

Golder Associates Ltd.

32 Steacie Drive
Kanata, Ontario, Canada K2K 2A9
Telephone 613-592-9600
Fax 613-592-9601



REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN REPORT
CARLING AVENUE EASTBOUND OVERPASS
BRIDGE WIDENING
STRUCTURE SITE 3-44
HIGHWAY 417
W.P. 4058-01-00**

Submitted to:

McCormick Rankin Corporation
300-1145 Hunt Club Road
Ottawa, Ontario
K1V 0Y3

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PART A

**FOUNDATION INVESTIGATION REPORT
CARLING AVENUE EASTBOUND OVERPASS BRIDGE WIDENING
STRUCTURE SITE 3-44
HIGHWAY 417
W.P. 4058-01-00**

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 Carling Avenue Eastbound (EB) including the bridge 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 dated December 7, 2005.

2.0 SITE DESCRIPTION

The Carling Avenue EB Bridge is a slab-on-steel girder overpass structure for Highway 417 and is located within a commercial area of Ottawa.

Carling Avenue EB is a three lane one-way road with an urban cross-section, a sidewalk on the west side, and a gravel boulevard adjacent to the east abutment. 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 bridge 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 less than approximately 19% of the exposed face. From a foundation perspective, the bridge is understood to be performing adequately.

The existing approach embankments are about 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 1957. The results of that investigation are contained in the report titled "Report on Foundation Investigation at Ottawa Queensway and Carling Avenue, Structure No. 5, to Deleuw, Cather and Company of Canada Limited" (Geocres No. 58-F-221-C).

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out on May 16 to 18 and on May 26, 2006. On those days, four boreholes (Boreholes 06-5 to 06-8, inclusive) were put down at the locations shown on Drawing 1. The boreholes were drilled at the approximate locations of the ends of the proposed abutment widenings. 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 5.9 m to 7.3 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 3.2 to 4.6 m 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 was installed in Borehole 06-5 and 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 Limits testing, and grain size distribution analyses were carried out on selected soil samples.

The groundwater level was measured in the standpipe in Borehole 06-5 on June 12, 2006 about three weeks 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)
06-5	North-east abutment	5027120.8	364090.8	75.6
06-6	South-east abutment	5027111.6	364143.8	75.1
06-7	North-west abutment	5027104.4	364078.1	75.8
06-8	South-west abutment	5027097.9	364347.3	75.8

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 Carling Avenue EB bridge. The borehole locations and ground surface elevations are shown on Drawing 1 in Appendix A. Soil stratigraphy sections projected along the centreline of the bridge and across the abutment foundation areas are shown on Drawing 2 in Appendix A.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are given on the Record of Borehole sheets in Appendix B and Figures 1 and 2 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, as previously noted (Geocres No. 58-F-221-C), and the Record of Borehole sheets from that investigation are also attached in Appendix D.

In summary, the soils encountered during the current investigation within the limits of the widening consist of pavement structure and fill materials to depths of about 1.1 m to 1.7 m, over

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.

thin deposits of peat, sand, silt, and silty clay to clayey silt, underlain by glacial till extending to depths of about 2.4 to 2.8 m. These overburden materials are underlain by limestone bedrock. The upper 1.2 to 1.9 metres of bedrock contain weathered and fractured zones.

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

4.2.1 Pavement Structure

The pavement structure of Carling Avenue at Borehole 06-6 consists of about 50 millimetres of asphaltic concrete underlain by about 280 millimetres of Portland cement concrete. The asphaltic concrete and concrete are in turn underlain by a thin layer of crushed stone base course materials (about 75 millimetres in thickness).

4.2.2 Topsoil

Topsoil (fill) exists at the ground surface at Boreholes 06-7 and 06-8 where it is about 0.3 and 0.2 m thick, respectively.

4.2.3 Fill

Fill material associated with previous uses of the site and roadway construction underlies the pavement structure and topsoil at Boreholes 06-6, 06-7 and 06-8 and extends from the ground surface at Borehole 06-5.

At Borehole 06-5, a layer of crushed stone fill extends from the ground surface to a depth of about 0.2 m. The crushed stone is in turn underlain by fine to coarse sand fill to a depth of about 0.8 m, and then by sandy silt fill with trace amounts of peat to a depth of about 1.3 m (i.e., Elevation 74.3 m).

At the remainder of the boreholes the pavement structure and topsoil are underlain by fine sand fill materials, extending to depths ranging from about 1.1 to 1.3 m (i.e., Elevations 74.6 to 74.0 m). The fine sand fill material at Borehole 06-8 is underlain by about 0.5 m of sandy silt and silty sand fill.

4.2.4 Peat

The fill materials at all the boreholes are underlain by a layer of peat ranging in thickness from about 0.1 to 0.4 m. The measured natural water contents of two samples of the peat were 70 and 163 percent.

4.2.5 Silty Clay to Clayey Silt

The peat at Boreholes 06-5 and 06-7 is underlain by thin deposits of silty clay and clayey silt, which are about 0.1 to 0.2 m in thickness. The silty clay and clayey silt at these locations have been weathered to a grey-brown colour.

4.2.6 Sand and Silt

The silty clay at Borehole 06-5 is underlain by about 0.2 m of silty sand (extending to about Elevation 73.6 m) and the peat at Borehole 06-8 is underlain by about 0.5 m of silt (extending to about Elevation 73.5 m). Grain size distribution test results obtained from a sample of the silt at Borehole 06-8 are shown on Figure 1. The measured natural water content of the silt was 18 percent.

4.2.7 Silty Sand to Sandy Silt Till

A 0.4 m to 1.2 m thick layer of glacial till was encountered below the peat deposit in Borehole 06-6, the silty clay in Borehole 06-7, and the sand and silt materials in Boreholes 06-5 and 06-8. The surface of this till deposit ranges between about elevations 73.5 m and 74.1 m in the boreholes (at depths below ground surface ranging from 1.2 m to 2.3 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. Grain size distribution test results obtained from samples of the glacial till at Boreholes 06-6 and 06-7 are shown on Figure 2. It should be noted however that these samples were retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposit. Measured SPT "N" values of 20 to 79 blows per 0.3 m of penetration indicates the deposit to have a compact to very dense relative density, though the higher "N" values may reflect the cobble and boulder content. Rotary diamond drilling techniques were required to penetrate the boulders in Borehole 06-5. The measured natural water contents of three samples of the glacial till ranged from 5 to 8 percent.

4.2.8 Limestone Bedrock

Limestone bedrock was encountered beneath the glacial till at all of the abutment widenings. The following table summarizes the bedrock surface depth and elevation data as encountered at the locations of Boreholes 06-5 to 06-8, 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)
East Abutment	06-5	75.6	2.8	72.8
	3	75.0	0.9	74.1
	4	75.8	2.4	73.4
	06-6	75.1	2.4	72.7
West Abutment	06-7	75.8	2.7	73.1
	1	75.2	1.1	74.1
	2	76.1	2.3	73.8
	06-8	75.8	2.7	73.1

The limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded. Occasional shale interbeds were also noted in the rock core.

The upper portion of the bedrock, to depths ranging from about 1.2 m to 1.9 m below the rock surface, contains fractured and weathered zones, with the Rock Quality Designation (RQD) values ranging from about 19 to 50 percent indicating very poor to poor quality rock. In Borehole 06-7, voids and mud seams appear to exist in this upper portion of bedrock, over the interval from about 0.5 m to 1.9 m below the bedrock surface.

The remainder of the bedrock in all of the boreholes, beneath the fractured zones, is slightly weathered to fresh. The Rock Quality Designation (RQD) values measured on recovered bedrock core samples from this bedrock typically ranged from about 72 to 100 percent indicating fair to excellent quality rock. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes.

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.

4.2.9 Groundwater Conditions

A piezometer was installed in Borehole 06-5, sealed within the bedrock, and the water level measured in that piezometer is given in the following table:

Borehole No.	Borehole Location	Date	Depth (m)	Elevation (m)
06-5	Northeast abutment	June 12, 2006	2.4	73.2

Borehole 06-6 remained dry during the short time it remained open following the overburden drilling and prior to commencing bedrock coring operations. The water levels in the casing in Boreholes 06-7 and 06-8 after completion of coring are summarized in the following table:

Borehole No.	Borehole Location	Depth (m)	Elevation (m)
06-7	Northwest abutment	2.4	73.4
06-8	Southwest abutment	1.9	73.9

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 of Ottawa. The field portions were supervised by Mr. James Samotowka and 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. This report was reviewed by Mr. Fintan J. Heffernan, P.Eng, the designated MTO contact for this project.

GOLDER ASSOCIATES LTD.

William Cavers, P.Eng
Geotechnical Group



Fintan J. Heffernan, P.Eng.
Designated MTO Foundations Contact



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PART B

**FOUNDATION DESIGN REPORT
CARLING AVENUE EASTBOUND OVERPASS BRIDGE WIDENING
STRUCTURE SITE 3-44
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 Carling Avenue EB 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 Carling Avenue EB 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 by about 11.4 metres and to the south by about 5.2 metres and will be converted to semi-integral type abutments. In addition, the roadway profile of Carling Avenue EB will be lowered by about 200 mm to maintain the existing vertical clearance between Carling Avenue EB and the widened overpass structure.

Wing (retaining) walls extending about 15 to 18 m back from the abutments have been proposed. It is anticipated that the height of additional fill placed on the existing side slopes will range from about 3 to 4 metres. Embankments with 2H:1V side slopes may extend past the ends of the bridge approach retaining 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 may be considered for the widening:

- Shallow foundations supported on the native glacial till soils.
- Shallow foundations supported by the 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 so as to be consistent with the existing

abutments (which is understood to be important from a structural perspective), then the pile cap level would be no more than 1 metre above the bedrock surface. Shallow foundations on bedrock are 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. The till deposit is only 0.4 metres thick at the southwest abutment widening and is only modestly thicker (at about 0.8 to 1.2 metres thick) at the remainder of the widenings. The much higher resistance available from the underlying bedrock with only slightly deeper excavation make 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 foundation option 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 the 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 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. For footings on bedrock, the borehole information indicates bedrock surface levels at the abutments as follows:

Foundation Element	Bedrock Surface Elevation (metres)
West Abutment – North Widening	73.1
West Abutment – South Widening	73.1
East Abutment – North Widening	72.8
East Abutment – South Widening	72.7

The above design 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 73.2 metres. It is recommended that the new foundations be founded at this same elevation.

The borehole results indicate that the bedrock surface is 0.1 to 0.5 m below the existing founding elevation of 73.2 metres. The surficial portion of the bedrock is fractured and weathered in some locations (i.e., at the south widenings in particular) and additional excavation may be required. It is not recommended to step the footing down. Rather, provision should 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. A Non Standard Special Provision has been included in Appendix E of this report for excavation immediately adjacent to the existing overpass footings.

6.2.1.1 Limits States Factored Geotechnical Resistance

The bridge abutments may be supported on spread footings placed on the limestone bedrock. Some fractured and weathered bedrock was encountered at the rock surface, particularly at the south widenings, and subexcavation of that bedrock may be required prior to construction of the footings. 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.

In the boreholes at the north widenings, fractured and weathered bedrock was also encountered, but beneath more competent bedrock. Removal of the overlying rock to also excavate the deeper fractured and weathered rock is not considered necessary, unless possibly if voids are present as discussed below.

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 2,000 kPa. This factored ULS resistance value considers the fractured nature 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.

Voids are indicated to potentially exist within the upper 2 m of the bedrock in at least one borehole (Borehole 06-7). Excessive settlement of the widening structure (i.e., greater than 25 mm) may result if the lateral extent of the void areas in the bedrock are sufficient to collapse under the foundation loads and this resulting settlement would be entirely differential with respect to the existing bridge structure. Probe holes (50 mm diameter drilled holes) should be advanced at a 2 m spacing and to depths of about 2 m below founding elevation at all foundation elements and should be inspected by the Quality Verification Engineer to ensure that the underlying bedrock is free of void areas. If the voids are found to be present, three options may be considered:

1. The bedrock may be subexcavated to below the voids and replaced with mass concrete;
2. The factored ULS bearing resistance may be reduced; or,
3. The bedrock may be grouted.

A Non Standard Special Provision has been included in Appendix E of this report for carrying out rock probing.

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 (e.g., 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. The unfactored ULS lateral bearing resistance of the limestone may be taken as the lesser of 30 MPa or the compressive strength of the Portland cement grout. 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 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 of 0.8 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, that any voids in the rock have been grouted as noted in Section 6.2.1.1 and that the bearing surface is vertical and undisturbed. 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 can, 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. A Non Standard Special Provision has been included in Appendix E of this report for excavation immediately adjacent to the existing overpass footings.

Dowelling is typically more labour intensive in comparison to the relatively limited rock excavation required for a shear key. However, the rock surface is generally below the anticipated founding depth of 73.2 metres at these widenings; dowel installation through mass concrete may be more practical than rock excavation. Additionally, 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. Considering the anticipated rock excavation depths for shear keys and the potential for disturbing the rock supporting the existing footings at the anticipated excavation depths, it is recommended that dowels be provided at the widening locations if additional lateral resistance is required.

6.2.1.3 Frost Protection

The presence of mud seams at some locations within about the upper 2 m of the bedrock at this site means that the rock is potentially frost susceptible and it is recommended that 1.8 m of earth cover for frost protection purposes be provided.

It is understood that the grade above the existing bridge foundations may be lowered by about 200 mm, to achieve a change in the profile of Carling Avenue. Design drawings for the existing bridge indicate that the footings for that bridge are founded on the bedrock surface, at depths ranging from about 2 to 2.5 m. That rock below the existing footings is also potentially frost

susceptible and therefore frost protection (i.e., insulation of the foundations) is recommended if the grade changes would result in less than 1.8 m of earth cover as discussed in Section 6.7.3.

6.3 Bridge Retaining Wall Foundation Options

Bridge retaining walls extending 15 to 18 metres back from the abutments and founded on separate foundations have been proposed.

It is considered that cantilevered reinforced concrete retaining walls supported on shallow foundations or RSS walls may be used at this location.

The choice of retaining 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 high appearance criteria is recommended for the bridge retaining walls at this site.

Settlements at this location should be less than the allowable limits for RSS walls (considered to be less than 1/500 of differential settlement or 40 mm of differential settlement per 20 m of wall length) and the founding conditions are suitable for support of reinforced concrete retaining walls on shallow footings. From a foundation perspective, either option is feasible and practical at this location.

The reinforcing strips for a RSS wall are typically about 0.8 of the height of the wall in length and therefore some subexcavation of the existing embankment fill may be required 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 the removal of the existing embankment fill materials. However RSS walls would still typically be less expensive than concrete walls.

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

- Shallow foundations supported on the native soils.
- Shallow foundations supported by the 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 as noted in Section 6.2 above.

It is considered feasible to support the bridge retaining walls on spread footings on or within the native soils at this site. Larger settlements would however occur for the 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 retaining walls supported on the native soils are considered, the walls should be provided with an articulated joint with the bridge abutments.

The 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 bridge 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 bridge retaining walls.

The bridge retaining walls can also be founded on the native soils (i.e., silty clay, silt, sand or glacial till) at about elevation 74 m. Higher bearing resistances may be realized by founding the walls on footings supported entirely on the glacial till at slightly lower elevations. The existing fill materials and peat are not suitable for support of retaining wall footings and need to be removed from the foundation areas.

6.3.1.1 Limits States Factored Geotechnical Resistance

The bridge retaining walls can be supported on spread footings placed on the limestone bedrock. The surficial portion of the bedrock is, in local areas, of poor quality (i.e., at 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 highly 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 2,000 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.

The retaining walls can be supported on spread or strip footings placed on the undisturbed native soils (i.e., silty clay, silt, sand and 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 factored geotechnical resistance at SLS (for 25 mm of settlement) of 150 kPa may be used for the design of walls placed on properly prepared native (i.e., silty clay, silt, sand or glacial till) soils immediately below the fill materials and peat. For retaining walls supported entirely on or within the glacial till, after removal of the less competent native soils (i.e., silty clay, silt or sand), a factored geotechnical resistance at ULS of 300 kPa and a factored geotechnical resistance at SLS (for 25 mm of settlement) of 200 kPa may be used for design.

RSS walls, designed with the geotechnical resistances given above for cantilevered retaining walls, may 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 A or Granular B Type 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 bridge 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 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 providing a shear key or dowelling into the bedrock. Considering the depth to bedrock and the anticipated founding depth of 73.2 m it is recommended that dowels be provided if additional lateral resistance is required.

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. The unfactored ULS lateral bearing resistance of the limestone may be taken as the lesser of 30 MPa or the compressive strength of the Portland cement grout. 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 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-east retaining wall	73.9 – 73.7	Weathered Silty Clay	0.43	0	50
	73.7 – 73.6	Silty Sand	0.45	0	
	73.6 – 72.8	Till	0.50	0	
South-east retaining wall	73.9 – 72.7	Till	0.50	0	
North-west retaining wall	74.3 – 74.1	Silty Clay and Clayey Silt	0.43	0	50
	74.1 – 73.1	Till	0.50	0	
South-west retaining wall	74.0 – 73.5	Silt	0.45	0	
	73.5 – 73.1	Till	0.50	0	

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

For footings on 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.

Where bridge 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

The native soils are potentially frost susceptible. The presence of mud seams at some locations within about the upper 2 m of the bedrock at this site means that the rock is also potentially frost susceptible. Therefore 1.8 m of earth cover for frost protection purposes is recommended.

6.4 Site Coefficient

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 silt, 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.

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 wing (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 a 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 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 Retaining Wall Design and Construction

The proposed work will require widening the existing approach embankments laterally by 11 to 12 metres on the north side of the highway and by about 5 to 6 metres on the south side of the highway. The existing embankments are about 5 to 6 metres in height. Wing (retaining) walls for the widened embankments will also be required extending about 13 to 14 metres from each abutment face along Highway 417. These wing (retaining) walls may be constructed as concrete cantilevered walls or as RSS walls founded on the native soils or engineered fill. Beyond the ends of the wing walls and parallel to the wing walls, the embankments will have 2 horizontal to 1 vertical side slopes.

Based on the borehole results, the embankment subgrade soils will consist of fill materials, peat, and thin discontinuous layers of silty clay, clayey silt, silt and silty sand. These materials are in turn underlain by compact to very dense silty sand to sandy silt till. The fill materials overlying the peat are generally composed of sand with varying amounts of silt and gravel.

6.6.1 Subgrade Preparation and Embankment Construction

Any surficial 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. All subgrade soils should be proof-rolled prior to fill placement.

A thin layer of peat is present beneath the existing embankment fill materials and underlying the fill materials beyond the toe of the existing slope. Some compression of the peat should be expected beneath the widened portion of the embankment due to the additional fill weight. Some limited settlement (less than 15 millimetres) of the new lanes, which will be built over the existing embankment side slope, could occur. However removal of the peat beneath that portion of the widening that underlies the new lanes is not considered practical since shoring of the existing lanes would be required to excavate the embankment side slope for removal of the peat. The peat beneath that portion of the embankment has also already been previously loaded and, considering its relatively limited thickness, the potential settlement is likely limited in magnitude. Some increased, but still limited settlement (less than 25 millimetres), may occur for the embankment widening constructed over the peat beyond the toe of the existing embankment. However, this peat is not anticipated to underlie the new roadway and the resulting settlement due to compression of the peat will have less impact on that surface. Much of the settlement would also occur fairly rapidly, prior to final paving and it is therefore considered more practical to leave the peat in place over the full width of the widening.

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.

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.

Concrete cantilever wing (retaining) walls on the north side of the highway, 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 have a factor of safety greater than 1.3 against deep seated slope instability. On the south side of the highway, due to the presence of a loose silt layer, concrete cantilever wing (retaining) walls, up to 6 m in height, will have a reduced factor of safety against instability if backfilled to their full height without any restraint/backfill in front of the wall. These walls should therefore be provided with a 2H:1V toe slope extending down from about elevation 77 m on the outside of the wall to increase the factor of safety to 1.3. This backfill in front of the wall should be placed before the backfill grade behind the wall is raised above Elevation 77 m. Alternatively, the 0.5 m thick silt layer could be removed and the approach retaining wall founded on the underlying glacial till.

RSS walls, up to 6 m in height, founded on a pad of engineered fill on the native soils after removal of the existing fill materials and surficial topsoil will 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.

The 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°	
Weathered Silty Clay and Clayey Silt	17.5		80
Sand and Silt	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, peat, and relatively thin native overburden soils.

Provided that the new embankment fill material consists of 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 the existing fill materials and peat. The coefficient of consolidation of the peat is relatively high due to the fibrous nature of the material. The subgrade settlements resulting from compression of the peat, which should be modest in magnitude (in the range of 15 to 25 mm), would be expected to occur quite rapidly, likely entirely during embankment construction. Similarly, the coefficient of consolidation of the weathered silty clay and clayey silt, typically being a fissured soil and being stressed within its re-compression limits, is relatively high. These deposits are also very thin. Therefore the subgrade settlements resulting from compression of the weathered silty clay and clayey silt, which should also be modest in magnitude (less than 25 mm), would be expected to occur quite rapidly, also likely entirely during embankment construction. The subgrade settlements resulting from compression of the silt, silty sand and 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 peat and native subgrade soils would be expected to not exceed 50 mm.

Based on the estimated settlements, it is recommended that the widened embankments be constructed as early as possible in the contract to allow the maximum amount of time available for settlement prior to paving of Highway 417 widening.

6.7 Design and Construction Considerations

6.7.1 Existing Utilities

Existing utilities at this bridge location include a 406 mm watermain extending parallel to Highway 417 south of the bridge location and which is about 6 metres from the edge of the widened structure. This watermain is outside the zone of the influence of the retaining wall footings and embankments and is likely outside the zone of disturbance due to construction.

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 2 m of existing sandy fill materials, overlying less than 1.3 m of peat, silty clay, clayey silt, silt, silty sand, and compact to very dense silty sand to sandy silt till. The groundwater level at the site is typically less than 1 m above 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 above the groundwater level are classified as Type 1 soils and as Type 3 soils where below the groundwater level, according to the OHSA. The existing fill materials and peat above the groundwater level are also classified as Type 3 soils. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

The above guidelines are also applicable for excavations advanced at the existing abutment face and footings for refacing of the abutment.

It is also understood that excavations up to 2 m in depth may be required behind the ballast walls. Although boreholes were not advanced at these locations, the approach embankment fill materials are above the groundwater level and would likely be classed as Type 3 soils, according to OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these fill materials should therefore be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

6.7.3 Existing Foundations – Frost Protection

It is understood that the profile grade of Carling Avenue EB may be lowered by about 200 mm and that this may result in a reduction to less than the minimum of 1.8 m of earth cover recommended for frost protection of the existing bridge foundations.

The existing bridge foundations may be insulated by placing insulation over the existing footings and extending the insulation horizontally from the base of the footing as shown in Figure 3. The levelling pad for the insulation should be constructed with mortar sand meeting the gradation limits of Table 1 in Canadian Standards Association (CSA) standard A17-04 (clause 5.3.2.2).

It is also 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.

6.7.4 Temporary Shoring

It is anticipated that temporary roadway protection will be required along Highway 417 and Carling Avenue to permit construction of the abutment widenings and new foundations.

It is understood that the design of the shoring will be the responsibility of the contractor. To the expected depths of excavation, it is not expected that basal heaving or basal instability will be a concern. 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 embankment fill and 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 to 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. Cantilevered soldier piles socketed into the underlying bedrock might also be feasible.

Soldier pile and lagging would likely be the preferred shoring type along Carling Avenue EB 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 for these excavations.

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

6.7.5 Construction Staging

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

6.7.6 Decommissioning of Boreholes

The standpipe in Borehole 06-5 will be decommissioned.

6.7.7 Groundwater and Surface Water Control

The groundwater level at the site is typically less than 1 metre above the bedrock surface. Excavations to expose the bedrock surface for founding of spread footings will likely involve modest 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. This report was reviewed by Mr. Fintan J. Heffernan P.Eng, the designated MTO contact for this project.

GOLDER ASSOCIATES LTD.

William Cavers, P.Eng.
Geotechnical Group



Fintan J. Heffernan, P.Eng.
Designated MTO Foundations Contact



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TABLE 1
COMPARISON OF BRIDGE FOUNDATION ALTERNATIVES
HIGHWAY 417 CARLING AVENUE EASTBOUND OVERPASS BRIDGE WIDENING
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 1.2 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 practical for bridge abutment widenings 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 for support of all foundation elements 	<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 BRIDGE RETAINING WALL FOUNDATION ALTERNATIVES
CARLING AVENUE EASTBOUND 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 1.2 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

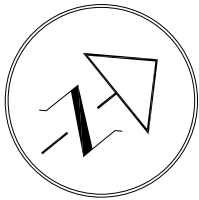
Drawing 1 Carling Avenue Eastbound, Borehole Locations

Drawing 2 Carling Avenue Eastbound, Soil Strata


HWY. 417

WP No. WP 258-98-00

CARLING AVENUE
EAST BOUND BRIDGE
BOREHOLE LOCATIONS



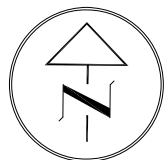
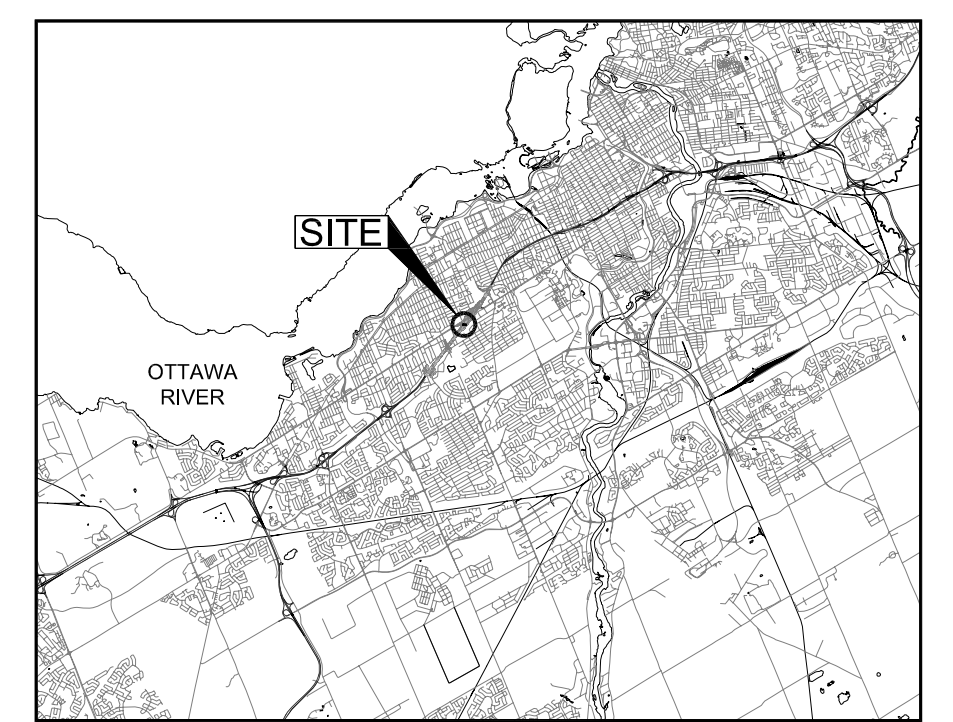
SHEET



Golder Associates



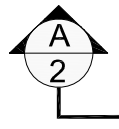
Golder Associates Ltd.

OTTAWA, ONTARIO, CANADA



KEY PLAN

LEGEND

-  Borehole — Current Golder Associates Ltd. Investigation
-  Borehole — Previous MTO Investigation
Geocres No. 58-F-221-C
-  Location of cross-section

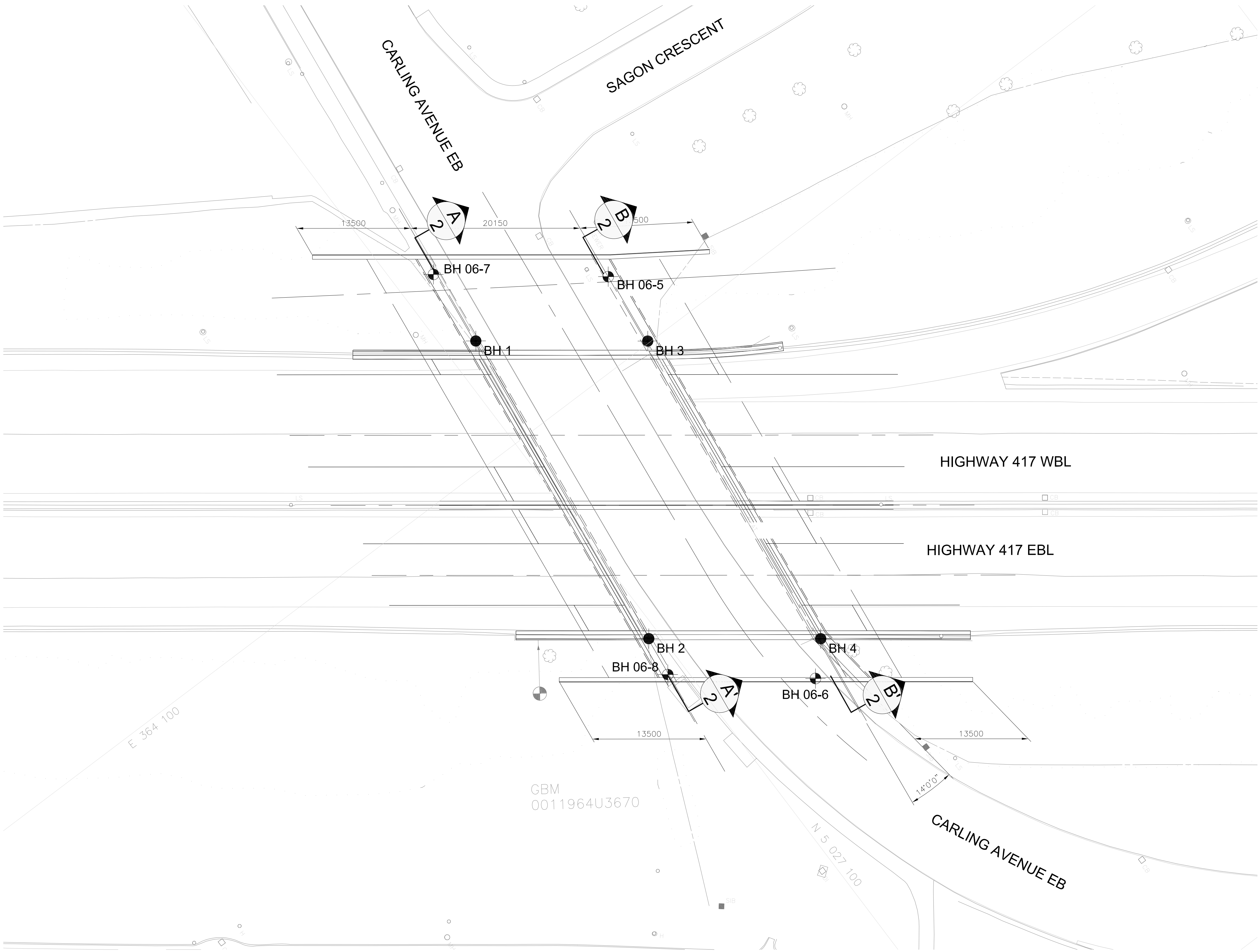
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-5	75.6	5027120.8	364090.8
06-6	75.1	5027111.6	364143.8
06-7	75.8	5027104.4	364078.1
06-8	75.8	5027097.9	364347.3
1	75.2	5027103.6	364087.4
2	76.1	5027098.7	364128.0
3	75.0	5027119.9	364099.7
4	75.8	5027115.0	364140.4

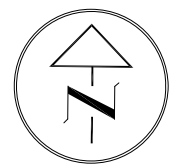
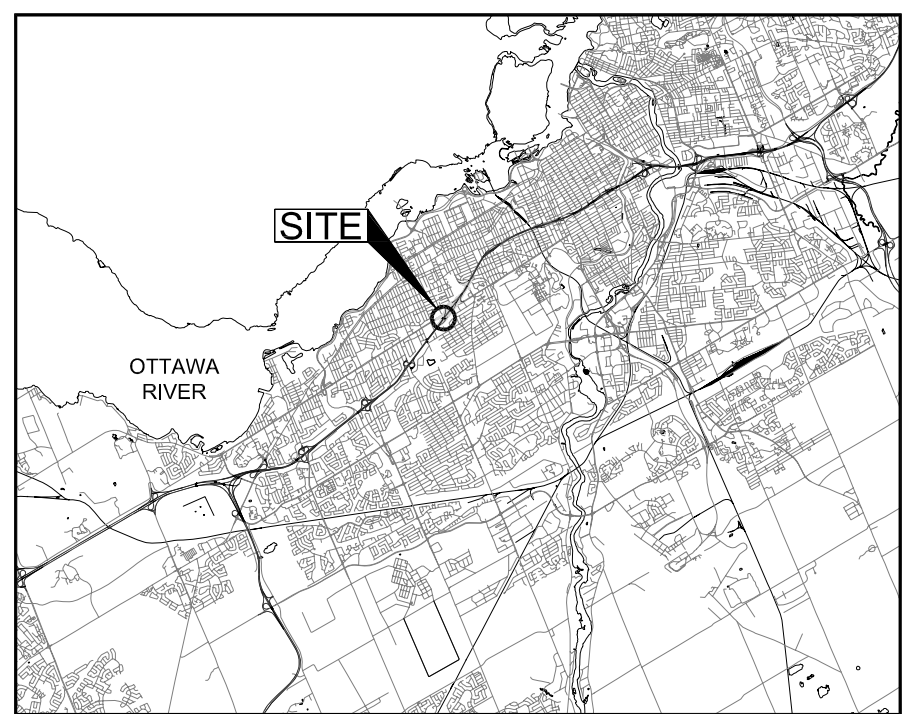
NOTES

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. 3105-219			
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

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





KEY PLAN

- LEGEND
-  Borehole – Current Golder Associates Ltd. Investigation
 -  Borehole – Previous MTO Investigation Goecres No. 58-F-221-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
06-5	75.6	5027120.8	364090.8
06-6	75.1	5027111.6	364143.8
06-7	75.8	5027104.4	364078.1
06-8	75.8	5027097.9	364347.3
1	75.2	5027103.6	364087.4
2	76.1	5027098.7	364128.0
3	75.0	5027119.9	364099.7
4	75.8	5027115.0	364140.4

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

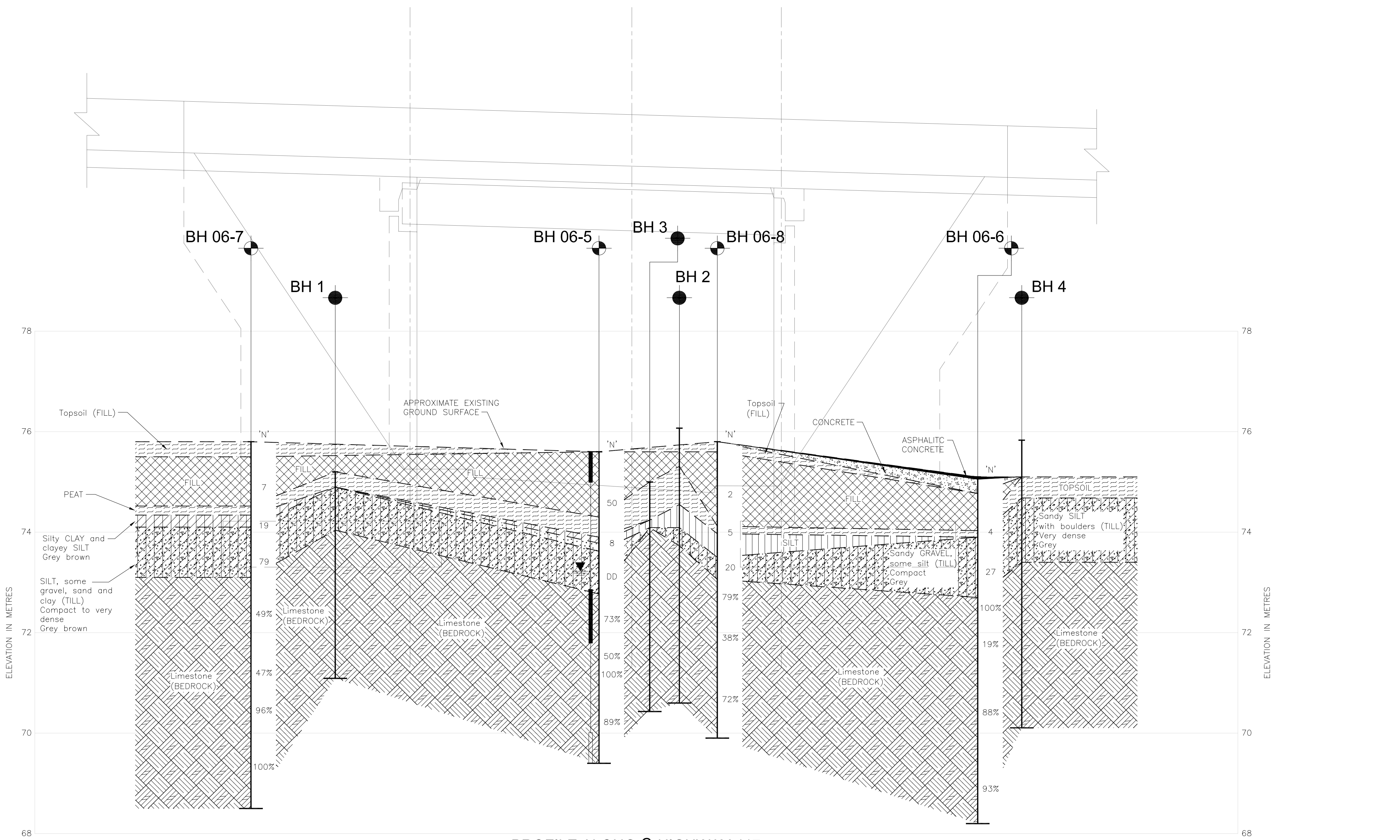
NOTES

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.

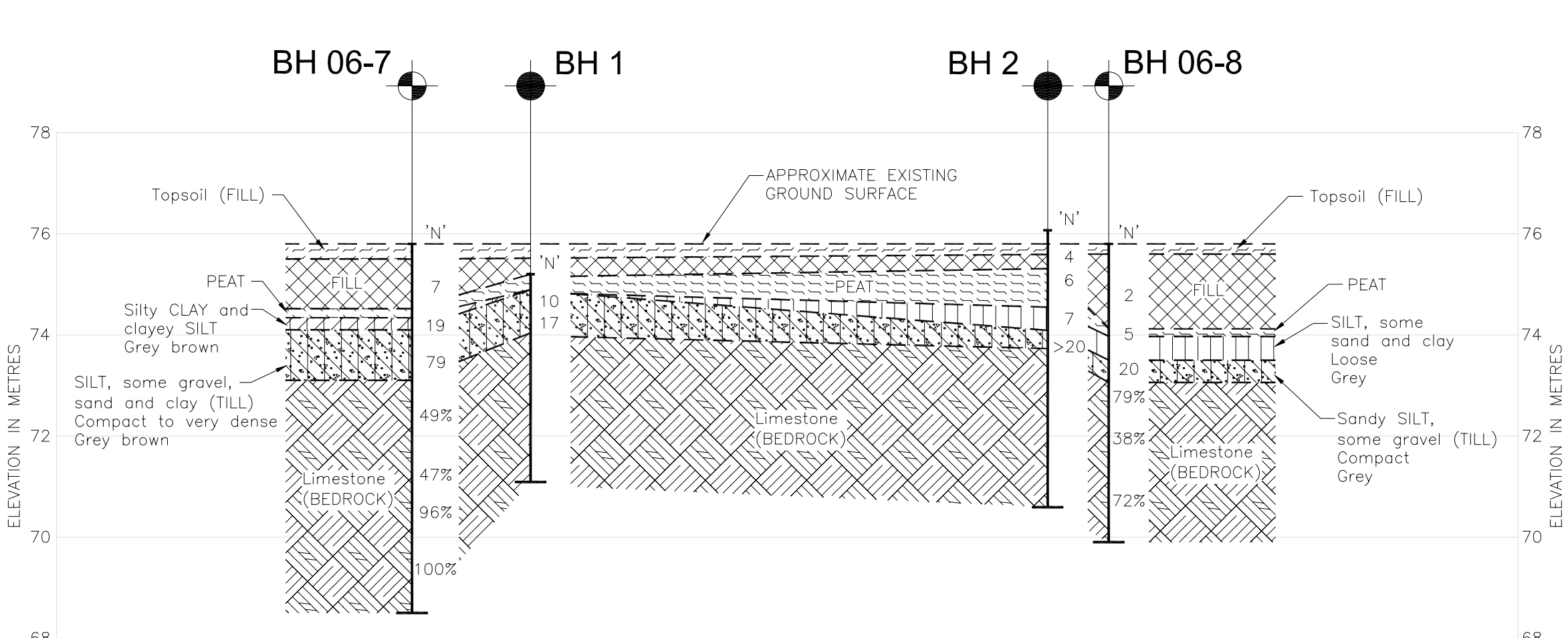
Base plan provided in electronic format by McCormick Rankin Corporation

NO.	DATE	BY	REVISION

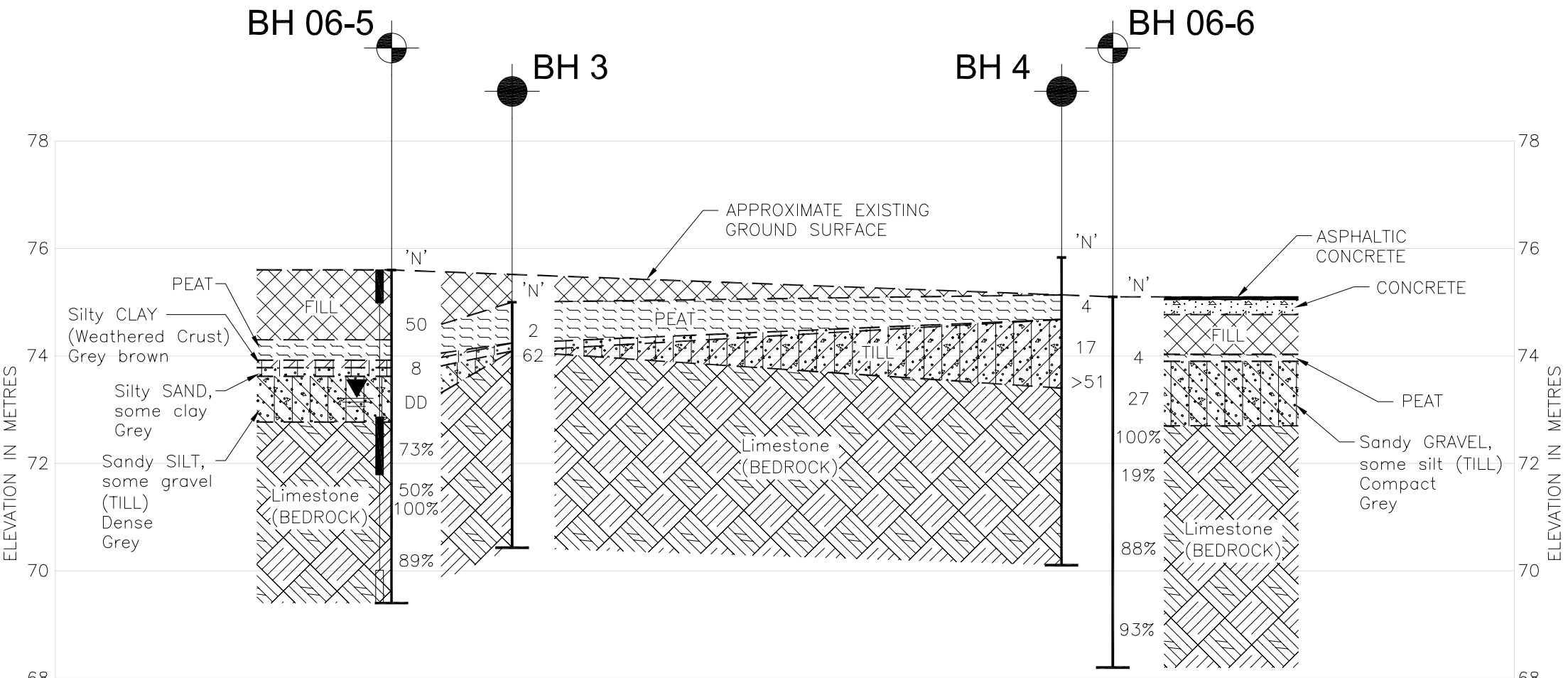
Geocres No. 31G5-219			
HWY. 417	PROJECT NO. 05-1120-210-2000	DIST.	
SUBM'D. W.C.	CHKD. M.I.C.	DATE: SEPTEMBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 2



PROFILE ALONG C HIGHWAY 417



SECTION A-A'



SECTION B-B'



APPENDIX B

Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Records of Boreholes 06-5 to 06-8

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.) DD- Diamond Drilling

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

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 ($D_R = \rho_s/\rho_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density \times 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 $= \sigma'_p/\sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
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 $= (\text{Compressive strength})/2$

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

RECORD OF BOREHOLE No 06-5

1 OF 1

METRIC

PROJECT 05-1120-210-2000

W.P. 4058-01-00

LOCATION N 5027120.8; E 364090.8

ORIGINATED BY D.J.S.

DIST HWY 417

BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger

COMPILED BY J.M.

DATUM Geodetic

DATE May 16, 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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
75.6	GROUND SURFACE						20	40	60	80	100				
0.0	Crushed stone (FILL)														
75.4	Grey														
0.2	Fine to coarse sand, some silt, trace gravel (FILL)														
74.8	Brown Moist														
0.8	Sandy silt, some gravel, trace peat (FILL)		1	SS	50										
74.3	Dense Grey Moist														
1.3	PEAT														
73.9	Moist														
	Silty CLAY (Weathered Crust)		2	SS	8										
	Grey brown Moist to wet														
2.0	Silty SAND, some clay														
	Grey Moist to wet														
	Sandy SILT, some gravel, trace clay (TILL)		3	NQ RC	DD										
72.8	Dense Grey Wet		4	SS											
2.8	Limestone (BEDROCK)														
	Fresh Thinly to medium bedded Grey Medium strong		5	NQ RC	DD										
72.0	Limestone (BEDROCK)														
3.6	Fractured Thinly to medium bedded Grey Medium strong		6	NQ RC	DD										
71.6	Limestone (BEDROCK)		7	NQ RC	DD										
4.0	Fractured Thinly to medium bedded Grey Medium strong														
	Bedrock cored between 2.8m 6.2m depth. For bedrock coring details refer to Record of Drillhole 06-5.		8	NQ RC	DD										
69.4	End of Borehole														
6.2	Note: Water level in standpipe at 2.4m depth below ground surface on June 12, 2006														

MISS_MTO 05-1120-210-2000.GPJ ON MOT.GDT 12/21/07

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

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Marathon Drilling

[illegible]

DEPTH SCALE

1 : 50

LOGGED: D.J.S.

CHECKED: W.C.

MISS MTO 05-1120-210-2000.GPJ ON MOT.GDT 12/21/07

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

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-6

SHEET 1 OF 1

LOCATION: N 5027111.6; E 364143.8

DRILLING DATE: May 26, 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	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT R-ROUGH SH-SHEAR P-POLISHED ST-STEPPED VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED										SM-SMOOTH FL-FLEXURED BC-BROKEN CORE UE-UNEVEN MB-MECH. BREAK B-BEDDING										DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
									RECOVERY			FRACT INDEX PER 0.3	DISCONTINUITY DATA			HYDRAULIC CONDUCTIVITY K, cm/sec																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
									TOTAL CORE %	SOLID CORE %	R.Q.D. %		TYPE AND SURFACE DESCRIPTION			10 ⁶	10 ⁵	10 ⁴	10 ³	10 ²	10 ¹	10 ⁰	10 ⁻¹	10 ⁻²	10 ⁻³	10 ⁻⁴	10 ⁻⁵	10 ⁻⁶			10 ⁻⁷	10 ⁻⁸	10 ⁻⁹	10 ⁻¹⁰	10 ⁻¹¹	10 ⁻¹²	10 ⁻¹³	10 ⁻¹⁴	10 ⁻¹⁵	10 ⁻¹⁶	10 ⁻¹⁷	10 ⁻¹⁸	10 ⁻¹⁹	10 ⁻²⁰	10 ⁻²¹	10 ⁻²²	10 ⁻²³	10 ⁻²⁴	10 ⁻²⁵	10 ⁻²⁶	10 ⁻²⁷	10 ⁻²⁸	10 ⁻²⁹	10 ⁻³⁰	10 ⁻³¹	10 ⁻³²	10 ⁻³³	10 ⁻³⁴	10 ⁻³⁵	10 ⁻³⁶	10 ⁻³⁷	10 ⁻³⁸	10 ⁻³⁹	10 ⁻⁴⁰	10 ⁻⁴¹	10 ⁻⁴²	10 ⁻⁴³	10 ⁻⁴⁴	10 ⁻⁴⁵	10 ⁻⁴⁶	10 ⁻⁴⁷	10 ⁻⁴⁸	10 ⁻⁴⁹	10 ⁻⁵⁰	10 ⁻⁵¹	10 ⁻⁵²	10 ⁻⁵³	10 ⁻⁵⁴	10 ⁻⁵⁵	10 ⁻⁵⁶	10 ⁻⁵⁷	10 ⁻⁵⁸	10 ⁻⁵⁹	10 ⁻⁶⁰	10 ⁻⁶¹	10 ⁻⁶²	10 ⁻⁶³	10 ⁻⁶⁴	10 ⁻⁶⁵	10 ⁻⁶⁶	10 ⁻⁶⁷	10 ⁻⁶⁸	10 ⁻⁶⁹	10 ⁻⁷⁰	10 ⁻⁷¹	10 ⁻⁷²	10 ⁻⁷³	10 ⁻⁷⁴	10 ⁻⁷⁵	10 ⁻⁷⁶	10 ⁻⁷⁷	10 ⁻⁷⁸	10 ⁻⁷⁹	10 ⁻⁸⁰	10 ⁻⁸¹	10 ⁻⁸²	10 ⁻⁸³	10 ⁻⁸⁴	10 ⁻⁸⁵	10 ⁻⁸⁶	10 ⁻⁸⁷	10 ⁻⁸⁸	10 ⁻⁸⁹	10 ⁻⁹⁰	10 ⁻⁹¹	10 ⁻⁹²	10 ⁻⁹³	10 ⁻⁹⁴	10 ⁻⁹⁵	10 ⁻⁹⁶	10 ⁻⁹⁷	10 ⁻⁹⁸	10 ⁻⁹⁹	10 ⁻¹⁰⁰	10 ⁻¹⁰¹	10 ⁻¹⁰²	10 ⁻¹⁰³	10 ⁻¹⁰⁴	10 ⁻¹⁰⁵	10 ⁻¹⁰⁶	10 ⁻¹⁰⁷	10 ⁻¹⁰⁸	10 ⁻¹⁰⁹	10 ⁻¹¹⁰	10 ⁻¹¹¹	10 ⁻¹¹²	10 ⁻¹¹³	10 ⁻¹¹⁴	10 ⁻¹¹⁵	10 ⁻¹¹⁶	10 ⁻¹¹⁷	10 ⁻¹¹⁸	10 ⁻¹¹⁹	10 ⁻¹²⁰	10 ⁻¹²¹	10 ⁻¹²²	10 ⁻¹²³	10 ⁻¹²⁴	10 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DEPTH SCALE

1 : 50



LOGGED: D.G.

CHECKED: W.C.

MIS-ROCK 001 05-1120-210-2000-ROCK GPJ GLDR CAN GDT 12/21/07 JM

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-7		1 OF 1		METRIC							
W.P. 4058-01-00		LOCATION N 5027104.4; E 364078.1		ORIGINATED BY D.J.S.									
DIST HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.									
DATUM Geodetic		DATE May 18, 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		WATER CONTENT (%)		γ	GR SA SI CL
75.8	GROUND SURFACE							20 40 60 80 100	○ UNCONFINED + FIELD VANE	25 50 75			
0.0	Topsoil (FILL)							● QUICK TRIAXIAL x REMOULDED					
75.5													
0.3	Fine sand (FILL) Loose Brown Moist		1	SS	7		75						
74.5													
74.3	PEAT Moist												
74.1	Silty CLAY and clayey SILT Grey brown Moist		2	SS	19		74						21 22 38 19
1.7	SILT, some gravel, sand and clay (TILL) Compact to very dense Brown and grey brown Moist		3	SS	79	▽							
73.1													
2.7	Limestone (BEDROCK) Slightly weathered Thinly to medium bedded Grey						73						
72.6	Medium strong												
72.4	Limestone with voids (BEDROCK) Highly fractured and weathered, with mud seams		4	NQ RC	DD								
72.1	Thinly to medium bedded Grey						72						
3.7	Medium strong												
	Limestone (BEDROCK) Weathered Thinly to medium bedded Grey												
71.2	Medium strong												
4.6	Limestone with voids (BEDROCK) Highly fractured and weathered, with mud seams		5	NQ RC	DD		71						
	Thinly to medium bedded Grey												
	Medium strong												
	Limestone with occasional thin black shale interbeds (BEDROCK) Fresh Thinly to medium bedded Grey		6	NQ RC	DD		70						
	Medium strong												
	Bedrock cored between 2.7m 7.3m depth. For bedrock coring details refer to Record of Drillhole 06-7.		7	NQ RC	DD		69						
68.5													
7.3	End of Borehole												
	Note: Water level in casing at 2.4m depth below ground surface upon completion of coring.												

MISS MTO 05-1120-210-2000.GPJ ON MOT GDT 12/21/07

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-7

SHEET 1 OF 1

LOCATION: N 5027104.4; E 364078.1

DRILLING DATE: May 18, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH	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					
									SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY		B-BEDDING					
									RECOVERY			FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec							
									TOTAL CORE %	SOLID CORE %	R.Q.D. %		DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION								
		ROCK SURFACE		73.10																		
		Limestone (BEDROCK)		2.70																		
		Slightly weathered																				
		Thinly to medium bedded																				
		Grey		72.60																		
		Medium strong		3.20																		
		Limestone with voids (BEDROCK)		72.40																		
		Highly fractured and weathered, with		3.40	1																	
		mud seams		72.10																		
		Thinly to medium bedded		3.70																		
		Grey																				
		Medium strong																				
		Limestone (BEDROCK)																				
		Weathered																				
		Thinly to medium bedded																				
		Grey																				
		Medium strong		71.20	2																	
		Limestone with voids (BEDROCK)		4.60																		
		Highly fractured and weathered, with																				
		mud seams																				
		Thinly to medium bedded																				
		Grey																				
		Medium strong																				
		Limestone with occasional thin black																				
		shale interbeds (BEDROCK)																				
		Fresh																				
		Thinly to medium bedded																				
		Grey																				
		Medium strong																				
		End of Drillhole		68.50																		
				7.30																		

DEPTH SCALE

1 : 50



LOGGED: D.J.S.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-2000-ROCK GPJ GLDR CAN GDT 12/21/07 JM

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-8		1 OF 1	METRIC
W.P. 4058-01-00		LOCATION N 5027097.9; E 364347.3		ORIGINATED BY D.J.S.	
DIST _____ HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.	
DATUM Geodetic		DATE May 17, 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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
							20 40 60 80 100	20 40 60 80 100						
75.8	GROUND SURFACE													
75.0	Topsoil (FILL)													
0.2	Fine sand (FILL) Loose Brown Moist													
74.6			1	SS	2									
1.3	Sandy silt and silty sand, some gravel and peat (FILL)													
74.1	Loose Dark brown													
1.8	PEAT Moist		2	SS	5									
73.5	SILT, some sand and clay, occasional silty clay seam													
2.3	Loose Grey Moist		3	SS	20									
73.1	Sandy SILT, some gravel, trace clay (TILL)													
2.9	Compact Grey Wet		4	NQ RC	DD									
72.3	Limestone (BEDROCK) Fractured and weathered Thinly to medium bedded													
3.5	Medium strong Limestone, occasional grey green shaley interbed (BEDROCK)		5	NQ RC	DD									
71.3	Fresh Thinly to medium bedded Grey Medium strong													
4.5	Limestone (BEDROCK) Fractured and weathered Thinly to medium bedded Grey Medium strong													
69.9	Bedrock cored between 2.7m 5.9m depth. For bedrock coring details refer to Record of Drillhole 06-8.		6	NQ RC	DD									
5.9	End of Borehole													

Note:
Water level in casing at 1.9m
depth below ground surface
upon completion of coring.

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-8

SHEET 1 OF 1

LOCATION: N 5027097.9; E 364347.3

DRILLING DATE: May 17, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	COLOR % RETURN	FR/FX-FRACTURE F-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				NOTES WATER LEVELS INSTRUMENTATION
									CL-CLEAVAGE				J-JOINT				R-ROUGH				UE-UNEVEN				
									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		DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec			DIAMETRAL CORE LOAD INDEX (MPa)					
									TOTAL CORE %	SOLID CORE %									10 ⁰						10 ¹
		ROCK SURFACE		73.10																					
3	Rotary Drill NO Core	Limestone (BEDROCK)		2.70																					
		Fractured and weathered		72.90																					
		Thinly to medium bedded		2.90																					
4		Medium strong																							
		Limestone, occasional grey green shaley																							
		interbed (BEDROCK)																							
5		Fresh		72.30																					
		Thinly to medium bedded		3.50																					
		Grey																							
6		Medium strong																							
		Limestone (BEDROCK)																							
		Fractured and weathered																							
7		Thinly to medium bedded																							
		Grey																							
		Medium strong																							
8		Limestone (BEDROCK)		71.30																					
		Fractured and weathered		4.50																					
		Thinly to medium bedded																							
9		Grey																							
		Medium strong																							
		Limestone (BEDROCK)																							
10		Fractured and weathered																							
		Thinly to medium bedded																							
		Grey																							
11		Medium strong																							
		Limestone (BEDROCK)																							
		Fractured and weathered																							
12		Thinly to medium bedded																							
		Grey																							
		Medium strong																							
13		Limestone (BEDROCK)																							
		Fractured and weathered																							
		Thinly to medium bedded																							
14		Grey																							
		Medium strong																							
		Limestone (BEDROCK)																							
15		Fractured and weathered																							
		Thinly to medium bedded																							
		Grey																							
16		Medium strong																							
		Limestone (BEDROCK)																							
		Fractured and weathered																							
17		Thinly to medium bedded																							
		Grey																							
		Medium strong																							
18		Limestone (BEDROCK)																							
		Fractured and weathered																							
		Thinly to medium bedded																							
19		Grey																							
		Medium strong																							
		Limestone (BEDROCK)																							
20		Fractured and weathered																							
		Thinly to medium bedded																							
		Grey																							
21		Medium strong																							
		Limestone (BEDROCK)																							
		Fractured and weathered																							
22		Thinly to medium bedded																							
		Grey																							
		Medium strong																							
23		Limestone (BEDROCK)																							
		Fractured and weathered																							
		Thinly to medium bedded																							
24		Grey																							
		Medium strong																							
		Limestone (BEDROCK)																							
25		Fractured and weathered																							
		Thinly to medium bedded																							
		Grey																							
26		Medium strong																							

DEPTH SCALE

1:50



LOGGED: D.J.S.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-2000-ROCK GPJ GLDR CAN GDT 12/21/07 JM

APPENDIX C

Figure 1 Grain Size Distribution Test Result – Silt

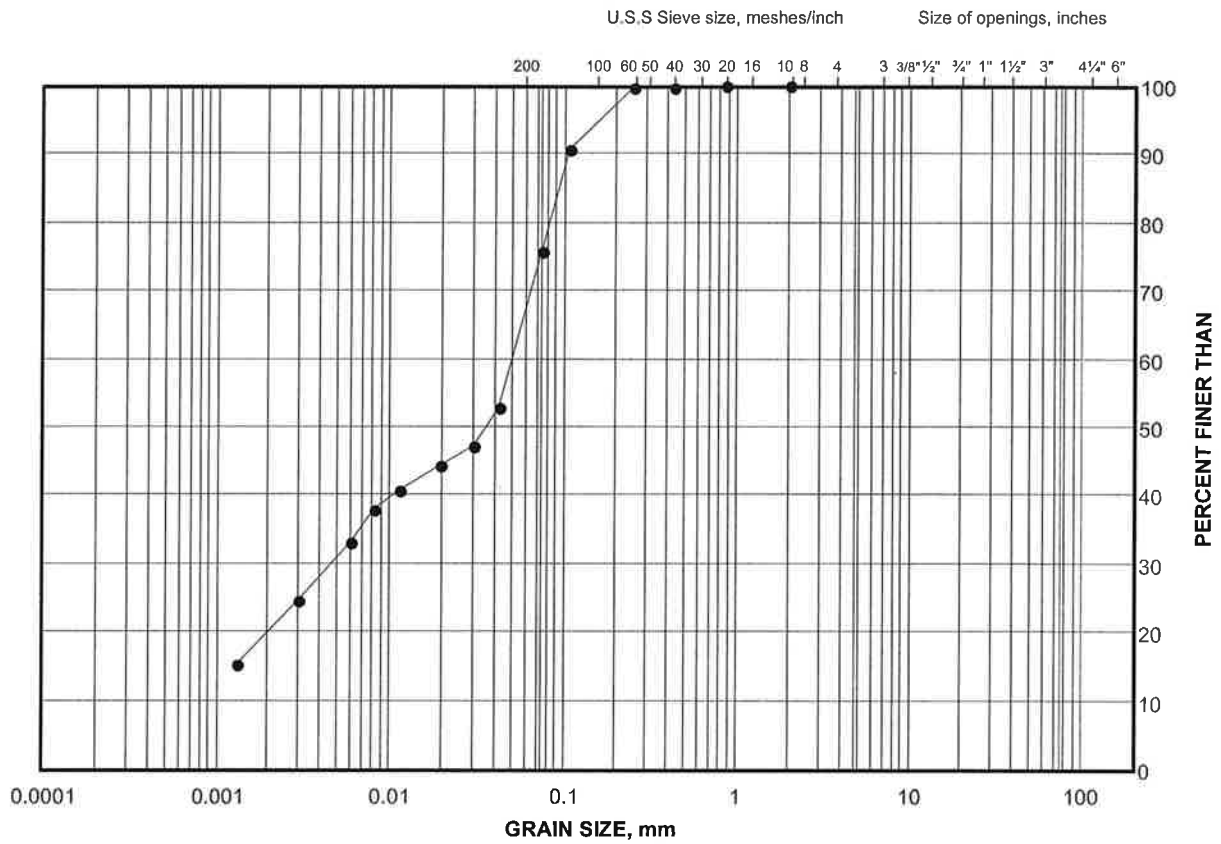
Figure 2 Grain Size Distribution Test Results – Glacial Till

Figure 3 Footing Insulation Detail

GRAIN SIZE DISTRIBUTION

Silt

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)
•	06-8	2	1.8-2.1

Project Number: 05-1120-210

Checked By: _____

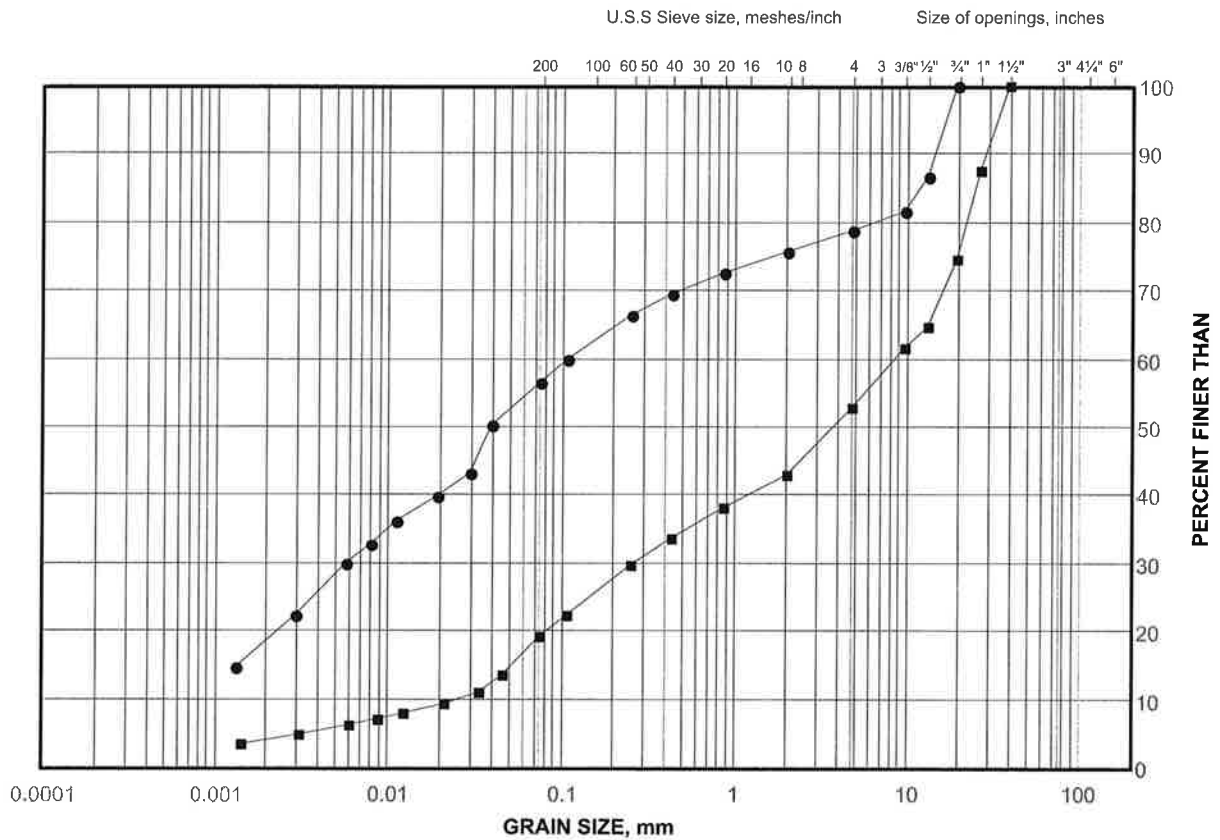
Golder Associates

Date: 22-Jun-06

GRAIN SIZE DISTRIBUTION

Gravel, Sand and Silt, Trace to Some Clay (Till)

FIGURE 2



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

LEGEND

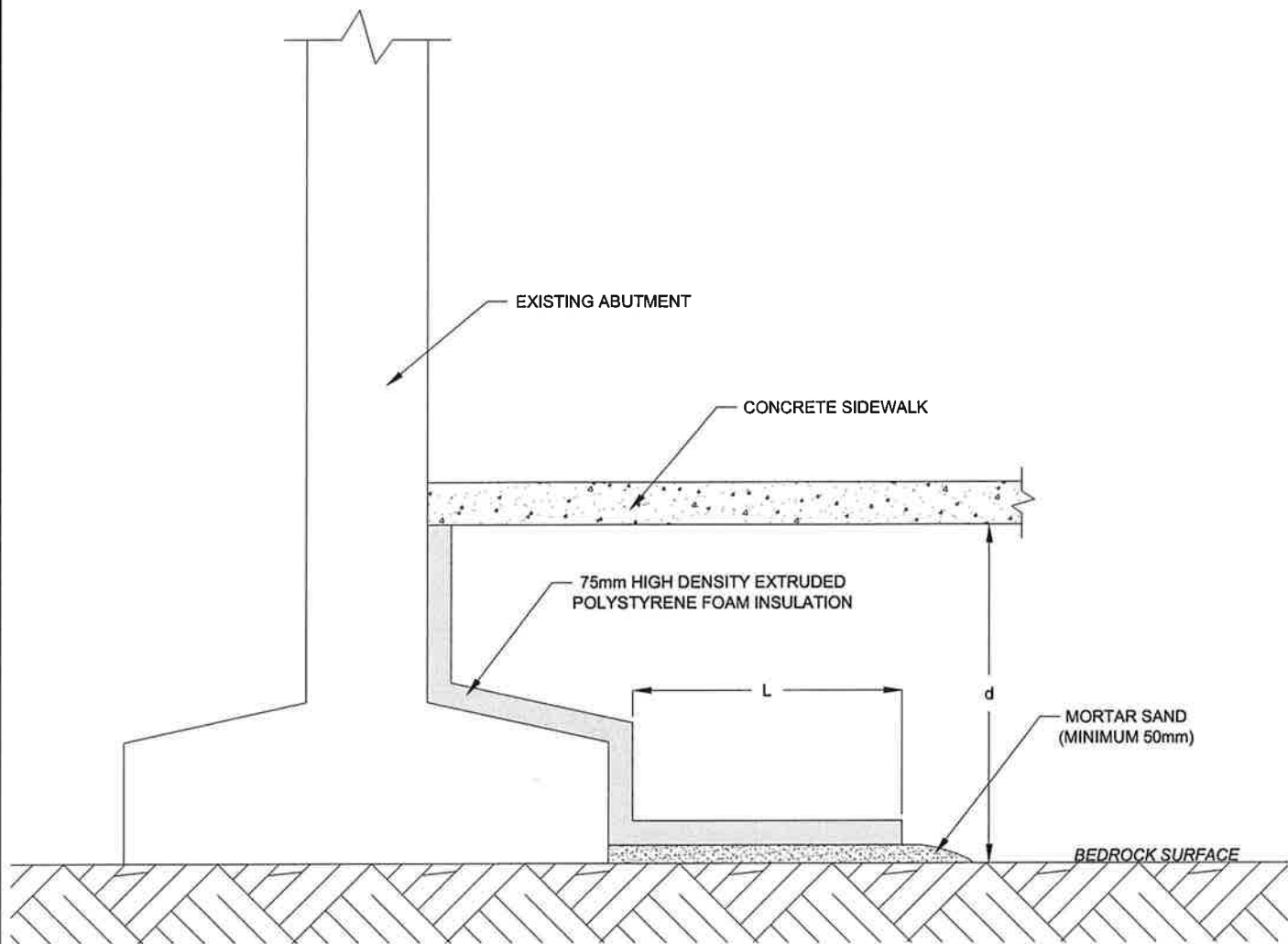
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	06-7	2	1.40 - 2.00
■	06-6	4	1.70 - 2.10

Project Number: 05-1120-210

Checked By: _____

Golder Associates

Date: 24-Oct-06



LEGEND:

- d THICKNESS OF EARTH COVER ABOVE BOTTOM OF FOOTING
- L PROJECTED LENGTH OF INSULATION

NOTES:

- 1) INSULATION JOINTS TO BE GLUED AND/OR LAPPED
- 2) FOR ADEQUATE FROST PROTECTION $d + L \geq 1.8\text{m}$
- 3) INSULATION SHOULD CONSIST OF DOW HIGHLOAD 60 OR EQUIVALENT
- 4) MORTAR SAND SHOULD MEET THE GRADATION LIMITS IN TABLE 1 IN THE CANADIAN STANDARDS ASSOCIATION (CSA) STANDARD A17-04 (CLAUSE 5.3.2.2)

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT



FILE No. 051120210-2000-03.dwg

PROJECT No. 05-1120-210 REV. 0

SCALE	NTS
DATE	05/03/07
DESIGN	W.C.
CADD	J.M.
CHECK	
REVIEW	

TITLE

FOOTING INSULATION DETAIL

CARLING AVENUE EASTBOUND BRIDGE

FIGURE

3

APPENDIX D

Records of Previous Boreholes 1 to 4 (Geocres No. 58-F-221-C)

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QUEENSWAY & CARLING AVE (EASTWARD)
BRIDGE #5

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 246.6' (Geodetic Datum)
REMARKS Ref: QUEENSWAY B.M. # 11-11 (el: 246.08')

HOLE NO. 1

DATE Nov 30/57

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT. ²	SMALL SCALE PENETROMETER KIPS/FT. ²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PENETRATION TEST	
						LB. HAMMERINCH DROP	NO CASINGINCH DIA. ROD
							BLOWS PER FOOT	
				GROUND SURFACE				
				Topsoil	0'	246.6'		
			1-1	Loose Till	1'	245.6'		
			2-1	Medium Dense Till	2.5'	244.1		
					3.8'	242.8'		
				Limestone (drilled) (CORE RECOVERY 95%)				
					8.5'	238.1'		
				Limestone (drilled) (CORE RECOVERY 83%)				
					13.5'	233.1'		
				Bottom of Hole				
							% WATER CONTENT	
							PLATE 2	

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QUEENSWAY & CARLING AVE (EASTWARD)
BRIDGE #5

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 248.8' (Geodetic Datum)
REMARKS Ref: QUEENSWAY B.M. #11-11 (el. 246.08')

HOLE NO.

4

DATE Nov. 30/57

UNCONFINED COMPRESSIVE STRENGTH Kips/Ft. ²	SMALL SCALE PENETROMETER KIPS/FT. ²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PENETRATION TEST			
						LB. HAMMER	NO CASINGINCH DROPINCH DIA. ROD
							BLOWS PER FOOT			
				GROUND SURFACE	0'	248.8'				
			2 for 6"	Fill						
			5 1-4							
			2 for 6"	ORGANIC	2.5'	246.3'				
			4 2-4							
				Boulders in Medium Dense Till	4'	244.8'				
			6 for 6"							
			17 3-4							
				Very Dense Till	7.5'	241.3'				
			51 for 9"							
					8.2'	240.6'				
				Limestone (drilled) (CORE RECOVERY 83%)						
				Limestone (drilled) (CORE RECOVERY 92%)	13.7'	235.1'				
				Limestone (drilled) (CORE RECOVERY 93%)	16.8'	232.0'				
					19'	229.8'				
				Bottom of Hole						
							% WATER CONTENT			
							PLATE 5			

APPENDIX E

Non Standard Special Provisions

Mass Concrete

Dowels into Rock

Rock Excavation Adjacent to Existing Footings

Non-Standard Special Provision

Scope of Work

This Special Provision covers the requirements for rock probes advanced under the abutment and/or retaining wall footings. The purpose of the rock probes is to locate voids within the bedrock under the footings.

Construction

Work under this item shall adhere to the following requirements:

- The surface of the footing founding rock shall be exposed, cleaned and any loose rock removed.
- Probe holes (50 mm diameter drilled holes) are to be advanced at 2 m spacing and to depths of about 2 m as per the contract drawings and documents.
- The Quality Verification Engineer (QVE) shall inspect the probe holes for the presence of voids using a rod probe or downhole camera.
- If the QVE determines that voids exist, grouting, or other remedial measures as directed by the Contract Administrator, shall be undertaken as indicated in the contract drawings and documents.

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.

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Scope of Work

This Special Provision covers the requirements for mass concrete under the abutment and/or retaining wall footings. The purpose of the mass concrete pad is to provide a level working surface at the appropriate elevation for the construction of the abutment and/or retaining wall footings.

Construction

Work under this item shall adhere to the following requirements:

- The surface of the footing founding rock shall be exposed, cleaned and any loose rock removed.
- The mass concrete shall have a minimum 28 day strength of 30 MPa.
- The mass concrete shall be placed on the exposed cleaned founding rock surface as per the contract drawings and documents.
- Thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface and on the underside of footing elevation. A nominal thickness and a footprint plan view area has been specified on the contract drawings and documents.
- Unwatering of the excavation for the footing construction, including the construction of the mass concrete pad, might be required and is covered under a separate Tender Item. The dewatering scheme shall be done in such a manner as to prevent any disturbance to the surrounding original soil.

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\insspsp-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.

Rock Excavation Adjacent to Existing Footings – Item No.

Non-Standard Special Provision

Scope of Work

This Special Provision covers the requirements for sub-excavation immediately adjacent to existing footings.

Construction

Work under this item shall adhere to the following requirements:

- Excavations immediately adjacent to the existing overpass foundations that extend *less than 0.5 m* below the underside of existing foundation elevation may be provided with a vertical face.
- Excavations immediately adjacent to the existing overpass foundations that extend *more than 0.5 m* below the underside of the existing foundation shall be stepped down at no less than 1H:1V.
- The above excavation guidelines are applicable provided that the bedrock below the existing foundation is not disturbed; good construction practices, such as closely spaced pre-drilling at the existing foundation face, will be required.
- The bedrock adjacent to the existing foundation shall be inspected by the Quality Verification Engineer (QVE) prior to excavation.
- If the QVE determines that the bedrock is sufficiently fractured or weathered to indicate a risk of disturbing the bedrock underlying the existing footing during excavation, the contractor shall adjust his excavation methods, limits or slopes as directed by the Contract Administrator.

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.

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