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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN REPORT
CARLING AVENUE WESTBOUND OVERPASS
BRIDGE WIDENING
STRUCTURE SITE 3-46
HIGHWAY 417
W.P. 4058-01-00**

Submitted to:

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PART A

**FOUNDATION INVESTIGATION REPORT
CARLING AVENUE WESTBOUND OVERPASS BRIDGE WIDENING
STRUCTURE SITE 3-46
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 WB including the bridge retaining walls and approach embankment widening. A separate report addresses the retaining walls located outside of the bridge approaches.

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

2.0 SITE DESCRIPTION

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

Carling Avenue WB is a three lane one-way road with an urban cross-section, a sidewalk on the east side and a gravel boulevard adjacent to the west abutment. The surrounding land on either side of Highway 417 is relatively flat and level.

The existing bridge has concrete abutments supported on piles 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 of Highway 417) with the abutments separated by a 25 mm joint.

It is understood that the abutment stem walls are in fair condition with spalls and delaminations covering less than 10% of the exposed face. From a foundation perspective, the bridge is understood to be performing adequately.

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

A previous investigation was conducted for the design of the existing bridge by McRostie & Associates for MTO in 1958. The results of that investigation are contained in the report titled "Report on Foundation Investigation at Ottawa Queensway and Carling Ave. Westbound, Structure No. 7 to Deleuw, Cather and Company of Canada Limited" (Geocres No. 58-F-222-C).

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out on May 11, 15, and 16, 2006. On those days, four boreholes (Boreholes 06-13 to 06-16, 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, where possible. Borehole 06-16, at the north-west widening, was advanced in the travelled lane of Carling Avenue WB, about 4 metres from the widening end, due to the presence of underground utilities near the end of the proposed abutment widening. The boreholes were advanced using track and truck mounted drill rigs supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. The boreholes were advanced to depths which vary from 14.1 m to 14.6 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 2.6 to 3.8 metres, after practical refusal to augering had been reached. The water levels in the open boreholes were observed throughout the drilling operations, and one standpipe was installed to monitor the groundwater level at the site. The standpipe was installed in Borehole 06-14 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 Limit testing and grain size distribution analyses were carried out on selected soil samples.

The groundwater level was measured in the standpipe in Borehole 06-14 on June 12, 2006 about one month after completion of drilling.

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

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-13	South-east abutment	5027555.1	364488.3	75.1
06-14	South-west abutment	5027540.4	364478.1	75.2
06-15	North-east abutment	5027562.8	364435.8	75.9
06-16	North-west abutment	5027549.4	364418.3	75.3

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 or near the limits of the foundation elements for the proposed widening of the Carling Avenue WB bridge. The borehole locations and ground surface elevations are shown on Drawing 1 in Appendix A. Soil stratigraphy sections projected along the highway centreline and across the abutment foundation areas are shown on Drawing 2 in Appendix A.

The detailed subsurface soil, bedrock, and groundwater conditions encountered in the boreholes and the results of the in-situ and laboratory testing are given on the Record of Borehole sheets in Appendix B and on Figures 1 to 5 in Appendix C. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Four boreholes had been previously advanced at the present bridge abutment locations on behalf of the Ministry in 1958, as previously noted, (Geocres No. 58-F-222-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 sand to silty clay fill materials to depths of about 1.5 m to 1.9 m, underlain

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.

by silty clay to clay, over glacial till and sands extending to depths of about 10.3 m to 12.0 m. These overburden materials are underlain by limestone bedrock.

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

4.2.1 Pavement Structure

The pavement structure of Carling Avenue at Borehole 06-16 consists of about 80 millimetres of asphaltic concrete underlain by about 220 millimetres of concrete. The asphaltic concrete and concrete are in turn underlain by a layer of crushed stone base course material which is about 210 millimetres thick.

4.2.2 Topsoil and Peat

Topsoil (fill) exists at the ground surface at Boreholes 06-13, 06-14 and 06-15 and is about 0.2 m thick.

The fill materials at Borehole 06-16 (discussed below) are underlain by a layer of peat which is about 0.2 m thick.

4.2.3 Fill

Fill material associated with previous uses of the site and roadway construction underlies the pavement structure and topsoil at all the boreholes.

The fill materials extend to depths ranging from about 1.5 m to 1.9 m below ground surface (i.e., Elevations 74.3 m to 73.4 m). The fill materials range in composition from silty clay to sand with varying amounts of clay, silt, sand, gravel, organic matter and cinders. Grain size distribution test results obtained from one sample of the fill materials at Borehole 06-13 are shown on Figure 1.

4.2.4 Clay

The peat at Borehole 06-16 and the fill materials at the remainder of the boreholes are underlain by a deposit of clay, which ranges in thickness from about 3.4 to 4.8 m.

Weathered Clay Crust

The upper portion of the clay deposit at Boreholes 06-13 to 06-15 has been weathered to a grey-brown colour. This weathered zone extends to depths ranging from about 2.1 to 2.9 m (i.e., Elevations 73.1 m to 73.0 m). Measured SPT “N” values in this deposit ranged from 2 to 4 blows per 0.3 m of penetration. In situ vane testing carried out within the weathered clay measured undrained shear strengths of 92 and greater than 96 kPa. These test results indicate that the weathered clay has a stiff to very stiff consistency.

The results of Atterberg limit testing on one selected sample of the weathered clay indicate a plasticity index of 54 percent and a liquid limit of 84 percent. These results, summarized on the plasticity chart on Figure 2, confirm that this material is a clay of high plasticity. The measured natural water content of one sample of the weathered clay was 55 percent.

Unweathered Clay

The clay below the depth of weathering at Boreholes 06-13 to 06-15 and the full depth of the deposit at Borehole 06-16 is grey in colour and extends to depths ranging from about 4.9 to 6.4 m (i.e., Elevations 70.2 m to 68.8 m). In situ vane testing carried out within this unweathered deposit measured undrained shear strengths ranging from 42 to 80 kPa. These results indicate that the unweathered clay has a firm to stiff consistency. The results of grain size distribution testing carried out on one sample of the unweathered clay are shown on Figure 3.

The results of Atterberg limit testing on three selected samples of the unweathered clay indicate plasticity indices ranging from 32 to 57 percent and liquid limits ranging from 54 to 85 percent. These results, summarized on the plasticity chart on Figure 4, confirm that this material is a clay of high plasticity. The measured natural water contents of the unweathered clay ranged from 57 to 83 percent, which were generally at or in excess of the measured liquid limits.

A thin deposit of layered silty clay and clayey silt with sand and gravel layers underlies the clay deposit at Borehole 06-15.

4.2.5 Sand and Sand and Gravel

Sand to sand and gravel deposits were encountered overlying the glacial till at Borehole 06-15 and underlying the till at Boreholes 06-14 and 06-15.

The silty sand and gravel overlying the till at Borehole 06-15 is about 0.3 metres thick and a measured SPT “N” value of 2 blows per 0.3 metres of penetration indicates a very loose state of packing.

The sands and gravels underlying the till at Boreholes 06-14 and 06-15 are about 0.7 and 1.2 metres thick, respectively. Measured “N” values in these deposits ranging from 17 to 55 blows per 0.3 m of penetration indicate compact to very dense states of packing.

4.2.6 Silty Sand to Sandy Silt Till

Glacial till was encountered below the silty sand and gravel at Borehole 06-15 and beneath the clay deposit in the remainder of the boreholes. The surface of this till deposit was encountered between about elevations 68.6 to 70.2 m (at depths below ground surface ranging from about 4.9 to 7.3 m). The glacial till deposit ranges in thickness from about 1.8 to 6.1 m.

Based on the samples retrieved as well as local experience and observations of the drilling resistance, the glacial till is considered to consist of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand, sand and sandy silt, with a trace of clay. Grain size distribution test results obtained for four samples of the glacial till are shown on Figure 5. 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 ranging from 6 to 120 blows per 0.3 m of penetration indicate the deposit to have a loose to very dense relative density. However the higher N values may reflect the presence of cobbles and boulders rather than the state of packing of the soil matrix. The measured natural water contents of three selected samples of the glacial till ranged from 7 to 9 percent.

4.2.7 Limestone Bedrock

Limestone bedrock was encountered at all of the boreholes.

The following table summarizes the bedrock surface depths and elevations as encountered at the locations of Boreholes 06-13 to 06-16, 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	06-13	75.1	11.0	64.1
	3	74.7	9.9	64.8
	2	75.5	10.7	64.8
	06-15	75.9	10.3	65.6
West Abutment	06-14	75.2	11.4	63.8
	1	74.7	9.8	64.9
	4	75.4	11.0	64.4
	06-16	75.3	12.0	63.3

The limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly- to medium-bedded. Rock Quality Designation (RQD) values measured on recovered bedrock core samples were quite variable and ranged from about 0 to 97 percent.

Low RQD values, ranging from 0 to 40 percent indicating very poor to poor quality rock, were recorded over almost the full cored depth in Boreholes 06-14 and 06-15 and in the upper 0.7 metres of bedrock in Borehole 06-13. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted in the core from Borehole 06-15. Mud seams were also encountered within the bedrock at Borehole 06-15.

RQD values ranging from 50 to 97 percent were recorded in the remainder of the bedrock cored in these boreholes and for the full core in Borehole 06-16, indicating fair to excellent quality rock.

A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.3 Groundwater Conditions

A piezometer was installed in Borehole 06-14, sealed within the bedrock. The water level measured in that piezometer is summarized in the following table:

Borehole No.	Borehole Location	Date	Depth (m)	Elevation (m)
06-14	West abutment	June 12, 2006	3.3	71.9

The water level in Borehole 06-15 was measured at 3.8 m depth during the short time the borehole remained open following the overburden drilling and prior to commencing the bedrock coring operations.

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

5.0 CLOSURE

The investigation was carried out using equipment supplied and operated by Marathon Drilling. The field portions were supervised by Mr. 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.

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WC/MIC/FJH/ch

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PART B

**FOUNDATION DESIGN REPORT
CARLING AVENUE WESTBOUND OVERPASS BRIDGE WIDENING
STRUCTURE SITE 3-46
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 WB 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 WB overpass bridge is a single span structure supported on piles founded on bedrock. The proposed work is to include complete replacement of the existing bridge deck and girders. Rapid replacement techniques will likely be used for this work. The existing abutments will also be extended to the north and south by about 5 metres and will be converted to semi-integral type abutments. In addition, the roadway profile of Carling Avenue WB will be lowered by about 200 mm to maintain the existing vertical clearance between Carling Avenue WB and the widened overpass structure.

Bridge retaining walls extending 13 to 14 metres back from the abutments have been proposed for the bridge approaches. It is anticipated that the height of additional fill placed on the existing side slopes will be up to 4 metres. Embankments with 2H:1V side slopes extending beyond and parallel to the ends of the wing walls are also understood to be proposed.

6.2 Bridge Foundation Options

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

- Shallow foundations supported on the native clay soils.
- Foundations supported on steel H-piles founded on, or socketed into, the bedrock.
- Foundations supported on caissons founded on, or socketed into, the bedrock.

Spread footings supported on the underlying glacial till, or on engineered fill supported on the glacial till, has not been considered as a feasible or practical option due to the 5 to 6 metre deep excavation that would be required.

The first option, using shallow foundations supported on the native clay soils, is not considered practical or appropriate for this site since the bearing resistance of these highly plastic soils of limited strength would be insufficient for support of the abutment loads and the settlement of the foundations would be excessive. For shallow foundations supported on the silty clay subgrade, the foundation settlement would be entirely differential with respect to the existing pile supported abutments.

It is considered that the most feasible and cost-effective options for the widened bridge abutments are foundations supported on piles, founded on the bedrock. This option is consistent with the existing bridge foundation construction.

It is understood that semi-integral abutments are under consideration for this bridge widening. From a foundation perspective, fully integral abutments are not considered to be feasible at this location since the widening and existing structure must be consistent and it would be impractical to refit the foundations of the existing structures to allow sufficient movement for integral abutments.

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

6.2.1 Steel H-Pile Foundations

Steel H-piles driven to found on the limestone bedrock may be used for support of the bridge abutments. Based on the existing grade at about Elevation 75 m and the requirement for a minimum 1.8 m of frost cover, it is assumed that the pile cap base will be at about Elevation 73 m. The bedrock surface is at about Elevation 63 to 66 m and the pile length will therefore be about 7 to 10 m.

The underside of the existing pile cap is at about Elevation 73.5 m and therefore the underside of the pile caps for the widenings could be about 0.5 m lower than existing, based on the assumed pile cap elevation above. This difference in pile cap elevation, between the existing and widened abutments, should be acceptable since the existing foundation loads are fully supported by the piles.

It could be necessary to socket the piles into the bedrock to resist lateral or seismic forces. The limestone bedrock is generally medium strong and this would require socket formation using coring or churn drilling to advance the hole.

6.2.1.1 Axial Geotechnical Resistance

For construction of the new widenings, the following factored axial resistances at Ultimate Limit States (ULS) may be assumed for design of piles driven to found on the bedrock, or socketted at least 2 m into the bedrock:

Pile Size	Factored ULS Resistance (kPa)
HP 310 x 110	2,000
HP 360 x 132	2,400
HP 360 x 152	2,750

The above values represent structural limitations for the piles rather than geotechnical limitations. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

For HP 310 x 110 piles driven to found on the bedrock surface at this location, the unfactored ULS uplift resistance would be 300 kN; in accordance with the CHBDC, a resistance factor of 0.3 is to be applied.

Consideration must be given to the presence of cobbles and boulders within the glacial till and sand which underlies the silty clay and existing fill materials at this site. In this regard, vertically driven piles should be equipped with Type I flange reinforcement as per OPSD 3000.100. Any battered piles should be equipped with suitable driving points (such as Titus standard rock bearing point or equivalent) to ensure adequate seating of the piles on the bedrock.

Pile installation should be in accordance with SP903S01. For this site, the piles will essentially be driven to practical refusal on the bedrock. The drawings should incorporate the appropriate note stating that the piles should be equipped with flange reinforcement and/or rock points and should be driven to bedrock. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles are not overdriven and to avoid possible damage to the piles. In this regard, for piles driven to refusal on bedrock, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A Non Standard Special Provision for vibration monitoring should be included in the contract documents and has been included in Appendix E of this report.

A maximum peak particle velocity of 100 mm/s is recommended at the existing abutments and bridge retaining walls. The piles further from the existing structure should be driven first, in order to check the vibration level at the existing structures and, if necessary, alter the pile driving criteria for the remaining piles.

The piles for the widened abutments may be driven in close proximity to the battered piles supporting the existing abutments and wing walls. These existing piles may be offset from their intended location or alignment and the potential exists for conflicts when driving the new piles. Current construction practice generally limits the acceptable pile offset *at the surface* to 50 mm and the deviation from the design inclination to 2%. However, even for piles installed meeting these construction limits, the tip offset *at depth* may be greater and it is considered that, for piles less than 10 m in length such as at this site, the tip offset *at depth* may be as much as 10% of the pile length.

The spacing between the new and existing piles may be less than this tolerance and therefore the potential exists for interference during driving of the new piles. For new piles driven within the potential zone of interference with the existing abutment or wing wall piles (defined as a distance around the existing pile centre equal to 20% of the pile length) the driving operations shall be continuously monitored by the QVE and the contractor shall cease driving of the pile if the QVE indicates that the driven pile may have come in contact with an existing pile. It may be necessary to extract and re-drive piles if contact between the new and existing piles is believed to exist. A Non Standard Special Provision for driving piles adjacent to existing battered piles should be included in the contract documents and has been included in Appendix E of this report.

The construction of the approach embankments will raise the effective stress level in the grey unweathered clay which, based on the results of oedometer testing elsewhere on this project (see Appendix F), would lead to some compression of the deposit. As discussed subsequently in Section 6.6.3 of this report, the embankment subgrade settlements are estimated at up to about 150 mm (from both primary consolidation and secondary compression). The elastic shortening of the piles will be significantly less, likely less than 5 mm under service loads, and therefore the differential settlements would be sufficient to generate downdrag forces.

In calculating the magnitude of the downdrag force, the methods described in both the Canadian Foundation Engineering Manual as well as the US Transportation Research Board's report, "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" [Briaud and Tucker (1994)] were considered. Considering the larger predicted settlement of the silty clay deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the underside of the silty clay deposit.

Based on the above, the unfactored downdrag load acting on a single HP 310 x 110 pile over the length of pile within the native soils is estimated to be 150 kN. The structural capacity of the piles

must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the CHBDC.

It is not known whether downdrag loads were considered for the design of the existing piles. However, ground movements will occur at the existing piles closest to the widening and will likely be large enough in magnitude to generate downdrag forces on those piles.

It is understood that the piles supporting the existing bridge are equivalent in size to HP 360 x 108 and that the front row of these piles is likely battered at 4V:1H. The front row of piles at the existing wing walls is understood to be battered at 6V:1H. The CHBDC indicates that for structural steel where plans and mill certificates are not available, and coupons have not been taken for testing (such as at this bridge), the strength of the structural steel shall be assumed as indicated in Table 14.6.3.1 of the CHBDC. The CHBDC indicates that for a structure such as this bridge built between 1933 and 1975, the yield strength of the structural steel shall be taken as 230 MPa. Based on this assumed yield strength and assuming that the piles have been driven to found upon the bedrock, the Ultimate Limit States resistance of the existing piles would be 1,200 kN. This value represents a structural limitation for the piles rather than a geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions govern for this foundation type.

6.2.1.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles.

Alternatively, the resistance to lateral loading will have to be derived from the soil in front of the piles and it may be assumed that this resistance will be nearly the same for vertical and inclined piles as indicated in Section C6.8.7.2 of the Commentary to the CHBDC.

The resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , is based on the equations given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition).

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where}$$

n_h is the constant of horizontal subgrade reaction, as given below
 z is the depth (m)
 B is the pile diameter/width (m)

For cohesive soils:

$$k_h = \frac{67s_u}{B} \quad \text{where} \quad \begin{array}{l} k_h \text{ is the coefficient of horizontal subgrade reaction;} \\ s_u \text{ is the undrained shear strength of the soil (kPa); and} \\ B \text{ is the pile diameter/width (m)} \end{array}$$

The following ranges for the values of n_h and s_u may be assumed in the structural analysis. The range in values reflects the variability in the subsurface conditions.

<i>Soil Unit</i>	<i>n_h (MPa)</i>	<i>s_u (kPa)</i>
Unweathered clay below about Elev. 73 m:		
South-east Widening: Above elevation 70.2 m.	—	54
South-west Widening: Above elevation 68.8 m.	—	42
North-east Widening: Above elevation 68.9 m.	—	42
North-west Widening: Above elevation 69.2 m.	—	50
Sands and glacial till below about Elevation 70 to 69 m:		
South-east Widening: Approximately 6.1 m thick between Elev. 70.2 m and bedrock surface at about Elev. 64.1 m	5	
South-west Widening: Approximately 5 m thick between Elev. 68.8 m and bedrock surface at about Elev. 63.8 m	8	
North-east Widening: Approximately 3.3 m thick between Elev. 68.9 m and bedrock surface at about Elev. 65.6 m	5	
North-west Widening: Approximately 5.9 m thick between Elev. 69.2 m and bedrock surface at about Elev. 63.3 m	8	

Note: Underside of pile cap level assumed to be at about Elevation 73 metres.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action should be considered for new and existing piles provided that the widened structure allows the new and existing piles to function as a unit. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

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

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

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.8.7 of the *Commentary to the CHBDC*. For individual piles in cohesive soils (i.e., silty clay and clay) the lateral resistance is assumed to vary linearly with a magnitude of $2S_u$ at the surface of the deposit and to a magnitude of $9S_u$ at a depth equal to three pile diameters below the underside of the pile cap where S_u is the undrained shear strength. Below a depth equal to 3 pile diameters, the lateral resistance is assumed to be constant at $9S_u$.

The lateral resistance from the sands and till should be neglected since, in these non-cohesive soils, the CHBDC Commentary (Section C6.8.7.1) suggests that resistance only be considered within a depth of six pile diameters below pile cap level; these soils are below that depth.

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

The S_u values for the unweathered silty clay at each widening location given in the above table may be assumed for estimating the ULS geotechnical lateral resistances.

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

A maximum lateral resistance of 120 kN at ULS, and a maximum lateral resistance of 35 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310 x 110 piles. These values are based on the "Assessed Horizontal Passive Resistance Values for Various Pile Types" provided in Table C6.8.7.1(a) of the *Commentary to the CHBDC*.

Additional lateral resistance can be provided by socketing the piles into the bedrock. For piles socketed at least 1 m into bedrock, the ultimate (unfactored) bearing resistance of the limestone may be taken as the lesser of 30 MPa or the compressive strength of the Portland cement grout or concrete placed in the bedrock socket.

6.2.1.3 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

6.2.2 Caisson Foundations

Caissons founded on or socketed into the limestone bedrock may be used for support of the bridge abutments. Based on the existing grade at about Elevation 75 m and the requirement for a minimum 1.8 m of frost cover, it is assumed that the pile cap base will be at about Elevation 73 m. The bedrock surface is at about Elevation 63 to 66 m and the caisson length will therefore be about 7 to 10 m.

The native marine (Champlain Sea) clay at this site is a sensitive soil. The disturbed clay could “flow” into the auger hole during caisson installation if left unsupported. The use of a liner or casing will be required in order to advance the caissons with minimal loss of ground. Additionally, it will be difficult to clean the bedrock surface, even with the use of liners, unless the liner is socketed into the bedrock; once disturbed, the sensitive clay soils, as well as the sandy till, could flow under the casings, at the interface with the bedrock. It may therefore be more practical to socket the caissons into the rock, rather than found on the bedrock surface.

The limestone bedrock at the site is moderately strong. If socketing of the caissons into the bedrock is required, the sockets will have to be advanced by rock coring or churn drilling.

6.2.2.1 Axial Geotechnical Resistance

Caissons founded on the surface of the limestone bedrock, or socketed nominally (less than 1 m) into the bedrock, should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 4 MPa should be used. Serviceability Limit States resistances do not apply to caissons founded on or socketed in the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Construction of the approach embankments will raise the effective stress level in the grey silty clay deposit at depth close to its estimated preconsolidation pressure. That stress increase will lead to some consolidation of the deposit and will result in downdrag forces on caissons supporting the abutments and wing walls. The unfactored downdrag load acting on a single 0.9 m or 1.5 m diameter caisson over its length is estimated to be 400 or 600 kN, respectively. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*. The assumptions and methods used in assessing that downdrag force are the same as those described in Section 6.2.1 of this report with respect to steel H-piles.

6.2.2.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 6.2.1.2.

6.2.2.3 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

6.3 Bridge Retaining Wall Foundation Options

Bridge retaining walls are planned to be constructed adjacent to all four of the abutment widenings extending 13 to 16 metres back from the abutments. The following options have been considered for the wing (retaining) wall foundations:

- Shallow foundations supported on the native silty clay soils.
- Foundations supported on steel H-piles founded on, or socketed into, the bedrock.
- Foundations supported on caissons founded on, or socketed into, the bedrock.

Spread footings supported on the underlying glacial till, or on engineered fill supported on the glacial till, has not been considered as a feasible or practical option due to the 5 to 6 metre deep excavation that would be required.

The first option, using shallow foundations supported on the native soils, is not considered practical or appropriate for this site since the bearing resistance provided by these highly plastic soils would be low and the settlement of the foundations would be excessive. As discussed further in Section 6.6.3, embankment subgrade settlements of up to 150 mm may occur beneath the widenings, and therefore for shallow foundations as well. This settlement would be entirely differential with respect to the pile or caisson supported abutments. Even with articulated joints between the retaining walls and abutments, this level of differential settlement is considered excessive.

It is considered that the most feasible and cost-effective options for the bridge retaining walls are foundations supported on piles or caissons, founded on or socketed into the bedrock. These options are also consistent with the bridge abutment foundation construction.

The geotechnical recommendations for the design of foundations for the abutments as described in Sections 6.2.1 and 6.2.2 are equally applicable to the design of the wing wall foundations.

A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the bridge retaining wall foundation options is presented in Table 2 following the text of this report.

6.4 Site Coefficient & Seismic Liquefaction

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

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

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

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

The following conditions are more prone to experiencing seismic liquefaction:

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

The assessment of the potential seismic liquefaction hazard at this site involves comparing the cyclic shear stresses applied to the soil by the design earthquake (represented by the cyclic stress ratio, CSR) to the cyclic shear strength offered by the soil (represented as the cyclic resistance ratio, CRR). The CSR is primarily a function of the effective overburden pressure, the design ground acceleration, and the earthquake magnitude and ground acceleration specific to the site. The CRR is primarily related to the relative density of the soil and its gradation.

The silty clay and glacial till soils at this site are too fine-grained to be potentially liquefiable. Portions of the fill material are coarser in gradation, however the fill materials are located above the groundwater level and are therefore also not liquefiable. Only the sand deposits encountered in Boreholes 06-14 and 06-15 are sufficiently coarse grained and below the groundwater level to therefore require further evaluation. However, with SPT 'N' values ranging from 17 to 55, these materials are compact to dense and, considering the seismicity of the Ottawa area, are also not considered to be liquefiable.

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 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 Special Provision SP105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with MTO's Special Provision 105S10. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the walls (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_0	0.43	0.43

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- Seismic loading will result in increased lateral earth pressures acting on the abutment stem. The stem 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, up to 50% amplification could be expected for the ground conditions at this site, resulting in an increase in the design ground surface acceleration to 0.3. The seismic lateral earth pressure coefficients given below have therefore been derived based on a design zonal acceleration ratio of $A = 0.3$.
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary*, for structures which do not allow lateral yielding the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.45$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.15$).

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	0.42	0.34	0.34
Non-yielding wall	0.86	0.68	0.68

- The above K_{AE} values for yielding walls are applicable provided that the calculated wall displacement is more than 250A (mm), where A is the design zonal acceleration ratio of 0.30. This corresponds to displacements of up to approximately 75 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/m^3),
as given previously;
d is the depth below the top of the wall (m); and
H is the total height of the wall (m).

6.6 Approach Embankment Design and Construction

Embankment widening beyond the ends of the four retaining walls will, as currently proposed, be accomplished using conventional 2H:1V embankment side slopes.

Based on the borehole results, the embankment widening subgrade soils will consist of fill materials, with a buried peat layer at the north-west widening at Borehole 06-16. The existing fill materials range in composition from silty clay to sandy silt to sand. These materials are underlain by silty clay to clay, which are in turn underlain by loose to very dense sand and silty sand to sandy silt till.

6.6.1 Subgrade Preparation and Approach 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.

The existing fill material within the footprint of the widening can generally be left in place beneath the embankment widening, provided some modest settlement (i.e., less than 15 mm) of the subgrade can be tolerated. However the subgrade surface should be proof rolled and compacted to 95 percent of the standard Proctor maximum dry density. This guideline is appropriate where earth filling will be used to construct the embankment 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 in accordance with MTO Special Provision 105S10. 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 Approach Embankment and Bridge Retaining Wall Stability

Bridge retaining walls up to 6 m in height will have a factor of safety of greater than 1.3 against deep-seated global instability. Pseudo-static seismic stability analyses also indicate that the retaining walls will have factors of safety greater than 1.1 against deep-seated global instability, based on an acceleration of 0.1g.

With appropriate subgrade preparation and proper placement of earth or granular soils, the 5 to 6 m high approach embankments, beyond the ends of the wingwalls, with side slopes maintained at 2 horizontal to 1 vertical, founded on the existing fill materials and native soils, will have a factor of safety greater than 1.3 against deep seated slope instability.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the 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 wingwall and slope stability analyses were carried out using the following parameters:

Material	Bulk Unit Weight (kN/m³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Earth or Granular Embankment Fill	21	32°	
Existing Sand Fill	19	29°	
Existing Silty Clay Fill	17.5		15 - 20
Peat	19		25
Weathered Silty Clay	17.5		80
Silty Clay	16.5		42
Till	22	32°	

6.6.3 Approach Embankment Settlement

Settlement of the approach embankments will occur as a result of compression of the new embankment fill itself, compression of the existing fill material, as well as consolidation of the clayey soils on which the approaches will be founded.

Provided that the new embankment fill material consists of granular fill, Select Subgrade Material or clean earth fill, the settlement of the embankment fill itself is expected to be less than about 25 mm. The use of granular fill for the new embankment construction would reduce the magnitude of post-construction settlement (likely to less than 10 mm), since the majority of settlement of granular fills will occur during construction.

The embankment fill materials will be underlain by about 1.5 to 1.9 m of existing fill materials. The subgrade settlement due to compression of the existing fill materials should be minor in magnitude (i.e., less than 25 mm) provided the subgrade surface is proof-rolled. Most of this settlement would also likely occur entirely during construction.

Some settlement of the embankment subgrade can be expected due to compression of the clay soils (i.e., the weathered clay crust and, in particular, the underlying grey silty clay to clay). The effective stress level in the clayey deposits will likely approach and potentially exceed the deposit's preconsolidation pressure, at least near the bottom of the deposit. The resulting consolidation settlements therefore correspond to recompression of the clayey deposits and some potential consolidation in the virgin compression range.

The total estimated magnitude of the primary consolidation settlement ranges from about 40 to 50 mm. It is estimated however that most of the primary consolidation settlement should be completed within about three to six months.

More significantly however, up to approximately 100 mm of additional secondary compression at all the widenings is anticipated over a twenty-year time span, by which time resurfacing of the highway might be expected. The relatively higher magnitude of secondary compression versus primary consolidation is typical of Champlain Sea clay when loaded to about its preconsolidation pressure, but not greatly in excess of it.

The secondary compression index was estimated using Mesri's correlation with the primary compression index. The primary compression index at the anticipated stress level was inferred from the oedometer consolidation test results and the secondary compression index was estimated from that value.

The secondary compression settlement magnitudes were then estimated to be about 100 mm. This estimated secondary settlement magnitude is about 2 to 3% of the clay layer thickness which, based on local experience, is consistent with long term settlements of Champlain Sea clay subjected to loads modestly in excess of the clay's preconsolidation pressure.

Based on the above, the total magnitude of the settlements due to primary and secondary compression of the underlying clayey deposits is anticipated to be up to about 150 mm.

Based on the estimated settlements, it is recommended that the approach embankments be constructed as early as possible in the contract to allow the maximum amount of time available for settlement prior to paving of the highway. However, it would still be necessary to pad and overlay the widened approach embankments for the bridge in the years following paving. If this maintenance is considered unacceptable, the following options could be considered for mitigation of post-construction settlement:

- Excavate the silty clay and replace with engineered fill.
- Preload the widened embankment and allow the settlements to occur prior to paving.
- Surcharge the widened embankment to increase the magnitude of settlement during the preload period, prior to paving.
- Install wick drains to accelerate the consolidation settlement within the silty clay to clay.
- Employ lightweight fill in the construction of the approach embankments to reduce the magnitude of primary consolidation settlement.
- Provide retaining wall foundations of sufficient width to support the entire weight of additional fill material.

Excavating the silty clay and replacing it with engineered fill is not considered to be feasible at this site due to the required excavation depth. Additionally, the excavation would need to extend across the full width of the existing side slope and impractically deep roadway protection would be required.

Preloading without a surcharge is also not considered to be feasible at this site. Preloading without a surcharge would reduce the magnitude of primary consolidation settlements but would not reduce the considerably higher long term settlement magnitudes due to secondary compression.

Preliminary analyses indicate that a 1.5 m high surcharge would result in approximately 100 mm of primary consolidation settlement within about 3 to 6 months. This would limit the post-paving settlement of the highway to approximately 50 mm.

Wick drains could be used to accelerate the settlements and reduce the preload time but it may be difficult to install wick drains along the widened embankment through the existing side slope. The wick drains should extend slightly past the toe of the widened slope and it would likely be necessary to build the widened slope in a series of benches to provide a working platform over the full width of the widened embankment for the installation equipment. The wick drains would then be installed through the existing and new embankment fills resulting in increased lengths and cost. If difficulties were encountered installing the drains through the embankment fills, pre-augering may also be required which would further increase the costs. Overhead hydro lines are also immediately adjacent to and potentially over the area where wick drains would be installed; there may be clearance difficulties between the hydro lines and the wick drain rig.

The amount of time-dependent settlement and the associated roadway maintenance may be reduced by employing lightweight fill materials below the pavement structure. Lightweight fill could be used in place of conventional earth fill to reduce the applied loading to below the pre-consolidation range.

Three types of lightweight fill are available for use:

- Extruded polystyrene (EPS) fill, with a bulk unit weight of less than 1 kN/m³;
- Ultra-lightweight slag fill from Hamilton (Litex-143), with a bulk unit weight of about 11.5 kN/m³; and,
- Lightweight slag fill (Superior Slag) from Sault Ste. Marie or from Hamilton (Litex-149), with a bulk unit weight of about 14 kN/m³.

If, for example, EPS fill is adopted for construction of the approach embankments a 1.5 metre thick layer of EPS behind retaining walls and abutments would reduce the applied load sufficiently to limit the post-paving, primary consolidation settlement to less than 25 mm.

Ultra-lightweight slag fill (Litex-143) could also be used to construct the embankment. Constructing the widened embankment entirely with ultra-lightweight slag fill would reduce the loading due to the weight of additional fill sufficiently to limit the post-paving, primary consolidation settlement to less than 25 mm. However, ultra-lightweight slag fill is relatively costly, particularly in small quantities, and this option may be uneconomical.

The actual magnitude of the settlements will depend greatly on the geometry of the additional embankment filling and of the retaining wall foundations. The preliminary widened embankment cross sections indicate that the greatest fill thicknesses may be placed in close proximity to the wing wall, above the heel of the pile supported foundations. Much of the load from the embankment widening may therefore be transmitted to the piles and not to the silty clay deposit. The increase in load on the clayey deposits may therefore be much less and the resulting settlements may be less than 25 millimetres.

The final foundation geometry may be sufficient to reduce the settlements as discussed above or, if considered necessary, the pile cap may be extended sufficiently to reduce the load imposed by the weight of the additional fill.

The settlement estimates and potential mitigation options should be refined as the design progresses and more information becomes available.

A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the settlement mitigation options is presented in Table 3 following the text of this report..

6.7 Design and Construction Considerations

6.7.1 Existing Utilities

An existing 1220 millimetre watermain extends parallel to Highway 417 and in close proximity to the proposed south abutment widenings and new wing walls. It is understood that this watermain will be removed prior to construction of the foundations. However, if that is not the case, special measures will be required during foundation construction to avoid damage to the watermain. In particular, the impact of the potential subgrade settlements beneath the widening will need to be considered.

6.7.2 Existing Foundations and Wing Walls – Frost Protection

It is understood that the profile grade of Carling Avenue WB 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 pile cap and extending the insulation horizontally from the base of the pile cap as shown in Figure 6 in Appendix C. 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.

The embankment settlements discussed in section 6.6.3 are sufficiently large to induce downdrag loads on those portions of the existing wing walls left within the widened embankment. The unit, unfactored, downward stress acting on each side of the embedded wing walls would be 35 kPa.

6.7.3 Excavations

Given that the pile caps will require a minimum of 1.8 metres of earth cover as frost protection, it is anticipated that excavation will extend to about 2 metres depth (about Elevation 73 to 74 m). That excavation would therefore extend through the surficial fill materials and topsoil, extending into the weathered silty clay to clay. The groundwater level is indicated to be at depths ranging from about 2 to 4 metres (about Elevation 72 to 73 m).

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 fill materials and weathered silty clay to clay are classified as Type 3 and 1 soils, respectively, according to the OHSA. 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 pile cap 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).

6.7.4 Construction Staging

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

6.7.5 Temporary Shoring

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

It is understood that the design of the shoring will be entirely the responsibility of the contractor. The shoring will have to be designed to resist lateral earth pressures that are controlled by the flexibility of the shoring and its method of support. However, conceptually, the temporary protection could consist of either soldier piles and lagging or steel sheet piling. It may be feasible to embed soldier piles or sheet steel piling sufficiently into the overburden without additional lateral support for excavations up to 3 m in depth (i.e., it may be feasible to cantilever the shoring). For deeper excavations or where steel sheet piles cannot be driven to sufficient depth due the presence of cobbles and boulders within the glacial till, it may be necessary to provide lateral support using either rakers supported on footings or piles within the excavation or using tie-backs grouted into the soil or bedrock behind the shoring.

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

The temporary excavation support should be in accordance with MTO Special Provision 105S19. The temporary system for the embankment and roadway protection at the abutment widenings should be designed to Performance Level 2 as defined in SP 105S19, provided that any buried utilities that may be present adjacent to the excavations(s) can tolerate this magnitude of deformation

6.7.6 Decommissioning of Boreholes

The standpipe in Borehole 06-14 will be decommissioned.

6.7.7 Groundwater and Surface Water Control

The groundwater level at the site generally ranges between about 2 to 4 m depth below the natural ground surface. Excavations for the construction of the pile caps will likely involve minimal groundwater and surface water control. It should be possible to handle ground and surface water inflows by pumping from well filtered sumps established in the floor of the excavations.

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



WC/MIC/FJH/ch

n:\active\2005\1120\geotechnical\05-1120-210 mrc hwy 417 bridges maitland to island park drive\foundations\05-1120-210-2000-4 rpt-004 - apr 2008.doc

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

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread footings supported on native silty clay soil	<ul style="list-style-type: none"> Not feasible 	<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
Steel H-pile foundations founded on or socketed into bedrock	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> High bearing resistance Negligible settlement Compatible with existing bridge construction or foundations Allows for semi-integral abutments 	<ul style="list-style-type: none"> If lateral / seismic loading conditions merit, pile toe may have to be socketed into medium strong bedrock, which would require coring or churn drilling Possibility of encountering cobbles or boulders during installation If sockets required, temporary liner necessary Existing abutment piles cannot be refitted to accommodate integral abutments Piles may interfere with existing battered piles. 	<ul style="list-style-type: none"> May be less expensive than caisson option 	<ul style="list-style-type: none"> Possibility of piles being driven mis-aligned due to boulders in glacial till
Caissons founded on or socketed into bedrock	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> High bearing resistance Negligible settlement Compatible with existing bridge construction or foundations 	<ul style="list-style-type: none"> Liners required to minimize disturbance to surrounding soils Possibility of encountering cobbles or boulders during installation Socketing of liner may be required to permit cleaning and inspection Coring or churn drilling will be required to form rock socket in medium strong bedrock May not be consistent with even semi-integral abutments 	<ul style="list-style-type: none"> May be more expensive than steel H-pile option, particularly if rock sockets are necessary, due to larger socket diameter 	<ul style="list-style-type: none"> May not be able to dewater socket for cleaning and inspection

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

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread footings supported on native soils	<ul style="list-style-type: none"> Not feasible 	<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
Steel H-pile foundations founded on or socketted into bedrock	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> High bearing resistance Negligible settlement Compatible with existing and proposed bridge foundations 	<ul style="list-style-type: none"> If lateral / seismic loading conditions merit, pile toe may have to be socketted into medium strong bedrock, which would require coring or churn drilling Possibility of encountering cobbles or boulders during installation If sockets required, temporary liner necessary 	<ul style="list-style-type: none"> May be less expensive than caisson option 	<ul style="list-style-type: none"> Possibility of piles being driven mis-aligned due to boulders in glacial till
Caissons founded on or socketted into bedrock	<ul style="list-style-type: none"> Feasible for support of all foundation elements 	<ul style="list-style-type: none"> Very high bearing resistance Negligible settlement Compatible with existing and proposed bridge foundations 	<ul style="list-style-type: none"> Temporary liners required to minimize disturbance to surrounding soils Possibility of encountering cobbles or boulders during installation If rock socket required, coring or churn drilling will be required to form rock socket in medium strong bedrock 	<ul style="list-style-type: none"> More expensive than steel H-pile option and spread footings 	<ul style="list-style-type: none"> May not be able to dewater socket for cleaning and inspection

TABLE 3
COMPARISON OF SETTLEMENT MITIGATION ALTERNATIVES
HIGHWAY 417 CARLING AVENUE WB
OVERPASS BRIDGE WIDENING
W.P. 4058-01-00

Settlement Mitigation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Allow embankments to settle and plan to pad/overlay roadway following construction	<ul style="list-style-type: none"> Feasible, if can tolerate interim settlements 	<ul style="list-style-type: none"> No impact on construction schedule 	<ul style="list-style-type: none"> Requires post-construction maintenance Possible interim safety issue, between overlays, due to settlement 	<ul style="list-style-type: none"> Relatively low costs, but must consider short term post-construction maintenance costs 	<ul style="list-style-type: none"> Excessive roadway settlement in short term
Excavate and replace silty clay	<ul style="list-style-type: none"> Not Feasible 	<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
Pre-load without surcharge	<ul style="list-style-type: none"> Not Feasible 	<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
Pre-load with surcharge	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> No post-construction maintenance 	<ul style="list-style-type: none"> Delays paving 	<ul style="list-style-type: none"> Slightly higher cost 	<ul style="list-style-type: none"> Some uncertainty about schedule, since can not start construction until monitoring indicates sufficient settlement has occurred.
Pre-loading with wick-drains	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Reduce the pre-loading time 	<ul style="list-style-type: none"> Mobilizing specialty subcontractor for very small amount of work May be difficult or impractical to install wick drains near or under overhead hydro lines 	<ul style="list-style-type: none"> Higher cost 	<ul style="list-style-type: none"> Limited uncertainty about schedule
Light weight fill	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> No post-construction maintenance. Minimal impact on schedule 	<ul style="list-style-type: none"> Expensive 	<ul style="list-style-type: none"> Expensive 	<ul style="list-style-type: none"> Low risk alternative, but contractor may successfully propose one of other options as change order
Fill loads transferred to retaining wall foundations	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> No impact on construction schedule No post construction maintenance 	<ul style="list-style-type: none"> May be additional cost to extend foundations Additional foundation loads may require additional piles 	<ul style="list-style-type: none"> May be more expensive 	<ul style="list-style-type: none"> Low risk alternative

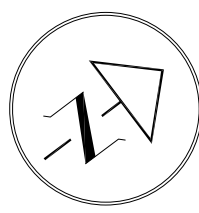
APPENDIX A

DRAWING 1 - CARLING AVENUE WB, BOREHOLE LOCATIONS
DRAWING 2 - CARLING AVENUE WB, SOIL STRATA


HWY. 417

WP No. WP 258-98-00

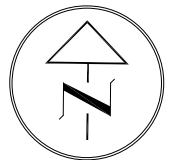
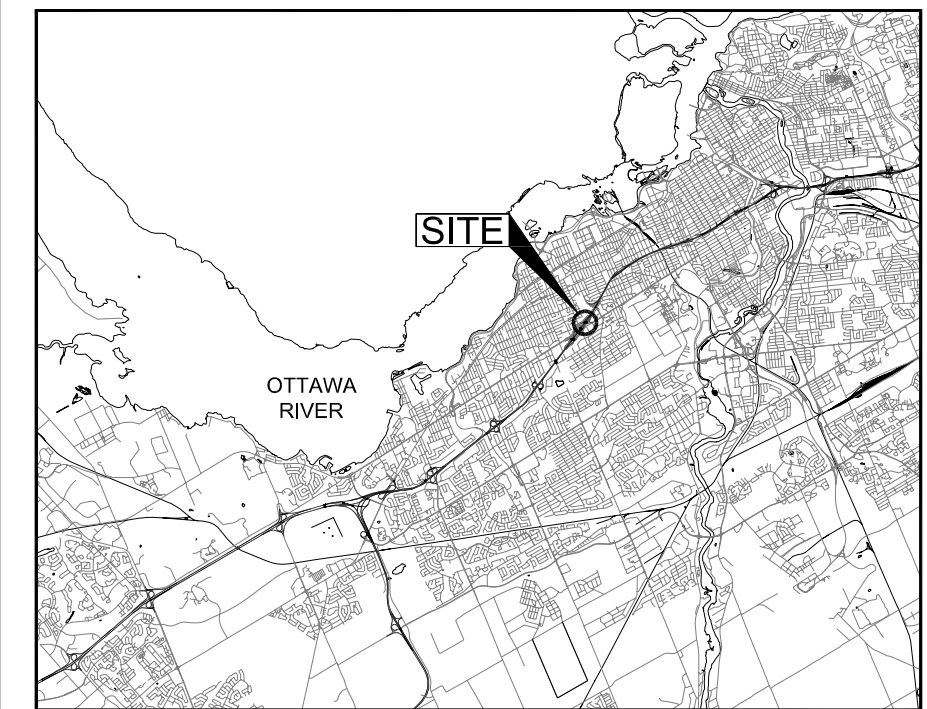
CARLING AVENUE
WEST BOUND BRIDGE
BOREHOLE LOCATIONS



SHEET



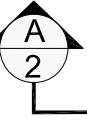
Golder Associates

Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

LEGEND

-  Borehole – Current Golder Associates Ltd. Investigation
-  Borehole – Previous MTO Investigation
Goecres No. 58-F-222-C
-  Location of cross-section

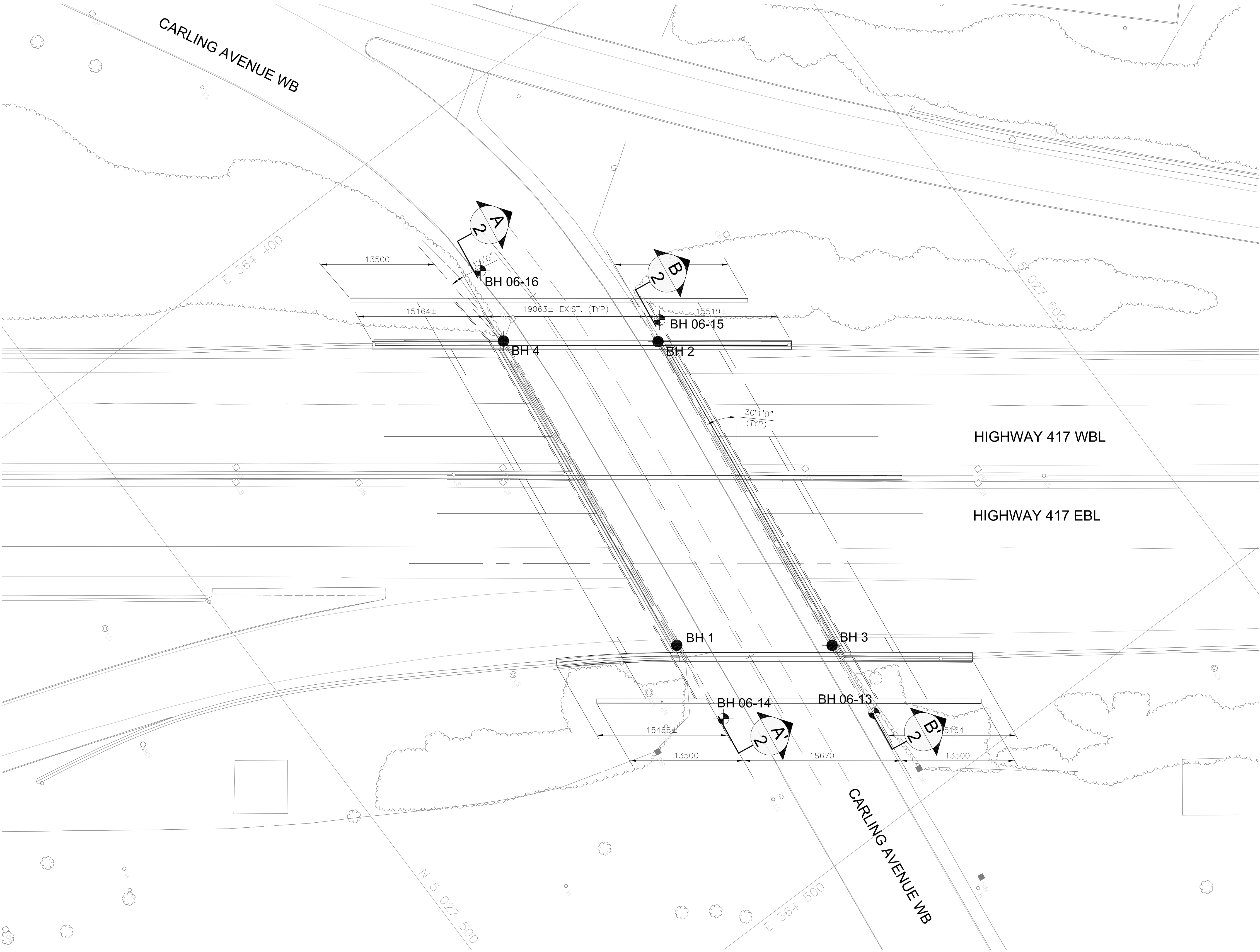
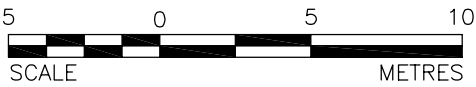
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-13	75.1	5027555.1	364488.3
06-14	75.2	5027540.4	364478.1
06-15	75.9	5027562.8	364435.8
06-16	75.3	5027549.4	364418.3
1	74.7	5027541.3	364467.9
2	75.5	5027561.1	364437.7
3	74.7	5027555.9	364478.9
4	75.4	5027546.5	364426.6

NOTES

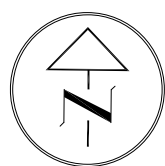
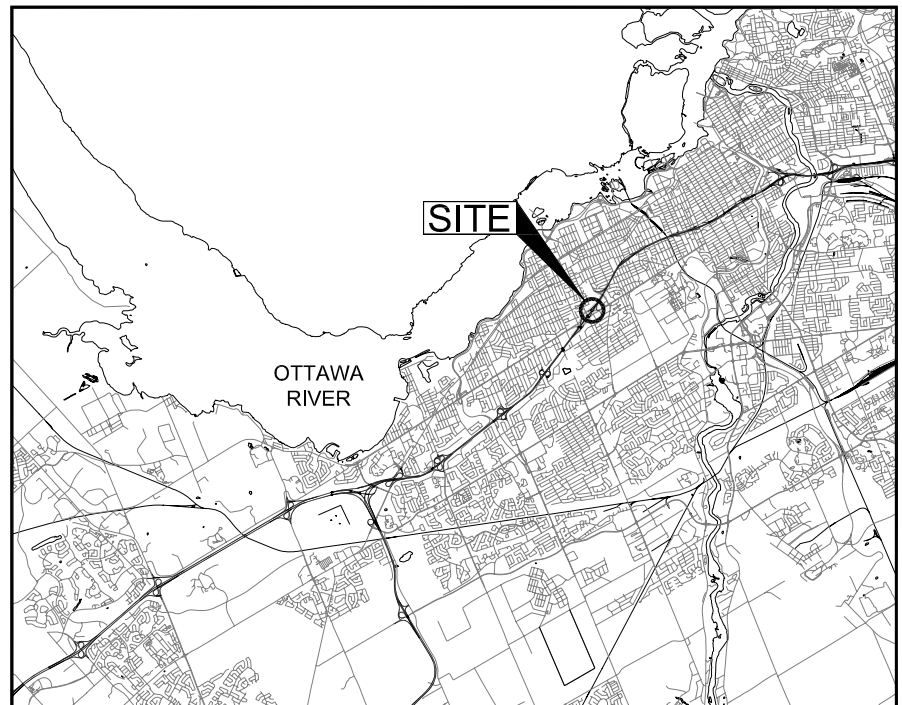
This drawing is for subsurface information only. Any surface details are for conceptual illustration.
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.
Base plan provided in electronic format by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

NO.	DATE	BY	REVISION
Geocres No. 3165-220			
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
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AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN












05-1120-210-4000-01.dwg



KEY PLAN

LEGEND

-  Borehole – Current Golder Associates Ltd. Investigation
-  Borehole – Previous MTO Investigation Goecres No. 58–F–222–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
-  WL in open borehole, measured upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06–13	75.1	5027555.1	364488.3
06–14	75.2	5027540.4	364478.1
06–15	75.9	5027562.8	364435.8
06–16	75.3	5027549.4	364418.3
1	74.7	5027541.3	364467.9
2	75.5	5027561.1	364437.7
3	74.7	5027555.9	364478.9
4	75.4	5027546.5	364426.6

METRIC

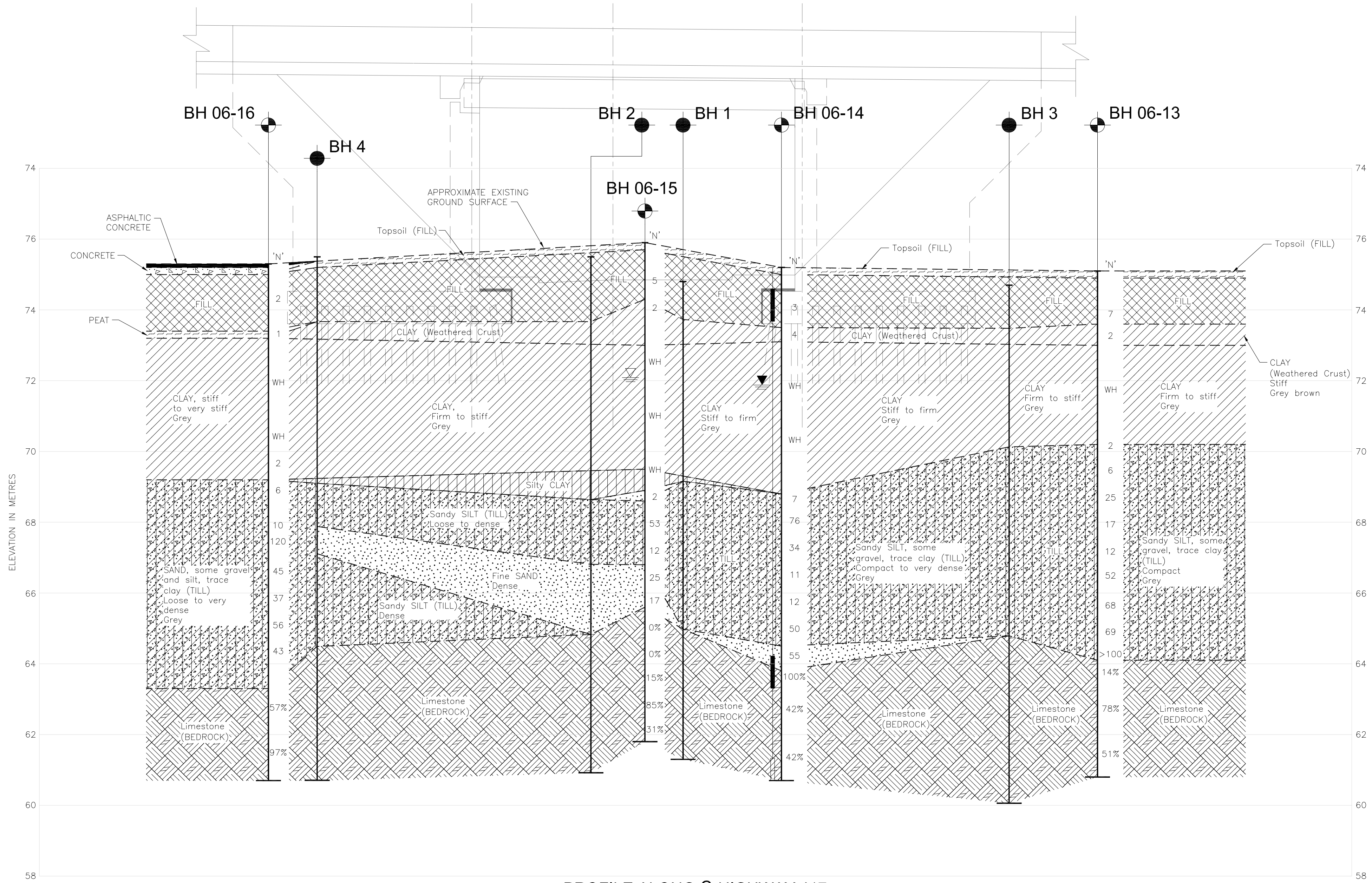
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AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

NOTES

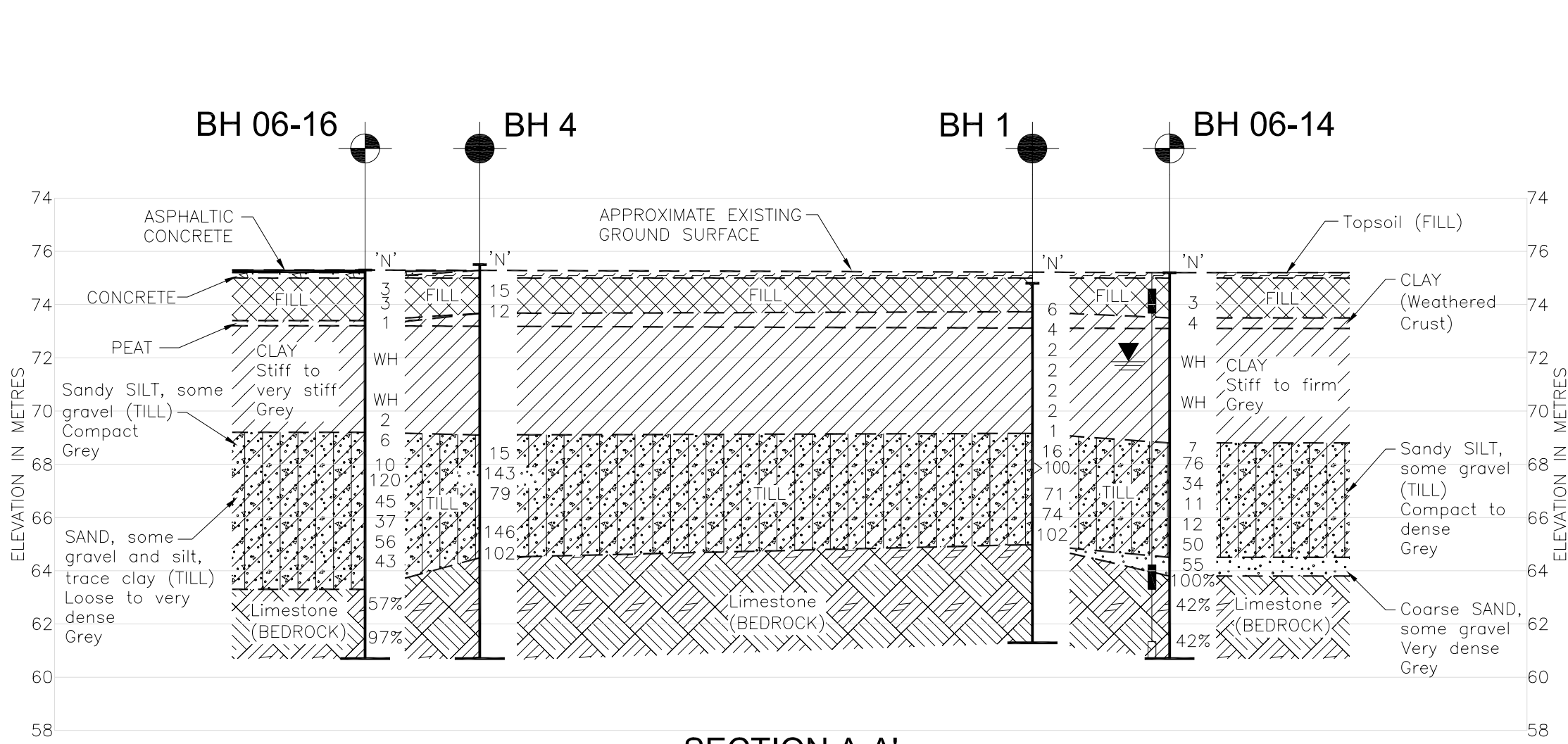
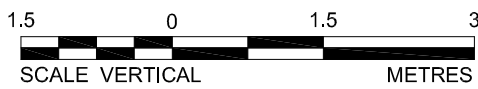
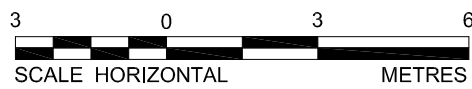
This drawing is for subsurface information only. Any surface details are for conceptual illustration.
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

Base plan provided in electronic format by McCormick Rankin Corporation

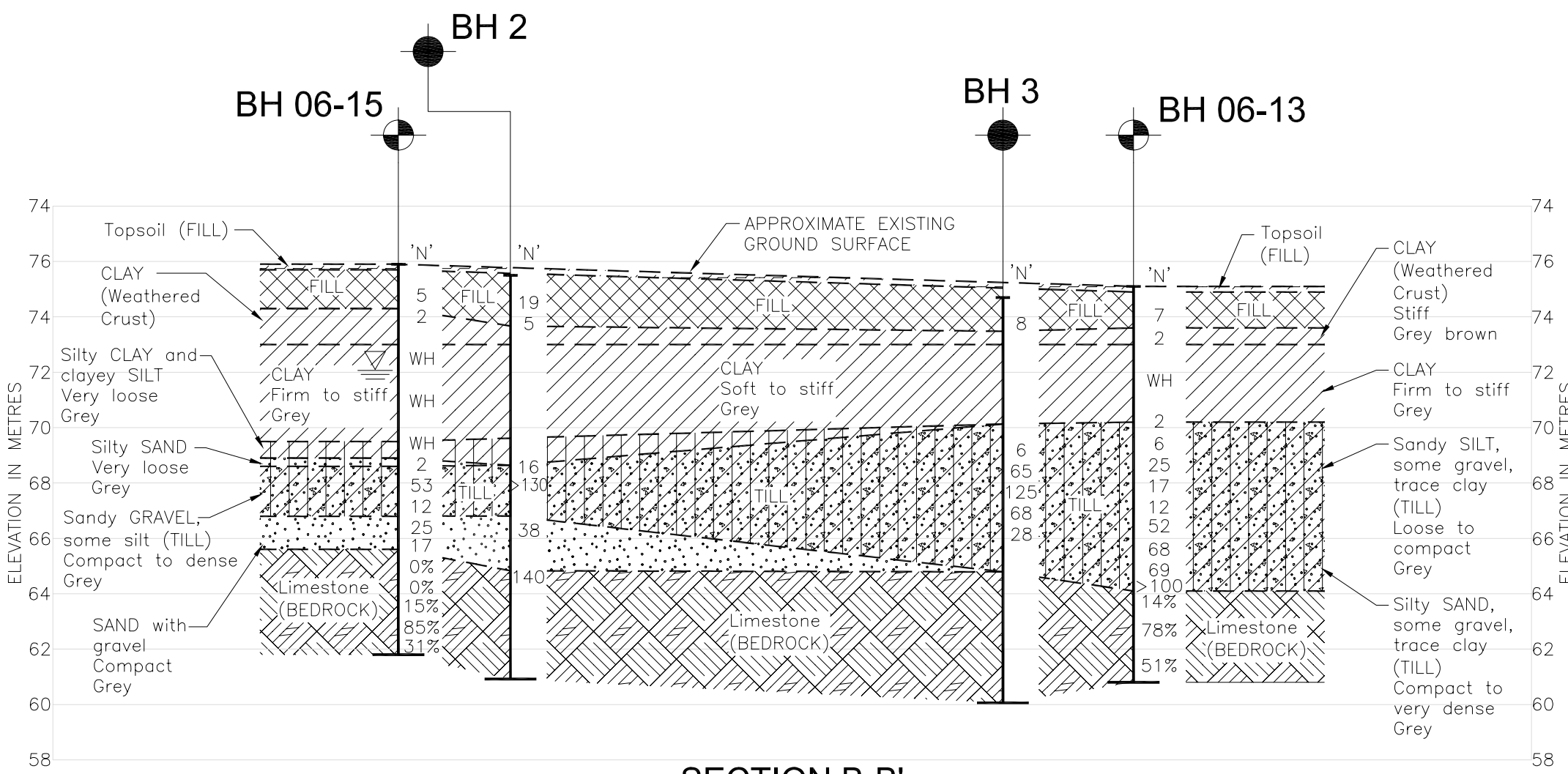
NO.	DATE	BY	REVISION
Geocres No. 31G5–220			
HWY. 417	PROJECT NO. 05–1120–210–2000		DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: SEPTEMBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD.	DWG. 2



PROFILE ALONG Q HIGHWAY 417



SECTION A-A'



SECTION B-B'



APPENDIX B

**LISTS OF ABBREVIATIONS AND SYMBOLS
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION
TERMINOLOGY
RECORDS OF BOREHOLES 06-13 TO 06-16**

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE		III. SOIL DESCRIPTION		
AS	Auger sample	(a)	Cohesionless Soils	
BS	Block sample			
CS	Chunk sample		Density Index	
DO	Drive open		(Relative Density)	
DS	Denison type sample		N	
FS	Foil sample		Blows/300 mm	
RC	Rock core		Or Blows/ft.	
SC	Soil core		Very loose	
ST	Slotted tube		Loose	
TO	Thin-walled, open		Compact	
TP	Thin-walled, piston	(b)	Cohesive Soils	
WS	Wash sample		Consistency	
			C _u 2S _u	
			Kpa	
			Psf	
			Very soft	
			Soft	
			Firm	
			Stiff	
			Very stiff	
		Hard		
		Over 200		
		Over 4,000		

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 x acceleration due to gravity)

(a) Index Properties (cont'd.)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

2. Shear strength = (Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

O:\ Templates\Rock Description Terminology

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

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

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

Description and Notes

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

Abbreviations

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

PROJECT 05-1120-210-2000

RECORD OF BOREHOLE No 06-13

1 OF 2

METRIC

W.P. 4058-01-00

LOCATION N 5027555.1; E 364488.3

ORIGINATED BY D.G.

DIST HWY 417

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

COMPILED BY J.M.

DATUM Geodetic

DATE May 11, 2006

CHECKED BY M.I.C.


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
								20 40 60 80 100			25 50 75				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							
75.1	GROUND SURFACE														
0.0	Topsoil (FILL)														
74.9															
74.6	Fine sand (FILL) Brown Moist		1	A.S.											
0.5	Silty clay with cinders (FILL) Grey brown Moist		2	A.S.											
74.3															
0.8	Fine sand, trace silt and clay (FILL) Loose Brown Wet		3	SS	7									0 91 5 4	
73.6															
1.5	CLAY (Weathered Crust) Stiff Grey brown Wet		4	SS	2										
73.0															
2.1	CLAY Firm to stiff Grey Wet														
			5	SS	WH										

MISS MTO 05-1120-210-4000.GPJ ON MOT.GDT 1/26/07

Continued Next Page

+ 3, X 3. Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-13		2 OF 2		METRIC															
W.P. 4058-01-00		LOCATION N 5027555.1; E 364488.3		ORIGINATED BY D.G.																	
DIST _____ HWY 417		BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.																	
DATUM Geodetic		DATE May 11, 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED					W _p — W — W _L WATER CONTENT (%)			γ kN/m ³			GR SA SI CL		
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100													
64.1	Sandy SILT, some gravel, trace clay (TILL) Very dense Grey Wet		13	SS	69		65														
			14	SS	>100																
11.0	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		15	NQ RC	DD		64														
			16	NQ RC	DD		63														
62.1	Limestone with thin shale interbeds (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong						62														
13.1																					
	Bedrock cored between 11.0m 14.3m depth. For bedrock coring details refer to Record of Drillhole 06-13.		17	NQ RC	DD		61														
60.8																					
14.3	End of Borehole																				

MISS_MTO 05-1120-210-4000.GPJ ON MOT.GDT 1/26/07

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-13

SHEET 1 OF 1

LOCATION: N 5027555.1; E 364488.3

DRILLING DATE: May 11, 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	FRFX-FRACTURE-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
								CL-CLEAVAGE		J-JOINT		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK		B-BEDDING							
								SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY											
								VN-VEIN		S-SLICKENSIDED		PL-PLANAR		C-CURVED											
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY											
								TOTAL CORE %	SOLID CORE %			DIP w/1 CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 ⁻⁶	K _f cm/sec	10 ⁻³	10 ⁻²	10 ⁻¹	10 ⁰						
11	Rotary Drill NQ Core	ROCK SURFACE		64.10																					
		Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		11.00	1																				
12					2																				
13		Limestone with thin shale interbeds (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		62.00 13.10																					
14		End of Drillhole		60.80 14.30																					
15																									
16																									
17																									
18																									
19																									
20																									
21																									

DEPTH SCALE

1 : 50



LOGGED: D.G.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-4000-ROCK GPJ GLDR CAN.GDT 12/6/07

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

MISS MTO 05-1120-210-4000.GPJ ON MOT.GDT 1/23/07

PROJECT		RECORD OF BOREHOLE		No 06-14		2 OF 2		METRIC					
W.P. 4058-01-00		LOCATION		N 5027540.4; E 364478.1		ORIGINATED BY		D.G.					
DIST		HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY					
J.M.		DATE		May 15, 2006		CHECKED BY		M.I.C.					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	GR SA SI CL
--- CONTINUED FROM PREVIOUS PAGE ---													
64.5	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact to very dense Gray Wet		11	SS	50		65						
10.7	Coarse SAND, some gravel with cobbles and boulders Very dense Gray Wet		12	SS	55		64						
63.8	Limestone (BEDROCK) Fresh Thinly to medium bedded Gray Medium strong Bedrock cored between 11.4m 14.5m depth. For bedrock coring details refer to Record of Drillhole 06-14.		13	NQ RC	DD		63						
11.4			14	NQ RC	DD		62						
60.7			15	NQ RC	DD		61						
14.5	End of Borehole Note: Water level in standpipe at 3.3m depth below ground surface on June 12, 2006												

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-14

SHEET 1 OF 1

LOCATION: N 5027540.4; E 364478.1

DRILLING DATE: May 15, 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	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT												SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
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RECOVERY		R.Q.D.		FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec		DIAMETRAL POINT LOAD INDEX (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
TOTAL CORE %	SOLID CORE %	%					10 ⁶	10 ⁵	10 ⁴	10 ³	1	0.5	0.1	0.05																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
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DEPTH SCALE

1 : 50



LOGGED: D.G.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-4000-ROCK.GPJ GLDR. CAN GDT 1/23/07

MISS_MTO 05-1120-210-4000.GPJ ON_MOT_GDT 1/23/07

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

PROJECT		05-1120-210-2000		RECORD OF BOREHOLE No 06-15		2 OF 2		METRIC										
W.P.		4058-01-00		LOCATION		N 5027562.8; E 384435.8		ORIGINATED BY										
DIST		HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY										
DATUM		Geodetic		DATE		May 15, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20						40	60	80	100	25
65.6	--- CONTINUED FROM PREVIOUS PAGE ---		10	SS	17													
10.3	SAND and GRAVEL, trace silt Compact Grey Wet																	
	Limestone with mud seams (BEDROCK) Fractured Thinly to medium bedded Grey and dark grey Medium strong		11	NQ RC	DD													
			12	NQ RC	DD													
			13	NQ RC	DD													
63.0																		
12.9	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong		14	NQ RC	DD													
62.5																		
13.4	Limestone (BEDROCK) Slightly fractured Thinly to medium bedded Grey Medium strong		15	NQ RC	DD													
61.8																		
14.1	Bedrock cored between 10.3m 14.1m depth. For bedrock coring details refer to Record of Drillhole 06-15. End of Borehole																	
	Note: Water level in open borehole at 3.8m depth upon completion of augering, and prior to commencing coring operations																	

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-15

SHEET 1 OF 1

LOCATION: N 5027562.8; E 364435.8

DRILLING DATE: May 15, 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 % 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										BC-BROKEN CORE MB-MECH. BREAK B-BEDDING			DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec								
									TOTAL CORE %	SOLID CORE %			DIP w/1 CORE AXIS	TYPE AND SURFACE DESCRIPTION									
									0 0														

PROJECT		05-1120-210-2000		RECORD OF BOREHOLE No 06-16		1 OF 2		METRIC				
W.P.		4058-01-00		LOCATION		N 5027549.4; E 364418.3		ORIGINATED BY D.G.				
DIST		HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY J.M.				
DATUM		Geodetic		DATE		May 16, 2006		CHECKED BY M.I.C.				
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
75.3	GROUND SURFACE											
0.0	ASPHALTIC CONCRETE											
75.0	CONCRETE											
74.8	Crushed stone (BASE)		1	A.S.			75					
0.5	Grey Moist Silly clay, some sand (FILL)											
74.4	Grey brown Moist		2/3	SS	2		74					
0.9	Fine to medium silty sand (FILL) Very loose Brown Moist		4	SS	1		73					
73.4	PEAT Black Moist CLAY Stiff to very stiff Grey Wet											
2.1			5	SS	WH		72					
			6	SS	WH		71					
			7	SS	2		70					
69.2												
6.1	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Loose Grey Wet		8	SS	6		69					
68.4												
6.9	SAND, some gravel and silt, trace clay (TILL) Loose to very dense Grey Wet		9	SS	10		68					
			10	SS	120		67					
			11	SS	45		66					
			12	SS	37							

Continued Next Page

+3, X3 Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT		05-1120-210-2000		RECORD OF BOREHOLE No 06-16		2 OF 2		METRIC						
W.P.		4058-01-00		LOCATION		N 5027549.4; E 364418.3		ORIGINATED BY						
DIST		HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY						
DATUM		Geodetic		DATE		May 16, 2006		CHECKED BY						
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						
	--- CONTINUED FROM PREVIOUS PAGE ---													
63.3	SAND, some gravel and silt, trace clay (TILL) Loose to very dense Grey Wet		13	SS	56		65							
			14	SS	43		64							37 48 12 3
			15	SS										
12.0	Limestone (BEDROCK) Fresh Thinly to medium bedded Grey Medium strong Bedrock cored between 12.0m 14.6m depth. For bedrock coring details refer to Record of Drillhole 06-16.		16	NQ RC	DD		63							
			17	NQ RC	DD		62							
60.7							61							
14.6	End of Borehole													

+ 3, × 3

Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

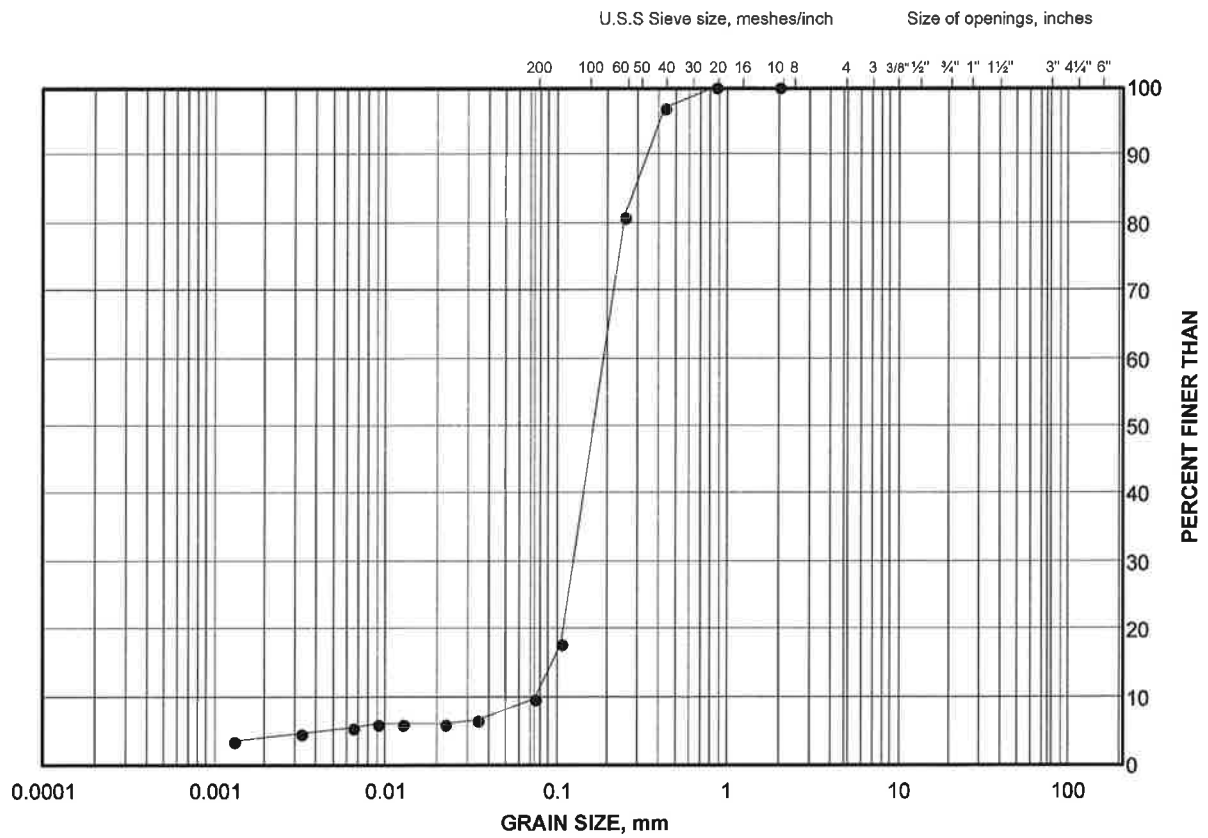
APPENDIX C

- FIGURE 1 - GRAIN SIZE DISTRIBUTION TEST RESULT – FILL
- FIGURE 2 - PLASTICITY CHART – CLAY (WEATHERED CRUST)
- FIGURE 3 - GRAIN SIZE DISTRIBUTION TEST RESULTS – CLAY
- FIGURE 4 - PLASTICITY CHART – CLAY
- FIGURE 5 - GRAIN SIZE DISTRIBUTION TEST RESULTS – TILL
- FIGURE 6 - FOUNDATION INSULATION DETAIL

GRAIN SIZE DISTRIBUTION

Sand, trace silt and clay (Fill)

FIGURE 1



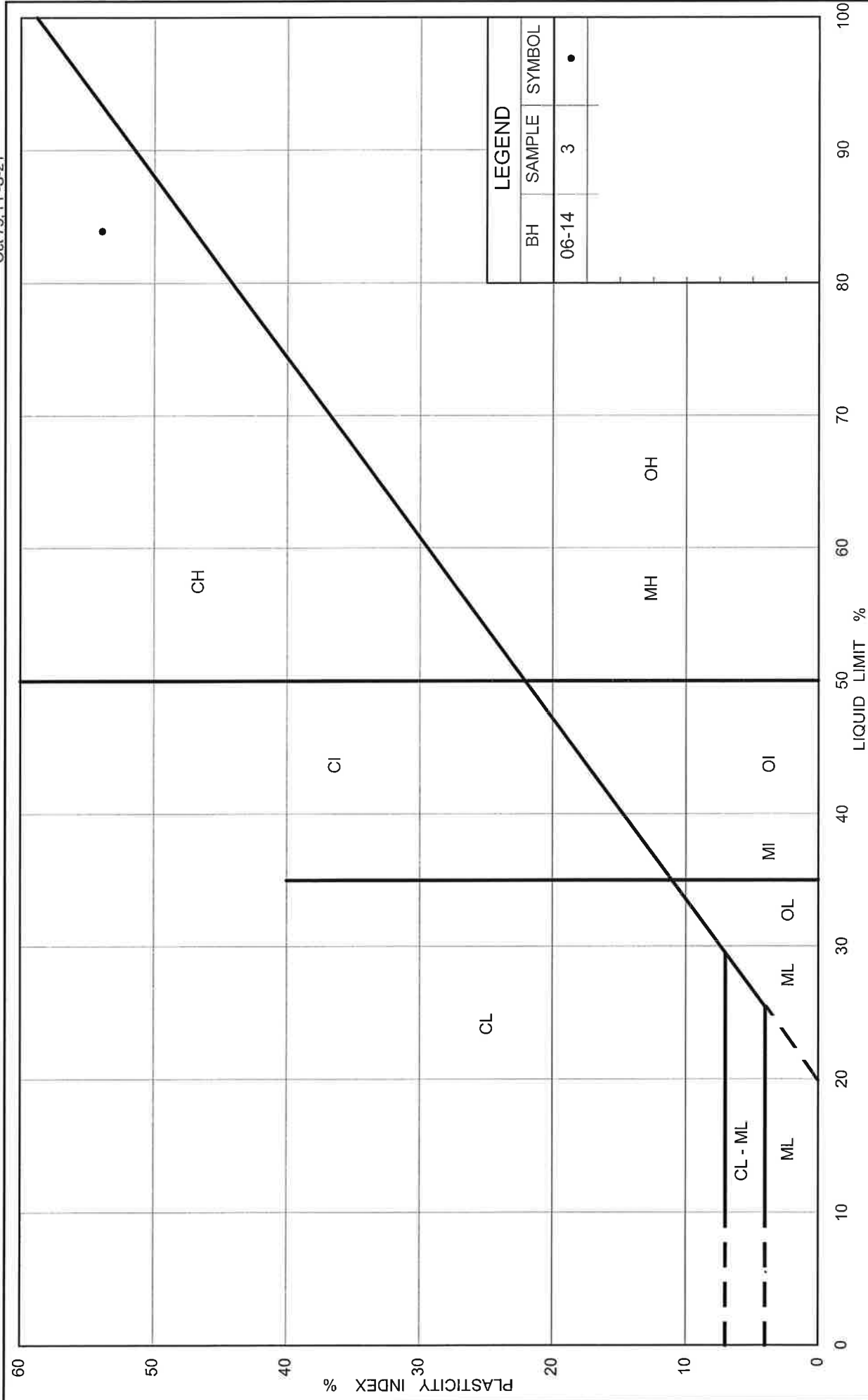


FIG No.2

PLASTICITY CHART Clay (Weathered Crust)

Ministry of Transportation



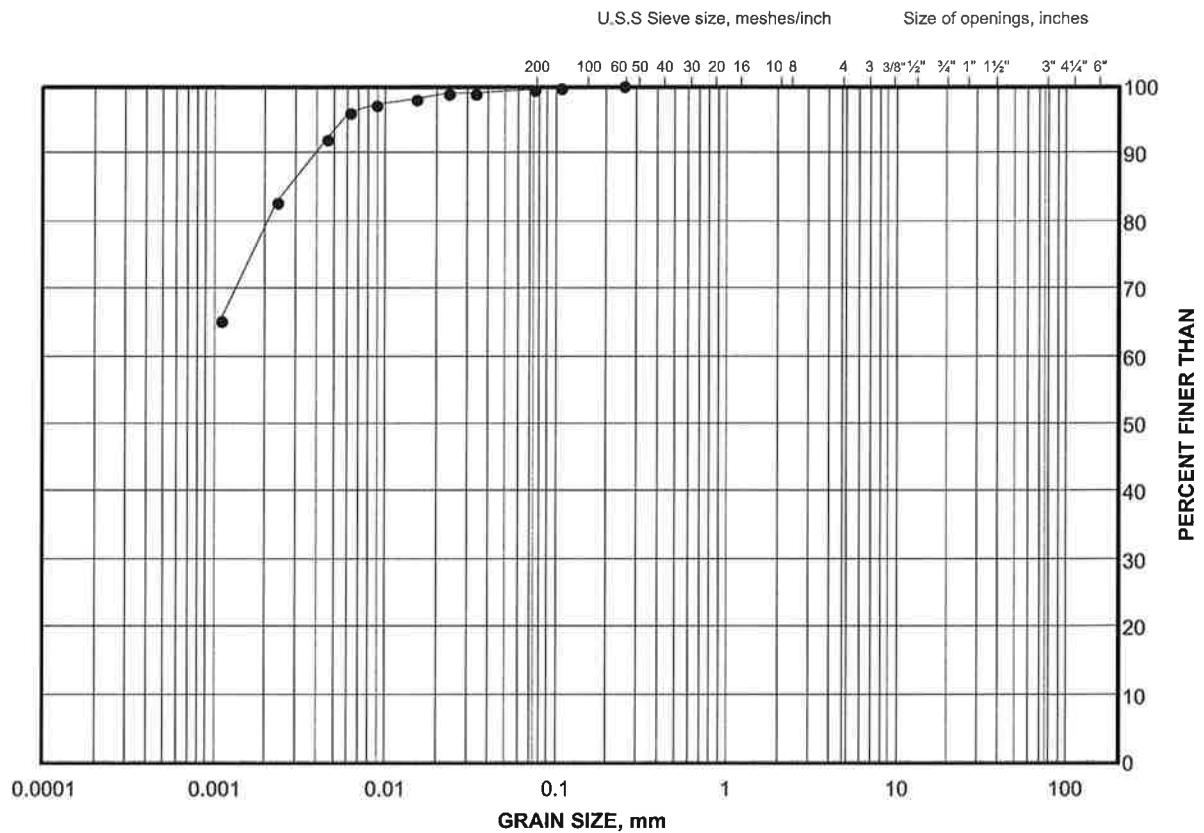
Ontario

Project No. 05-1120-210 - 2000

GRAIN SIZE DISTRIBUTION

Clay

FIGURE 3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-14	4	3.10 - 3.70

Project Number: 05-1120-210

Checked By: _____

Golder Associates

Date: 15-Jan-07

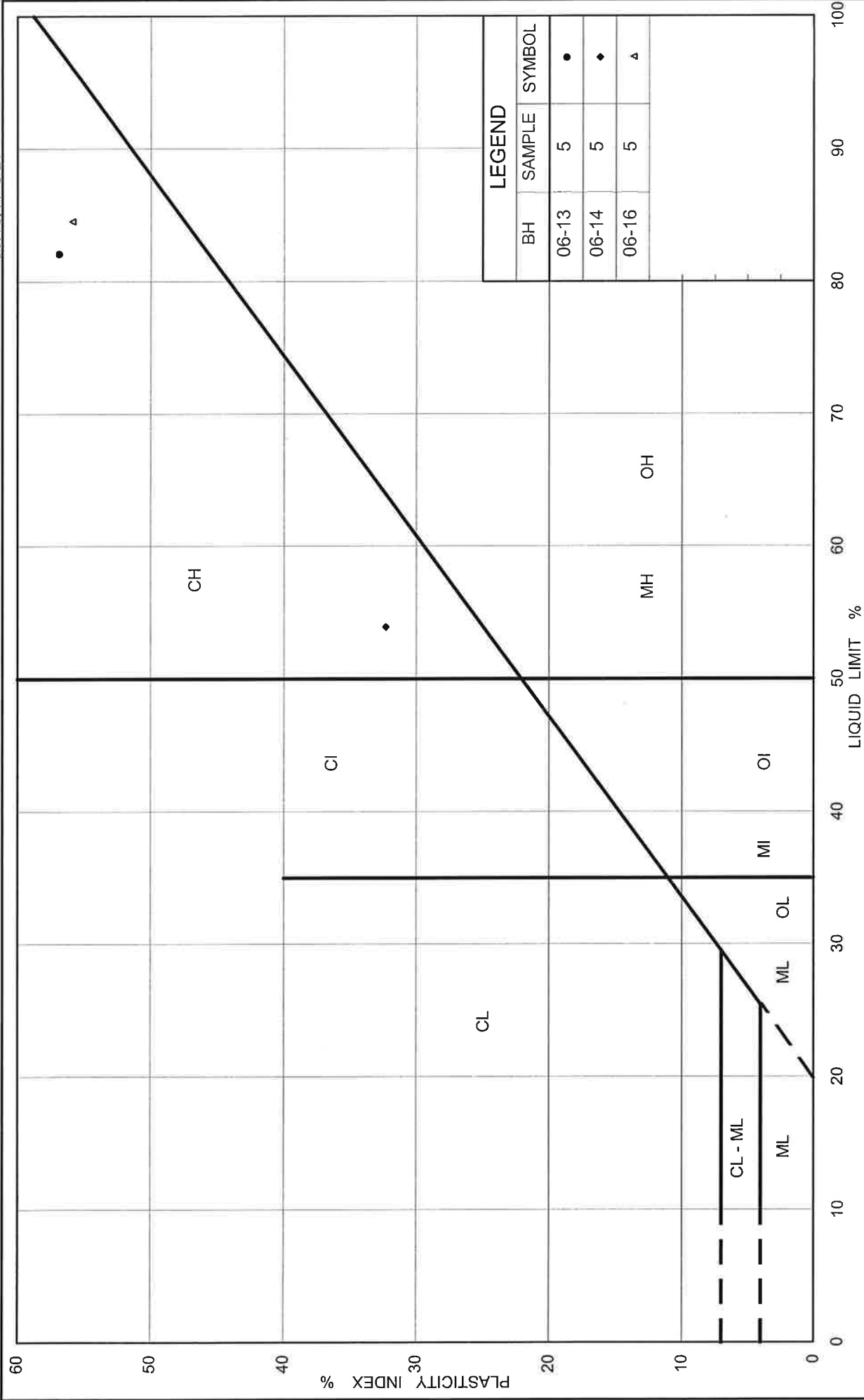


FIG No. 4

PLASTICITY CHART Clay

Ministry of Transportation



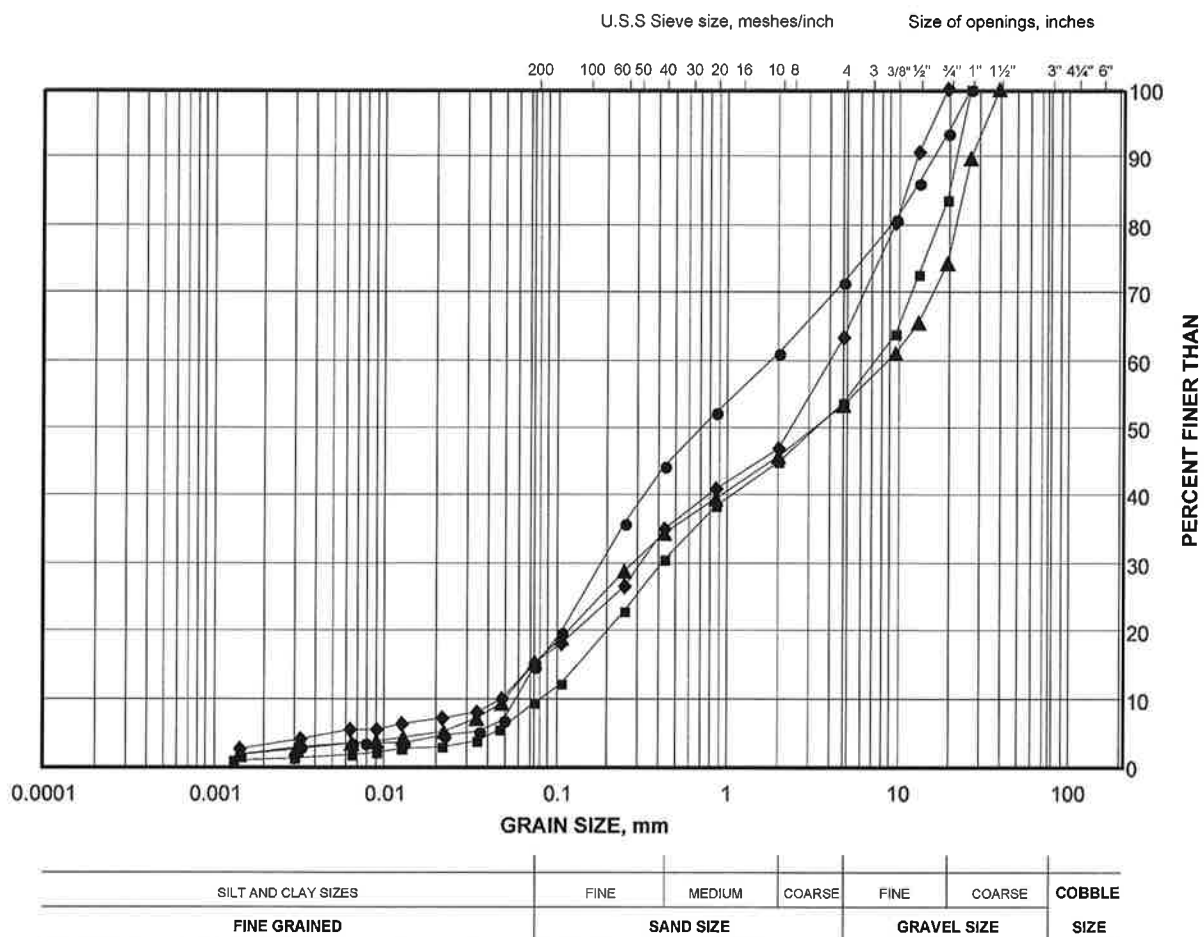
Ontario

Project No. 05-1120-210 - 2000

GRAIN SIZE DISTRIBUTION

Gravel, Sand and Silt, trace clay (Till)

FIGURE 5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	06-16	11	8.40 - 9.0
■	06-13	11	8.40 - 9.0
◆	06-16	14	10.70 - 11.20
▲	06-15	7	7.60 - 8.20

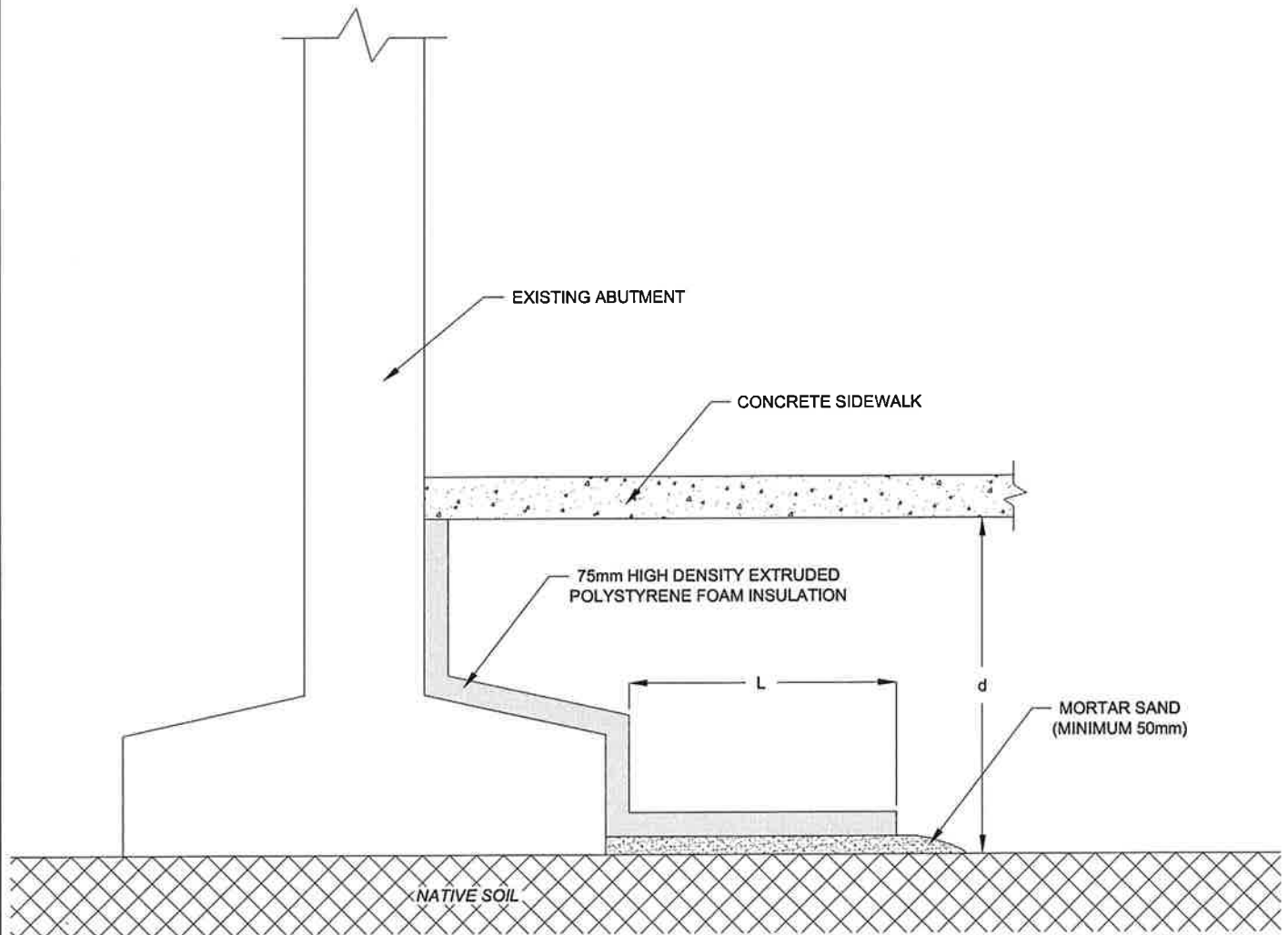
Project Number: 05-1120-210

Checked By: _____

Golder Associates

Date: 14-Apr-08

Drawing file: 051120210-4000-06.dwg Jun 07, 2007 - 8:58am



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 $d + L \geq 1.8m$
- 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. 051120210-4000-06.dwg

PROJECT No. 05-1120-210 REV. 0

SCALE	NTS
DATE	5 JUNE 07
DESIGN	W.C.
CADD	J.M.
CHECK	
REVIEW	

TITLE

FOOTING INSULATION DETAIL

CARLING AVENUE WESTBOUND BRIDGE

FIGURE

6

APPENDIX D

RECORDS OF PREVIOUS BOREHOLES 1 TO 4 (GEOCRES NO. 58-F-222-C)

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

Queensway & Carling (Westbound)
Bridge #7

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 245.1 (Geodetic Datum)
REMARKS ref: B.M. #11-11 el. 246.1' Top M.H. 20'S. of CARLING. Inters. of CARLING
& Rly.

HOLE NO.

1

DATE Feb 1/58

PENETRATION TEST

.....LB. HAMMER NO CASING
.....INCH DROPINCH DIA. ROD
BLOWS PER FOOT

				DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PENETRATION TEST							
UNCONFINED COMPRESSIVE STRENGTH KIPS/FT. ²	SMALL SCALE PENETROMETER KIPS/FT. ²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER				LB. HAMMER	INCH DROP	NO CASING	INCH DIA. ROD				
				GROUND SURFACE			BLOWS PER FOOT							
FROM SPLIT BARREL SAMPLES					Topsoil	0'	245.1'							
						1.0'	244.1'							
				5.7	3 for 6"	6	1-1	Loose Fine Sand with Silt	3.5'	241.6'				
				5.0	2 for 6"	4	1-2	Very Stiff Fissured Brownish Gray Clay	6.5'	238.6'				
				1.1	2 for 18"		1-3	Medium Soft Fissured Gray Clay						
				1.3	2 for 18"		1-4		11.5'	233.6'				
				0.75	2 for 18"		1-5	Soft						
				0.8	2 for 18"		1-6	Silty Gray Clay						
					1 for 6"	1	1-7	Very Loose Till	18.5'	226.6'				
					6 for 6"	16	1-8	Medium Dense Till	20.0'	225.1'				
					120 for 6"		1-9		22.5'	222.6'				
						71	1-10							
						74	1-11	Dense Sandy Till						
						102	1-12							
								Shaley Limestone & interbedded shale CORE RECOVERY - 78%	32.2'	212.9'				
								Shaley Limestone CORE RECOVERY - 89%	36.7'	208.4'				
								Shaley Limestone CORE RECOVERY - 76%	39.7'	205.4'				
									43.9'	201.2'				
								Bottom of Hole						
											% WATER CONTENT			
							PLATE 2							

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

Queensway Carling (Westbound)
Bridge #7

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 242.7' (Geodetic Datum)
REMARKS Ref: B.M. #11-11 el. 246.1' Top M.H. 20'S. of CARLING, INTERS. of CARLING
f Rly. DATE Feb 1/58

HOLE NO.
2

				PENETRATION TEST		
				LB. HAMMER	NO CASING	
				INCH DROP	INCH DIA. ROD	
				BLOWS PER FOOT		
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
				GROUND SURFACE	0'	247.7'
				Top Soil	1.0'	246.7'
		6 for 6"	2-1	Medium Dense Fine Sand	4.0'	243.7'
		5 for 6"	2-2	Loose Fine Sand	6.0'	241.7'
	5.8			Very Stiff Brownish GRAY Clay	7.5'	240.2'
2.0	3.1, 2.8 3.2, 3.1		2-3	Stiff Fissured Brownish GRAY Clay	10.0'	237.7'
1.7	1.4, 1.1 1.4, 1.2		2-4	Medium Soft		
1.5	1.4, 1.8 1.0, 1.2		2-5	Fissured GRAY Clay	15.0'	232.7'
1.8	1.5, 1.8 1.5, 1.8		2-6	Medium Soft Fissured GRAY Clay with some Silt	17.5'	230.2'
1.6	1.9, 1.6 1.5, 1.5		2-7	Medium Soft Fissured Silty GRAY Clay		
1.3	1.5		2-8		22.0'	225.7'
		3 for 6"	2-9	Loose SANDY Till	22.5'	224.2'
		13 for 6"	2-10	Dense SANDY Till	26.2'	221.5'
				Boulders in Dense SANDY Till	28.5'	219.2'
		38	2-11	Dense Fine Sand with some COARSE SAND & A few STONES		
		20 for 6"	2-12	Weathered Shaley Limestone	35.0'	212.7'
	140			Shaley Limestone & interbedded shale CORE RECOVERY - 94%	37.0'	210.7'
				Shaley Limestone CORE RECOVERY - 83%	42.5'	205.4'
				Shaley Limestone CORE RECOVERY - 100%	45.8'	201.9'
					47.8'	199.9'
				Bottom of Hole		
				% WATER CONTENT		PLATE 3

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QUEENSWAY & CARLING (Westbound)
Bridge #7

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 245.0' (Geodetic DATUM)
REMARKS Ref: B.M. #11-11 el. 246.1' Top M.H. 20'S. of CARLING INTERS. of CARLING
f Rly.

HOLE NO.

3

DATE Feb. 1/58

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 CASING		
						INCH DROP	INCH DIA. ROD		
							BLOWS PER FOOT				
				GROUND SURFACE							
				Topsoil	0'	245.0'					
					1.0'	244.0'					
		8	3-1	Loose Fine Sand	4.0'	241.0'					
2.1	4.5, 4.5 4.5		3-2	Very Stiff Fissured Brownish Gray Clay	7.5'	237.5'					
1.0	1.5, 1.6 1.8, 1.7		3-3	Medium Soft Fissured Gray Clay	10.0'	235.0'					
							Over-Night Water Level - 8.6'				
1.8	1.5, 1.5 1.2, 1.4		3-4	Medium Soft Fissured Silty Gray Clay	15.0'	230.0'					
1.5	1.8, 2.3 1.5, 1.7		3-5								
			3-6								
		5 for 6 6	3-7	Loose Till							
		65	3-8		20.0'	225.0'					
		65 for 6 125	3-9	Boulders in							
		27 for 6 68	3-10	Dense Sandy Till							
		28 for 6	3-11								
					32.5'	212.5'					
				Shaley Limestone + interbedded shale CORE RECOVERY - 57%	37.5'	207.5'					
				Shaley Limestone CORE RECOVERY - 91%	43.0'	202.0'					
				Shaley Limestone CORE RECOVERY - 80%	48.0'	197.0'					
				Bottom of Hole							
							% WATER CONTENT				
							PLATE 4				

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

Queensway & Carling (Westbound)
Bridge #7

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 247.5' (Geodetic Datum)
REMARKS Ref: B.M. #11-11 el. 246.1' Top M.H. 20'S. of CARLING. INTERS. of CARLING
Rly.

HOLE NO.
4

DATE Feb. 1/58

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 CASING		
							INCH DROP		INCH DIA. ROD		
							BLOWS PER FOOT				
				GROUND SURFACE →	0'	247.5'					
				Top soil	1.0'	246.5'					
				Loose Fine Sand	3.0'	244.5'					
				Medium Dense Fine Sand	6.0'	241.5'					
				Stiff Fissured							
				Brownish Gray Clay	10.0'	237.5'					
				Medium Soft Fissured							
				GRAY CLAY	15.0'	232.5'					
				Medium Soft Slightly Fissured	17.5'	230.0'					
				Silty GRAY CLAY	21.0'	226.5'					
				Medium Soft Fissured Silty	23.0'	224.5'					
				GRAY CLAY with a few sand	25.0'	222.5'					
				Layers of a few stones	27.5'	220.0'					
				Loose Till							
				Medium Dense Sandy Till							
				Dense Fine Sand with some							
				Coarse Sand & a few stones							
				Boulders							
				in							
				Dense Sandy Till							
				Shaley Limestone							
				Core recovery - 34%							
				Interbedded Shale							
				Shaley Limestone							
				Core recovery - 90%							
				Shaley Limestone							
				Core recovery - 93%							
				Bottom of Hole							
							% WATER CONTENT				PLATE 5

APPENDIX E

NON STANDARD SPECIAL PROVISIONS VIBRATION MONITORING DRIVING PILES ADJACENT TO EXISTING BATTERED PILES

VIBRATION MONITORING - Item No.

Special Provision

Scope

This special provision describes requirements for vibration monitoring during pile installation works.

Definitions

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

Submission Requirements

The Contractor shall submit details of the vibration monitoring plan to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Qualifications of vibrations monitoring specialist.
- Proposed instrumentation.
- Proposed location of instruments.
- Proposed frequency of readings.
- Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

Monitoring

The Contractor shall take readings during driving of each pile. The readings should be taken and recorded during the entire length of driving and during seating of the pile on the bedrock.

The pile(s) furthest from the monitored structure or utility should be driven first to assess the the vibration level at the existing structures. If necessary, the contractor must alter the pile driving procedures for the remaining piles. The revised procedure shall be submitted to the Contract Administrator for approval prior to driving the remaining piles.

The measured vibrations shall not exceed 100 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile has been driven prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next piles with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures until the vibrations are within acceptable levels. The above process must be repeated for each pile.

Basis of Payment

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

END OF SECTION

Non-Standard Special Provision

Scope of Work

This Special Provision covers the requirements for driving piles within close proximity to existing battered piles (i.e., where the anticipated distance between the new pile tip at depth and the existing battered pile tip at depth is less than 20% of the existing pile length.)

Definitions

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

Construction

Work under this item shall adhere to the following requirements:

- For new piles driven within the potential zone of interference with the existing abutment or wing wall piles (defined as a distance around the existing pile tip at depth equal to 20% of the pile length) the driving operations shall be continuously monitored by the QVE.
- The contractor shall cease driving of the pile if the QVE indicates that the driven pile may have come in contact with an existing pile.
- If contact between the new and existing piles is believed to exist the contractor shall take remedial action as directed by the Contract Administrator, which may include extracting the pile and re-driving or replacing the pile.

Basis of Payment

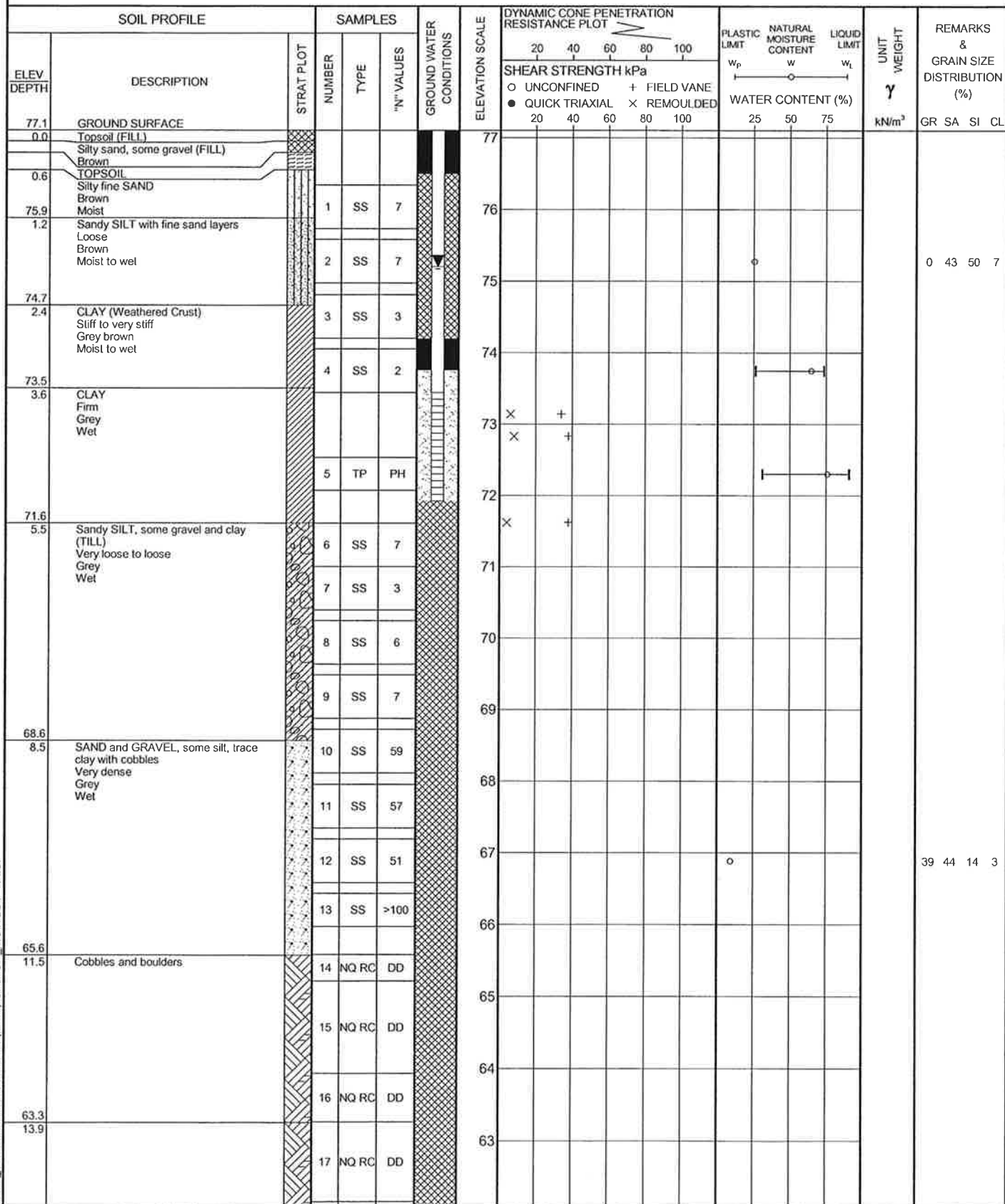
Payment at the contract price for the above noted Tender Item includes full compensation for all labour, equipment and materials to do the required work.

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APPENDIX F

RECORD OF BOREHOLE 06-208 CONSOLIDATION TEST RESULTS – BH 06-208

PROJECT <u>05-1120-210-2700</u>		RECORD OF BOREHOLE No 06-208		1 OF 2	METRIC
W.P. <u>4058-01-00</u>	LOCATION <u>N 5027482.9; E 364336.4</u>	ORIGINATED BY <u>D.J.S.</u>			
DIST <u>HWY 417</u>	BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem Auger</u>	COMPILED BY <u>J.M.</u>			
DATUM <u>Geodetic</u>	DATE <u>November 13, 2006</u>	CHECKED BY <u>M.I.C.</u>			



MISS_MTO_051120210-2700-2 (EDIT) GPJ ON MOT GDT 7/4/07

Continued Next Page

+ 3, x 3

Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE



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

MISS_MTO 051120210-2700-2 (EDIT).GPJ ON_MOT.GDT 7/4/07

PROJECT: 05-1120-210-2700

RECORD OF DRILLHOLE: 06-208

SHEET 1 OF 1

LOCATION: N 5027482.9; E 364336.4

DRILLING DATE: Nov. 10, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		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
				DEPTH (m)	RECOVERY					R.O.D. %	FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K _f cm/sec							
												TOTAL CORE %	SOLID CORE %		DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION					
		ROCK SURFACE		65.00																	
		Cobbles and boulders		11.50	1																
12					2																
13					3																
14	Rotary Drill NO Core	Limestone with black shale interbeds (BEDROCK) Fresh Grey		63.20 13.90	4																
15					5																
16		End of Drillhole		60.70 16.40																	
17																					
18																					
19																					
20																					
21																					
22																					
23																					
24																					
25																					
26																					

MIS-RCK-001 051120210-2700-2-ROCK GPJ GLDR CAN.GDT 7/4/07 J.M.

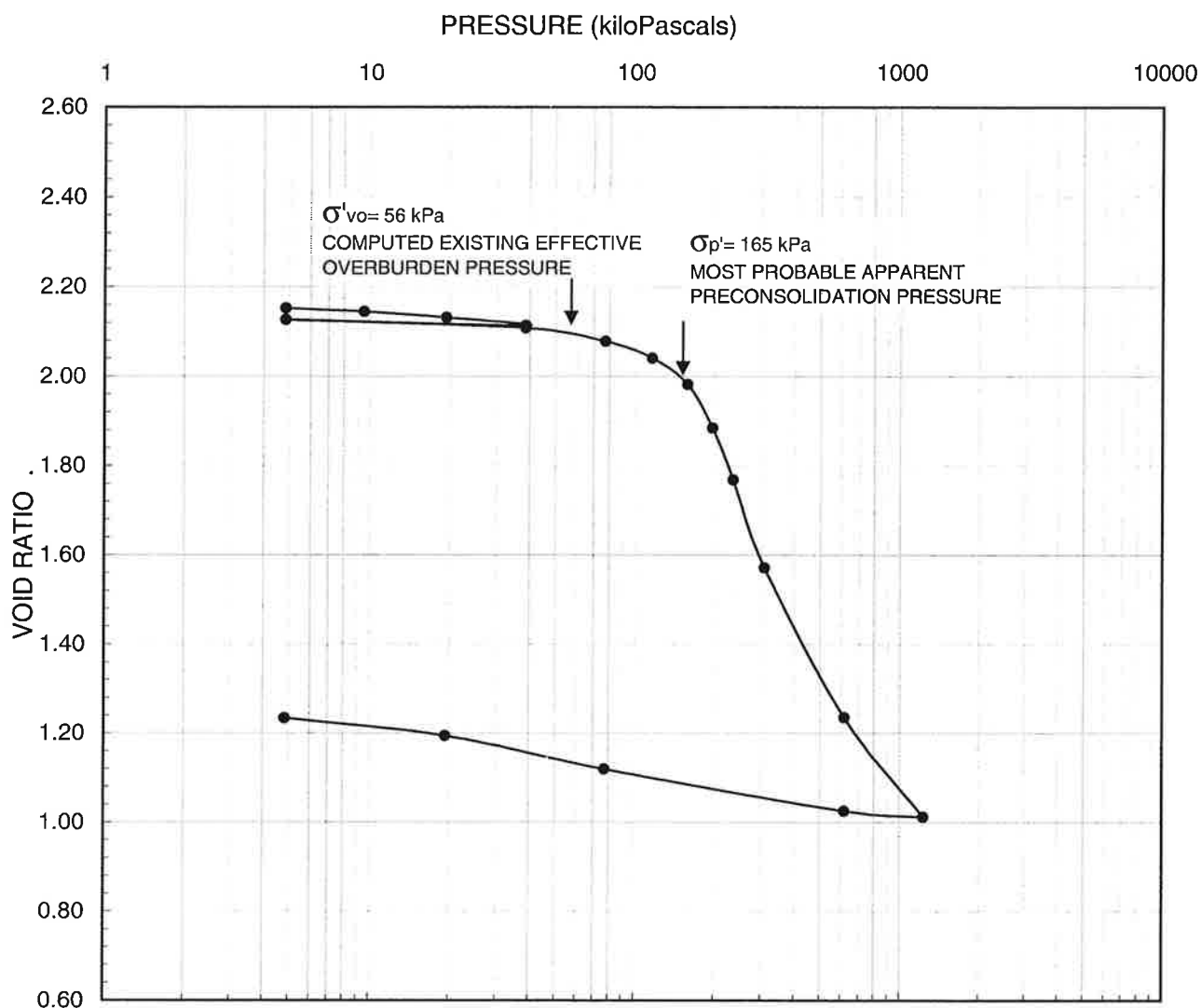
DEPTH SCALE

1 : 75



LOGGED: D.J.S.

CHECKED: W.C.



LEGEND

Borehole: 06-208	$w_i = 76.0\%$	$S_o = 97\%$
Sample: 5	$w_f = 47.0\%$	$C_c = 1.66$
Depth (m): 4.8m	$w_l = 90.8\%$	$C_r = 0.012$
	$w_p = 30.6\%$	



SCALE	AS SHOWN
DATE	07/13/07
DESIGN	NA
CADD	NA
CHECK	
REVIEW	

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary
PROJECT No. 05-1120-210-2700-2 REV. 0

FIGURE 10