



February 2017

REPORT ON

Foundation Investigation and Design Report Merivale Road Overpass Bridge Widening Structure 3-47 Highway 417 W.P. 4058-01-00

Submitted to:

MMM Group Limited
300-1145 Hunt Club Road
Ottawa, Ontario
K1V 0Y3

REPORT



Geocres Number: 31G5-271

Report Number: 05-1120-210-2000-5

Distribution:

1 copy - MMM Group Limited, Ottawa
5 copy - Ministry of Transportation, Kingston
1 copy - Ministry of Transportation, Downsview
1 copy - Golder Associated Ltd., Ottawa





Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	2
3.0 INVESTIGATION PROCEDURES.....	3
4.0 SITE GEOLOGY AND STRATIGRAPHY.....	4
4.1 Regional Geological Conditions	4
4.2 Site Stratigraphy.....	4
4.2.1 Fill	5
4.2.2 Topsoil	5
4.2.3 Silty Clay to Clay.....	5
4.2.4 Sand	6
4.2.5 Silty Sand to Sandy Silt Till.....	6
4.2.6 Limestone Bedrock	6
4.3 Groundwater Conditions.....	7
5.0 CLOSURE	8

PART B – FOUNDATION DESIGN REPORT

6.0 ENGINEERING RECOMMENDATIONS	9
6.1 General	9
6.2 Bridge Foundation Options.....	9
6.2.1 Steel H-Pile Foundations	10
6.2.1.1 Axial Geotechnical Resistance	11
6.2.1.2 Resistance to Lateral Loads.....	13
6.2.1.3 Frost Protection	15
6.2.2 Caisson Foundations	15
6.2.2.1 Axial Geotechnical Resistance	15
6.2.2.2 Resistance to Lateral Loads.....	16
6.2.2.3 Frost Protection	16
6.3 Bridge Retaining Wall Foundation Options.....	16



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

6.4	Site Coefficient	17
6.5	Lateral Earth Pressures for Design	18
6.6	Approach Embankment Design and Construction	20
6.6.1	Subgrade Preparation and Approach Embankment Construction	20
6.6.2	Approach Embankment and Retaining Wall Stability	21
6.6.3	Approach Embankment Settlement	22
6.7	Design and Construction Considerations	24
6.7.1	Existing Utilities	24
6.7.2	Existing Wing Walls – Frost Protection	25
6.7.3	Excavation	26
6.7.4	Temporary Shoring	26
6.7.5	Groundwater and Surface Water Control	26
7.0	CLOSURE	27

TABLES

Table 1	Comparison of Foundation Alternatives, Highway 417 Merivale Road Overpass Bridge Widening, W.P. 4058-01-00
Table 2	Comparison of Bridge Retaining Wall Alternatives, Highway 417 Merivale Road Overpass Bridge Widening, W.P. 4058-01-00
Table 3	Comparison of Settlement Mitigation Alternatives

APPENDICES

APPENDIX A

Drawing 1	Merivale Road, Borehole Locations
Drawing 2	Merivale Road, Soil Strata

APPENDIX B

Lists of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Records of Boreholes 06-17 to 06-18

APPENDIX C

Figure 1	Plasticity Chart – Weathered Clay
Figure 2	Grain Size Distribution Test Results – Silty Clay
Figure 3	Plasticity Chart – Clay to Silty Clay
Figure 4	Summary of Engineering Properties – Silty Clay to Clay



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

APPENDIX D

Records of Previous Boreholes 1 to 6 (Geocres No. 58-F-229-C)

APPENDIX E

Figures

Figure 5 - Foundation Insulation Detail

Figures 6 to 9 - Results of Slope Stability Analysis

Non Standard Special Provisions

Vibration Monitoring

Driving Piles Adjacent to Existing Battered Piles

Pile Driving over Storm Sewer Tunnel

APPENDIX F

Drawings 9B and 10 (Report Number 05-1120210-6)

Record of Borehole 06-141

Consolidation Test Results – BH 06-141

Record of Borehole 06-143

Record of Borehole 06-143A

Consolidation Test Results – BH 06-143A



PART A

**FOUNDATION INVESTIGATION REPORT
MERIVALE ROAD OVERPASS BRIDGE WIDENING
STRUCTURE SITE 3-47
HIGHWAY 417
W.P. 4058-01-00**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group Limited (MMM) 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 were 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.

The bridge widenings at Clyde Avenue, Carling Avenue EB, Kirkwood Avenue and Carling Avenue WB were constructed in 2008, 2010, and 2013 under separate bridge rehabilitation contracts. Rehabilitation of the Merivale Road Structure and construction of the retaining walls is part of the remaining work under W.P. 4058-01-00.

This report addresses the proposed widening of the bridge over Merivale Road 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 Merivale Road bridge is an overpass structure for Highway 417 and is located within a commercial area of Ottawa.

Merivale Road is a two lane road with an urban cross-section and sidewalks on both sides. The surrounding land on either side of Highway 417 is relatively flat and level.

The existing bridge is a rigid frame concrete structure supported on piles founded on bedrock. The bridge consists of two separate bridges (one for each of the eastbound and westbound lanes of Highway 417) with the two abutments separated by a 25 mm joint.

It is understood that the abutment stem walls are in good condition with spalls and delaminations covering less than 1 percent of the exposed face. From a foundation perspective, the existing bridge is performing adequately.

The existing approach embankments are 4 to 5 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.

There are numerous utilities in the area of the bridge structure, including (but not limited to) a 2,100 mm diameter storm sewer tunnel in the rock to the west of the bridge which crosses under the southwest abutment footing, an 1,800 mm diameter storm sewer located east of the structure, a 1,050 mm sanitary sewer and a number of hydro ducts (to be relocated) beneath Merivale Road, and a decommissioned 1,220 mm diameter watermain in the area of the south abutment.

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 Merivale Rd., Bridge No. 36, to Deleuw, Cather and Company of Canada Limited" (Geocres No. 58-F-229-C).



3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out on May 10 and 11, 2006. On those days, two boreholes (Boreholes 06-17 and 06-18) were put down at the locations shown on Drawing 1. The boreholes were drilled near or at the approximate locations of the ends of the proposed abutment widenings. The boreholes were advanced using a truck mounted drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of 11.6 and 10.2 m below present ground surface.

Samples of the overburden were obtained at 0.6 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 of 3.5 and 3.0 m, after practical refusal to augering had been reached. One standpipe was installed (in Borehole 06-18) to monitor the groundwater level at the site. The standpipe consists of 20 mm outside diameter HDPE tubing with a 0.6 m long slotted tip. The boreholes were backfilled with bentonite mixed with soil cuttings. The site conditions were restored following completion of the field work.

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

The groundwater level was measured in the standpipe in Borehole 06-18 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 MMM 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 in Appendix A.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
06-17	South-west abutment	5027932.3	364740.8	74.1
06-18	South-east abutment	5027945.9	364749.6	73.7



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, two boreholes were advanced within or near the limits of the foundation elements for the proposed widening of the Merivale Road bridge. The borehole locations are shown on Drawing 1 in Appendix A. Soil stratigraphy sections projected along the highway centreline and across the abutment foundation areas are shown on Drawing 2 in Appendix A.

The detailed subsurface soil, bedrock, and groundwater conditions encountered in the boreholes and the results of the in-situ and laboratory testing are given on the Record of Borehole sheets in Appendix B and on Figures 1 to 3 in Appendix C. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Six 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-229-C) and the Record of Borehole sheets from that investigation are also attached in Appendix D.

Golder Associates carried out an investigation in the area of this site, the results of which are included in a report to MMM titled "Foundation Investigation and Design, Retaining Walls, Maitland Ave to Island Park Drive, Highway 417, W.P. 4058-01-00" dated January 2008 (report number 05-1121-210-2000-6). Borehole location plans, select record of boreholes sheets (i.e., BH 06-141 and 06-143A), and the results of two consolidation tests from the previous investigation are presented in Appendix F, and are used herein solely to describe the silty clay conditions. The two relevant boreholes are located about 140 and 50 m to the west and east of the Merivale bridge structure, respectively. Very similar ground conditions were encountered and the results of the consolidation testing on the silty clay deposit are representative of the silty clay deposit located at the Merivale overpass structure.

In summary, the soils encountered during the current investigation within the limits of the widening consist of topsoil and fill materials extending to depths of about 1.2 to 1.3 m, underlain by some 5.5 to 6 m of clay, over 1.3 m of glacial till at Borehole 06-17, with the overburden extending to depths of about 7.2 to 8.1 m. These overburden materials are underlain by limestone bedrock.

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.



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

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 Fill

Topsoil (fill) exists at the ground surface at both Boreholes 06-17 and 06-18 and is about 0.1 and 0.3 m thick, respectively.

Fill materials associated with previous uses of the site and roadway construction underlies the topsoil at both boreholes.

The fill materials at Boreholes 06-17 and 06-18 extend to depths of about 1.0 and 1.2 m, respectively, below ground surface. At Borehole 06-18, the upper 0.2 m of fill is composed of sand with some gravel. The remaining fill materials at both boreholes are composed of silty clay with traces of sand or gravel. An SPT “N” value of 9 blows per 0.3 m of penetration was measured in the silty clay fill at Borehole 06-18.

4.2.2 Topsoil

A buried layer of topsoil, about 0.3 m thick, was also encountered beneath the fill materials at Borehole 06-17 at a depth of about 1.0 m.

4.2.3 Silty Clay to Clay

The buried topsoil layer at Borehole 06-17 and the fill materials at Borehole 06-18 are underlain by a deposit of silty clay to clay, which is about 5.5 and 6.0 m thick (including a thin sand layer at Borehole 06-18), respectively. The upper portion of the deposit has been weathered to depths below the clay surface of about 2.3 and 2.4 m to a grey-brown colour. Measured SPT “N” values in this weathered zone ranged from 1 to 15 blows per 0.3 m of penetration. These test results indicate that the weathered portion of the deposit has a stiff to very stiff consistency.

The results of Atterberg limit testing on one selected sample of the weathered portion of the deposit indicate a plasticity index of 52 percent and a liquid limit of 79 percent. These results, shown on the plasticity chart on Figure 1, confirm that this material is a clay of high plasticity. The measured natural water contents of two samples of the weathered silty clay to clay were 51 and 61 percent.

The silty clay to clay below the depth of weathering at both boreholes is grey in colour and extends to depths of about 6.8 and 7.2 m below ground surface. In situ vane testing carried out within this unweathered deposit measured undrained shear strengths generally ranging between 42 and 61 kPa. These test results indicate that the unweathered silty clay to clay has a firm to stiff consistency. In situ vane testing carried out on remoulded grey silty clay to clay gave undrained shear strengths ranging from 5 to 10 kPa, reflecting a sensitive material (sensitivities ranging from 6 to 8).

The results of a grain size distribution test carried out on a sample of the unweathered silty clay from Borehole 06-17 are shown on Figure 2.

The results of Atterberg limit testing on two selected samples of the unweathered silty clay to clay indicate plasticity index values of 28 and 37 percent and liquid limit values of 48 and 58 percent. These results, also shown on the plasticity chart on Figure 3, confirm that this material is a silty clay to clay of intermediate to high plasticity. The measured natural water content of the unweathered silty clay ranged from 55 to 61 percent, which is generally in excess of the measured liquid limit.



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

The results of two oedometer consolidation tests carried out on samples from nearby boreholes (06-141 and 06-143A) which bracket the Merivale Road site indicate the clay deposit is preconsolidated by about 75 to 125 kPa above the existing overburden pressure. The results of the oedometer consolidation tests are shown in Appendix F.

A summary of the measured engineering properties of the silty clay to clay with depth is presented on Figure 4, which includes the measured undrained shear strengths, natural water contents and Atterberg limits.

4.2.4 Sand

A thin layer of sand, about 0.2 m thick, separates the weathered silty clay and the unweathered clay at Borehole 06-18 at a depth of about 3.5 m.

4.2.5 Silty Sand to Sandy Silt Till

A deposit of glacial till was encountered below the unweathered silty clay in Borehole 06-17. The surface of this till deposit was encountered at about elevation 67.3 m in this borehole (at a depth below ground surface of about 6.8 m) and the glacial till deposit is about 1.3 m thick.

The deposit was not encountered in Borehole 06-18.

Based on local experience and observations of the drilling resistance, the glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand to sandy silt, with a trace of clay. Rotary diamond drilling techniques were required to penetrate the boulders in the till.

4.2.6 Limestone Bedrock

The bedrock encountered at the abutment widenings consists of limestone with thin shale interbeds.

The following table summarizes the bedrock surface depths and elevations as encountered at the locations of Boreholes 06-17 and 06-18, and as encountered at the previous boreholes 1 to 6. The bedrock was cored in all eight of these boreholes.

Borehole Location	Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
West Abutment	06-17	74.1	8.1	66.0
	6	75.2	8.7	66.5
	4	74.5	8.9	65.6
	2	73.3	8.4	64.9
East Abutment	06-18	73.7	7.2	66.5
	5	73.5	6.9	66.6
	3	74.5	8.5	66.0
	1	73.6	8.6	65.0



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

The limestone bedrock at the site is a member of the Gull River Formation; it is medium-strong and thinly to medium-bedded. Thin shale interbeds were also present in the rock core. Rock Quality Designation (RQD) values measured on recovered bedrock core samples typically ranged from about 25 to 94 percent, generally increasing with depth. The lowest RQD values were recorded for the upper 0.5 and 1.5 m in Boreholes 06-17 and 06-18, respectively. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although some vertical fracturing was noted. 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 06-18, sealed within the bedrock. The water level measured in that piezometer is summarized in the following table:

Borehole Number	Borehole Location	Date	Depth (m)	Elevation (m)
06-18	East abutment	June 12, 2006	3.0	70.7

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



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

5.0 CLOSURE

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

GOLDER ASSOCIATES LTD.

Kim Lesage, P.Eng.
Geotechnical Engineer



Bill Cavers, P.Eng.
Associate, Geotechnical Engineer

Fintan Heffernan, P.Eng.
Designated MTO Foundations Contact



WC/KSL/MIC/FJH/ob

\\golder.gds\gal\ottawa\active\2005\1120\geotechnical\05-1120-210 mrc hwy 417 bridges maitland to island park drive\foundations\05-1120-210-2000-5 rpt-005 - fidr feb 2017.docx

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation.



PART B

**FOUNDATION DESIGN REPORT
MERIVALE ROAD OVERPASS BRIDGE WIDENING
STRUCTURE SITE 3-47
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 Merivale Road overpass structure on Highway 417 in Ottawa, Ontario, including recommendations for the structure foundations, approach embankments and related earth/rock works. 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 Merivale Road overpass bridge is a single span rigid frame structure supported on piles founded on bedrock. The proposed work is to include rehabilitation of the existing structure. The existing abutments will also be extended to the south by about 5 m and new retaining walls will be constructed on the south side of the bridge. In addition, the roadway profile of Merivale Road will be lowered by about 200 mm to maintain the existing vertical clearance between Merivale Road and the widened overpass structure.

Bridge retaining walls extending 11 to 12 m 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 5 m. Embankments with 2H:1V side slopes extending beyond and parallel to the ends of the retaining walls are also understood to be proposed.

The bridge widening is being designed in accordance with the 2006 Canadian Highway Bridge Design Code (CHBDC).

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 silty clay to clay soils.
- Foundations supported on steel H-piles founded on, or socketted into, the bedrock.
- Foundations supported on caissons founded on, or socketted into, the bedrock.

Spread footings supported on the underlying glacial till or bedrock, or on engineered fill supported on the glacial till or bedrock, have not been considered as a feasible or practical option due to the about 7 m 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 of these intermediate to highly plastic soils would be insufficient to support the anticipated abutment loads and the settlement of the foundations would be excessive. The settlement of abutment widenings supported on shallow foundations would be entirely differential with respect to the existing pile supported abutments. Detailing of the structural elements to accommodate the anticipated settlements may not be feasible.



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

Geotechnical recommendations for the design of foundations for the bridge abutments are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the 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 abutments. Based on the existing grade at about Elevation 74 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 72 m. The bedrock surface is at about Elevation 66 m and the pile length will therefore be about 5 to 6 m.

The existing pile caps are stepped along their length and the elevation of the underside of the pile caps ranges from about 71.8 to 72.9 m. It is understood that the foundations for the widenings will be placed at about the same elevation as the adjacent portions of the existing foundations. Slight differences in pile cap elevation, between the existing and widened abutments, should also be acceptable since the existing foundation loads are fully supported by the piles.

It could also 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.

Pile installation should be in accordance with the Ontario Provincial Standard Specifications, Provincial Oriented, (OPSS.PROV) 903 (Deep Foundations). For this site, the piles will essentially be driven to practical refusal on (or within) 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 wingwalls. 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. Additional recommendations are provided in section 6.7.1 for the pile driving in the area of the southwest footing in relation to the vibration levels that may occur at the existing storm sewer tunnel.



The piles for the widened abutments may need to be driven in close proximity to the battered piles supporting the existing abutments and wing walls, although this is not anticipated to be the case based on the 90 percent construction drawings. It is understood that the new piles have been placed to avoid conflict with the existing. The following recommendations are therefore intended to be used only if the location of the proposed piles changes significantly. 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 75 mm and the deviation from the design inclination to 2 percent. 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 percent of the pile length. If the spacing between the new and existing piles is less than this tolerance, there would be a potential 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 percent of the pile length) the driving operations should be continuously monitored by the QVE and the contractor should 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.

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 the design of piles driven to found on (or within) 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 resistances for the piles rather than geotechnical resistances. 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 240 kN; in accordance with the CHBDC, a resistance factor of 0.3 is to be applied.

There is some potential for piles to 'hang up' on boulders in the glacial till. However, given the short length of these piles (such that most of the driving energy will reach the toe and the piles are quite likely to displace boulders) and the limited thickness of the glacial till, it is considered that the risk of the piles not achieving the design capacity is relatively low. If a pile did 'hang-up', and could not be driven to achieve the design capacity, additional piles could be installed (with or without extracting the previously driven pile).



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

The glacial till layer in the boreholes at the widening is indicated to be relatively thin, ranging from non-existent to about 1.3 metres in thickness, and rotary diamond drilling was required to penetrate past boulders in the deposit. Based on these conditions, it is recommended that Titus standard bearing points would be preferred for the vertical piles (rather than Type I flange reinforcement per OPSD 3000.100). For battered piles, injector points (e.g., Titus HD Rock Injector) should be used to ensure adequate seating of the piles on the bedrock and to therefore reduce the risk of the battered pile tips deflecting along the bedrock surface.

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 to be in excess of 200 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 250 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 understood that the piles supporting the existing bridge are equivalent in size to HP 310 x 79 and that the front and back rows of these piles are likely battered at 3V:1H. The front and middle rows of piles of the existing retaining walls are understood to be battered at between 6V:1H and 3V:1H while the back row is vertical. 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 14.1 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 1,300 kN. This value represents a structural limitation for the piles rather than a geotechnical limitation. The geotechnical resistance at 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.

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. The unfactored downdrag load acting on a single existing pile (assumed equivalent to a HP 310 x 79) over the length of pile within the native soils is estimated to be 250 kN. In the absence of information on the construction sequence used to design the existing bridges, it can be assumed that this magnitude of loading is already acting on all of the existing piles. The estimated 250 kN is a limiting value and is not to be considered as an addition to any existing downdrag load which may be acting on the piles.



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

The potential downdrag force is limited by the shear resistance of the soil-pile interface and therefore further settlement/movement of the soil would not result in an increase to any existing downdrag forces.

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:

$$k_h = \frac{n_h z}{B}$$

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

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where: k_h is the coefficient of horizontal subgrade reaction;
 s_u is the undrained shear strength of the soil (kPa); and,
 B is the pile diameter/width (m)

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

SLS		
Soil Unit	n_h	s_u
Weathered silty clay and clay crust above Elevation 70 m – Approximately 2 m thick	–	100 kPa
Unweathered silty clay and clay below Elevation 70 m:		
East Widening: Above Elevation 66.5 m	–	52 kPa
West Widening: Above Elevation 67.3 m	–	42 kPa
Glacial till below Elevation 67.3 m:		
West Widening (only): Approximately 1.3 m thick between about Elevation 67.3 and Elevation 66.0	4 MPa/m	–

Note: Underside of pile cap level assumed to be at about Elevation 72 m.



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

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

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

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

The ULS *geotechnical* resistance to lateral loading (acting over a bearing width of one pile diameter) may be calculated using the 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 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 following values for S_u may be assumed for estimating the ULS geotechnical lateral resistances:

ULS	
Soil Unit	S_u (kPa)
Weathered silty clay and clay crust above Elevation 70 m – Approximately 2 m thick	80 kPa
Unweathered clay below about Elevation 70 m:	
East Widening: Above Elevation 66.0 m	52
West Widening: Above Elevation 67.3 m	42

Note: Underside of pile cap assumed to be at about Elevation 72 m.



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 79 piles. These values are based on the "Assessed Horizontal Passive Resistance and Geotechnical Reaction at SLS" provided in Table C6.4 of the *Commentary* to the CHBDC.

Additional lateral resistance can be provided by socketing the piles into the bedrock. For example, for piles socketed at least 1.2 m into bedrock within a 600 mm diameter concrete filled socket (i.e., socketed for a depth of at least twice the socket diameter), the ultimate (unfactored) lateral 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.

It is understood that the profile grade of Merivale Road 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 and widening foundations.

The 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 4 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).

6.2.2 Caisson Foundations

Caissons founded on or socketted into the limestone bedrock may be used for support of the abutments. Based on the existing grade at about Elevation 74 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 72 m. The bedrock surface is at about Elevation 66 m and the caisson length (to the rock surface) will therefore be about 5 to 6 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 temporary 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 socketted into the bedrock; once disturbed, the sensitive clay soils, as well as the sandy silt till, could flow under the casings at the interface with the bedrock. It is therefore recommended to socket the caissons into the rock, rather than found directly on the bedrock surface.

The limestone bedrock at the site is moderately strong. If socketting of the caissons into the bedrock is required, the sockets may 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 socketted nominally (e.g., 0.5 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 nominally socketted in the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.



The upper 0.4 to 1.4 m of bedrock is indicated to be of poor quality, with RQD values of less than 25 percent. OPSS.PROV 903 (Deep Foundations) should be included in the Contract Documents requiring inspection and approval of the caisson hole and bearing area by the Quality Verification Engineer prior to caisson construction. It may be necessary to advance the caisson into the better quality rock 0.4 to 1.4 m below the bedrock surface (or a reduced bearing resistance can be used).

The caisson can alternatively be socketed to greater depth and designed based on the shear resistance along the socket. If the caissons were socketed to a depth of at least twice the socket diameter (in addition to whatever depth is needed to penetrate past the upper fractured bedrock), they could be designed using a factored unit side resistance in the rock sockets (below the upper fractured bedrock) of 1 MPa. A minimum socket diameter of 0.9 m is recommended in this case. It should be noted that more effort would also be required with constructing, cleaning, and inspecting the sockets for this design option.

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 retaining 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 600 or 950 kN, respectively (linear interpolation can be used for other diameters). 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.

It is understood that the profile grade of Merivale Road 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 and widening foundations.

The 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 5 in Appendix E. 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).

6.3 Bridge Retaining Wall Foundation Options

Bridge retaining walls are planned to be constructed adjacent to both of the abutment widenings and extending 14 to 15 m back from the abutments. The following options have been considered for the 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.



Supporting the retaining walls on spread footings founded on the underlying glacial till or bedrock, or on engineered fill supported on the glacial till or bedrock, is not considered to be a feasible or practical option due to the 5 to 6 m deep excavation that would be required.

The first option (i.e., using shallow foundations supported on the native soils) is not considered practical or appropriate for this site since, as discussed further in Section 6.6.3, embankment subgrade settlements in excess of 200 mm may occur beneath the widenings, and this would also apply for shallow foundations. 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 existing 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 retaining 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

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 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.

6.5 Lateral Earth Pressures for Design

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

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

- Select free-draining granular fill meeting the specifications of the provincial version of the Ontario Provincial Standard Specifications (OPSS.PROV) Granular 'A' or Granular 'B' should be used as backfill behind the walls. This fill should be compacted in accordance with OPSS.PROV 501 (Compacting). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3101.150 and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the abutment stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501 (Compacting). Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with a width equal to at least 1.8 m behind the back of the abutment stem (Case I) 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).
- 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:



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

	Granular 'A'	Granular 'B' Type II
Soil Unit Weight	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_p	3.70	3.70

- 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 percent 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



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

- 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:

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

Where: $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d, (kPa)

K is the static active earth pressure coefficient, K_a (to be used for yielding walls);

K is the static at-rest earth pressure coefficient, K_o (to be used for non-yielding walls);

K_{AE} is the seismic active earth pressure coefficient;

γ is the unit weight of the backfill soil (kN/m^3), as given previously;

d is the depth below the top of the wall (m); and,

H is the total height of the wall (m).

The appropriate values of K (K_a or K_o) and K_{EA} are provided above.

6.6 Approach Embankment Design and Construction

Embankment widening beyond the ends of the two southern 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 topsoil layer at the west widening at Borehole 06-17. The existing fill materials range in composition from silty clay to sand. These materials are underlain by silty clay to clay, which is in turn underlain by glacial till at the west widening and limestone bedrock at the east widening.

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 from the existing embankment sideslope and the new footprint. All subgrade soils should be proof-rolled prior to fill placement.

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

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 OPSS.PROV 501 (Compacting). 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. An alternative to this would be to use Turf Reinforcement Mats.

6.6.2 Approach Embankment and Retaining Wall Stability

Static and seismic (pseudo-static) slope stability analyses were completed for the proposed approach embankments and retaining walls using the commercially available slope stability analysis software, SlopeW (produced by Geo-Slope International Ltd.). The results of the slope stability analysis are provided on Figures 6 to 9 in Appendix E.

Retaining walls up to 5 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 bridge 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 4 to 5 m high approach embankments, beyond the ends of the retaining walls, 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 carried out for the above configuration 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 retaining wall and slope stability analyses were carried out using the following parameters:

Material	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Earth or Granular Embankment Fill	21	32°	
Existing Sand Fill	19	29°	
Existing Silty Clay Fill	17.5		50
Topsoil	18		20
Weathered Silty Clay	17.5		80
Silty Clay	16.5		40
Till	22	32°	



6.6.3 Approach Embankment Settlement

It is understood that up to about 5 m height of additional fill will need to be placed in order to construct the embankment widenings. Settlement of the approach embankment widenings will occur as a result of compression of the new embankment fill itself, as well as consolidation of the clayey soils on which the approaches will be founded.

Provided that the embankment material consists of select subgrade material or clean earth fill, the settlement of the embankment fill itself is expected to be less than 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 1.2 to 1.3 m of existing fill materials and buried topsoil. The subgrade settlement due to compression of the existing fill materials and topsoil should be minor in magnitude provided the subgrade surface is proof-rolled. Most of this settlement would also likely occur during construction.

Settlement of the embankment subgrade can be expected due to compression of the clay soils (i.e., the weathered clay crust, and in particular of the underlying grey silty clay to clay). The effective stress level in the clayey deposits will likely exceed the deposit's preconsolidation pressure based on the results of oedometer consolidation testing carried out elsewhere on this project (see Appendix F). The resulting consolidation settlements therefore correspond to recompression of the clayey deposits and consolidation in the virgin compression range.

The total estimated magnitude of the primary consolidation settlement ranges from about 70 to 120 mm. It is estimated that most of the primary consolidation settlement should be completed within about six to eighteen months.

Up to about 50 to 100 mm of additional secondary compression is anticipated over a twenty-year time span, by which time resurfacing of the highway might be expected.

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 220 mm.

To address these large expected settlements, one option would be to construct the approach embankments 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. This maintenance is considered to probably be unacceptable, and therefore the following additional options have therefore been considered for mitigation of post-construction settlement:

- 1) Excavate the silty clay and replace with engineered fill.
- 2) Preload the widened embankment and allow the settlements to occur prior to paving.
- 3) Preload and surcharge the widened embankment to increase the magnitude of settlement during the preload period, prior to paving.
- 4) Install wick drains to accelerate the consolidation settlement within the silty clay to clay.
- 5) Employ lightweight fill in the construction of the approach embankments to reduce the magnitude of primary consolidation settlement.



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

6) Provide retaining wall foundations of sufficient width to support the entire weight of additional fill material.

The first option, of 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.

The second option, of 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 sufficiently reduce the long term settlement magnitudes due to secondary compression.

For the third option, of preloading with a surcharge, preliminary analyses indicate that a 1.5 m high surcharge would result in approximately 200 mm of primary consolidation settlement within about 3 to 6 months. This program would therefore limit the post-paving settlement of the highway to less than 25 mm. The feasibility of constructing this surcharge adjacent to the operating highway would, however, need to be reviewed. It may also not be feasible to place a surcharge in the area directly behind the new abutment widening. If this option is to be considered further, a more detailed assessment of the settlements would need to be carried out based on the actual surcharge geometry which can feasibly be achieved, given the space constraints, to confirm the suitability of this option, and whether sufficient settlements could actually be achieved.

For the fourth option, 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. This option is considered only marginally feasible.

For the fifth option, the amount of time-dependent settlement and the associated roadway maintenance may be reduced by employing lightweight fill materials below the pavement structure. The lightweight fill would be used in place of conventional earth fill to reduce the applied loading to keep the stress level in the clay to below the pre-consolidation range.

Four types of lightweight fill are available for use:

- Extruded polystyrene (EPS) fill, with a bulk unit weight of less than 1 kN/m³;
- Cellular concrete, with a bulk unit weight ranging from about 4 to 8 kN/m³;
- Ultra-lightweight slag fill from Hamilton (Litex), with a bulk unit weight ranging from about 9 to 12 kN/m³; and,
- Lightweight slag fill (Superior Slag) from Sault Ste. Marie, with a bulk unit weight of about 14 kN/m³.

If, for example, EPS fill is adopted for construction of the approach embankments, a 2.5 m 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.



Cellular concrete or ultra-lightweight slag fill (Litex) could also be used to construct the embankment. Constructing the widened embankment entirely with cellular concrete or 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 and 50 mm, respectively. However, cellular concrete and ultra-lightweight slag fill is relatively costly, particularly in small quantities, and this option may be uneconomical.

Lightweight slag fill would not have a sufficiently low unit weight to be effective at reducing the settlements.

The sixth option would involve using the bridge structure foundations to support the weight of the widened embankment, at least in the area immediately behind the abutment. The preliminary widened embankment cross sections indicate that the greatest fill thicknesses may be placed in close proximity to the bridge retaining 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 actually be much less and the resulting settlements may be less than 25 mm.

The final foundation geometry may therefore 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.

If the settlements are not entirely avoided, such as by the use of lightweight fill or by structurally supporting the embankment, then downdrag forces would be induced on the new foundations, as described in Sections 6.2.1.1 and 6.2.2.1. New downdrag loads could also be induced on the existing piles, as described in Section 6.2.1.1. However, it should also be considered that downdrag loads could be imposed on the faces of the south wingwall which it is understood will remain in-place and be buried within the widened embankment. Further guidelines on this issue can be provided, if required.

6.7 Design and Construction Considerations

6.7.1 Existing Utilities

There are several utilities in the area of the proposed bridge widenings. If the settlements discussed in Section 6.6.3 will not be mitigated by the use of lightweight fill or structural support, the impact of the potential settlements to the existing utilities will need to be considered. The impact should be limited to a distance of about 3 m (in plan) from the footprint of the widening.

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing utilities are maintained below tolerable levels. Particular measures are recommended for the existing deep storm sewer (the Cave Creek collector sewer) which extends beneath the location of the southwest widening. Based on the drawings supplied by MMM, the obvert of the sewer is about 8.3 m below the surface of the limestone bedrock (and the bearing elevation of the piles).



The possible impacts to the sewer from the driving of the piles and the imposed pile loads are:

- Increased stress on the sewer lining from the pile loads; and,
- Damage to the sewer lining from the vibrations that will occur during driving.

The increase in stress on the sewer lining from the loads supported by the end bearing piles is likely to be low, considering the significant sewer depth below the surface of the rock; those forces should be significantly distributed and reduced by the tunnel level. However, the actual magnitude of those stresses could only be determined by more rigorous analysis (i.e., finite element modelling). The City of Ottawa may require that such analysis be carried out.

The potential for vibration damage to the sewer lining is likely a more significant risk for this project. Driving of the piles immediately above the sewer (i.e., for the southwest widening) could generate significant vibrations within the rock, especially once the piles reach the bedrock and are driven to meet the set criteria (i.e., after initial seating on the bedrock surface). The owner of the sewer should be consulted for their assessment of the potential restrictions on the vibrations experienced by this sewer (which will depend to a large extent on the sewer age, condition and liner type). In the absence of any information on the sewer condition, and as a preliminary guideline, a peak particle velocity of 50 mm/s may provide a conservative limit for the storm sewer. Vibration monitoring at the sewer level (either directly on the storm sewer or of the bedrock at the sewer level/elevation) should be carried out during pile installation for the southwest widening to ensure that the vibration levels at the existing storm sewer are maintained below tolerable levels. The piles furthest from the storm sewer should be driven first, in order to check the vibration level at the sewer and, if necessary, the pile driving criteria can be altered for the remaining piles for the southwest widening.

To limit the vibrations within the rock and at the sewer level, the piles should not be overdriven. Furthermore, the hammer energy should be reduced when the pile tip is approaching the bedrock surface elevation (i.e., reduce energy starting at about Elevation 67.5 m) and a reduced set criteria may also be used to limit vibrations at the sewer during seating of the pile on the rock surface. CAPWAP analyses should be carried out for any piles that need to be set using reduced energy. If the CAPWAP analyses indicate that the full capacity of the pile cannot be mobilized, then additional piles may be required. The final CAPWAP report should be stamped by an engineer licensed in the province of Ontario. It is recommended that the piling operations and vibration monitoring (in the area of the southwest footing) should be inspected on a full time basis by geotechnical personnel working for a firm registered in RAQS in the prime specialty of: Foundation Engineering: Geotechnical (Structures and Embankments) – High Complexity. A sample NSSP is included in Appendix E for pile driving over the storm sewer.

Pre and post construction surveys of the deep storm sewer should be carried out in addition to vibration monitoring.

6.7.2 Existing Wing Walls – Frost Protection

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



6.7.3 Excavation

Given that the pile caps will require a minimum of 1.8 m of earth cover as frost protection, it is anticipated that excavation will extend to about 2 m depth (relative to the grade in front of the abutments). That excavation would therefore extend through the surficial fill material and topsoil, extending into the weathered clay to silty clay crust. The groundwater level is indicated to be at a depth of about 3.0 m, at about Elevation 70.7 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 above the groundwater level and the weathered silty clay underlying the fill materials are classified as Type 3 soil. Temporary excavations (i.e. those which are only open for a relatively short period) through the fill materials and weathered silty clay to clay should therefore 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.

6.7.4 Temporary Shoring

It is anticipated that temporary roadway protection (i.e., excavation shoring) will be required along Highway 417 to permit construction of the abutment widenings, and will likely also be required along Merivale Road 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 such that it could be used 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 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/below the shoring.

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

The temporary roadway protection should be in accordance with OPSS.PROV 539 (Temporary Protection Systems) and should be designed to Performance Level 2 as defined in OPSS.PROV 539, provided that any buried utilities that may be present adjacent to the excavations(s) can tolerate this magnitude of deformation.

6.7.5 Groundwater and Surface Water Control

The groundwater level at the site was measured at about 3.0 m depth below the natural ground surface. Excavations for the construction of the pile caps will therefore 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 excavation.



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

7.0 CLOSURE

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

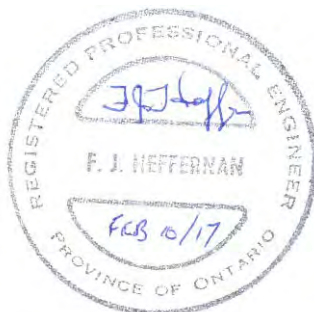
GOLDER ASSOCIATE

Kim Lesage, P.Eng.
Geotechnical Engineer



Bill Cavers, P.Eng.
Associate, Geotechnical Engineer

Fintan Heffernan, P.Eng.
Designated MTO Foundations Contact



WC/KSL/MIC/FJH/ob

\\golder.gds\gallottawa\active\2005\1120\geotechnical\05-1120-210-2000-5 rpt-005 - fidr feb 2017.docx

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation.



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

Table 1
Comparison of Foundation Alternatives

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 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 bridge construction or 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 could 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"> Less expensive than caisson option 	<ul style="list-style-type: none"> Possibility of piles being driven misaligned due to boulders in glacial till Possibility of contact between new and existing piles during driving
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"> High bearing resistance Negligible settlement Compatible with existing bridge construction or foundations 	<ul style="list-style-type: none"> Temporary liners required to minimize disturbance to surrounding soils Possibility of encountering cobbles or boulders during installation Socketting 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 Possible need to advance past upper fractured bedrock or to use reduced bearing resistance 	<ul style="list-style-type: none"> More expensive than steel H-pile option 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



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

Table 2
Comparison of Bridge Retaining Wall Alternatives

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 could 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"> Less expensive than caisson option 	<ul style="list-style-type: none"> Possibility of piles being driven misaligned due to boulders in glacial till Possibility of contact between new and existing piles during driving
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 Possible need to advance past upper fractured bedrock or to use reduced bearing resistance 	<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



FOUNDATION REPORT MERIVALE ROAD OVERPASS BRIDGE WIDENING

Table 3
Comparison of Settlement Mitigation Alternatives

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"> Not feasible, due to excessive settlements 	<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
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"> Marginally Feasible 	<ul style="list-style-type: none"> Minimal post-construction maintenance 	<ul style="list-style-type: none"> Could delay paving 	<ul style="list-style-type: none"> Slightly higher cost 	<ul style="list-style-type: none"> Some uncertainty about paving schedule
Pre-loading with wick-drains	<ul style="list-style-type: none"> Marginally Feasible May not be feasible directly behind abutment widening 	<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
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 – Merivale Road, Borehole Locations

Drawing 2 – Merivale Road, Soil Strata

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

HWY. 417

WP No. WP 4058-01-00

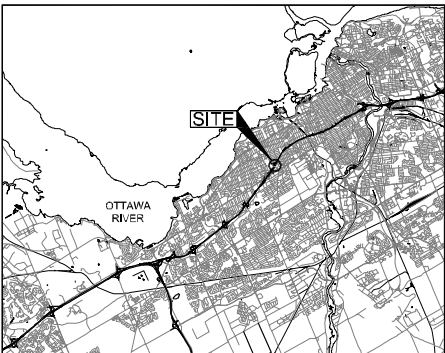
MERIVALE ROAD
BOREHOLE LOCATIONS



SHEET



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole - Current Golder Associates Ltd. Investigation
- Borehole - Previous MTO Investigation Goecres No. 58-F-229-C
- Location of cross-section

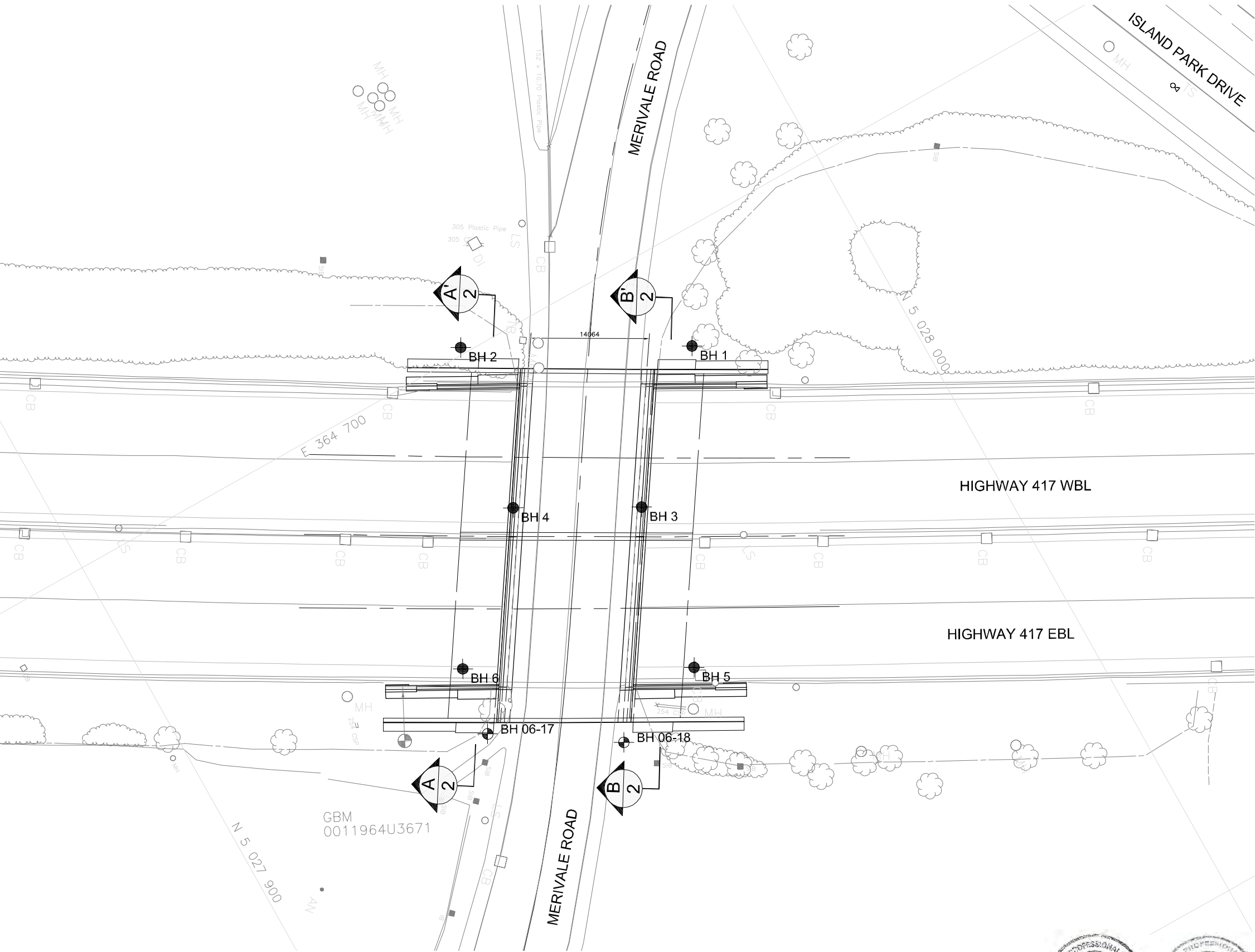
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-17	74.1	5027932.3	364740.8
06-18	73.7	5027945.9	364749.6
1	73.6	5027975.0	364716.5
2	73.3	5027952.0	364701.6
3	74.5	5027959.6	364729.2
4	74.5	5027946.8	364720.9
5	73.5	5027954.4	364748.5
6	75.2	5027931.3	364733.6

NOTES

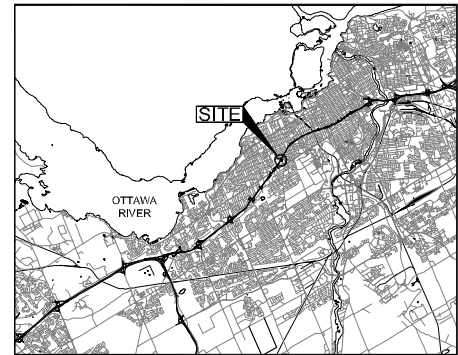
This drawing is for subsurface information only. Any surface details are for conceptual illustration. The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence. Base plan provided in electronic format by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

NO.	DATE	BY	REVISION

Geocres No. 3105-271			
HWY. 417	PROJECT NO. 05-1120-210-2000		DIST. EASTERN
SUBM'D. W.C.	CHKD. K.S.L.	DATE: SEPTEMBER 2006	SITE:
DRAWN: J.M.	CHKD. W.C.	APPD. F.J.H.	DWG. 1








05-1120-210-5000-01.dwg



KEY PLAN

LEGEND

-  Borehole – Current Golder Associates Ltd. Investigation
-  Borehole – Previous MTO Investigation Goecres No. 58-F-229-C
-  Seal
-  Piezometer
- N Standard Penetration Test value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
-  WL in piezometer, measured on June 12, 2006

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-17	74.1	5027932.3	364740.8
06-18	73.7	5027945.9	364749.6
1	73.6	5027975.0	364716.5
2	73.3	5027952.0	364701.6
3	74.5	5027959.6	364729.2
4	74.5	5027946.8	364720.9
5	73.5	5027954.4	364748.5
6	75.2	5027931.3	364733.6

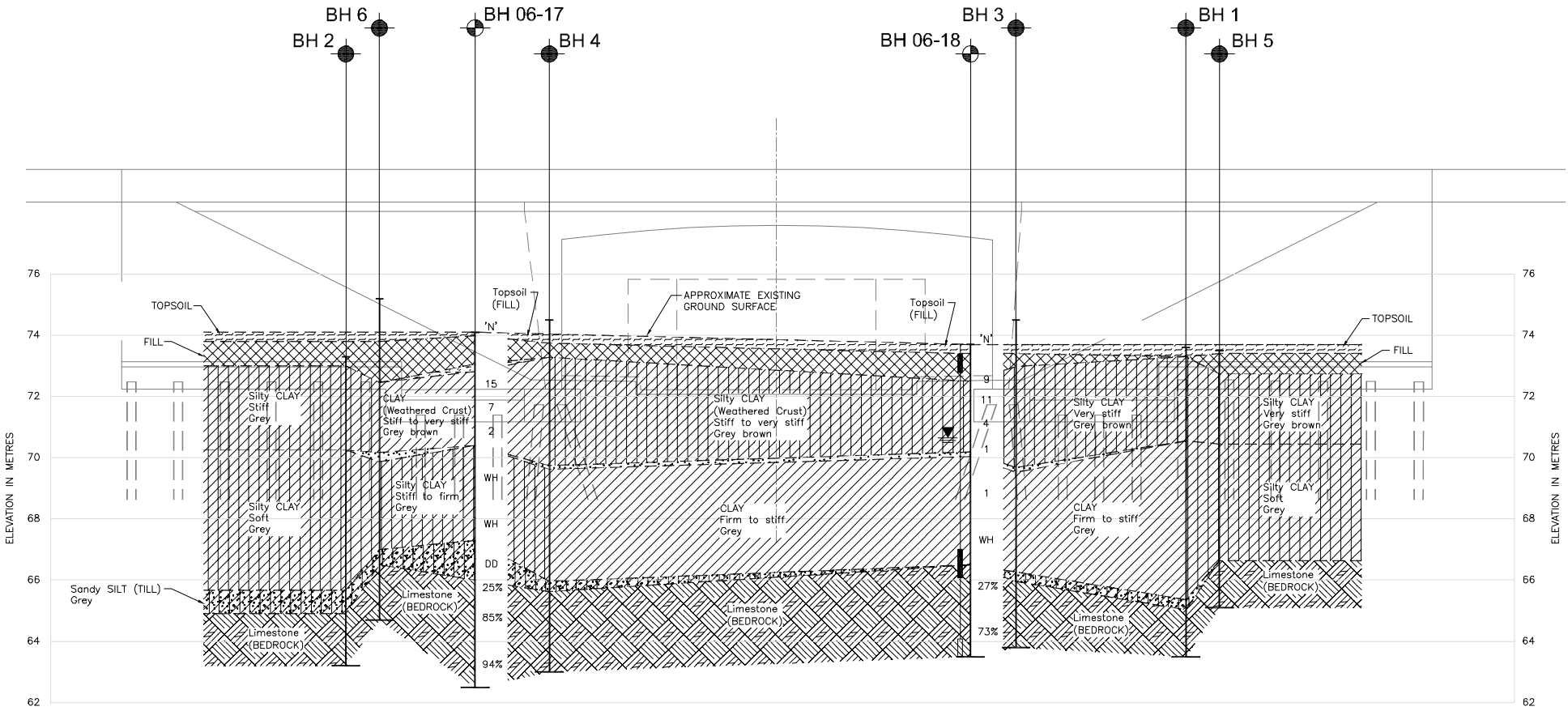
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

NOTES

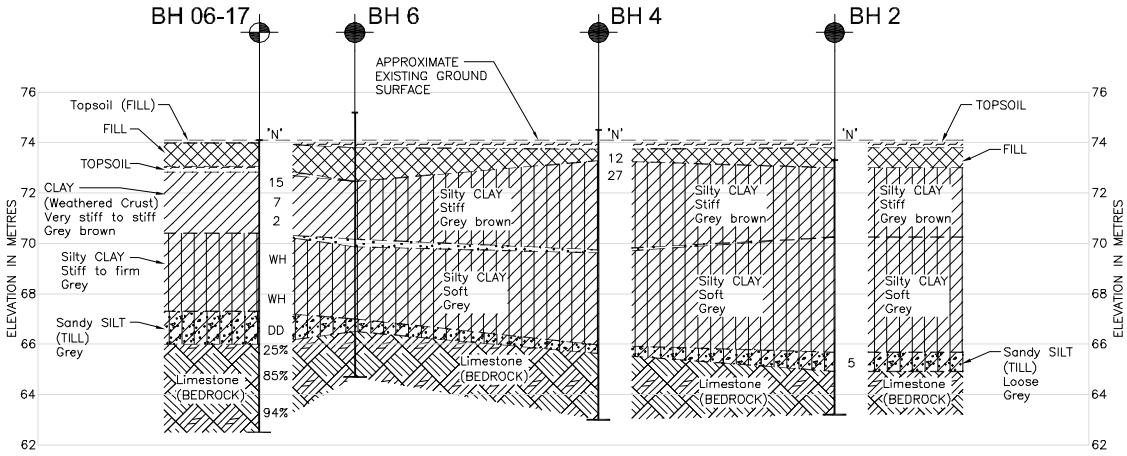
This drawing is for subsurface information only. Any surface details are for conceptual illustration. The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence. Base plan provided in electronic format by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

NO.	DATE	BY	REVISION

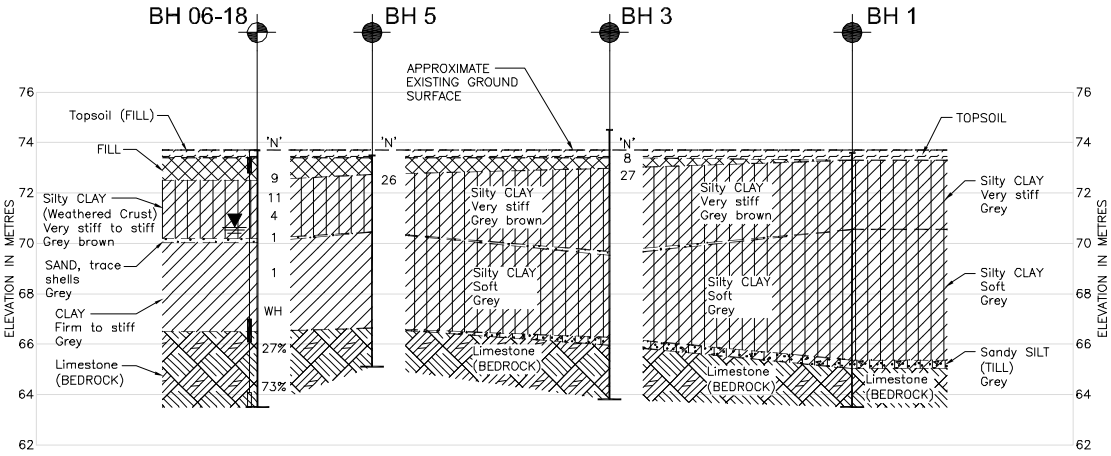
Geocres No. 3165-271		PROJECT NO. 05-1120-210-2000		DIST. EASTERN	
HWY. 417		DATE: SEPTEMBER 2006		SITE:	
SUBM'D. W.C.		CHKD. W.C.		APPD. F.J.H.	
DRAWN: J.M.		DWG. 2			



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 06-17 to 06-18



LIST OF SYMBOLS

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

I. GENERAL

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

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 stress (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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$
$$\text{shear strength} = (\text{compressive strength})/2$$



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

	C_u, S_u	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS 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 and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
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	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

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

CORE CONDITION

Total Core Recovery (TCR)

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 varied from 0% for completely broken core to 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 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 abbreviation 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

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	

PROJECT 05-1120-210-2000		RECORD OF BOREHOLE No 06-17		SHEET 1 OF 1		METRIC													
G.W.P. 4058-01-00		LOCATION N 5027932.3; E 364740.8		ORIGINATED BY D.J.S.															
DIST Eastern HWY		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			WATER CONTENT (%)			γ			GR SA SI CL		
74.1	GROUND SURFACE							20 40 60 80 100	○ UNCONFINED + FIELD VANE	W _p	W	W _L							
0.0	Topsoil (FILL)						74	20 40 60 80 100	● QUICK TRIAXIAL × REMOULDED	25 50 75									
0.1	Silty clay, trace gravel (FILL) Grey brown																		
73.1	TOPSOIL						73												
1.3	CLAY (Weathered Crust) Very stiff to stiff Grey brown Moist to wet		1	SS	15		72												
			2	SS	7		71												
			3	SS	2		70												
70.4	Silty CLAY Stiff to firm Grey Wet																		
3.7			4	SS	WH		69												
							68												
			5	SS	WH		67												
67.3	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Grey Wet						66												
6.8			6	NQ RC	DD		65												
66.0	Limestone with thin shale interbed (BEDROCK) Slightly weathered to fresh Grey Medium strong						64												
8.1	Bedrock cored between 8.1m 11.6m depth. For bedrock coring details refer to Record of Drillhole 06-17.						63												
			7	NQ RC	DD														
			8	NQ RC	DD														
			9	NQ RC	DD														
62.5	End of Borehole																		
11.6																			

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-17

SHEET 1 OF 1

LOCATION: N 5027932.3; E 364740.8

DRILLING DATE: May 11, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT		SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION		
									CL-CLEAVAGE		J-JOINT		R-ROUGH		UE-UNEVEN				MB-MECH. BREAK	
									SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY				B-BEDDING	
									VN-VEIN		S-SLICKENSIDED		PL-PLANAR		C-CURVED					
RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY												
TOTAL CORE %		SOLID CORE %				TYPE AND SURFACE DESCRIPTION		K, cm/sec												
80 60 40 20		80 60 40 20		80 60 40 20		5 10 15 20		DIP w.r.t. CORE AXIS		10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³										
80 60 40 20		80 60 40 20		80 60 40 20		5 10 15 20		0 30 60 90		2 4 6										
		ROCK SURFACE		66.00																
9	Rotary Drill NQ Core	Limestone with occasional thin shale interbed (BEDROCK) Slightly weathered to fresh Grey Medium strong		8.10	1															
10					2															
11					3															
12		End of Drillhole		62.50 11.60																
13																				
14																				
15																				
16																				
17																				
18																				
19																				
20																				
21																				
22																				
23																				

DEPTH SCALE

1 : 75



LOGGED: D.J.S.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-5000-ROCK GPJ GAL-MISS GDT 02/10/17 JM

PROJECT		RECORD OF BOREHOLE		No 06-18		SHEET 1 OF 1		METRIC						
G.W.P. 4058-01-00		LOCATION		N 5027945.9; E 364749.6		ORIGINATED BY		D.G.						
DIST Eastern HWY		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY		J.M.						
DATUM Geodetic		DATE		May 10, 2006		CHECKED BY		M.I.C.						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
73.7	GROUND SURFACE													
73.4	Topsoil (FILL)													
0.5	Sand, some gravel (FILL) Brown Moist		1	A.S.										
72.5	Silty clay, trace sand (FILL) Grey brown Moist		2/3	SS	9									
1.2	Silty CLAY (Weathered Crust) Stiff to very stiff Grey brown Moist to wet		4	SS	11									
			5	SS	4									
70.2			6/7	SS	1									
3.7	SAND, trace shells Grey Wet CLAY Firm to stiff Grey Wet													
			8	SS	1									
			9	SS	WH									
66.5														
7.2	Limestone with thin shale interbeds (BEDROCK) Fresh Grey Medium strong Bedrock cored between 7.2m 10.2m depth. For bedrock coring details refer to Record of Drillhole 06-18.		10	NQ RC	DD									
			11	NQ RC	DD									
63.5														
10.2	End of Borehole Note: Water level in standpipe at 3.0m depth below ground surface on June 12, 2006													

PROJECT: 05-1120-210-2000

RECORD OF DRILLHOLE: 06-18

SHEET 1 OF 1

LOCATION: N 5027945.9; E 364749.6

DRILLING DATE: May 10, 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			
									CL-CLEAVAGE			J-JOINT			R-ROUGH			UE-UNEVEN				MB-MECH. BREAK		
									SH-SHEAR			P-POLISHED			ST-STEPPED			W-WAVY				B-BEDDING		
									VN-VEIN			S-SLICKENSIDED			PL-PLANAR			C-CURVED						
RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY			DIAMETRAL POINT LOAD INDEX (MPa)															
TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴																
		ROCK SURFACE		66.50																				
8	Rotary Drill NG Core	Limestone with thin shale interbeds (BEDROCK) Fresh Grey Medium strong		7.20																				
9				1																				
10				2																				
10		End of Drillhole		63.50 10.20																				
11																								
12																								
13																								
14																								
15																								
16																								
17																								
18																								
19																								
20																								
21																								
22																								

DEPTH SCALE

1 : 75



LOGGED: D.G.

CHECKED: W.C.

MIS-RCK 001 05-1120-210-5000-ROCK GPJ GAL-MISS GDT 02/10/17 JM



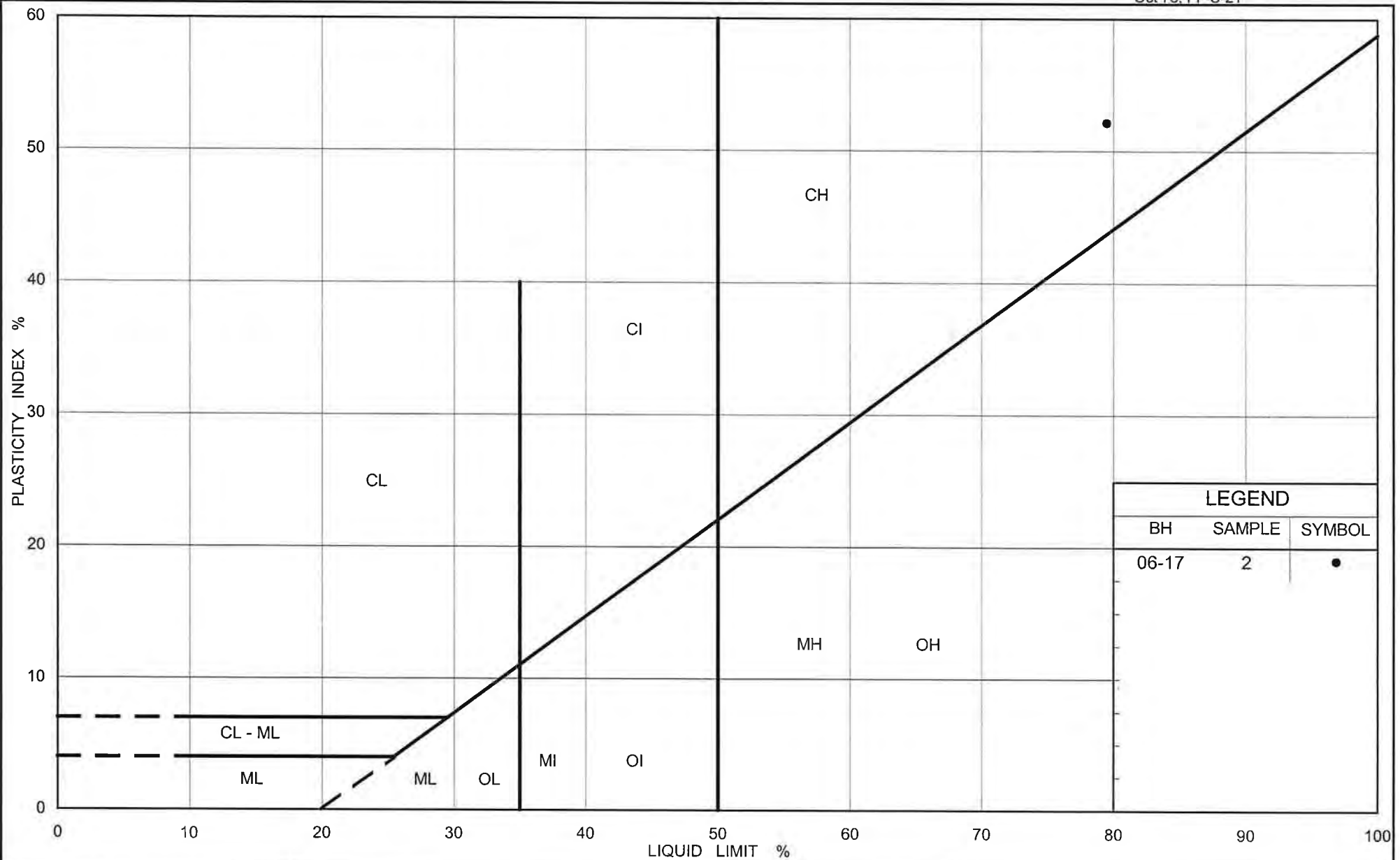
APPENDIX C

Figure 1 – Plasticity Chart – Weathered Clay

Figure 2 – Grain Size Distribution Test Results – Silty Clay

Figure 3 – Plasticity Chart – Clay to Silty Clay

Figure 4 – Summary of Engineering Properties – Silty Clay to Clay



Ministry of Transportation

Ontario

PLASTICITY CHART Weathered Clay

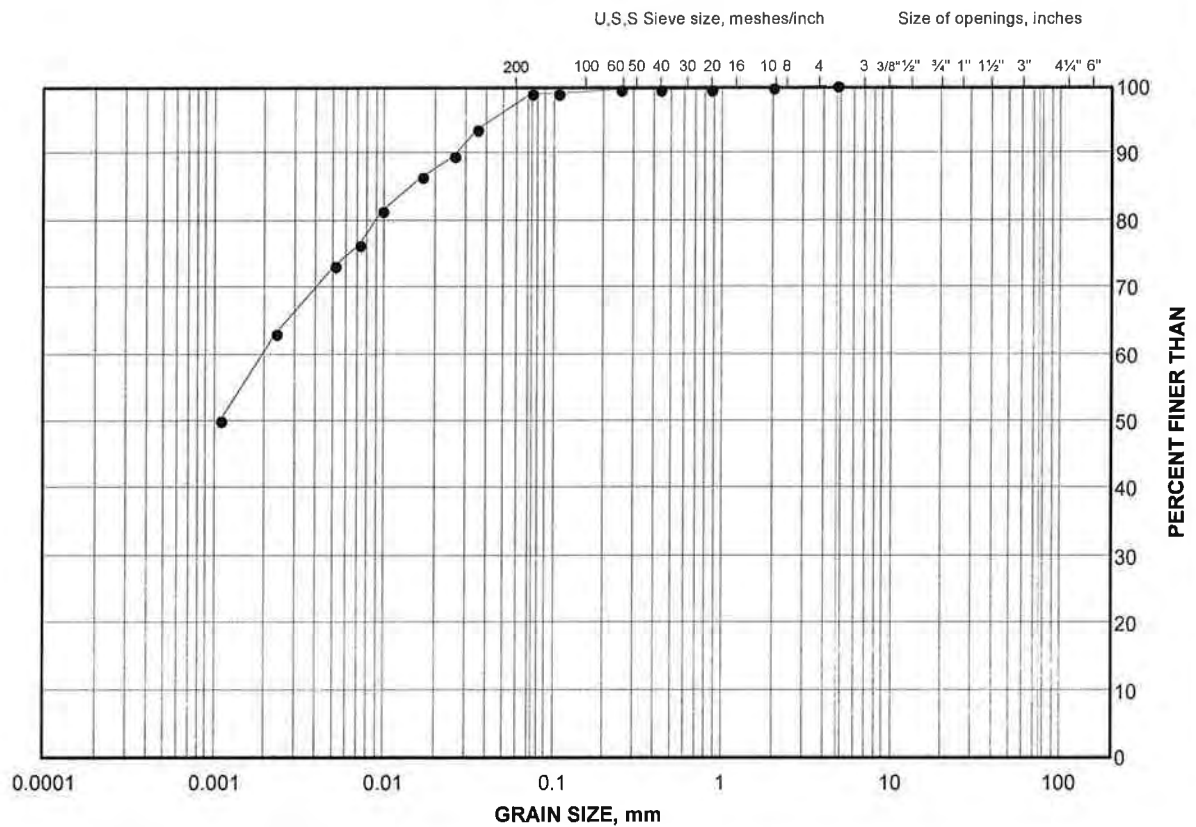
FIG No. 1

Project No. 05-1120-210 - 2700 2000

GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE 2



LEGEND

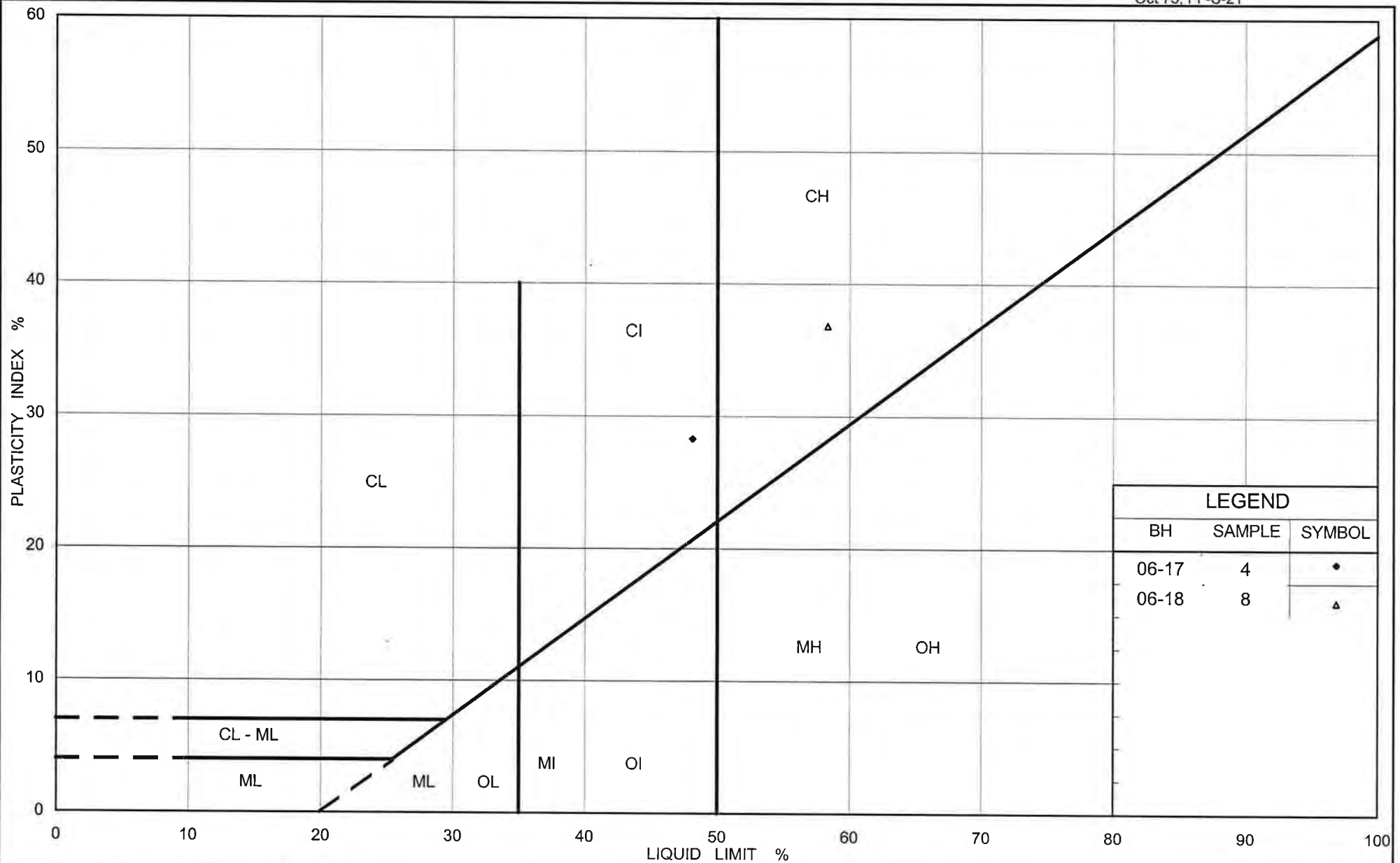
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	06-17	4	4.4-5.0

Project Number: 05-1120-210

Checked By: _____

Golder Associates

Date: 08-Jun-06



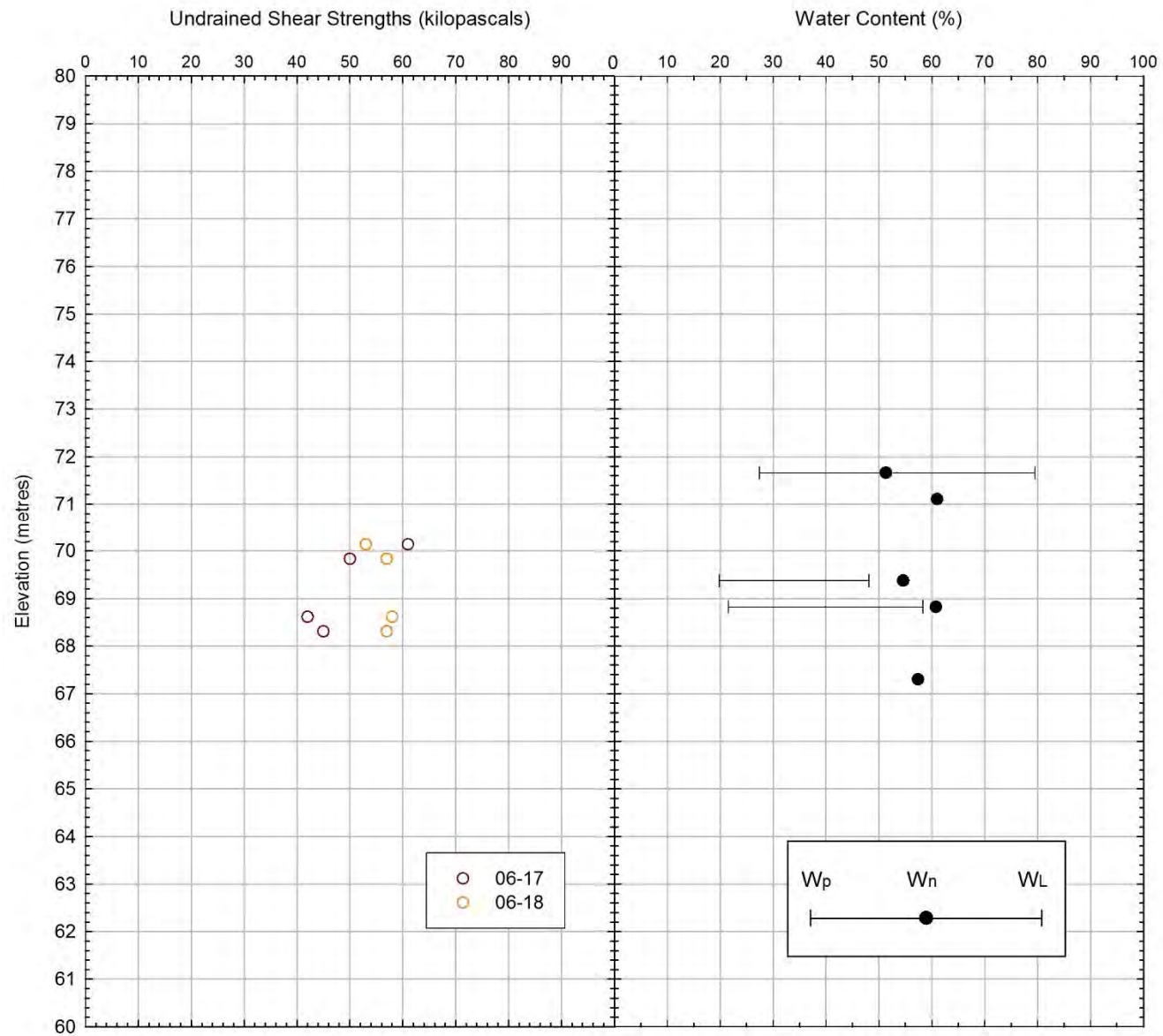
Ministry of Transportation

Ontario

PLASTICITY CHART Clay to Silty Clay

FIG No. 3

Project No. 05-1120-210 - 2700-2000



Summary of Engineering Properties

Silty Clay to Clay

Project No.	05-1120-210
Drawn:	WAM
Date:	2/8/2017
Checked:	KSL
Review:	WC

Figure 4



APPENDIX D

Records of Previous Boreholes 1 to 6 (Geocres No. 58-F-229-C)

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QWY, MERIVALE RD.

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 240.2'REMARKS See: plate #2

HOLE NO.

2

DATE 30/10/58

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 DROPINCH DIA. ROD
							BLOWS PER FOOT	
				GROUND SURFACE				
					0.0	240.2	OVERNIGHT WATER LEVEL 0.0'	
				TOP SOIL - 1.0'				
				VERY STIFF				
4.5	6.6-7.2 7.3-R-48		2-1	BROWNISH GRAY CLAY				
				STIFF BROWNISH				
				GRAY CLAY				
3.1	3.2-4.0 7.2-R-18		2-2	MEDIUM SOFT				
				BROWNISH GRAY CLAY				
1.1	2.8-1.6 1.7-R-0.2		2-3	MEDIUM SOFT				
				FISSURED GRAY	10.0	230.2		
4.2	1.4-1.1 1.6-R-0		2-4	SILTY CLAY				
				MEDIUM SOFT				
1.5	1.3-1.6 0.5-1.0 1.4-R-0		2-5	MEDIUM SOFT				
				GRAY CLAY				
1.6	1.0-0.6 1.1-1.2 1.0-R-0		2-6	WITH SANDY				
				CLAY LAYERS	20.0	220.2		
1.6	1.0-0.9 0.6-1.3 1.4-R-0		2-7	AND SILTY				
				CLAY LAYERS				
1.4	0.9-1.3 1.3-1.3 1.0-R-0		2-8	LOOSE TILL				
				MEDIUM DENSE TILL				
				SHALEY LIMESTONE	30.0	210.2		
				(CORE RECOVERY 92%) DROP				
					33.1	207.1		
				BOTTOM OF HOLE				
							0 20 40 60 80 100 % WATER CONTENT WATER CONTENT	
							PLATE 3	

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QWY / MERIVALE RD.

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 244.2'REMARKS See: plate #2.

HOLE No.

3

DATE Nov 5/53

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								
					0.0	0.0	244.2					
				FILL								
			3-1	(BROWNISH GRAY CLAY WITH SAND)								
					5.0'							
			27	HARD								
			3-2	BROWNISH GRAY CLAY								
					7.5'							
1.7	6.7-6.7 6.5-8.1.6		3-3	VERY STIFF FISSURED BROWNISH GRAY CLAY WITH SOME SILT	10.0	234.2						
					10.0							
			3-4	VERY STIFF BROWNISH GRAY CLAY WITH SOME SILT	12.5'							
1.8	5.2-4.2 3.8-1.7 R-0.4											
			3-5	MEDIUM SOFT FISSURED BROWNISH GRAY SILTY CLAY	15.8							
1.1	1.8-1.6 1.5-1.7 R-0.0											
			3-6	SAND & SILT MEDIUM SOFT FISSURED GRAY SILTY CLAY	18.2							
1.3	1.2-1.3 1.8-2.0 1.6-2.0.0											
			3-7	MEDIUM SOFT GRAY	20.0	224.2						
0.9	1.6-1.3 1.6-1.5 1.5-2.0.0											
			3-8	SILTY CLAY								
1.3	0.8-1.3 1.5-1.4 1.3-1.6 R-0.0											
			3-9	MEDIUM SOFT FISSURED GRAY CLAY SILT IN LAYERS	26.0							
0.5	0.9-1.4 1.2-1.4 R-0.0											
				TILL	28.0							
				SHALEY LIMESTONE 89%	28.7							
					30.0	214.2						
				DROP SHALEY LIMESTONE (CORE RECOVERY 88%)								
					34.9	209.3						
				BOTTOM OF HOLE								
							0	20	40	60	80	100
							% WATER CONTENT					PLATE
							① MOISTURE CONTENT					4

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QWY, MERIVALE RD.

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 244.3'

HOLE No.

REMARKS See plate #2 FOR MECHANICAL ANALYSIS SEE

4

PLATES 8, 29

DATE

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'	244.3						
				FILL								
		12	4-1		4.0'							
		27	4-2	HARD BROWNISH GRAY CLAY WITH								
			4-3	A LITTLE SILT								
3.7	8.9-7.8 R-20				10'	234.3						
2.1	4.0-3.5 2.6 R-0.4		4-4	STIFF SILTY BROWNISH GRAY CLAY								
1.2	1.9-2.1 2.5-0.8 R-0.0		4-5	MEDIUM SOFT BROWNISH GRAY CLAY WITH A LITTLE SILT								
	3.0-1.3 1.8-0.7				15.0'							
0.8	3.1-1.8 R-0.0		4-6	SOFT TO MEDIUM SOFT GRAY SILTY CLAY WITH TRACES OF SAND								
	1.5-1.0 1.8-1.8 1.5 R-0.0		4-7	MEDIUM SOFT GRAY SILTY CLAY WITH TRACES OF SAND	20.0	224.3						
1.1	1.6-1.5 0.8-1.0 1.3-1.5 R-0.0		4-8	MEDIUM SOFT GRAY SILTY CLAY WITH SANDY SILT LAYERS AND A FEW STONES	25.0							
			4-9	GRAY SILTY CLAY WITH A LITTLE SAND	29.0							
				LOOSE TILL	31.0							
				SHALEY LIMESTONE (CORE RECOVERY 62%)	31.4	214.3						
				SHALEY LIMESTONE (CORE RECOVERY 81%)								
					37.7	206.6						
				BOTTOM OF HOLE								
							0	20	40	60	80	100
							% WATER CONTENT					PLATE 5
							MOISTURE CONTENT					

REINFORCED = R

% WATER CONTENT

○ MOISTURE CONTENT

PLATE

5



APPENDIX E

Figures

Figure 5 - Foundation Insulation Detail

Figures 6 to 9 - Results of Slope Stability Analysis

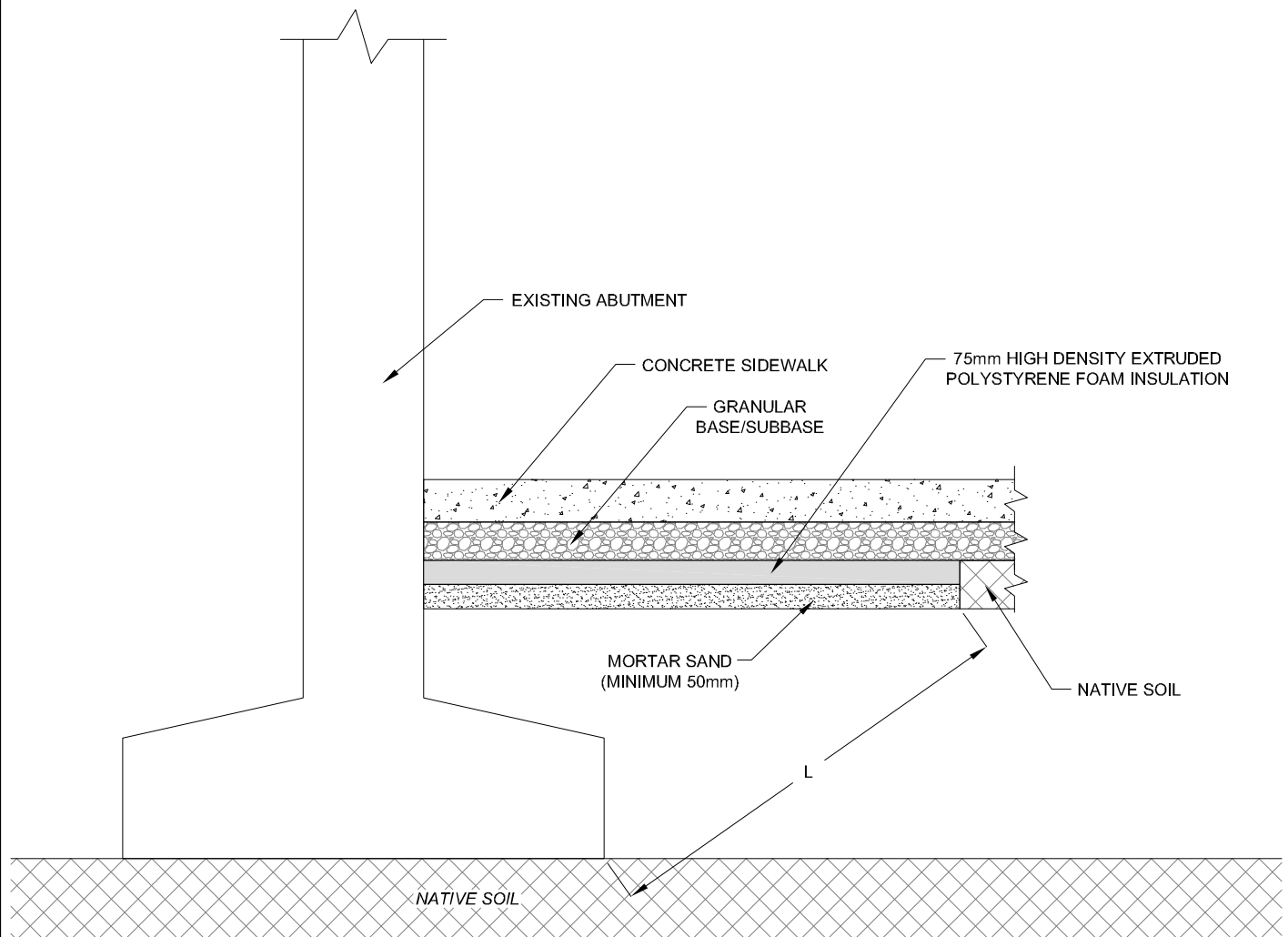
Non Standard Special Provisions

Vibration Monitoring

Driving Piles Adjacent to Existing Battered Piles

Pile Driving over Storm Sewer Tunnel

Drawing file: 051120210-5000-05.dwg Feb 10, 2017 - 3:37pm



NOTES:

- 1) INSULATION JOINTS TO BE GLUED AND/OR LAPPED
- 2) FOR ADEQUATE FROST PROTECTION $L \geq 1.8\text{m}$
- 3) INSULATION SHOULD CONSIST OF DOW HIGHLOAD 100 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



SCALE	NTS
DATE	3 JUNE 08
DESIGN	W.C.
CADD	J.M.
CHECK	K.S.L.
REVIEW	F.J.H.

TITLE

FOOTING INSULATION DETAIL

FILE No. 051120210-5000-05.dwg

PROJECT No. 05-1120-210 REV. 0

MERIVALE ROAD BRIDGE

FIGURE

5

Name: Embankment Fill
 Model: Mohr-Coulomb
 Unit Weight: 21
 Cohesion: 0
 Phi: 32
 Phi-B: 0

Name: Existing Fill
 Model: Mohr-Coulomb
 Unit Weight: 19
 Cohesion: 0
 Phi: 29
 Phi-B: 0

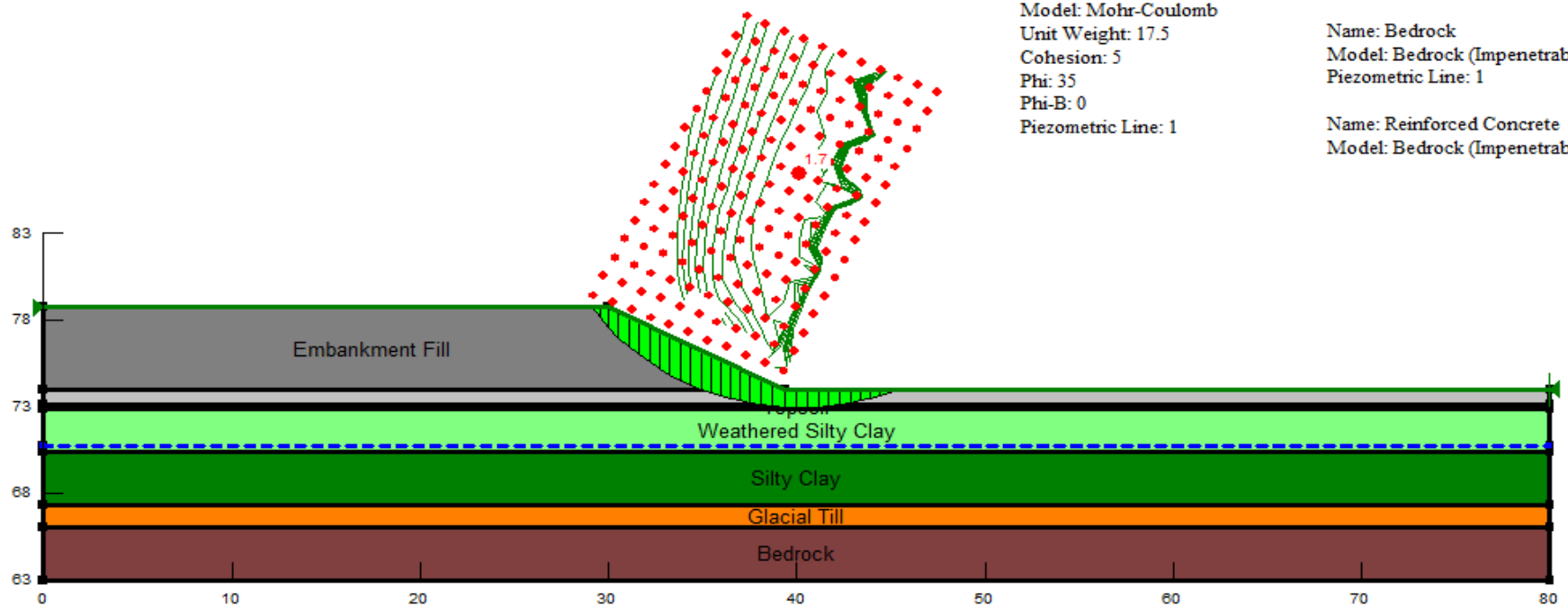
Name: Weathered Silty Clay
 Model: Mohr-Coulomb
 Unit Weight: 17.5
 Cohesion: 5
 Phi: 35
 Phi-B: 0
 Piezometric Line: 1

Name: Topsoil
 Model: Mohr-Coulomb
 Unit Weight: 18
 Cohesion: 0
 Phi: 25
 Phi-B: 0

Name: Silty Clay
 Model: Mohr-Coulomb
 Unit Weight: 16.5
 Cohesion: 7.5
 Phi: 31
 Phi-B: 0
 Piezometric Line: 1

Name: Bedrock
 Model: Bedrock (Impenetrable)
 Piezometric Line: 1

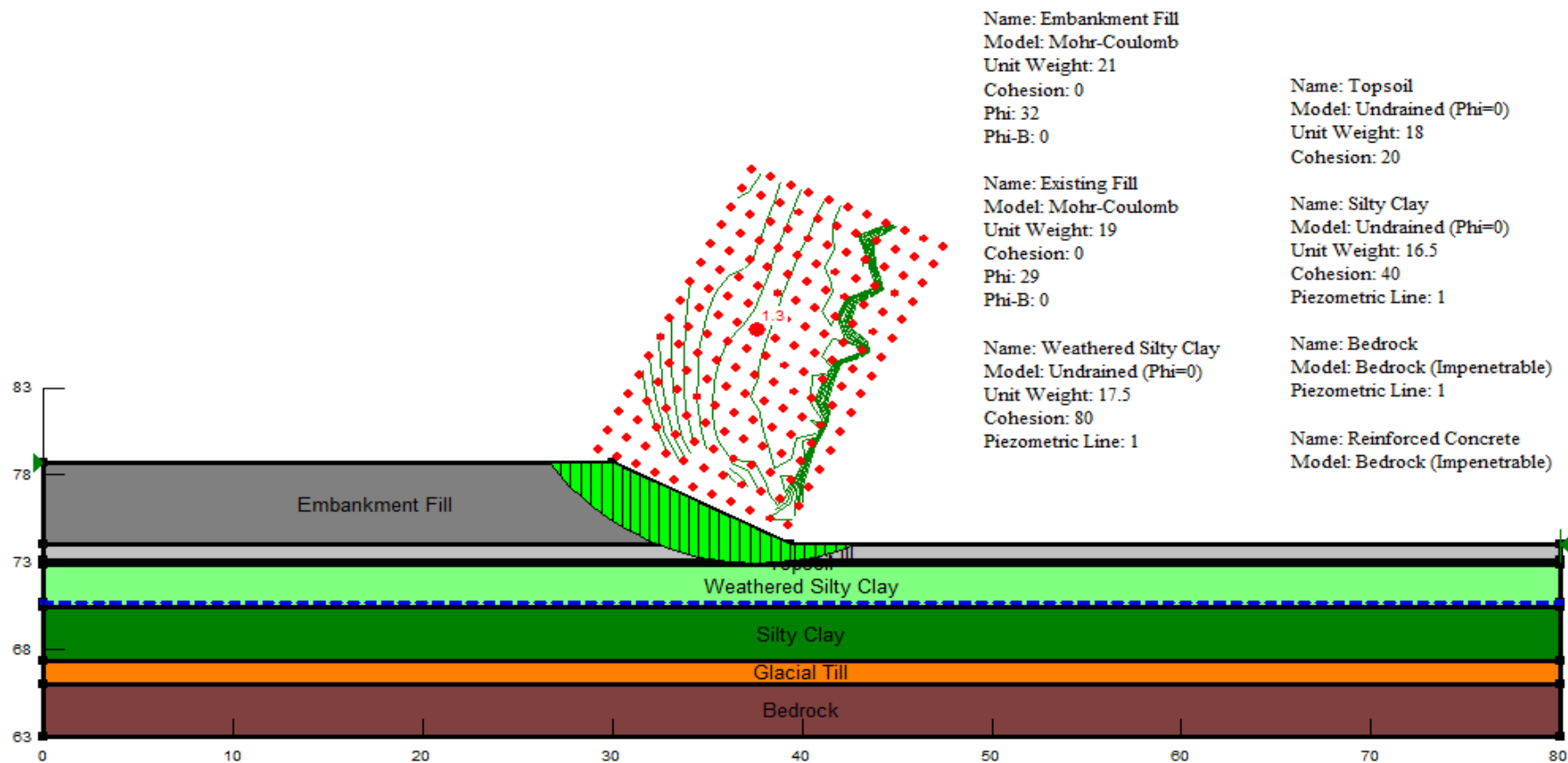
Name: Reinforced Concrete
 Model: Bedrock (Impenetrable)



Results of Slope Stability Analysis
Embankment - Static

Project No.	05-1120-210
Drawn:	WAM
Date:	2/8/2017
Checked:	KSL
Review:	WC

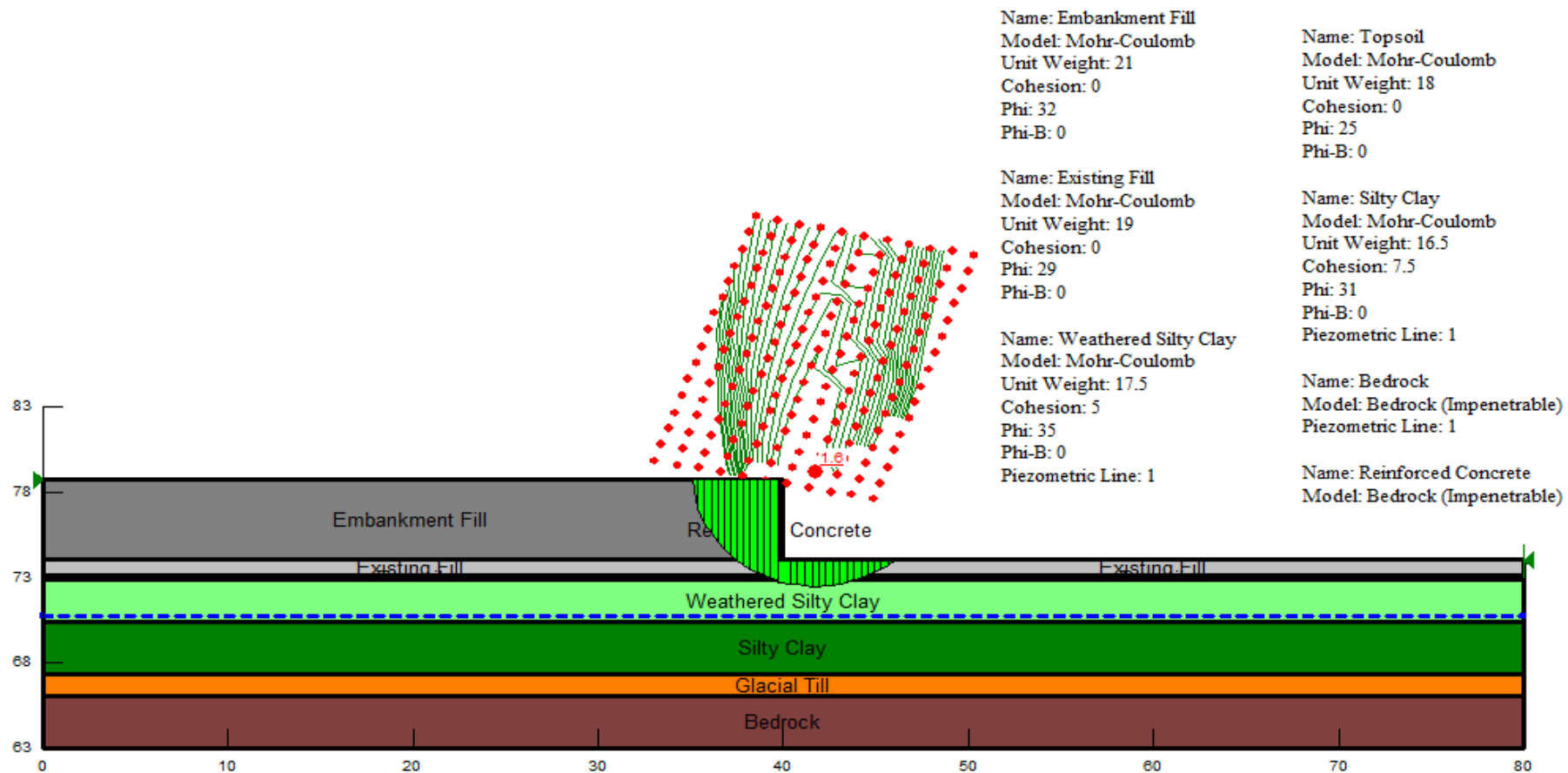
Figure 6



Results of Slope Stability Analysis
Embankment - Seismic

Project No.	05-1120-210
Drawn:	WAM
Date:	2/8/2017
Checked:	KSL
Review:	WC

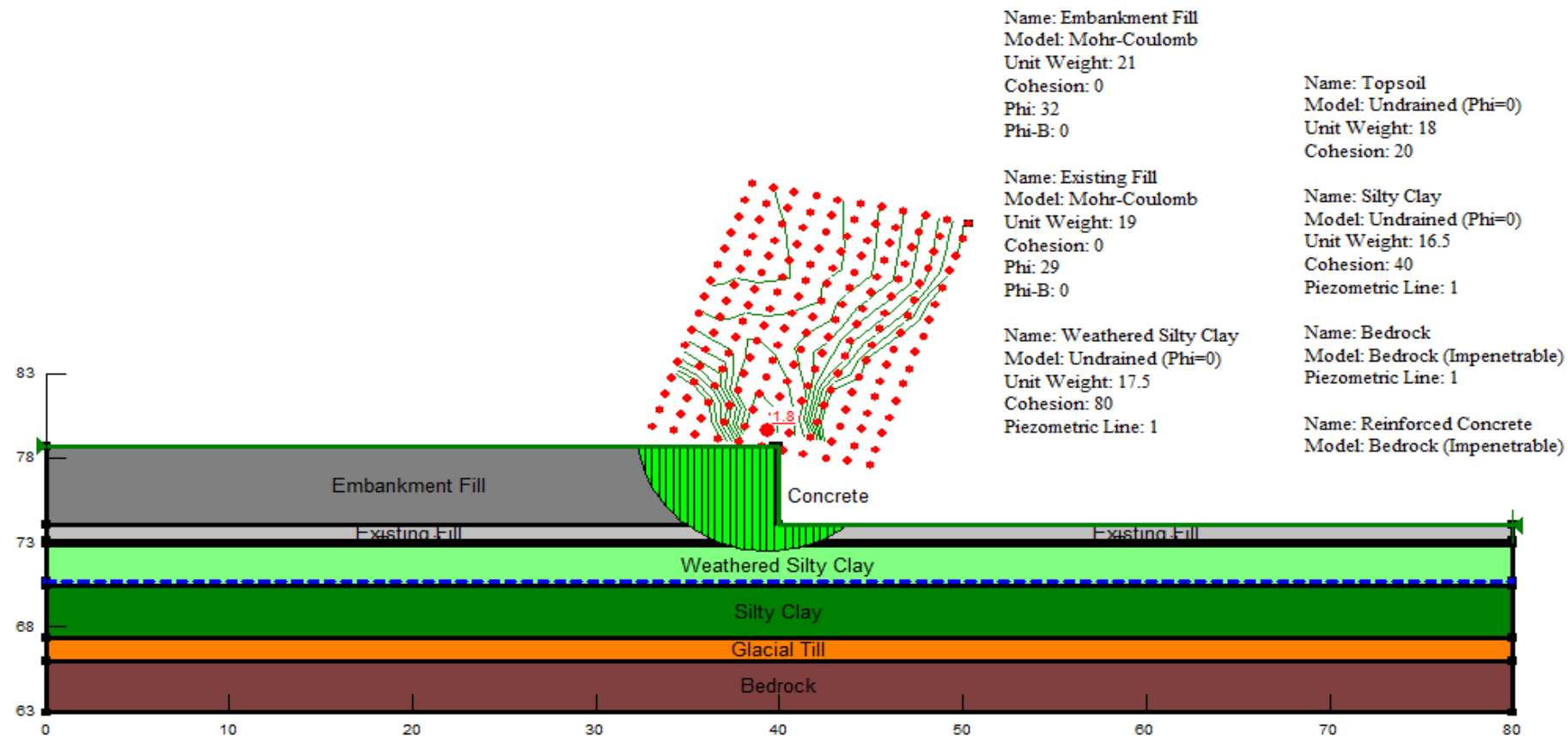
Figure 7



Results of Slope Stability Analysis
Retaining Wall - Static

Project No.	05-1120-210
Drawn:	WAM
Date:	2/8/2017
Checked:	KSL
Review:	WC

Figure 8



Results of Slope Stability Analysis
Retaining Wall - Seismic

Project No.	05-1120-210
Drawn:	WAM
Date:	2/8/2017
Checked:	KSL
Review:	WC

Figure 9

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, in real time. 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 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).

If it is not practical to drive the piles furthest from the existing structure first due to space constraints, the piles nearest the existing structure may be driven first but the measured vibrations in that case shall not exceed 50 mm/s.

The results shall be submitted to the Contract Administrator after each pile has been driven and 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

Driving Piles Adjacent to Existing Battered Piles – Item No.

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 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.

PILE DRIVING OVER STORM SEWER TUNNEL - Item No.

Special Provision

Scope

This special provision describes requirements for piling in the area of the southwest widening, above the storm sewer tunnel.

Construction

The Contractor shall monitor the vibrations at the level of the deep storm sewer tunnel (in accordance with the NSSP for vibration monitoring).

The measured vibrations at the sewer shall not exceed 50 mm/s (peak particle velocity).

The pile(s) furthest from the monitored utility should be driven first to assess the vibration level at the existing storm sewer.

To limit the vibrations within the rock and at the sewer level, the piles should not be overdriven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures, and possibly the set criteria, until the vibrations are within acceptable levels.

To reduce the vibrations at the sewer, the hammer energy should be reduced when the pile tip is approaching the bedrock surface elevation, at Elevation 67.5 m, and a reduced set criteria may be used during seating of the pile on the rock surface.

CAPWAP analyses should be carried out for any piles that need to be set using reduced energy.

If the CAPWAP analyses indicate that the full capacity of the pile cannot be mobilized, then additional piles may be required, with the approval of the Contract Administrator.

The final CAPWAP report should be stamped by an engineer licensed in the province of Ontario.

The piling operations and vibration monitoring in the area of the southwest footing should be inspected on a full time basis by geotechnical personnel working for a firm registered in RAQS in the prime specialty of: Foundation Engineering: Geotechnical (Structures and Embankments) – High Complexity.

Basis of Payment

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

END OF SECTION



APPENDIX F

Drawings 9B and 10 (Report Number 05-1120210-6)

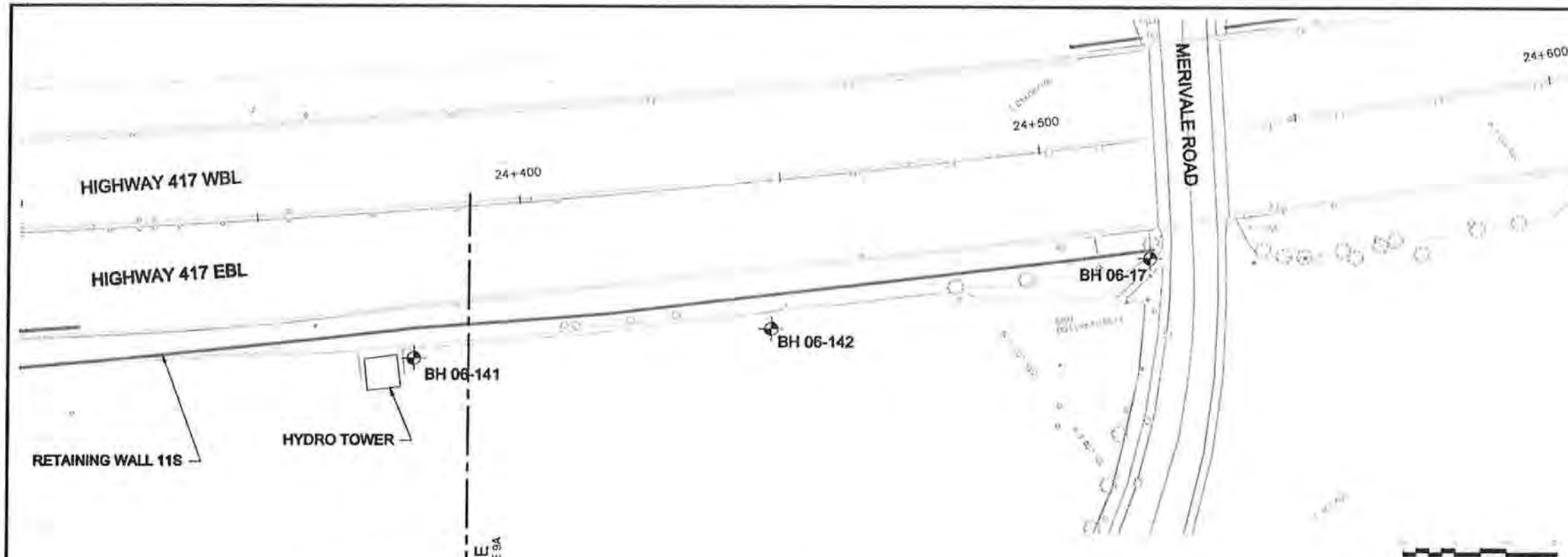
Record of Borehole 06-141

Consolidation Test Results – BH 06-141

Record of Borehole 06-143

Record of Borehole 06-143A

Consolidation Test Results – BH 06-143A



HWY. 417

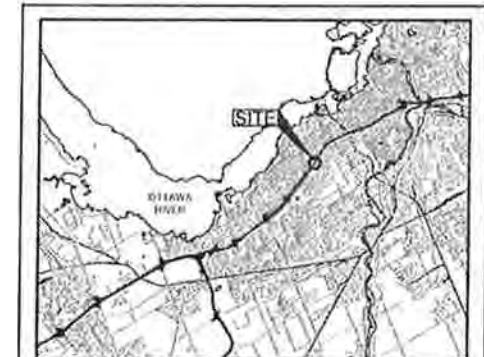
WP No. WP 4058-01-00

HIGHWAY 417 RETAINING WALL 11S PLAN AND PROFILE






2 of 2



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



LEGEND

-  Borehole - Current Golden Associates Ltd.
Investigation
-  Seal
-  Piezometer
- N Standard Penetration Test value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 1/blow)
- 100% Rock Quality Designation (RQD)
-  WL in piezometer
-  WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-138	76.2	5027626.2	364561.4
06-139	75.1	5027686.0	364595.5
06-140	74.3	5027733.3	364607.9
06-141	74.7	5027807.7	364671.7
06-142	74.7	5027865.9	364708.2
06-17	74.1	5027932.3	364740.8

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 geologist evidence.
Base plan provided in electronic format by McCormick Rankin Corporation

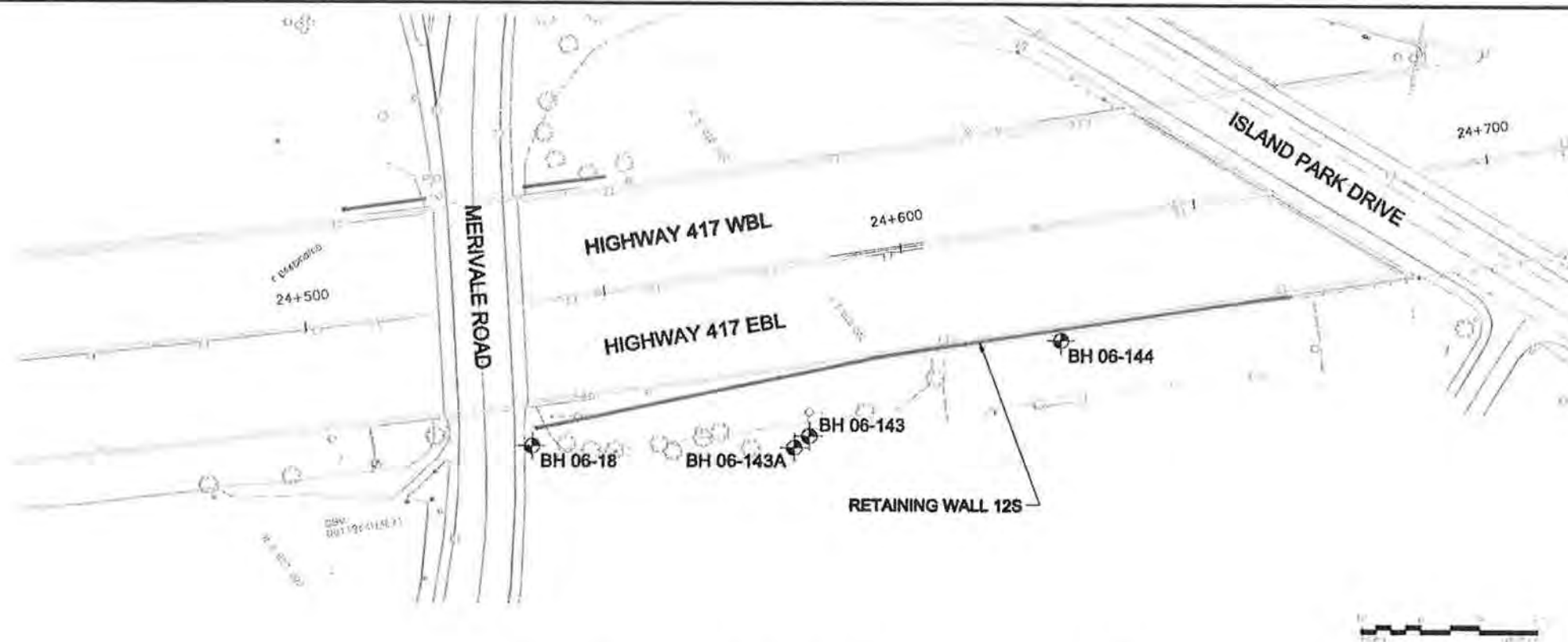
PROFILE ALONG RETAINING WALL 11S

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

NO.	DATE	BY	REVISION

Geocres No. 31G5-218

HWY. 417	PROJECT NO. 05-1120-210-2000	DIST.
SUBM'D. W.C.	CHKD. M.J.C.	DATE: OCTOBER 2005
DRAWN: J.W.	CHKD. W.C.	APPD. DWG. 98



HWY. 417

WP No. WP 4058-01-00

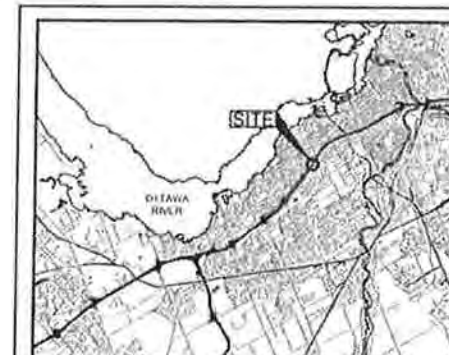
HIGHWAY 417
RETAINING WALL 12S
PLAN AND PROFILE



SHEET



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN

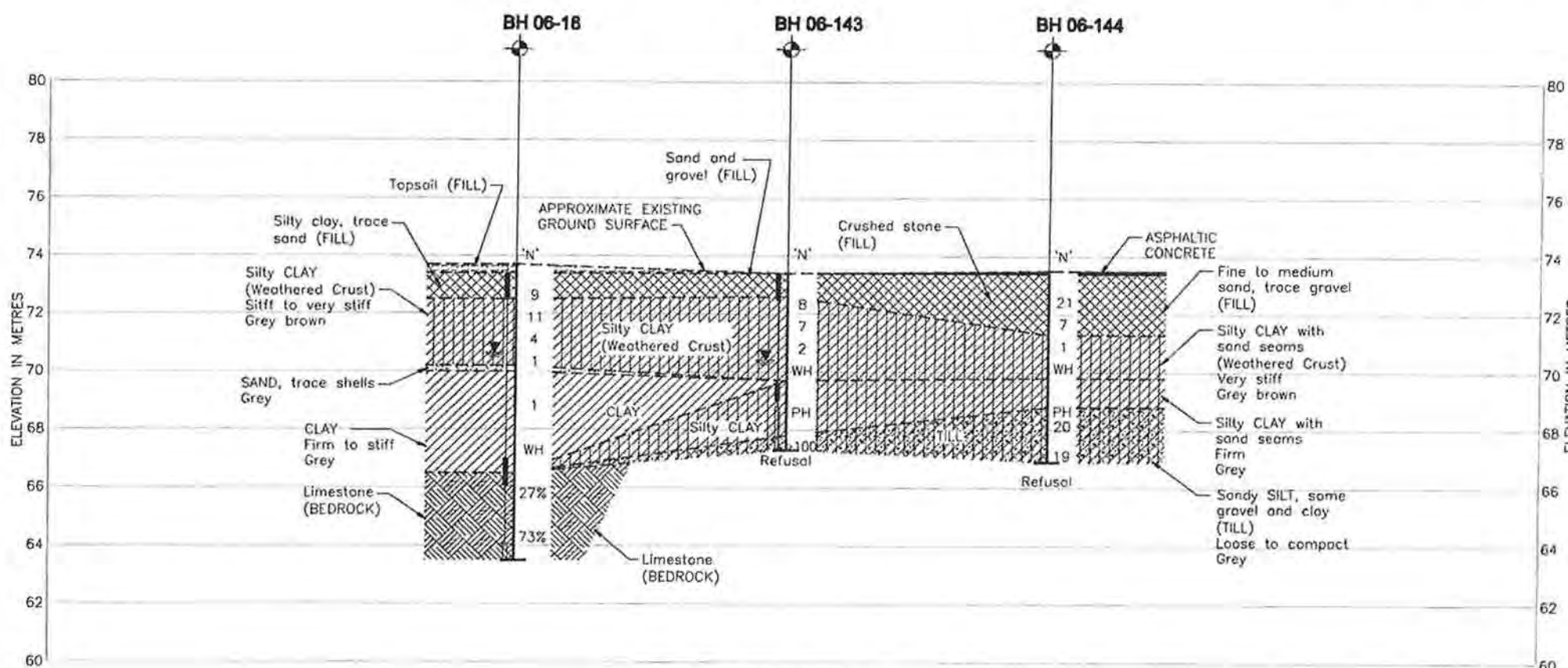
LEGEND

- Borehole - Current Golder Associates Ltd. Investigation
- Seal
- Piezometer
- N Standard Penetration Test value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 J/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer
- WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
06-143	73.4	5027983.9	364775.9
06-143A	73.4	5027980.8	364776.0
06-144	73.5	5028028.8	364786.5
06-18	73.7	5027945.9	364749.6

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



PROFILE ALONG RETAINING WALL 12S



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

NO.	DATE	BY	REVISION

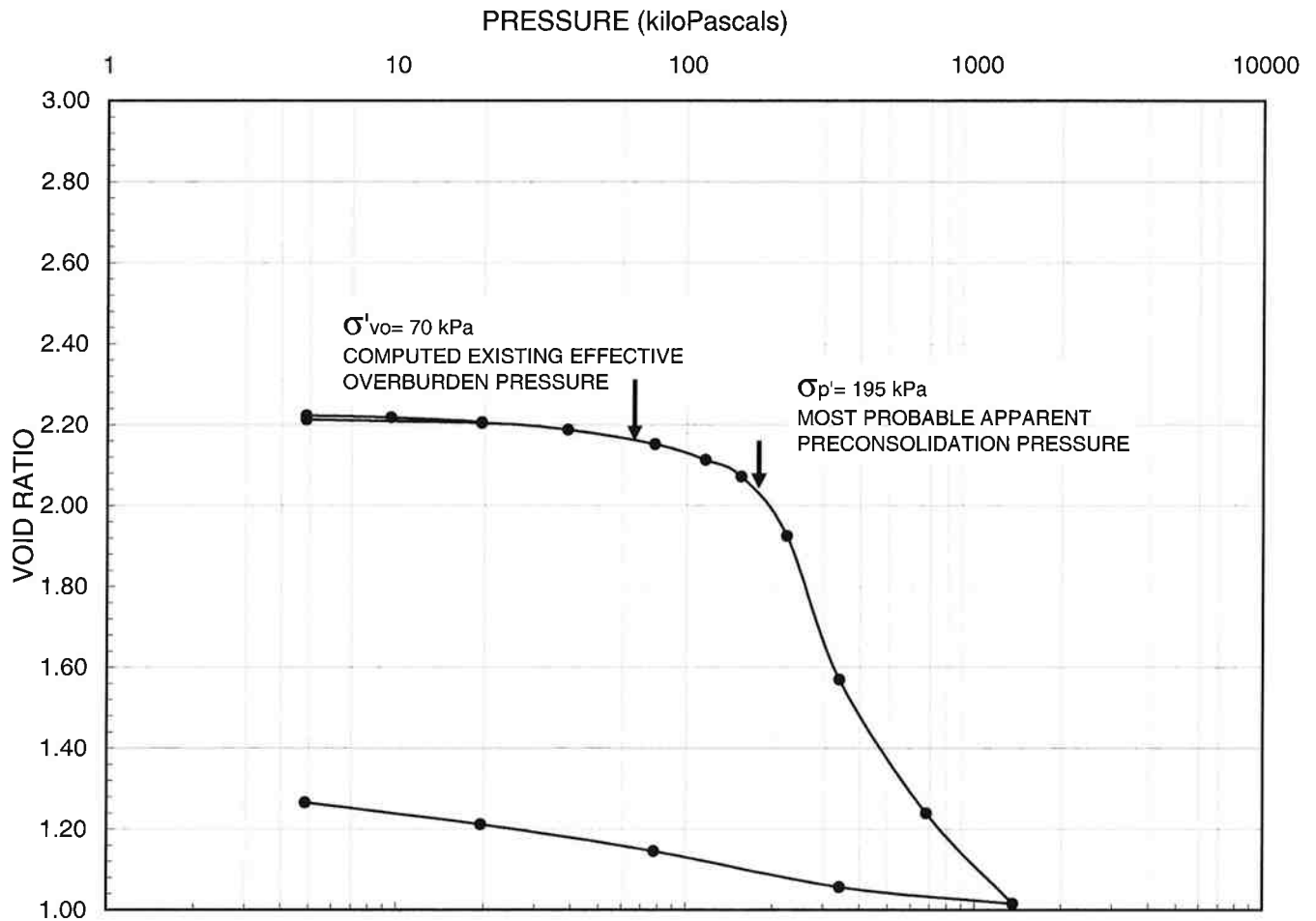
Geocres No. 3165-218

HWY. 417	PROJECT NO. 05-1120-210-2000	DIST.
SUBM'D. W.C.	CHKD. M.I.C.	DATE: OCTOBER 2006
DRAWN: J.M.	CHKD. W.C.	APPD.

SITE: DWG. 10

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

MISS_MTO 05-1120-210-6000 GPJ ON MOT GDT 4/10/07



LEGEND

Borehole: 06-141	$w_l = 73.4\%$	$S_o = 95\%$
Sample: 5	$w_l = 44.0\%$	$C_c = 1.95$
Depth (m): 4.90	$w_l = 58.1\%$	$C_r = 0.012$
	$w_p = 28.8\%$	



SCALE	AS SHOWN
DATE	10/10/07
DESIGN	NA
CADD	NA

CONSOLIDATION TEST RESULTS

FILE No. _____ Consolidation summary

PROJECT No. _____ 0 REV. 0

CHECK

REVIEW

Retaining Wall 11S

FIGURE

18

PROJECT 05-1120-210-2000

RECORD OF BOREHOLE No 06-143

1 OF 1

METRIC

W.P. 4058-01-00

LOCATION N 5027983.9; E 364775.9

ORIGINATED BY D.J.S.

DIST HWY 417

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

COMPILED BY J.M.

DATUM Geodetic

DATE July 4, 2006

CHECKED BY M.I.C.

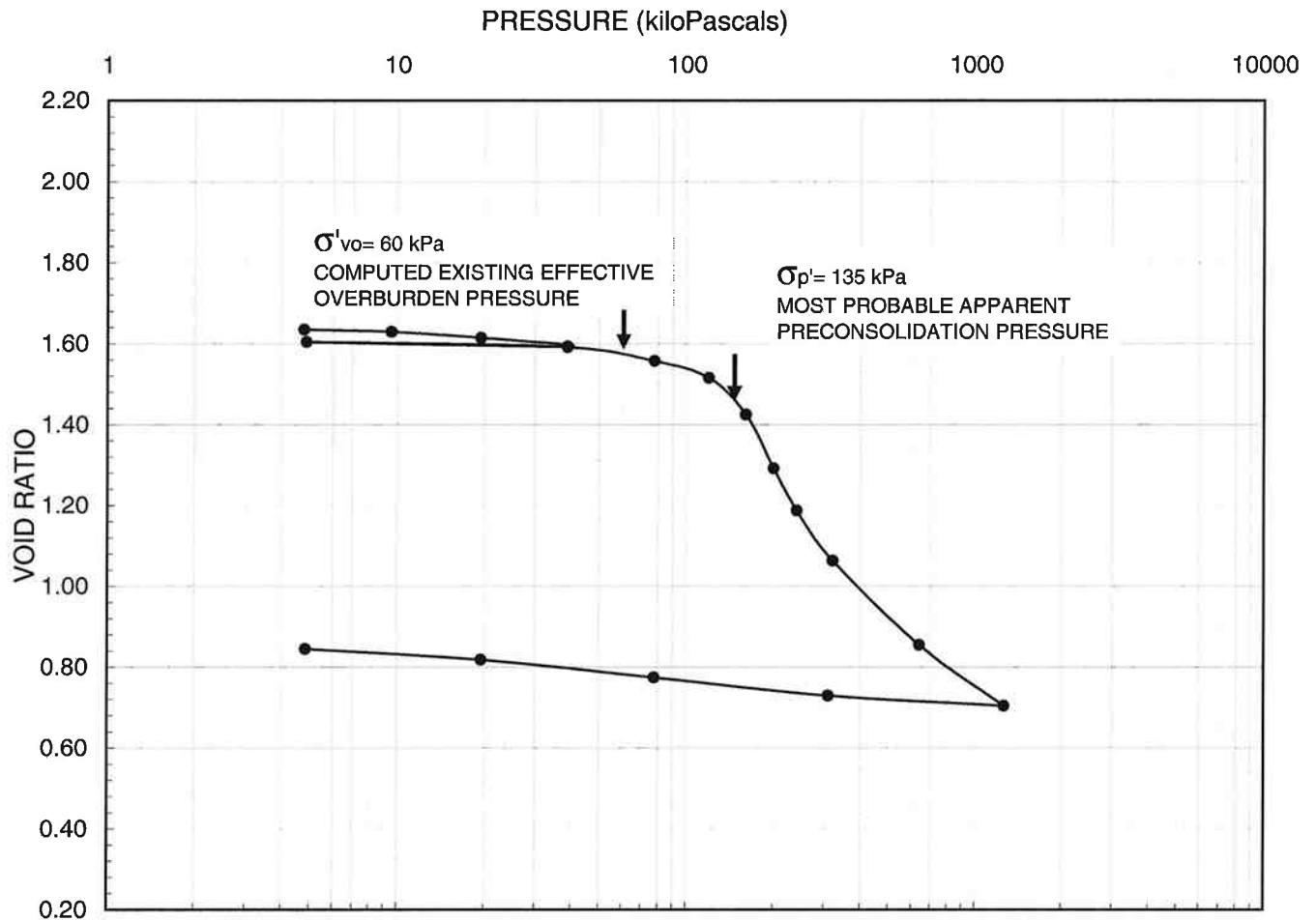
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
73.4	GROUND SURFACE							20 40 60 80 100						
0.0	Sand and gravel (FILL)													
73.2	Brown													
72.9	Silty clay, trace gravel (FILL)													
	Grey brown													
	TOPSOIL													
72.6	Sandy SILT													
0.8	Light brown													
	Moist													
	Silty CLAY (Weathered Crust)		1	SS	8									
	Very stiff													
	Grey brown													
	Moist to wet													
			2	SS	7									
70.7			3	SS	2									
2.7	Silty CLAY, some sand (Weathered Crust)													
70.4	Stiff													
3.1	Gray brown													
	Wet													
	Silty CLAY (Weathered Crust)		4	SS	WH									
	Firm													
69.7	Gray brown													
	Wet													
3.7	Silty CLAY													
	Firm													
	Grey													
	Wet													
			5	TP	PH									
67.9														
5.6	Sandy SILT, some gravel and clay (TILL)													
	Grey													
	Wet													
67.3														
6.1	End of Borehole Auger Refusal													
	Note: Water level in well screen at 3.0m depth below ground surface on Aug. 22, 2006.													

MISS_MTO 05-1120-210-6000 GPJ ON MOT GDT 4/10/07

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

PROJECT <u>05-1120-210-2000</u>	RECORD OF BOREHOLE No 06-143A		1 OF 1	METRIC
W.P. <u>4058-01-00</u>	LOCATION <u>N 5027980 8; E 364776 0</u>	ORIGINATED BY <u>J.A.S.</u>		
DIST <u> </u> HWY <u>417</u>	BOREHOLE TYPE <u>Power Auger 108 mm I.D. Hollow Stem Auger</u>	COMPILED BY <u>J.M.</u>		
DATUM <u>Geodetic</u>	DATE <u>Feb. 2, 2007</u>	CHECKED BY <u>MIC</u>		

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
73.4 0.0	GROUND SURFACE See Record of Borehole 06-143 for soil description.						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED						



LEGEND

Borehole: 06-143A	$w_i = 58.6\%$	$S_o = 100\%$
Sample: 1	$w_f = 32.1\%$	$C_c = 1.38$
Depth (m): 3.9m	$w_l = 57.1\%$	$C_r = 0.013$
	$w_p = 21.6\%$	



SCALE	AS SHOWN
DATE	10/10/07
DESIGN	NA
CADD	NA

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary

CHECK

PROJECT No. 0 REV. 0

REVIEW

Retaining Wall 12S

FIGURE

22

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

For more information, visit golder.com

Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 44 1628 851851
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Ltd.
1931 Robertson Road
Ottawa, Ontario, K2H 5B7
Canada
T: +1 (613) 592 9600

