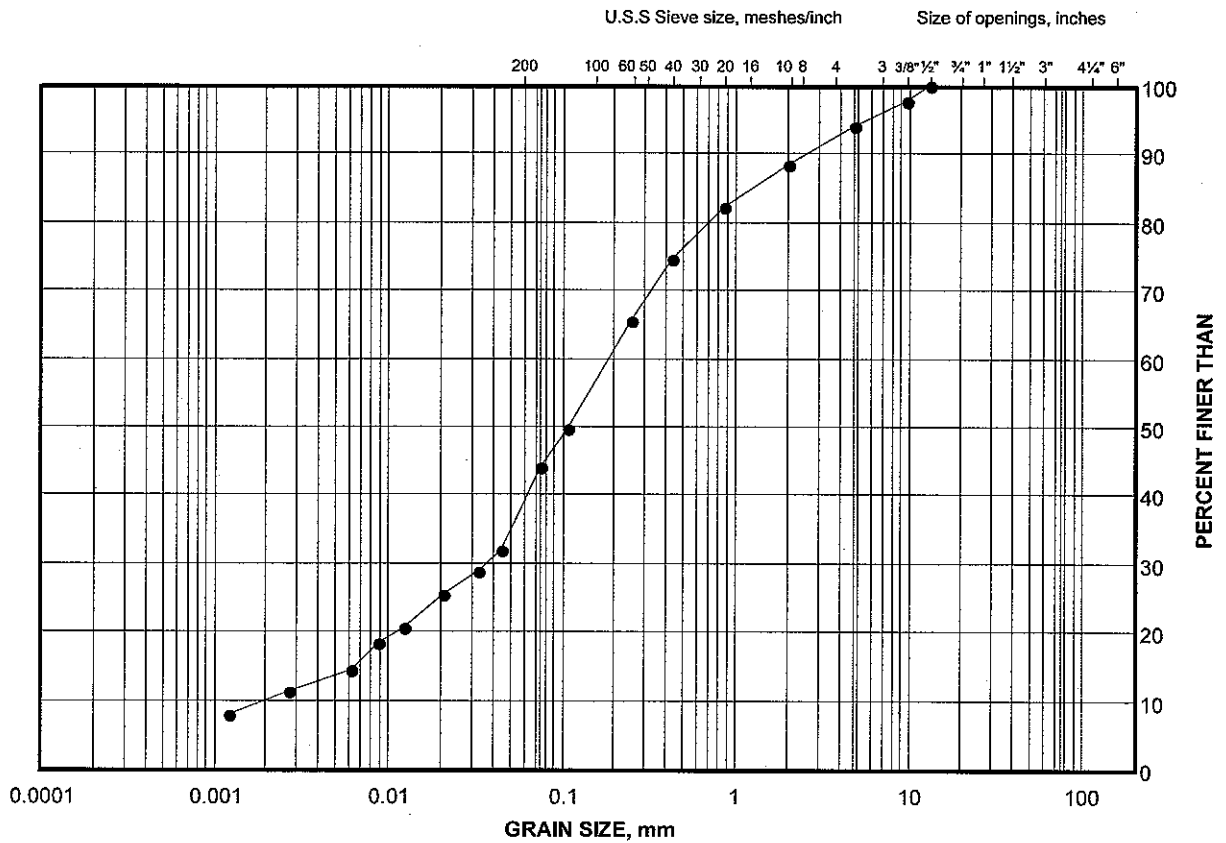
FIG No. 2

PLASTICITY CHART
Weathered Silty Clay

Project No. 05-1120-210 - 2000

GRAIN SIZE DISTRIBUTION Glacial Till

FIGURE 3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-1	3	1.7-2.1

Project Number: 05-1120-210

Checked By: _____

Golder Associates

Date: 07-Jun-06

APPENDIX D

Records of Previous Boreholes 1 to 4 (Geocres No. 58-F-224-C)

APPENDIX E

**Non Standard Special Provisions - Mass Concrete
- Dowels into Rock**

MASS CONCRETE – Item No.

Special Provision

Scope of Work

The scope of work for the above noted tender item includes the mass concrete under the abutment and/or retaining wall footings.

Basis of Payment

Payment at the contract price for the above noted tender item includes full compensation for all labour, equipment and materials to do the required work.

n:\active\2005\1120\geotechnical\05-1120-210 mrc hwy 417 bridges maitland to island park drive\foundations\unssp\retaining walls\sp-mass concrete.doc

DOWELS INTO ROCK – Item No.

Special Provision

May 22, 2007

1.0 GENERAL

1.1 Scope

The work for the above noted tender item shall be in accordance with OPSS 904, including all special provision, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the structure footings.

1.2 Instructions to Contractor

- 1.2.1 These instructions are to be read in conjunction with the Contract Drawings.
- 1.2.2 A total of 1 test Dowels into Rock are required for the Dowels into Rock at each structure footing.
- 1.2.3 Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

1.3 Qualifications

- 1.3.1 **Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock:** All work shall be performed under the direction of personnel experienced with all aspects associated with the installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.
- 1.3.2 **Qualifications of the Quality Verification Engineer:** A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.
- 1.3.3 **Qualifications of the Design Engineer:** A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of

experience of projects of similar nature and scope to the work required for this project.

1.4 Responsibilities of the Contractor

- 1.4.1 The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.
- 1.4.2 The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.
- 1.4.3 The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.
- 1.4.4 The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 1.6.

1.5 Definitions

- 1.5.1 Dowels into Rock: reinforcing steel bar and non-shrink grout.
- 1.5.2 Design Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.
- 1.5.3 Quality Verification Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

1.6 Submissions and Working Drawings

- 1.6.1 Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.
- 1.6.2 The Contractor shall submit Working Drawings to the Contract Administrator as follows:

- All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.
 - All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.
- 1.6.3 Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.
- 1.6.4 Working Drawings consisting of testing and installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.
- 1.6.5 Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:
- Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.
 - Test results verifying the 28 day strength of non-shrink grout.
 - The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
 - The procedures to verify hole length. Records of measurements that verify the hole length.
 - Records of all drilling procedures, rock conditions encountered, and installation times.
 - Test procedures for Dowels into Rock.
 - Drawings and design calculations for a suitable reaction system for the applied test loads.
 - Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.

- Drawings and details for reference system arrangement.
- Current calibration curves shall be provided for all gauges.
- Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.
- Remedial measures for unacceptable stressing results.

1.7 Subsurface Conditions

- 1.7.1 Rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

2.0 MATERIALS

The non-shrink grout shall be an approved DSM 9.10.35 non-shrink grout.

The Contractor shall provide the following information from the manufacturer for non-shrink grout:

- Data sheets for the non-shrink grout,
- installation procedures

3.0 EQUIPMENT

3.1 General

- 3.1.1 All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

- 3.1.2 The equipment shall not cause damage to the reinforcing steel bars.

4.0 INSTALLATION

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

4.1 Construction of Holes

- 4.1.1 The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the

method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.

4.1.2 The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.

4.1.3 At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

4.2 Installation of Reinforcing Steel Bar

4.2.1 Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.

4.2.2 Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.

4.2.3 Dowels shall extend through the tremie concrete for the footing and into sound bedrock.

4.2.4 Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

4.3 Grout

4.3.1 The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.

4.3.2 The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

4.3.3 Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

5.0 TESTING REQUIREMENTS

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

5.1 General Testing Requirements

- 5.1.1 Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.
- 5.1.2 The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.
- 5.1.3 The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the each structure location.
- 5.1.4 The Contractor shall submit Working Drawings that include proposed procedures for testing of the dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.
- 5.1.5 The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.
- 5.1.6 The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.
- 5.1.7 The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

5.2 Testing Location

- 5.2.1 The Contractor shall remove all loose rock down to sound bedrock at the test location.
- 5.2.2 The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator.

- 5.2.3 If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

5.3 Testing Equipment

- 5.3.1 The dowels into rock will be carried out generally in accordance with the prevailing requirements of A.S.T.M. (Designation D1143-81) superseded where applicable by the procedures specified in this document.
- 5.3.2 The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.
- 5.3.3 The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:
- The beams shall be independently supported with the support firmly embedded in the ground.
 - The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.
 - Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.
- 5.3.4 The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

5.4 Testing for Dowels Into Rock, and Report

- 5.4.1 At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.

- 5.4.2 Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.
- 5.4.3 Jacks used for reinforcing steel bars shall have a minimum ram dimension of 153 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.
- 5.4.4 Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

5.5 Testing Loading

- 5.5.1 The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the specified test load. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.
- 5.5.2 Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

5.6 Acceptance Criteria

- 5.6.1 The following acceptance criteria apply:

The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at each structure location.

Tests for Dowels into Rock shall have a capacity of at least insert value kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

6.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for

tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.