REPORT

Foundation Investigation and Design

Highway 417 Walkley Road Underpass Rehabilitation
Site No. 3-306
Highway 417, Ottawa, Ontario
W.P. 4116-01-01
Assignment No. 4016-E-0008

Submitted to:

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by WSP Canada Group Limited (WSP) (formerly MMM Group) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the detail design of numerous bridge replacements and rehabilitations including seismic retrofits, several culvert rehabilitations and culvert removals, and overhead sign support structures on Highway 417 from Aviation Parkway to Ramsayville Road and Highway 417 expansion from Ottawa Road 147 to Hunt Club Road in Ottawa, Ontario (Assignment number 4016-E-0008).

This report presents the results of the foundation investigation carried out for the rehabilitation of the Highway 417 Underpass at Walkley Road, Site No. 3-306 (G.W.P. No. 4074-11-00 and W.P. 4116-01-01). The rehabilitation of the structure is to be carried out in accordance with the current version of the Canadian Highway Bridge Design Code (CHBDC, S6-14).

The terms of reference and scope of work for the foundation investigation are outlined in the MTO's Request for Proposal, dated May 2016, and subsequent addenda. Golder's scope of work for foundation engineering services associated with the Highway 417 Underpass at Walkley Street is contained in Table 17.8.3 of WSP's Technical Proposal for this assignment dated June 28, 2016. The work has been carried out in accordance with Golder's Quality Control Plan for foundation engineering services for the project dated March 13, 2017.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 Site Description

The Highway 417 Underpass at Walkley Road is located just east of the Hawthorne Meadows/Sheffield Glen neighbourhoods within the City of Ottawa. The site is on Highway 417 (Ottawa Queensway) approximately 160 m east of the Hydro Easement. At this location, Highway 417 is a divided highway with two lanes in each direction separated by a grass swale median. One speed change lane is also present in each direction at the underpass location for oncoming traffic from the Walkley Road bridge on-ramps.

The existing underpass bridge structure was built in 1973 and consists of an 83.1 m two-span cast-in-place post-tensioned concrete slab structure. The average deck width is 31.2 m with a roadway width of 26.1 m. The bridge consists of four lanes (two lanes in each direction) and two on-ramp lanes.

Photographs of the east and west bridge abutments and approach embankments taken on June 19, 2019 are attached to this report (Photographs 1 and 2). Based on visual observations, no visible signs of foundation settlement and/or erosion/instability of the approach embankments were noted.

2.2 Regional Geology

As delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984), this section of Highway 417 lies within the minor physiographic region known as the Ottawa Valley Clay Plain, which 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 former 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 (Belanger, 1998). This region is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain at depth by igneous and metamorphic bedrock of the Precambrian Shield. Regional bedrock mapping indicates that the bedrock at this site is primarily shale of the Carlsbad Formation(Williams, Rae, and Wolf, 1984). The shales were described as thinly bedded and fine grained.

3.0 INVESTIGATION PROCEDURES

3.1 Current Investigation

The field work for the current subsurface investigation was carried out on June 19 and 20, 2018. During that time, one (1) borehole (numbered 18-2301) was advanced at the location shown in plan on Drawing 1. The borehole was located just south of the bridge structure on the left shoulder of eastbound Highway 417. The borehole was advanced using truck-mounted drilling equipment supplied and operated by George Downing Estate Drilling Limited. The borehole was advanced through the asphaltic concrete (asphalt) shoulder and overburden to a depth of about 10 m (Elevation 56.1 m) below the surface of the existing roadway. The bedrock was then cored to a depth of 13.2 m (Elevation 52.9 m) using HQ-size coring equipment.

Soil samples were obtained at vertical intervals of about 0.75 m, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. A grab sample within the upper 300 mm of the granular fill underlying the asphalt was also recovered. Field vane tests were carried out in the clay deposit using a MTO “N” Vane to obtain undrained and remoulded shear strengths

A monitoring well was installed to observe the groundwater level at the site. The monitoring well consists of 32 mm outside diameter PVC tubing with a 3.0 m long screen sealed at a selected interval within the borehole. The monitoring well installation was completed with a flushmount casing at the ground surface. The groundwater level was measured on July 26, 2018, some five weeks after installation. The site conditions were restored following completion of the field work. The monitoring well was decommissioned in accordance with Ontario Regulation 903 (as amended) following the field investigation.

One soil sample was submitted to Eurofins Environment Testing for chemical analysis related to potential corrosion of buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack).

In addition to the borehole investigation, shear wave velocity testing at the site was completed using the Multichannel Analysis of Surface Waves (MASW) technique. The MASW testing was conducted on July 17, 2018 by personnel from Golder’s Mississauga and Ottawa offices. A series of 24 low frequency (4.5 Hz) geophones were laid out at 3 m intervals. A 9.9 kg sledge hammer and 45 kg weight drop were used as the seismic sources. The source locations were offset at various distances from, and collinear to, the geophone array.

The field work was supervised by a member of Golder’s staff who located the borehole in the field, directed the drilling, sampling, and in-situ testing operations, and logged the borehole. The soil and bedrock samples were identified in the field, placed in labelled containers, and transported to Golder’s laboratory in Ottawa for further examination and testing. Index and classification tests consisting of water content determinations, grain size distribution analyses, and Atterberg Limits tests were carried out on selected soil samples at the Ottawa laboratory. An unconfined compressive strength test was carried out on a sample of the bedrock core at Golder’s Mississauga laboratory. The laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate.

The borehole elevation was surveyed by Golder using a Trimble R8 GPS unit. The borehole location in MTM NAD83 Zone 9 northing and easting coordinates, ground surface elevation referenced to geodetic datum and drilled depth are summarized below and are shown on Drawing 1. Northing and easting grid coordinates and latitude and longitude geographic coordinates are also indicated on the Record of Borehole.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
18-2301	Left shoulder of Highway 417 Eastbound	5029057.3	375809.7	66.1	13.2

3.2 Previous Investigations (1971 and 1972)

As part of the current assignment, previously collected subsurface information pertinent to the site was reviewed and compiled. This existing subsurface information was contained in the following report:

- Report prepared by the Ontario Department of Transportation and Communications titled "*Foundation Investigation Report for Proposed Structure at the Crossing of Hwy #417 and Walkley Road Extension Regional Municipality of Ottawa-Carleton, District #9 (Ottawa), W.O. 71-11125, W.P. 10-69-08*", dated May 16, 1972 (Geocres No. 31G05-113).

Five sampled boreholes accompanied by dynamic cone penetration tests (Boreholes 1, 2, 4, 5, and 7) and two dynamic cone penetration tests (Boreholes 3 and 6) were advanced at the site as part of the 1971 investigation. Four sampled boreholes with dynamic cone penetration tests (Boreholes 1A to 4A) along with one in situ field vane test hole (Borehole 5A) were advanced as part of the 1972 investigation. The approximate borehole and ground surface elevations are shown on the existing borehole records included in Appendix B and are also shown on Drawing 1. The locations of the previous boreholes should be considered approximate since the locations were referenced to an imperial borehole location plan rather than metric MTM coordinates.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 General

The Record of Borehole and Drillhole Sheets from the current investigation are presented in Appendix A. The results of the laboratory testing carried out during the current investigation are presented on the Record of Borehole and Drillhole Sheets as well as on Figures A1 and A2 in Appendix A. A photograph of the bedrock core recovered is shown on Figure A3 in Appendix A. The existing borehole record sheets from the 1971 and 1972 investigations are provided in Appendix B. The results of chemical testing carried out on a soil sample from Borehole 18-2301 is included in Appendix C.

The MASW test results and associated technical memorandum are presented in Appendix D and include the calculated shear wave velocity profile measured from the field testing and a graphical representation of the shear wave velocity profile with depth.

The borehole locations from the current and previous investigations along with the interpreted stratigraphic profile projected along the centreline of the Highway 417 underpass are shown on Drawing 1. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile are inferred from observations of drilling progress and non-continuous sampling and, therefore, represent transitions between soil and rock types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations. Further, the boreholes from the 1971 and 1972 investigations were put down prior to construction of the bridge and the ground surface conditions presented on those borehole records may not be representative of the post-construction subsurface conditions, particularly with respect to the composition and thickness of overburden and fill.

4.2 Overburden

In general, the subsurface condition at the borehole location (18-2301) consist of asphaltic concrete overlying granular fill, overlying clayey silt to clay, which is in turn underlain by glacial till over shale bedrock. Embankment fill materials at the west and east approaches are expected behind the abutments and retaining walls of the bridge structure, although they were not investigated as part of the current program. Due to the age of the structure, it is possible that remnants of temporary works abandoned after construction of the existing structure may be buried in the fill. The boreholes drilled in the 1971 and 1972 investigations appeared to be consistent with the native materials encountered in Borehole 18-2301.

A more detailed description of the soil deposit, bedrock and groundwater conditions encountered in the boreholes from the present and previous investigations is provided below.

4.2.1 Pavement Structure and Fill

A layer of asphaltic concrete was encountered at the ground surface at borehole 18-2301, which was advanced along the Highway 417 Eastbound Lanes paved shoulder, with a thickness of about 100 mm.

Granular fill was found underlying the asphaltic concrete surface of Borehole 18-2301 and extends to a depth of 1.5 m below existing ground surface. The fill consists of gravelly sand to sandy gravel, containing shale fragments.

The measured value of one Standard Penetration Test (SPT) “N” in the fill gave 23 blows per 0.3 m of penetration, indicated a compact state of packing.

4.2.2 Silty Clay

Underlying the fill at Borehole 18-2301, and at surface at previous boreholes BH 1 to BH 7, inclusively, and BH 1A to BH 5A, inclusively, there is a deposit of silty clay to clay. The base of the deposit was observed at depths of approximately 3.5 m to 5.5 m below the existing ground surface at the time of drilling (i.e. Elevations ranging from about 61.5 to 59.3 m).

At borehole 18-2301, the upper portion of the silty clay deposit has been weathered to a grey-brown crust. The weathered crust extends to a depth of about 3.1 m below the existing ground surface (i.e., Elevation of 63.1 m). SPT ‘N’ values measured in the weathered silty clay crust at borehole 18-2301 ranged from 4 to 6 blows per 0.3 m of penetration, indicating a stiff consistency.

The results of Atterberg limit testing on one sample of the weathered silty clay deposit gave a plasticity index value of about 39 percent and liquid limit value of 60 percent, indicating a clay of high plasticity. The results of the Atterberg limits testing are presented on Figure A1.

Below the depth of weathering at borehole 18-2301, and through the deposit at all previous boreholes BH 1 to BH7, inclusively, and BH 1A to BH 5A, inclusively, the silty clay is grey in colour. In situ shear vane testing carried out within this deposit measured undrained shear strengths ranging from about 40 to 115 kPa, but more generally around 50 to 80 kPa, indicating that the deposit has a firm to very stiff consistency.

The results of Atterberg limit testing on one sample of the grey silty clay deposit gave a plasticity index value of about 16 percent and liquid limit value of about 34 percent, indicating a deposit of low plasticity. The results of the Atterberg limit testing are provided on Figure A1. The measured water content on two samples of the grey silty clay deposit ranges from approximately 29 to 51 percent.

4.2.3 Glacial Till

A till deposit was encountered below the fill and/or silty clay at all of the borehole locations. The till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sand and silt to silty sand, with some clay, and gravel.

The till deposit was fully penetrated in Borehole 18-2301 and previous boreholes BH2, BH 4, BH 5, BH 1A and BH 2A. Where penetrated, the till ranges in thickness from about 3.1 to 5.2 m and extends to depths of about 8.4 to 10.4 m below existing ground surface (i.e. Elevations of 55.4 to 57.2 m). At previous boreholes BH 1, BH 3, BH 6, BH 7, BH 3A, BH 4A, and BH 5A, the glacial till was proven to depths of about 5.7 to 9.2 m below ground surface (Elevations 56.3 to 59.7m).

Measured SPT "N" values within the till range from 10 to 104 blows per 0.3 metres of penetration, indicating a generally compact to very dense state of compactness. Values greater than 50 are considered to likely reflect cobbles/boulders in the till or refusal on the bedrock surface rather than the relative density of the till.

The results of grain size distribution testing on two samples of the sand and silt to silty sand till are shown on Figure A2. It should be noted that the samples were retrieved using a 50 mm outside diameter sampler and therefore the samples do not properly reflect the cobble and boulder portions of the deposit.

The measured natural water content of the till ranges from about 6 to 12 to percent.

4.3 Bedrock

Shale bedrock was encountered beneath the glacial till layer in Borehole 18-2301 at a depth of about 10.0 m below the existing ground surface (i.e., Elevation 56.1m). The bedrock was cored between depths of about 10.0 and 13.2 m using HQ diamond drilling techniques. The following table summarizes the bedrock surface depths and elevations as encountered at the current borehole location as well as the previous Boreholes 1 to 7 and 1A to 5A (Geocres No. 31G05-113). Note that certain previous boreholes were not advanced to the bedrock; only those boreholes that reached bedrock are included below.

Borehole Number	Borehole Location	Existing Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
18-2301	Left shoulder of Highway 417 Eastbound	66.1	10.0	56.1
2	Intersection of Highway 417 and Walkley Road prior to underpass construction	65.5	9.0	56.5
4		65.6	8.4	57.2
5		65.5	9.4	56.1
7		65.5	9.2*	56.3*
1A		65.7	10.4	55.3
2A		65.7	8.9	56.8
3A		65.7	9.0*	56.7*
4A		65.7	8.8*	56.9*

Note: *Inferred bedrock surface based on auger refusal.

The shale bedrock at the site is a member of the Carlsbad Formation. It is fresh and thinly bedded. Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from about 90 to 100 percent below the surface at 0.5 m, indicating very good to excellent quality rock.

The result of an unconfined compressive strength (UCS) test carried out on one bedrock core sample is presented on the borehole record in Appendix A. The sample tested had a UCS value of about 42 MPa, indicating medium strong bedrock. A photograph of the bedrock core obtained during the current investigation is provided in Appendix A on Figure A3. A description 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 and Drillhole sheets included with this report.

4.4 Groundwater Conditions

The water level in the monitoring well installed in Borehole 18-2301 was measured on July 26, 2018 and is summarized in the following table.

Borehole Number	Borehole Location	Screened Interval	Groundwater Level Depth (m)	Groundwater Elevation (m)	Date of Reading
18-2301	Highway 417 Eastbound left shoulder	Sand and Silt to Silty Sand (Till)	2.1	64.0	July 26, 2018

Geocres Report No. 31G05-113 reported that the groundwater level was in the overburden between elevations 64.9 and 65.5 m which, at the time, corresponded to depths ranging from ground surface to about 0.6 m below the ground surface at the time of drilling. The water levels which were measured in the open boreholes are shown on the 1971 and 1972 borehole records (Appendix B).

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt and is expected to be higher during the spring and periods of precipitation.

5.0 CLOSURE

This report was prepared by Mr. Pierre-Philippe Levasseur, P.Eng., and Michael Snow, P.Eng., both Senior Geotechnical Engineers with Golder, and Mr. Fintan Heffernan, P.Eng., a Senior Consultant with Golder and the Designated MTO Foundations Contact for this project, conducted an independent quality control review of this report.

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PART B – FOUNDATION DESIGN

Highway 417 Underpass at Walkley Road Bridge Rehabilitation Site No. 3-306 Ottawa, Ontario Assignment No. 4016-E-0008 G.W.P. 4074-11-00 W.P. 4116-01-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical input to the seismic evaluation of the Highway 417 underpass bridge at Walkley Road (MTO Structure Site No. 3-306) in Ottawa, Ontario. The input provided herein is based on an interpretation of the factual data obtained from desktop study of the available GEOCRES information and the borehole advanced during the current subsurface investigation. The input includes static and seismic design considerations for assessment of the foundations as part of the seismic evaluation of the underpass bridge to be carried out in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC).

The foundation investigation report, discussion, and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Existing Foundations

Based on the original structural design drawings (W.P. 10-69-08, Dwg Nos. 3-306-1 to 3-306-4), the grade of Highway 417 is at about Elevation 65.8 m at both the westbound and eastbound lanes. The Walkley Road grade ranges from about Elevation 72.7 m at the west abutment to about 72.9 m at the midpoint and 72.7 m at the east abutment.

The existing Walkley Road Bridge is a two-span structure with a cast-in-place, post-tensioned rectangular voided concrete slab. The spans are each about 41.5 m in length. The average deck width is 31.2 m with a roadway width of 26.1 m. The pier is founded on steel HP 12x89 (HP310x132) piles bearing directly on bedrock. The pier piles are configured in four rows with the two outside rows battered at 1H:3V and 1H:4V. The abutments are supported on “perched” foundations on piles end bearing on bedrock. The abutment piles are steel HP 12x74 (HP 310x110) and configured in two rows with the front row piles battered at about 1H:5V towards Highway 417. The existing approach embankments are about 6 to 7 m high relative to the highway profile.

6.3 Seismic Design

6.3.1 Seismic Hazard and Importance Category

The CHBDC states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the Geological Survey of Canada (GSC). The current seismic hazard maps (referred to as the 5th generation seismic hazard maps) were developed by the GSC and were made available for public use in December 2015.

In accordance with Section 4.4.2 of the CHBDC, and as specified in the RFP by the MTO, the bridge structure has been given an importance category of “*Other*” bridge.

6.3.2 Seismic Site Classification

Multichannel Analysis of Surface Waves (MASW) geophysical testing was carried out on either side of the interchange between Walkley Road and Highway 417 in the grassed area adjacent to Highway 417. The two MASW locations were carried out in the vicinity of the bridge to evaluate the average shear wave velocity of the upper 30 m of soil/bedrock at the site. The shear wave velocities measured at the site are presented in a technical memorandum (see results in Appendix D) and indicate that the average shear wave velocity in the upper 30 m of the subsurface stratigraphy at the two MASW locations were 473 m/s and 582 m/s.

As outlined in the CHBDC, a Site Class C should be used for design based on the measured shear wave velocities.

However, Table 4.1 of the CHBDC also specifies circumstances for which a Site Class of F is applicable and a site-specific response evaluation must be carried out; the presence of liquefiable soils is one of those conditions. As presented below in Section 6.3.4, this site is not underlain by soils considered likely to undergo seismic liquefaction.

6.3.3 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the bridge (latitude 45.40 N and longitude 75.59 W), the following are the Site Class C (reference) peak seismic hazard values based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca).

Site Class C Spectral Values for Subject Site

Parameter	Value at Given Probability of Exceedance in 50 Years		
	10% (475-year)	5% (975-year)	2% (2,475-year)
PGA	0.11 g	0.17 g	0.30 g
T <= 0.2 s	0.17 g	0.27 g	0.47 g
T = 0.5 s	0.09 g	0.14 g	0.25 g
T = 1.0 s	0.05 g	0.07 g	0.12 g
T = 2.0 s	0.02 g	0.03 g	0.06 g
T = 5.0 s	0.005 g	0.008 g	0.015 g
T => 10.0 s	0.002 g	0.003 g	0.006 g

The fundamental period of the rehabilitated structure has yet to be confirmed and may depend on the design modifications to the superstructure. In consideration of the structure's "*Other*" importance category and the site-specific seismic hazard values given above, the bridge would fall in Seismic Performance Category 2, if the fundamental period of the structure is greater than or equal to 0.5 s, or Seismic Performance Category 3, if the fundamental period of the structure is less than 0.5 s, in accordance with Table 4.10 of the CHBDC.

This bridge is considered to be *regular* geometry and will be designed using a "force-based approach" as defined in the CHBDC, for both Seismic Performance Categories 2 and 3.

6.3.4 Liquefaction Assessment

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface settlements) and under undrained conditions generate excess pore pressures. The excess pore pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as "lateral spreading" or under certain conditions even catastrophic failure of the slope often referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

The methodology used to assess liquefaction potential at the site is consistent with the “simplified” approach presented outlined in the CHBDC and by Idriss and Boulanger (2008). It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

Liquefaction analyses were carried out for the abutments and pier locations. The analyses carried out for the abutments used the in-situ testing data collected at the boreholes advanced during the 1971 investigation carried out prior to construction of the bridge and considered the influence of the increased confining stress beneath the subsequently constructed approach embankments. The analyses carried out for the pier used the in-situ testing data collected at the boreholes advanced during the previous and current investigations adjacent to the pier.

The design groundwater level was established based on the groundwater elevations measured in the standpipe piezometer installed in Borehole 18-2301, and those measured in the open boreholes during the 1971 and 1972 investigations.

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

The results of the liquefaction analyses at the pier and abutments indicate that the fill surrounding and overlying the existing abutment and pier footings may be considered to be non-liquefiable for design. The results of the liquefaction assessment using the simplified method indicate that certain localized horizons within the glacial till at borehole 18-2301 at the site may be considered liquefiable during the 2,475-year design earthquake.

However, the liquefaction methodologies outlined in Idriss and Boulanger (2008) do not account for the additional cyclic resistance provided by the aging/cementation that may be a characteristic of the glacial till at the site. Although aging deposits is known to help resist seismic liquefaction, little research has been done in this area to quantify this. Based on Figure 9 presented in the work by Leon et al (2006), a correction increase of about 30% in the CRR profile would appear appropriate. While such aging/cementation corrections would potentially reduce the risk of liquefaction at this site given the age of the till of about 10,000 to 15,000 years, specific testing, assessment and research on he till at this site is not available.

Work done by Harpin et al (2017) on the site response of sites in eastern Canada and the site response analyses conducted by Golder Associates at the CR31 overpass (i.e., Site No. 31-204), CPR Overhead (i.e., Site No. 3-302/1-2 and Green’s Creek Structures (i.e., Site No. 3-311/1-2 and 3-310) would suggest that site-specific response analysis would also reduce the CSR profile, when compared to the simplified methodology outlined in Idriss and Boulanger (2008), such that an approximately 20% reduction could be expected.

In consideration of the beneficial effects of aging and the anticipated lower cyclic shear stresses in eastern Canada relative to the simplified method, higher CRR and lower CSR respectively, the extent and probability of liquefaction at the site is considered to be very small to the point of having little impact on the dynamic response of the site (i.e., Site Class C) and the performance of the foundation elements (i.e., no liquefaction settlement induced downdrag).

The susceptibility of the silty clay deposit to cyclic mobility was also assessed based on the methodology provided in Idriss and Boulanger (2008), in which the CRR for clay-like soil is calculated based on the undrained shear strength and approximate OCR of the soil. The CRR is equated with the CSR (for reference stress equal to 65% of peak shear stress) to calculate the factor of safety against cyclic softening that would be expected to result in greater than 3% shear strain. Based on the results of the analyses, the silty clay is not considered to be susceptible to cyclic softening.

6.4 Assessment of Existing Foundations

6.4.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the existing underpass structure and foundation system may be classified as having large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a "typical" consequence level associated with exceeding limits states design. Given the level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in Section 6.4.2.1.
For seismic design, the consequence factor, Ψ, and resistance factor, ϕ_{gu}, should be taken as unity, as per Section 4.6.3 of the CHBDC.

6.4.2 Steel H-Piles (Abutments)

Based on the original structural design drawings (W.P. 10-89-08, Dwg Nos. 3-306-1 to 3-306-3), the pier is founded on steel HP 12x89 (HP 310x132) piles bearing directly on bedrock. The pier piles are configured in four rows with the two outside rows battered at 1H:3V and 1H:4V. The abutments are supported on "perched" foundations on piles end bearing on bedrock. The abutment piles are steel HP 12x74 (HP 310x110) and configured in two rows with the front row piles battered at about 1H:5V towards Highway 417.

The pile caps should have a minimum embedment depth of 1.8 m for frost protection purposes, per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

6.4.2.1 Factored Geotechnical Axial Resistance

Based on the GEOCRES information and the pile installation methods likely used at the time of original construction, the HP 12x74 (HP 310x110) piles at the abutments may be considered to have a static factored geotechnical resistance of 1,500 kN at Ultimate Limit States (ULS). The HP 12X89 (HP310x132) piles at the pier may be considered to have a static factored geotechnical resistance of 1,800 kN at ULS. These factored ULS values assume that the piles are bearing directly on bedrock (as per the available information, see Section 6.4.2) and are based on a ULS consequence factor, Ψ, of 1.0 and a geotechnical resistance factor, ϕ_{gu}, of 0.50 based on Tables 6.1 and 6.2 of the CHBDC.Serviceability Limit States (SLS) resistances do not apply to piles founded on the bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

The silty clay deposit at the site may be subject to consolidation settlements if fill placement is carried out above the existing ground surface (i.e. grade raise). Consolidation settlements around piles will induce downdrag (negative skin friction) forces on the piles. It is our understanding that the approach embankments are not proposed to be raised/widened as part of the bridge rehabilitation works. Therefore, no additional downdrag loads are expected on the abutment piles.

6.4.2.2 Resistance to Lateral Loads

It is understood that lateral loading will be resisted fully or partially by the battered steel H-piles at the abutments and pier. Additional resistance to lateral loading may be derived from the soil in front of the piles.

For preliminary design of the seismic retrofit of the Walkley Road bridge, the SLS geotechnical response of the soil in front of the piles under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition). It may be assumed that this resistance (from the soil in front of the piles) will be nearly the same for vertical and inclined piles.

For cohesionless soils:

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

Where: n_h is the constant of horizontal subgrade reaction, as given below;

z is the depth (m); and,

B is the pile diameter/width (m).

For cohesive soils:

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

Where: s_u is the undrained shear strength of the soil (kPa); and,

B is the pile diameter/width (m).

The following ranges for the values of n_h and s_u may be used in the preliminary structural analysis. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behavior (such that k_h is a function of deflection).

Location	Elevation (m)	Soil Type	n_h (MN/m^3)	s_u (kPa)
East Abutment (BH 1A, BH 4A, BH 2)	65.7 – PCL[1]	Inferred Compact to Dense Sand/Gravel (Emb. Fill)	6.6	-
	60.4 – 65.7	Firm to very stiff Silty Clay (Below Water Table)	-	80
	55.3 – 60.4	Dense to very dense Glacial Till (Below Water Table)	4.4	-
	55.3	Bedrock	-	-
West Abutment (BH 2A, BH 5A)	65.7 – PC[L2]	Inferred Compact to Dense Sand/Gravel (Emb. Fill)	6.6	-
	59.1 – 65.7	Firm to very stiff Silty Clay (Below Water Table)	-	60
	56.5 – 59.1	Compact to very dense Glacial Till (Below Water Table)	4.4	-
	56.5	Bedrock	-	-
Central Pier (BH 3A, BH 4, 18-2301)	66.1 – PCL[3]	Inferred Compact to Dense Sand/Gravel (Fill)	6.6	-
	60.6 – 66.1	Firm to very stiff Silty Clay (Below Water Table)	-	80
	56.0 – 60.8	Dense to very dense Glacial Till (Below Water Table)	4.4	-
	56.0	Bedrock	-	-

Note: [1] PCL = Pile Cap Level, understood to be Elevation 68.6 m
[2] PCL = Pile Cap Level, understood to be Elevation 68.1 m
[3] PCL = Pile Cap Level, understood to be Elevation 63.4 m

Group action should be considered using the generalized p-multipliers (i.e. p-reduction factors) for a range of piles provided in Section C6.11.3.4 of CHBDC.

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.11.2.2.1 of the *Commentary to the CHBDC.*

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

6.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment and wing walls depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The available historical boreholes at the abutment locations from the available GEOCRES information were put down prior to embankment construction. Based on the original structural design drawings and the abutment backfill type typically used at the time of construction of the bridges, the abutment backfill was assumed to consist of compact sand.

For preliminary design, the static lateral earth pressures may be calculated using the lateral earth pressure coefficients provided in the table below (assuming a compact sand backfill with a unit weight of 20 kN/m^3 and a friction angle of 30 degrees).

Static Lateral Earth Pressure Coefficients

Coefficient Type	Coefficient Value
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_p	3.00

Where the abutment walls do not allow lateral yielding, at-rest earth pressures should be assumed for the design. Where the abutment walls allow lateral yielding of the stem, active earth pressures should be used in the design of the wall structure(s). The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the CHBDC.

The seismic active earth pressure acting on the abutment walls can be calculated using the seismic active earth pressure coefficients (K_{AE}) provided in the table below.

Seismic Lateral Earth Pressure Coefficients

Wall Type	Design Earthquake	Site PGA	Coefficient Value (K_{AE})
Yielding Wall	**475 Year**	0.11	0.37
	975 Year	0.17	0.39
	2,475 Year	0.30	0.43

Wall Type	Design Earthquake	Site PGA	Coefficient Value (K_{AE})
Non-Yielding Wall	**475 Year**	0.11	0.40
	975 Year	0.17	0.45
	2,475 Year	0.30	0.57

In accordance with Sections 4.6.5 and C.4.6.5 of the 2014 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 equal to the site adjusted PGA estimated at the ground surface (Site Class C). For structures which allow lateral yielding, k_h is taken as 0.5 times site adjusted PGA estimated at the ground surface.

It should be noted that the seismic earth pressure coefficients provided in the table above were calculated considering 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.

The K_{AE} value for a yielding wall is applicable provided that the wall can move up to 250k_h mm, where k_h is the site-specific PGA as given in the table above. This corresponds to displacements of about 30, 45, and 75 mm for the 475-year, 975-year, and 2,475-year design earthquakes 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:

$\sigma_h(z) = K_A\,\gamma\,z + (K_{AE} - K_A)\,\gamma\,(H\text{-}z)$ for yielding walls; and,

$\sigma_h(z) = K_0\,\gamma\,z + (K_{AE} - K_A)\,\gamma\,(H\text{-}z)$ for non-yielding walls;

Where: $\sigma_h(z)$ is the lateral earth pressure at depth ‘z’ (kPa);

K_A is the static active earth pressure coefficient;

K_0 is the static at-rest earth pressure coefficient;

K_{AE} is the seismic earth pressure coefficient;

γ is the unit weight of the backfill soil (kN/m^3), use 20 kN/m^3;

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

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

6.6 Corrosion and Cement Type

One soil sample from Borehole 18-2301 was submitted to Eurofins Environmental Testing for chemical analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The results of the testing are provided in Appendix C and are summarized in the table below.

The results indicate a low potential for concrete degradation due to the presence of sulphates, and that concrete made with Type GU Portland cement should be acceptable for substructures. However, the results also indicate a potential for corrosion of exposed ferrous metal which should be considered in the design.

Summary of Corrosivity of Sample

Borehole No.	Sample Depth (m)	Sample Type	Chloride (%)	pH	Electrical Conductivity (mS/cm)	Resistivity (ohm-cm)	Sulphate (%)
18-2301	4.6 to 5.2	Silty Clay	0.031	8.15	0.36	2780	0.02

6.0 CLOSURE

This report was prepared by Mr. Pierre-Philippe Levasseur, P.Eng., and Michael Snow, P.Eng., both Senior Geotechnical Engineers with Golder, and Mr. Fintan Heffernan, P.Eng., a Senior Consultant with Golder and the Designated MTO Foundations Contact for this project, conducted an independent quality control review of this report.

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https://golderassociates.sharepoint.com/sites/11263g/shared documents/01_foundations/6 - reports/1230 walkley/superseded/1662565-1230-001-rev0-walkley road fidr-june 2019.docx

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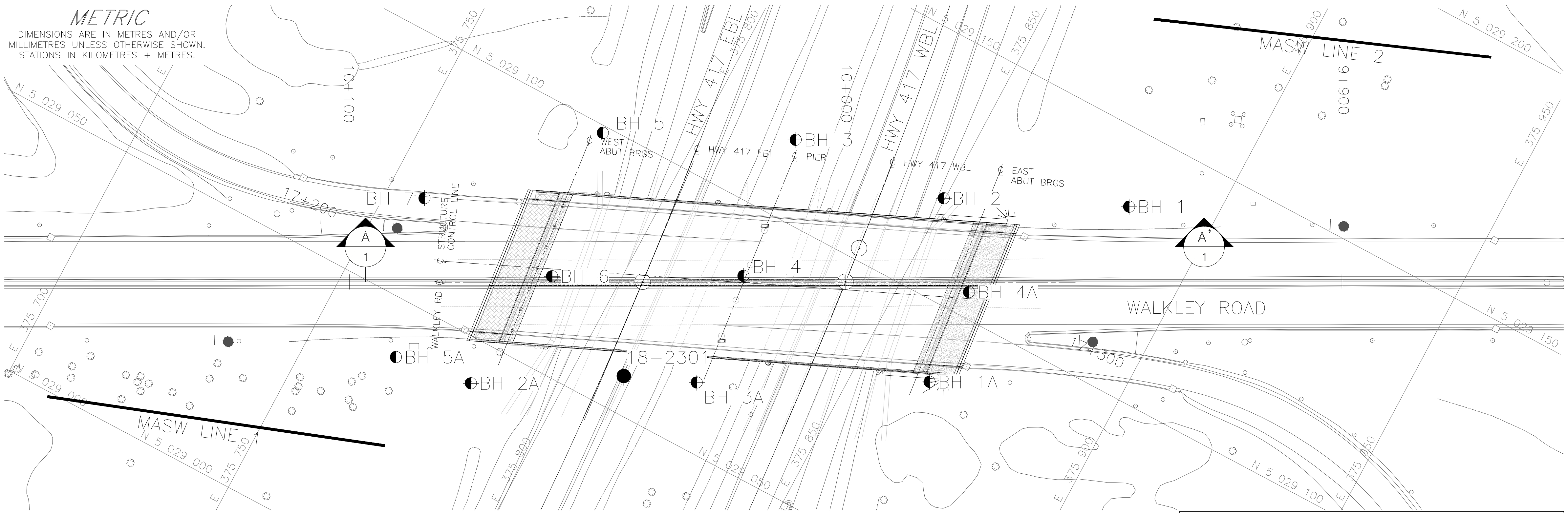
WALKLEY ROAD UNDERPASS

HIGHWAY 417
BOREHOLE LOCATIONS AND SOIL STRATA
LAT. 45.398123 LONG. -75.592854

SHEET

GOLDER

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



PLAN
SCALE
10 0 10 20 m

KEY PLAN
SCALE
1 0 1 2 km

NOTES

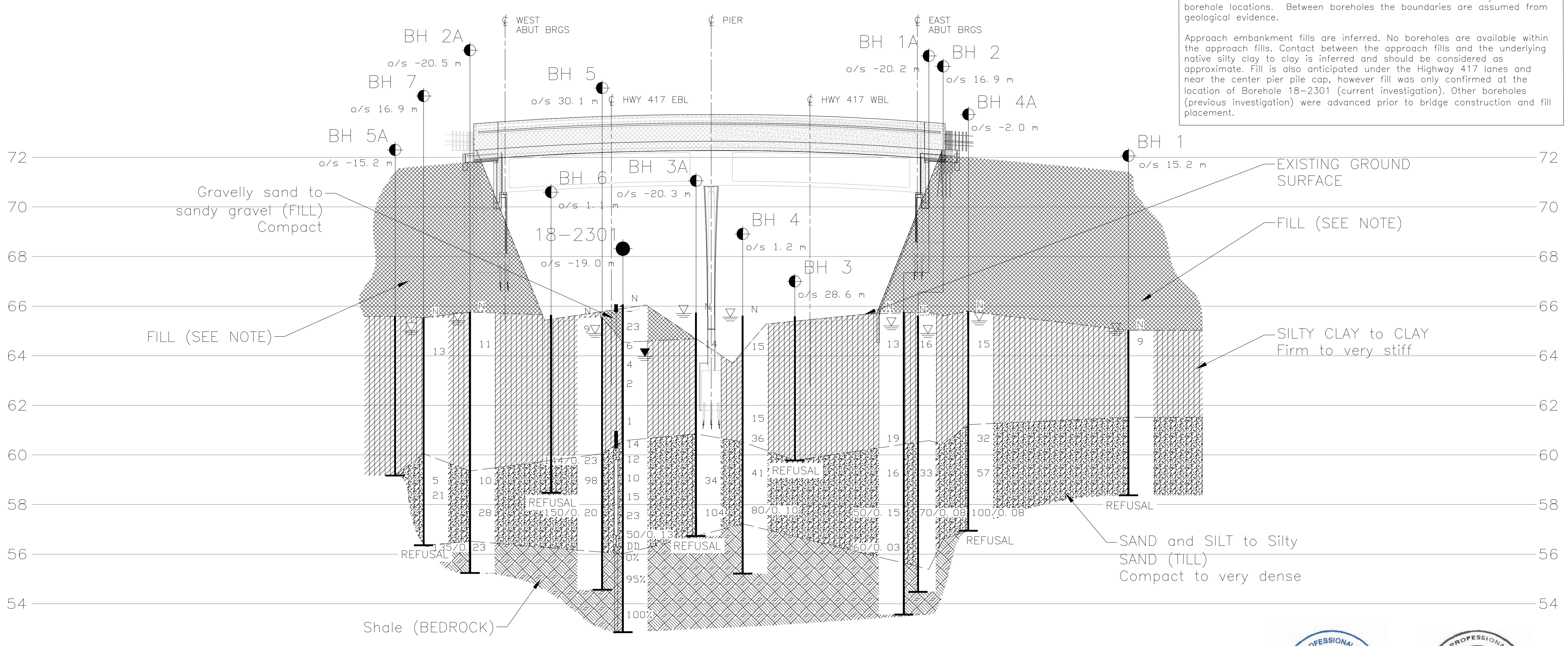
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

Approach embankment fills are inferred. No boreholes are available within the approach fills. Contact between the approach fills and the underlying native silty clay to clay is inferred and should be considered as approximate. Fill is also anticipated under the Highway 417 lanes and near the center pier pile cap, however fill was only confirmed at the location of Borehole 18-2301 (current investigation). Other boreholes (previous investigation) were advanced prior to bridge construction and fill placement.

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation (Geocres No. 31G05-113)
- 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 July 26, 2018
- WL in open borehole, measured during drilling



PROFILE A–A'
SCALE
10 0 10 20 m

BOREHOLE CO-ORDINATES (MTM ZONE 9)			
No.	ELEVATION	NORTHING	EASTING
18-2301	66.1	5029057.3	375809.7
BH 1	65.0	5029135.4	375883.5
BH 1A	65.8	5029085.3	375864.6
BH 2	65.6	5029119.3	375849.8
BH 2A	65.8	5029041.5	375783.2
BH 3	65.6	5029115.6	375817.9
BH 3A	65.7	5029063.1	375823.3
BH 4	65.6	5029086.5	375821.5
BH 4A	65.8	5029105.0	375863.1
BH 5	65.5	5029098.6	375782.9
BH 5A	65.6	5029039.1	375767.4
BH 6	65.6	5029068.3	375787.5
BH 7	65.5	5029070.1	375757.4

REFERENCE

Base plans provided in digital format by WSP, drawing file nos. XA1-NAD 83.dwg, XB1-NAD 83 (CSRS).dwg, received APR. 19, 2017 and S17M-00850-300-311-001GA.dwg, received JUN. 14, 2019.

LICENSED PROFESSIONAL ENGINEER M.S. SNOW PROVINCE OF ONTARIO
REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31G5-303

HWY. 417	PROJECT NO. 1662565		DIST. EASTERN
SUBM'D. BB	CHKD. BB	DATE: 6/20/2019	SITE: 3-306
DRAWN: JM	CHKD. MSS	APPD. FJH	DWG. 1

Photograph 1: East Abutment (looking southeast, June 19, 2019)

Photograph 2: West Abutment (looking south, June 19, 2019)

APPENDIX A

Borehole Records and Laboratory Test Results (Current Investigation)

Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Record of Borehole

Laboratory Test Results, Figures A1 to A2

Bedrock Core Photograph, Figure A3

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
υ	Poisson's ratio
	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C	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

Notes:
1. $\tau = c' + \sigma' \tan\phi'$
2. shear strength = (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, 60º 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 cm^2 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

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

(b) Cohesive Soils

Consistency	c_u, s_u kPa	c_u, s_u 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^{o} 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		

GOLDER

RECORD OF BOREHOLE No 18-2301 SHEET 1 OF 2 METRIC

PROJECT 1662565-1230			
G.W.P. 4116-01-01	LOCATION N 5029057.3; E 375809.7 NAD 83 MTM ZONE 9 (LAT. 45.397850; LONG. -75.593010)		ORIGINATED BY DG
DIST Eastern	HWY 417	BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, HQ Casing	COMPILED BY ZS
DATUM Geodetic	DATE June 19-20, 2018		CHECKED BY PPL

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
66.1	GROUND SURFACE
0.0	ASPHALTIC CONCRETE
0.1	(SP) Gravelly sand (FILL) Grey Moist
65.5 0.6	(GP) Sandy gravel, contains shale fragments (FILL) Compact Grey Moist
64.6 1.5	(CI/CH) SILTY CLAY to CLAY (WEATHERED CRUST) Stiff Grey-brown Moist
63.1 3.1	(CI/CH) SILTY CLAY to CLAY Stiff Grey Wet
60.6 5.5	(SM) SAND and SILT to Silty SAND, some gravel to gravelly, trace clay, contains cobbles and boulders (TILL) Compact Grey Wet
56.5 9.7	COBBLES and BOULDERS (TILL)

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	GRAB	-				
2	SS	23				
3	SS	6				
4	SS	4				
5	SS	2				
6	SS	1				
7	SS	14	12	40	38	10
8	SS	12				
9	SS	10				
10	SS	15	28	44	24	4
11	SS	23				
12	SS	50/0.13				
13	RC	DD				

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE: 66, 65, 64, 63, 62, 61, 60, 59, 58, 57

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

>96

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ kN/m³

Continued Next Page

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

GOLDER

RECORD OF BOREHOLE No 18-2301

SHEET 2 OF 2

METRIC

PROJECT 1662565-1230

G.W.P. 4116-01-01

LOCATION N 5029057.3; E 375809.7 NAD 83 MTM ZONE 9 (LAT. 45.397850; LONG. -75.593010)

ORIGINATED BY DG

DIST Eastern HWY 417

BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, HQ Casing

COMPILED BY ZS

DATUM Geodetic

DATE June 19-20, 2018

CHECKED BY PPL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
56.1 10.0	Shale (BEDROCK) Bedrock cored from depths of 10.0 m to 13.2 m For bedrock coring details refer to Record of Drillhole 18-2301		1	RC	REC 100%		56				RQD = 0%
			2	RC	REC 100%		55			UC=42 MPa	RQD = 95%
			3	RC	REC 100%		54				RQD = 100%
							53				
52.9 13.2	END OF BOREHOLE NOTES: 1. Water level in well screen at a depth of 2.1 m below ground surface (Elev. 64.0 m), measured on July 26, 2018.										

$+^{3}, \times^{3}$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF DRILLHOLE: 18-2301

PROJECT: 1662565-1230
LOCATION: N 5029057.3 ;E 375809.7
INCLINATION: -90° AZIMUTH: ---
DRILLING DATE: June 19-20, 2018
DRILL RIG: CME 75
DRILLING CONTRACTOR: Downing Drilling
SHEET 1 OF 1
DATUM: Geodetic

NOTE: For abbreviations, symbols and descriptions refer to **LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY**

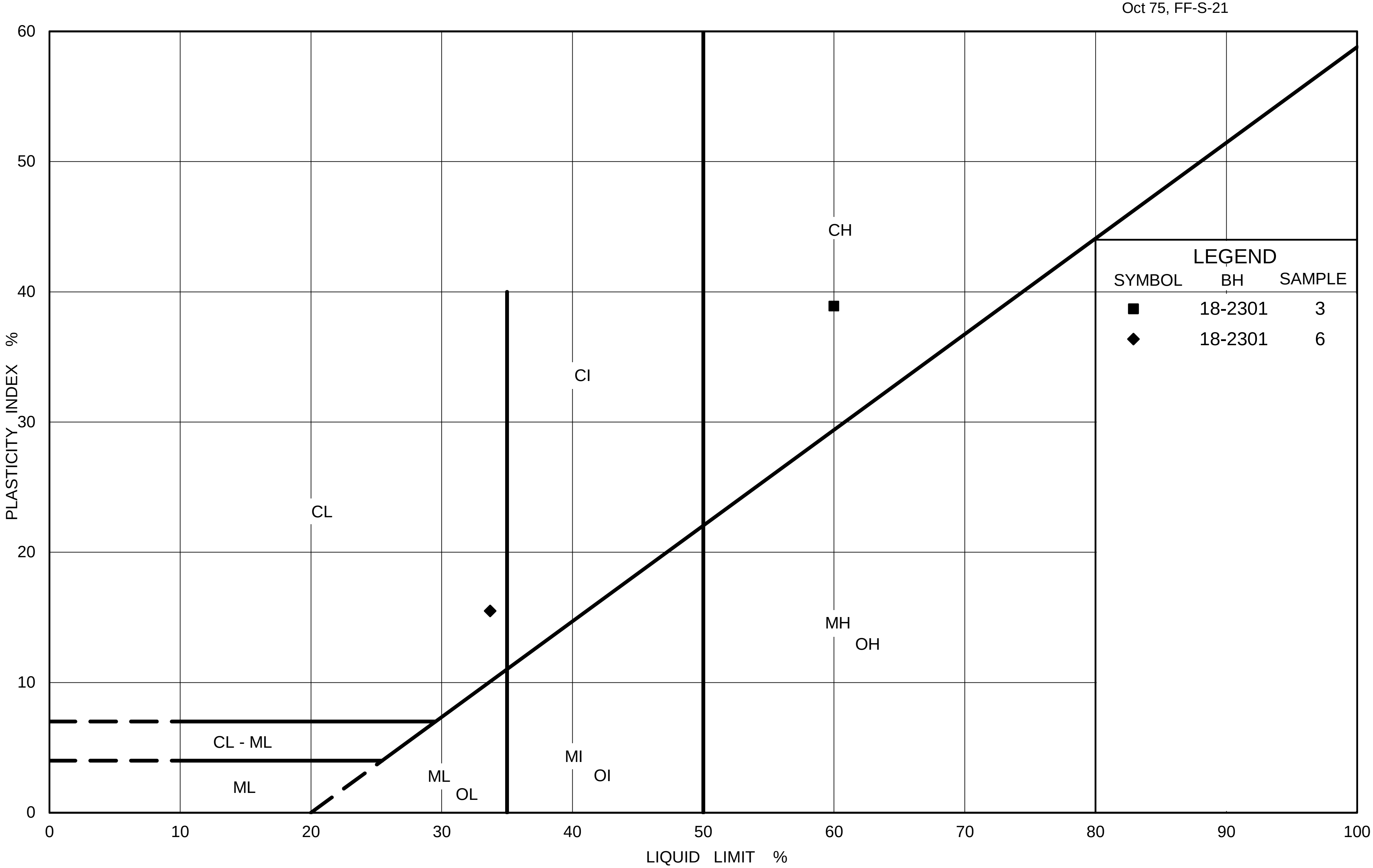
DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	WEATHERING INDEX	FEATURES
		BEDROCK SURFACE		56.02													
11	Rotary Drill, HQ Core	Shale (BEDROCK) Fresh Thinly bedded Fine grained Dark grey to black		10.04	1	100											
12					2	100											UC = 42 MPa
13				52.85 13.21	3	100											
		END OF DRILLHOLE															

Depth scale: 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

DEPTH SCALE
1 : 50

GOLDER

LOGGED: DG
CHECKED: PPL



Ministry of Transportation

Ontario

PLASTICITY CHART

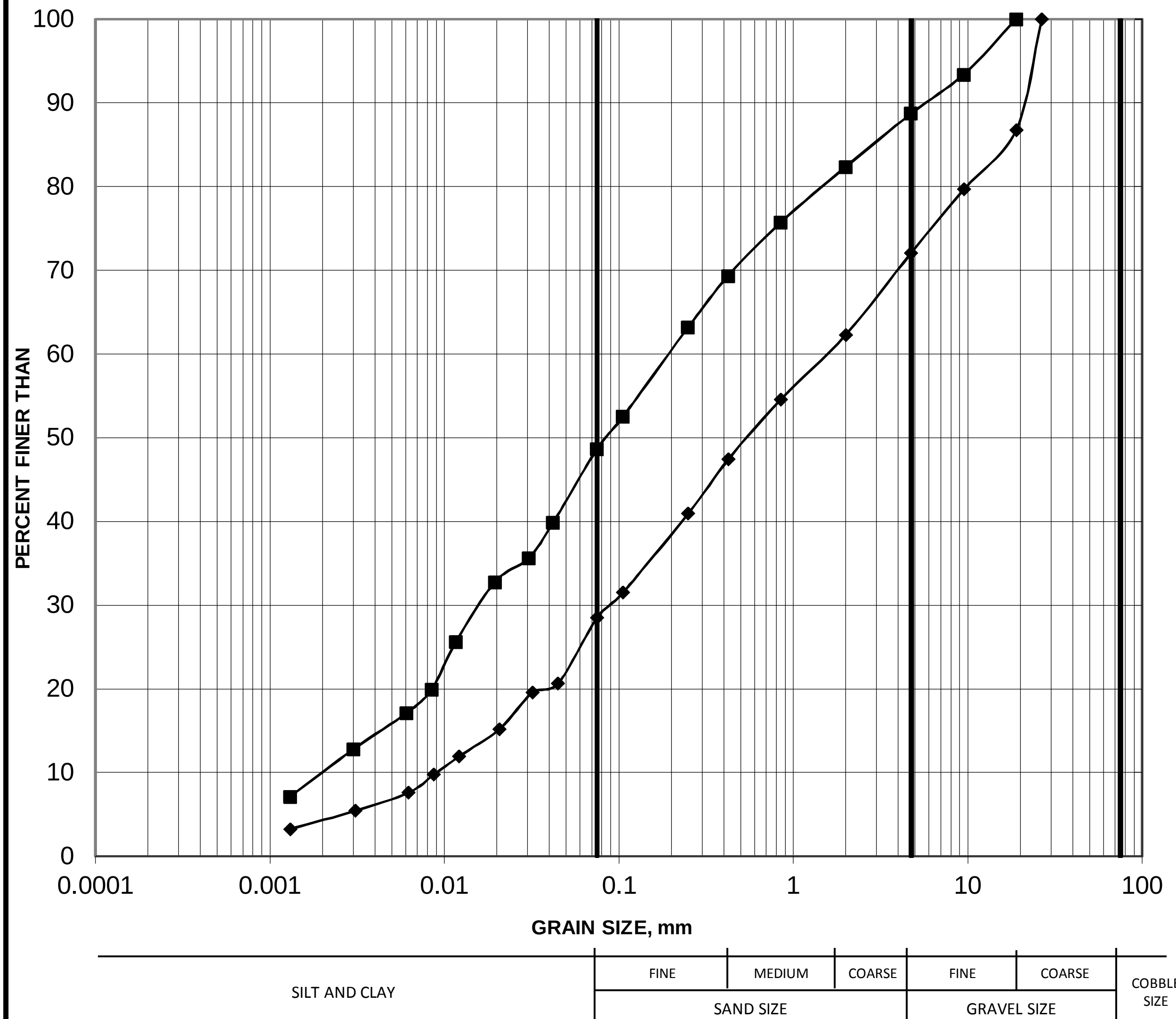
FIG No. A1

Project No. 1662565/1230

Compiled By : MI Checked By : CW

GRAIN SIZE DISTRIBUTION	FIGURE A2

(SM/ML) SAND AND SILT TO SILTY SAND (TILL)



Borehole	Sample	Depth (m)
18-2301	7	5.49-6.10
18-2301	10	7.62-8.23

Project: 1662565/1230

Golder Associates

Created by: MI
Checked by: AC

BH 18-2301 (Wet)
Cored Length of 10.08 to 13.21 metres
Core Box 1 to 2 of 2

10.08 m Top of Bedrock

13.24 m End of Borehole

CLIENT	PROJECT
MMM	Foundation Investigation and Design Highway 417 Walkley Road Underpass Rehabilitation

CONSULTANT		
GOLDER	DD/MM/YYYY	2018-08-16
	PREPARED	
	DESIGN	
	REVIEW	
	APPROVED	

TITLE
BOREHOLE 18-2301 (WET)
CORE PHOTOGRAPHS

PROJECT No	PHASE	Rev.	FIGURE
1662565	1230	A	A3

APPENDIX B

Borehole Records
(Previous Investigation, Geocres No. 31G05-113)

Records of Previous Boreholes BH 1 to BH 7 and BH1A to BH5A

FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 71-11125 LOCATION Co-ords. 499,055 N; 233,136 E. ORIGINATED BY JS

W.P. 10-69-08 BORING DATE Dec. 14 & 15, 1971 COMPILED BY SO

DATUM Geodetic BOREHOLE TYPE NX Washboring CHECKED BY

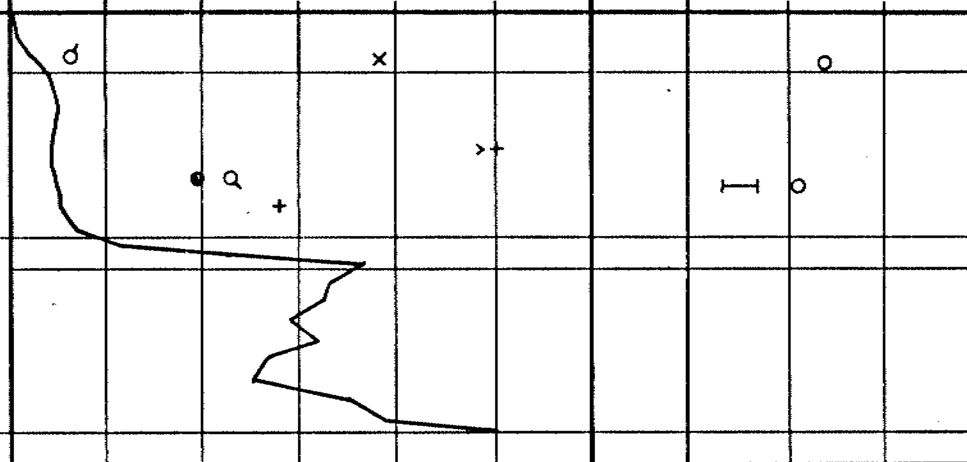
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
213.2	Ground Level							
0.0	Silty clay to clay. Stiff to Very Stiff Grey		1	SS	9		109	
			2	TW	PM	210		
			3	TW	PM			
			4	TW	PM		115	
201.7			5	TW	PM			
11.5	End of Borehole					200		
191.4								
21.8	End of Cone Test					190		

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100

SHEAR STRENGTH P.S.F.: O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE — 400 800 1200 1600 2000

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w

WATER CONTENT %: 15 30 45



FORM OD-MT - 126 (REV 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

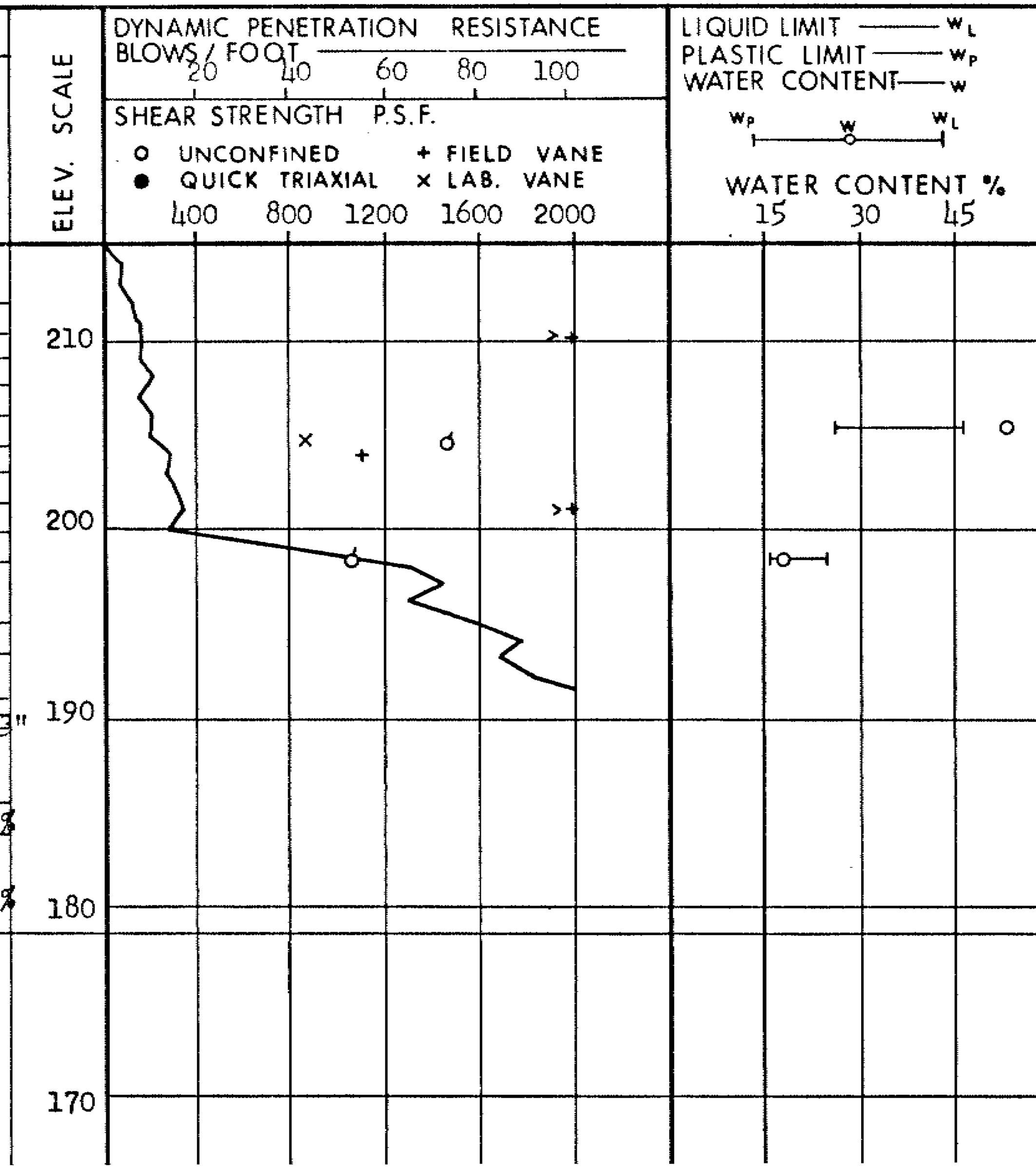
FOUNDATION SECTION

JOB 71-11125
W.P. 10-69-08
DATUM Geodetic

LOCATION Co-ords. 499,002 N; 233,025 E.
BORING DATE Nov. 16 & 17, 1971
BOREHOLE TYPE NX Washboring

ORIGINATED BY WH
COMPILED BY SO
CHECKED BY [signature]

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
215.1	Ground Level						
0.0	Silty clay to clay Very Stiff Grey		1	SS	16		▼213.0
			2	TW	PM		
			3	TW	PM	111.5	
			4	TW	PM		
198.1			5	TW	PM	135	
17.0	Het.mix.of silt,sand & gravel,trace of clay. Dense to Very Dense Grey		6	SS	33		
			7A	BX	17%		
			7	SS	70/3"		
185.6			8	BX	35%		
29.5	Shale Bedrock Sound Grey		9	BX	100%		
178.6			10	BX	100%		
36.5	End of Borehole						



FORM OD-MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

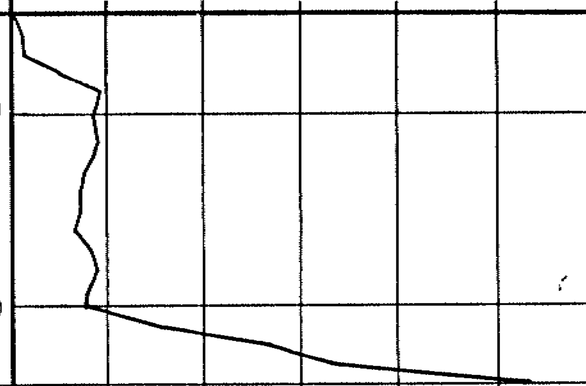
FOUNDATION SECTION

JOB 71-11125
LOCATION Co-ords. 498,990 N; 232,920 E.
ORIGINATED BY M.D.

W.P. 10-69-08
BORING DATE Dec. 18, 1971
COMPILED BY SO

DATUM Geodetic
BOREHOLE TYPE Cone Test Only
CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE				
215.0	Ground Level									
0.0						210				
						200				
196.0 19.0	End of Cone					190				



FORM OD-MT - 126 (REV 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

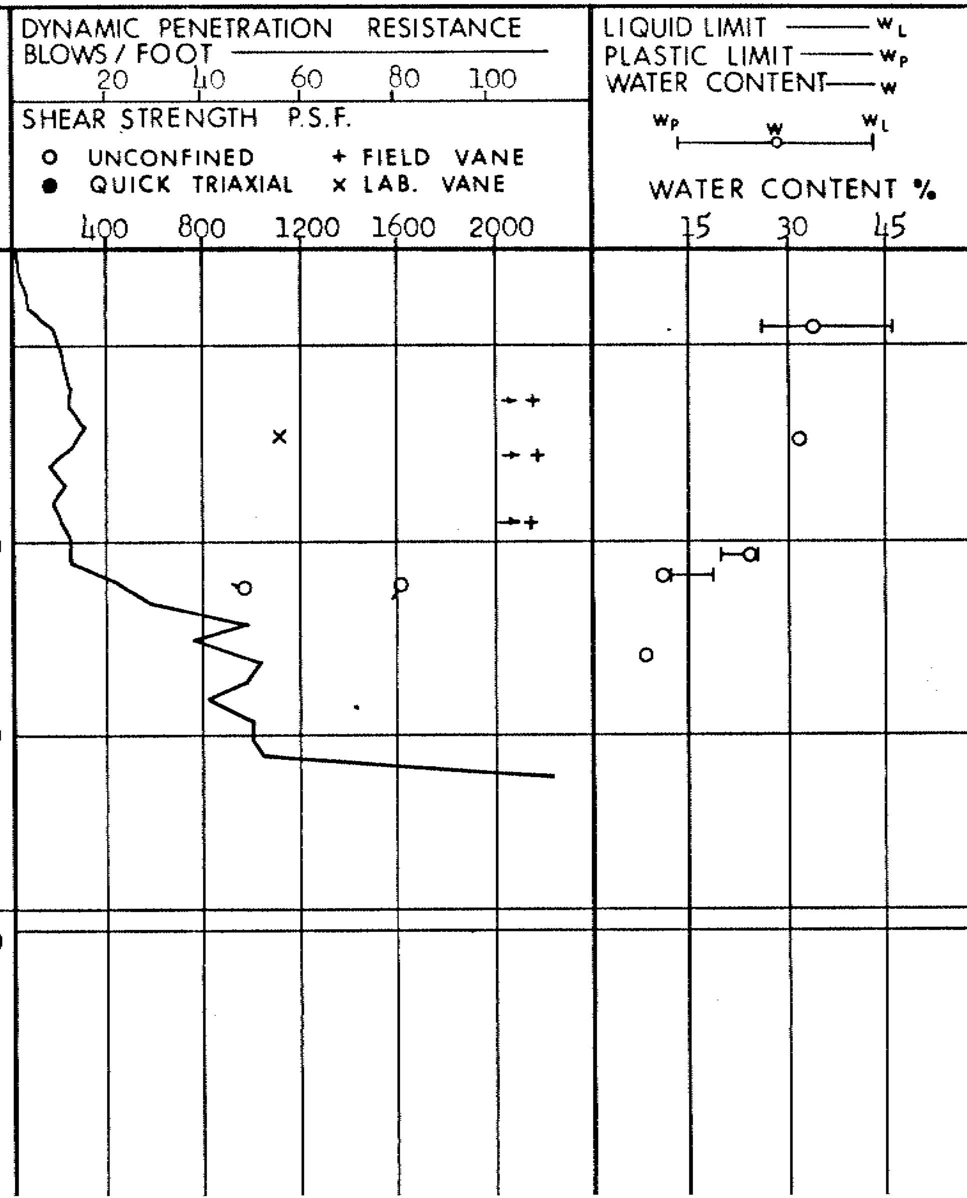
FOUNDATION SECTION

JOB 71-11125
W.P. 10-69-08
DATUM Geodetic

LOCATION Co-ords. 498,894 N; 232,932 E.
BORING DATE Dec. 15 & 16, 1971
BOREHOLE TYPE NX Washboring

ORIGINATED BY JS
COMPILED BY SO
CHECKED BY [signature]

SOIL PROFILE ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
215.1	Ground Level							
0.0	Silty clay to clay.		1	SS	15			0 11 56 33
	Very Stiff		2	TW	PM	210		
	Grey		3	TW	PM		112	
			4	TW	PM			
			5	SS	15			
199.4			6	TW	PM	200	128	
16.7	Het.mix.of silt,sand & gravel,trace of clay		7	SS	36			
	Glacial Till		8	SS	41			
	Dense to Very Dense		9	SS	80/4"	190		
187.6			10	BX	54%			
27.5	Shale Bedrock		11	BX	100%			
181.0	Sound, Grey		12	BX	100%			
34.1	End of Borehole					180		



FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

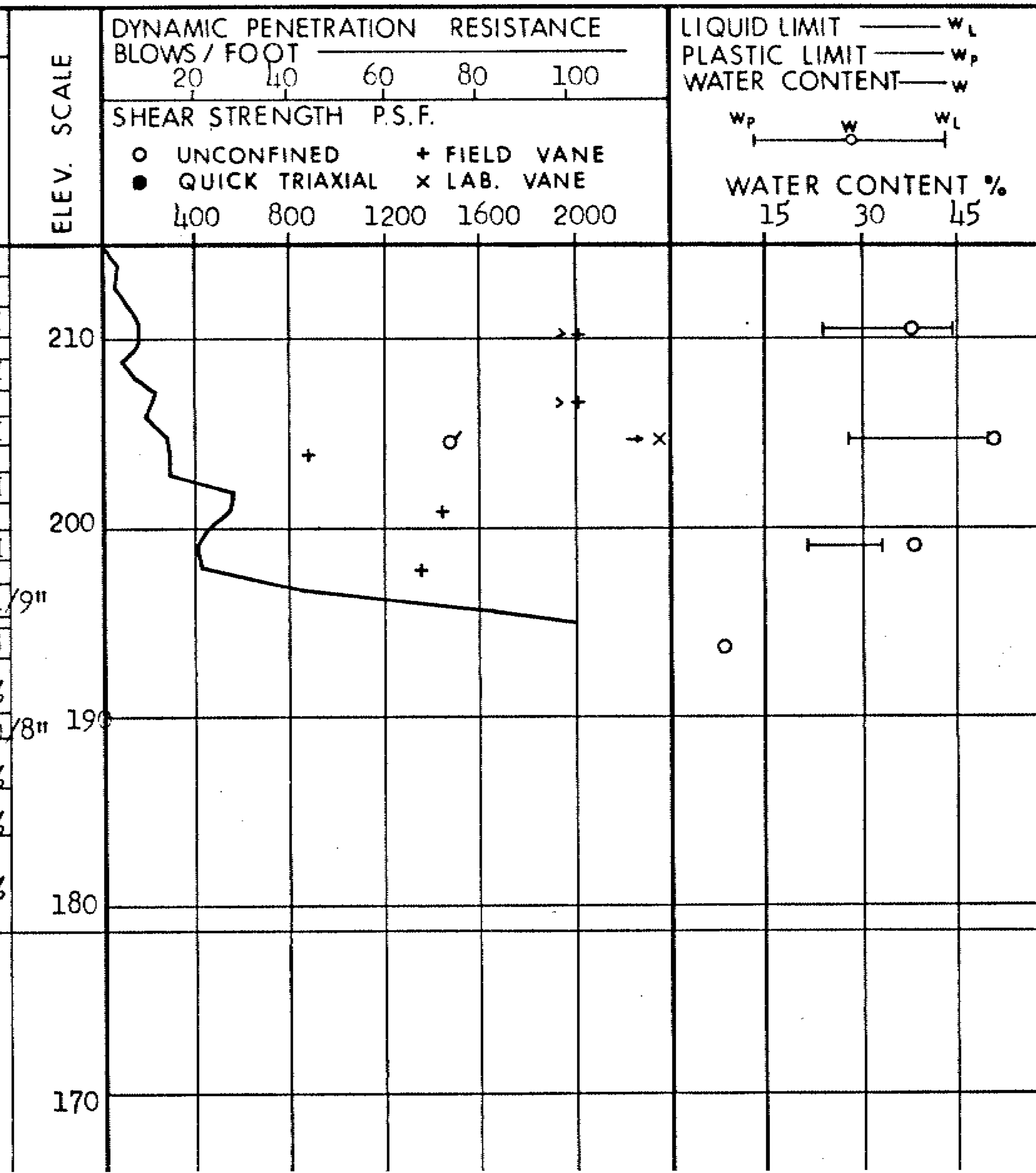
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

FOUNDATION SECTION

JOB 71-11125	LOCATION Co-ords. 498,934 N; 232,805 E.	ORIGINATED BY JS
W.P. 10-69-08	BORING DATE Dec. 16 & 17, 1971	COMPILED BY SO
DATUM Geodetic	BOREHOLE TYPE NX Washboring	CHECKED BY [illegible]

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
214.9	Ground Level					
0.0	Silty clay to clay Stiff to Hard Grey	1	SS	9		212.9
		2	TW	PM		0 6 59 35
		3	TW	PM		
		4	TW	PM		
		5	TW	PM		
196.9		6	TW	PM	117	
18.0	Het.mix.of silt,sand & gravel,trace of clay Occ. boulders up to 6" in size. Glacial Till	7	SS	144/9"		
		8	SS	98		26 35 34
		9	BX	30%		
		10	SS	150/8"		
		11	BX	15%		
183.9		12	BX	13%		
31.0	Shale Bedrock Sound Grey	13	BX	92%		
178.9						
36.0	End of Borehole					



FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

JOB 71-11125

LOCATION Co-ords. 498,834 N; 232,820 E.

ORIGINATED BY MD

W.P. 10-69-08

BORING DATE Dec. 18, 1971

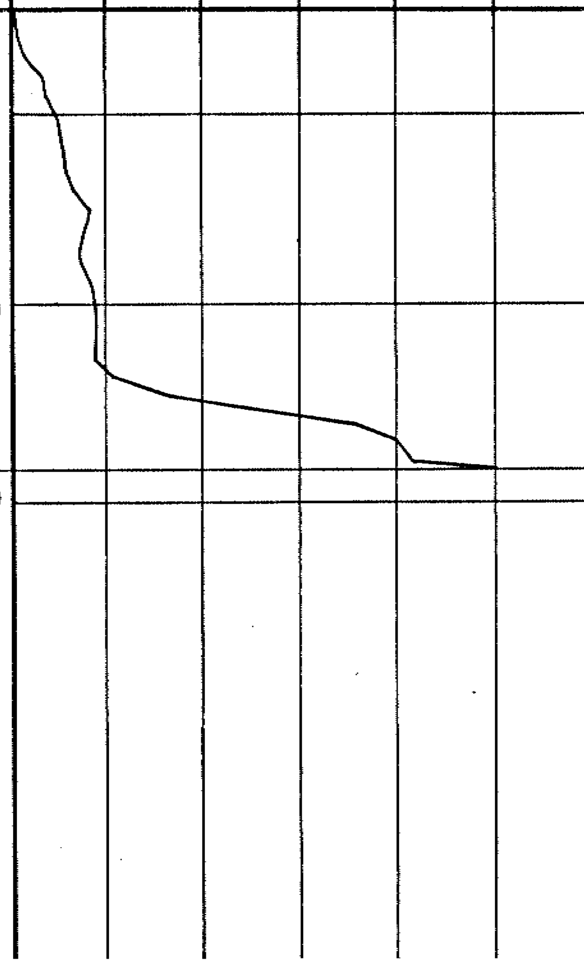
COMPILED BY SO

DATUM Geodetic

BOREHOLE TYPE Cone Test Only

CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE				
215.2	Ground Level									
0.0						210				
						200				
191.7										
23.5	End of Cone					190				



FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 7

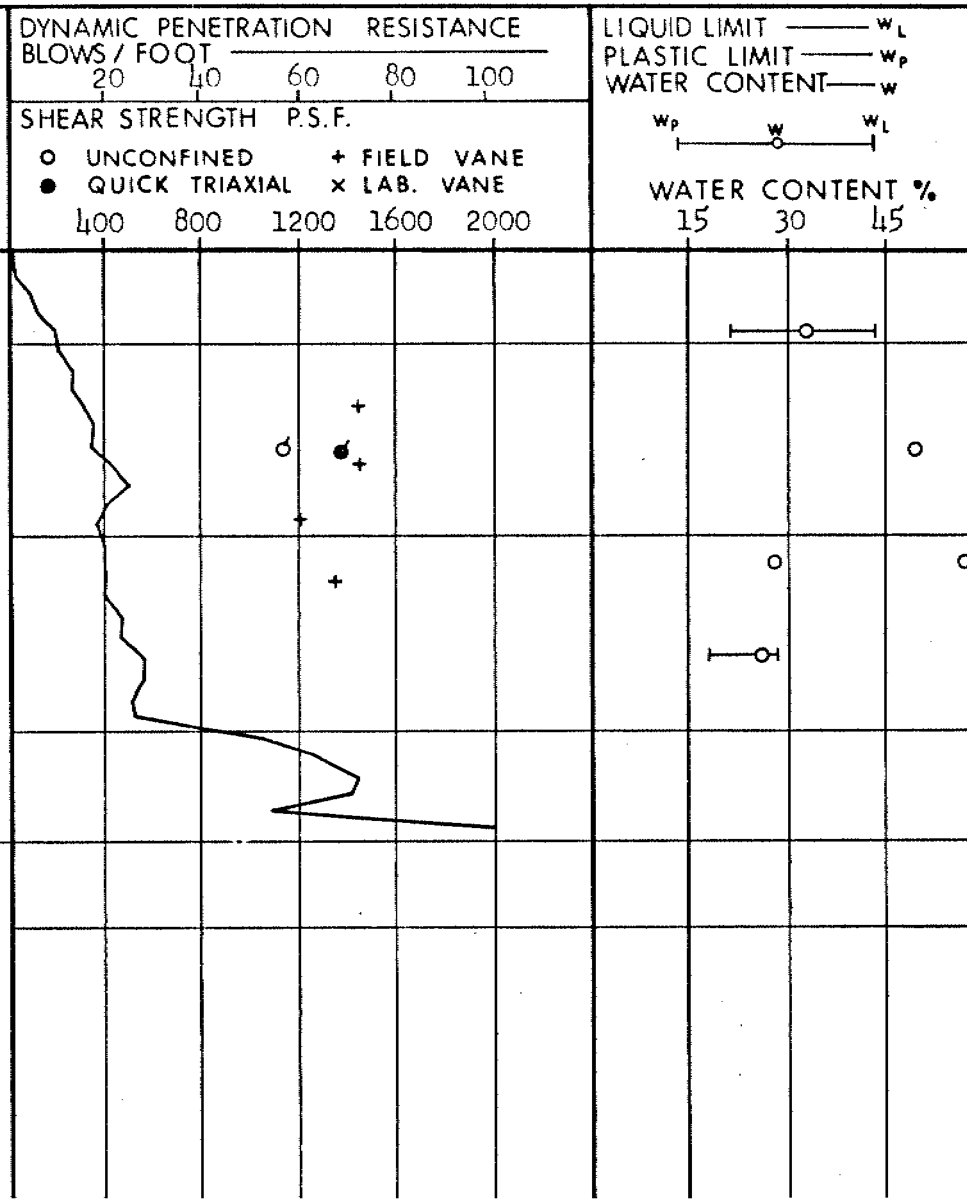
FOUNDATION SECTION

JOB 71-11125 LOCATION Co-ords. 498,840 N; 232,721 E. ORIGINATED BY

W.P. 10-69-08 BORING DATE Dec. 16, 1971 COMPILED BY S.O.

DATUM Geodetic BOREHOLE TYPE NX Washboring CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
214.9	Ground Level							
0.0	Silty clay to Clay							213.4
			1	SS	13	210		
	Stiff to Very Stiff		2	TW	PM			
			3	TW	PM		107	
	Grey		4	TW	PM			
			5	TW	PM	200	115	
196.9								
18.0	Het.mix.of silt,sand & gravel,trace of clay.		6	TW	PM			
			7	SS	5			6 18 54 22
			8	SS	21			17 25 40 18
	Glacial Till					190		
	Loose to Very Dense							
184.8	Probable Bedrock		9	SS	135/9"			
30.1	End of Borehole							
						180		



FORM OB-MT-12? (REV. 1971)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 1A

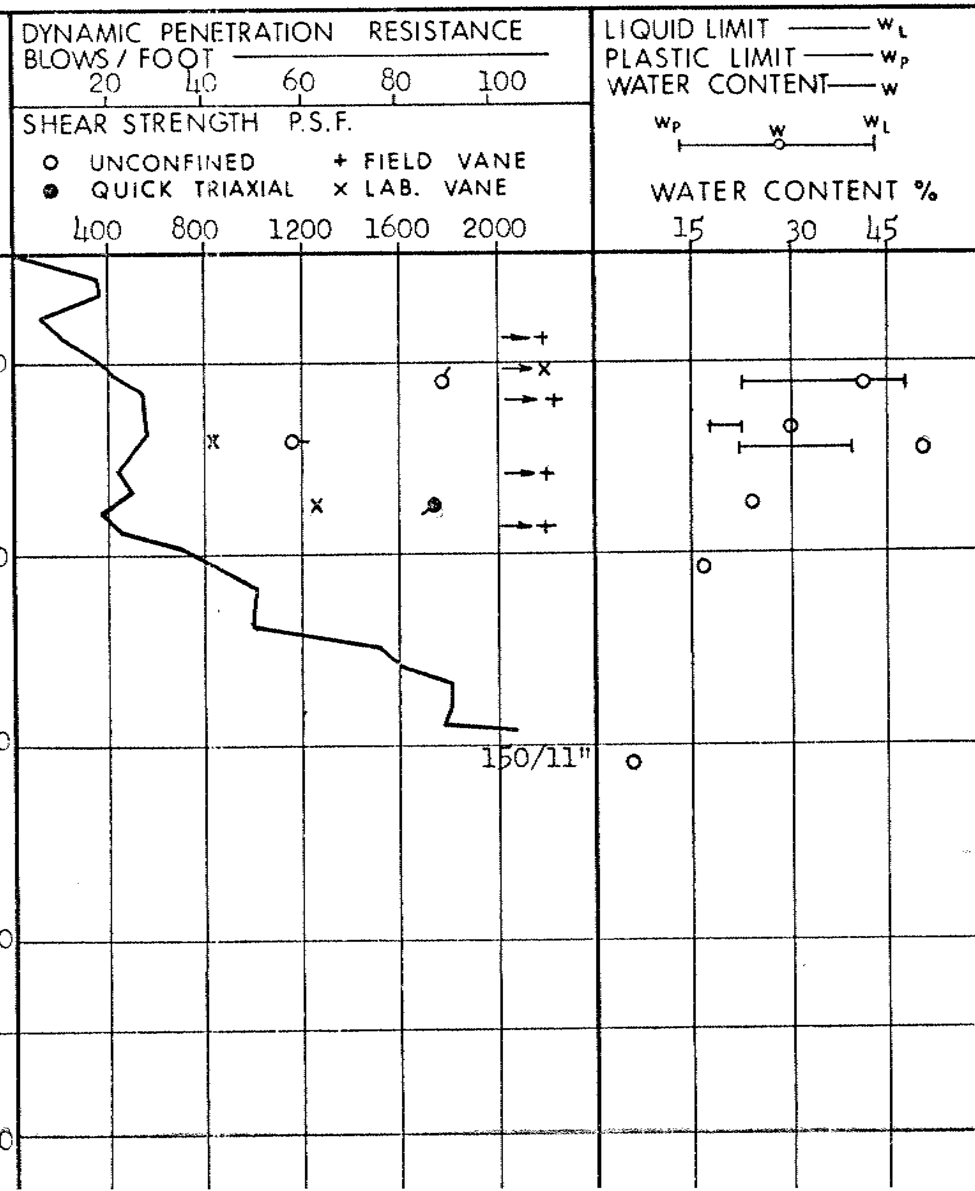
FOUNDATION SECTION

JOB 71-11125 LOCATION Co-ords. 16,498,890 N; 1,233,074 E. ORIGINATED BY SAA

W.P. 10-69-08 BORING DATE April 21, 1972 COMPILED BY SAA

DATUM Geodetic BOREHOLE TYPE Auger and BX Rock Core CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
215.6	Ground Level							
0.0	Silty clay, trace of sand. Stiff to Very Stiff Grey		1	SS	13			214.0 in open BH Apr.24/72
			2	TW	PH	210	112	
			3	TW	PH		117.5	
			4	TW	PH		127.5	
198.6			5	SS	19	200		
17.0	Het.mix.of silt,sand & gravel, trace of clay. Glacial Till Compact to Very Dense Grey		6	SS	16			
			7	SS	50/6"	190		41 31 22 6
			8	SS	60/1"			
181.6	Grey boulders up to 6" in size		9	BX	15%			
34.0	Sound Shale Bedrock		10	BX	10%	180		
175.6								
40.0	End of Borehole					170		



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 2A

FOUNDATION SECTION

JOB 71-11125

W.P. 10-69-08

DATUM Geodetic

LOCATION Co-ords. 16,498,746 N; 1,232,806 E.

BORING DATE April 24, 1972

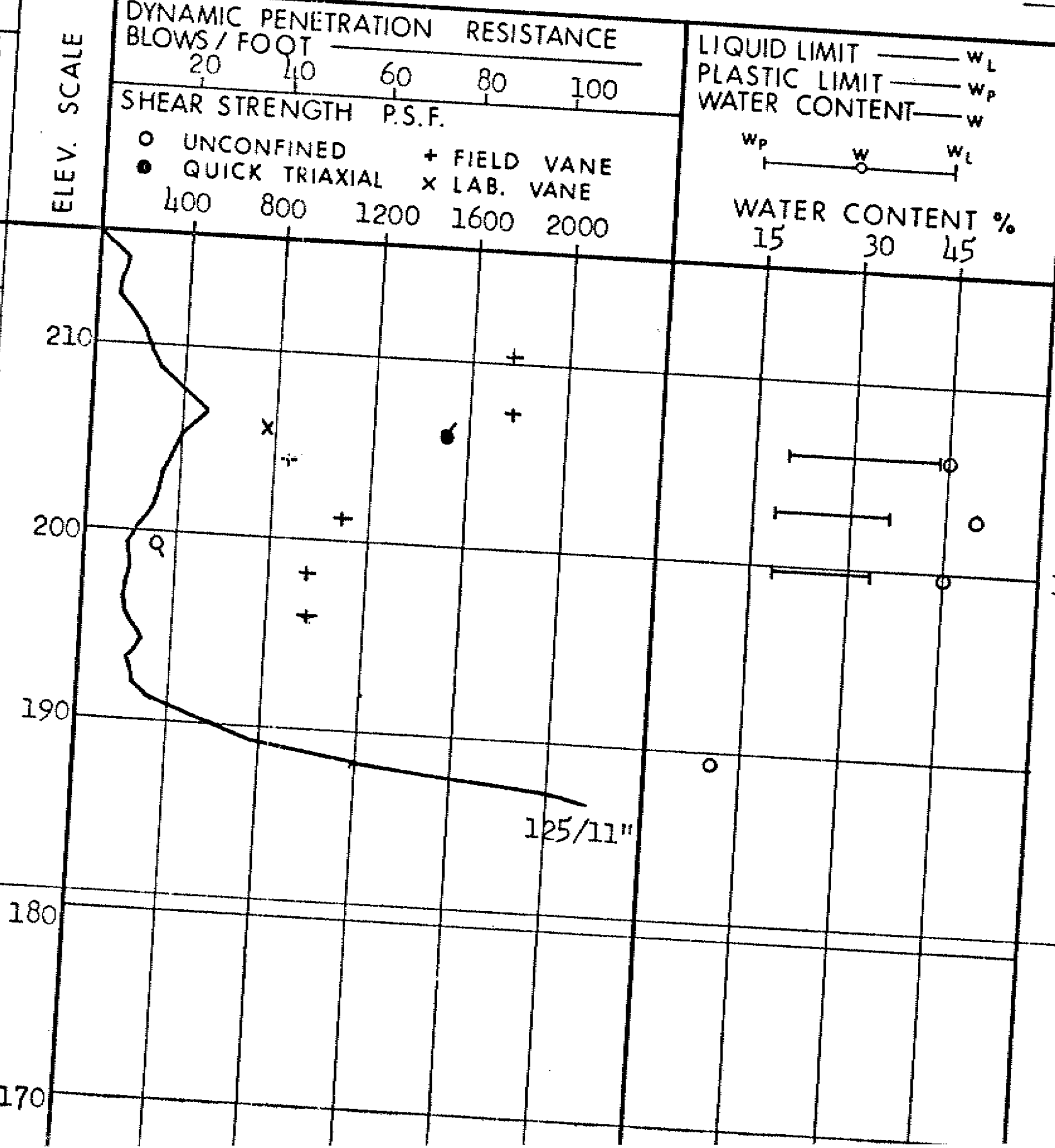
BOREHOLE TYPE Auger and BX Rock Core

ORIGINATED BY SAA

COMPILED BY SAA

CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT
15.6	Ground Level			
0.0	Silty clay, trace of sand and gravel. Firm to Stiff. Grey	1	SS	11
		2	TW	PH
		3	TW	PH
		4	TW	PH
		5	TW	PH
.6		6	TW	PH
.0	Het.mix.of silt,sand & gravel,trace of clay. Glacial Till. Compact. Grey	7	SS	10
		8	SS	28
.3				
.3	Shale Bedrock. Sound. Grey	9	RC	60%
.1		10	RC	100%
.5	End of Borehole			



BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
	▽ 214.6 in open BH Apr.24/72
110	0 3 51 46
111	
	16 33 46 5

FORM OB-MT-126 (REV. 1971)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 3A

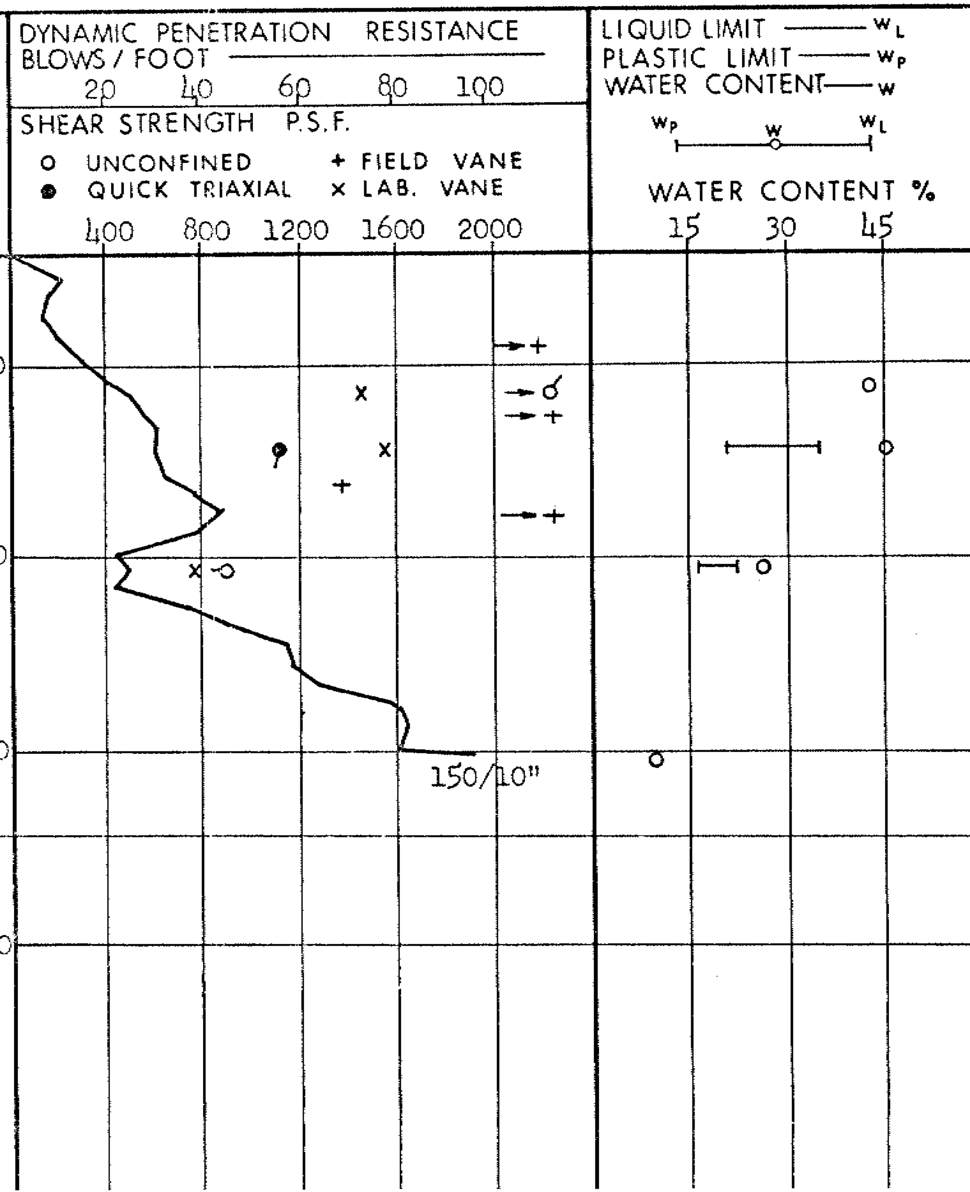
FOUNDATION SECTION

JOB 71-11125 | LOCATION Co-ords. 16,498,817 N; 1,232,938 E. | ORIGINATED BY SAA

W.P. 10-69-08 | BORING DATE April 25, 1972 | COMPILED BY SAA

DATUM Geodetic | BOREHOLE TYPE Auger and BX Rock Core | CHECKED BY

ELEV. DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
215.5	Ground Surface						▽ 215.5 in open BH Apr. 25/72
0.0	Silty clay, trace of sand and gravel	1	SS	14			
	Firm to Very Stiff	2	TW	PH	210		
	Grey	3	TW	PH		110	0 1 65 34
		4	TW	PH			
199.5		5	TW	PH	200		
16.0	Het. mix. of silt, sand & gravel, trace of clay. Glacial Till	6	SS	34			
	Dense to Very Dense Grey	7	SS	104	190		20 39 34 7
186.0	Probable Bedrock						
29.5	End of Borehole				180		



FORM OB-MT-126 (REV. 1971)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 4A

FOUNDATION SECTION

JOB 71-11125 — LOCATION Co-ords. 16,498,955 N; 1,233,069 E. — ORIGINATED BY SAA

W.P. 10-69-08 — BORING DATE April 25, 1972 — COMPILED BY SAA

DATUM Geodetic — BOREHOLE TYPE Auger - BX Rock Core — CHECKED BY [signature]

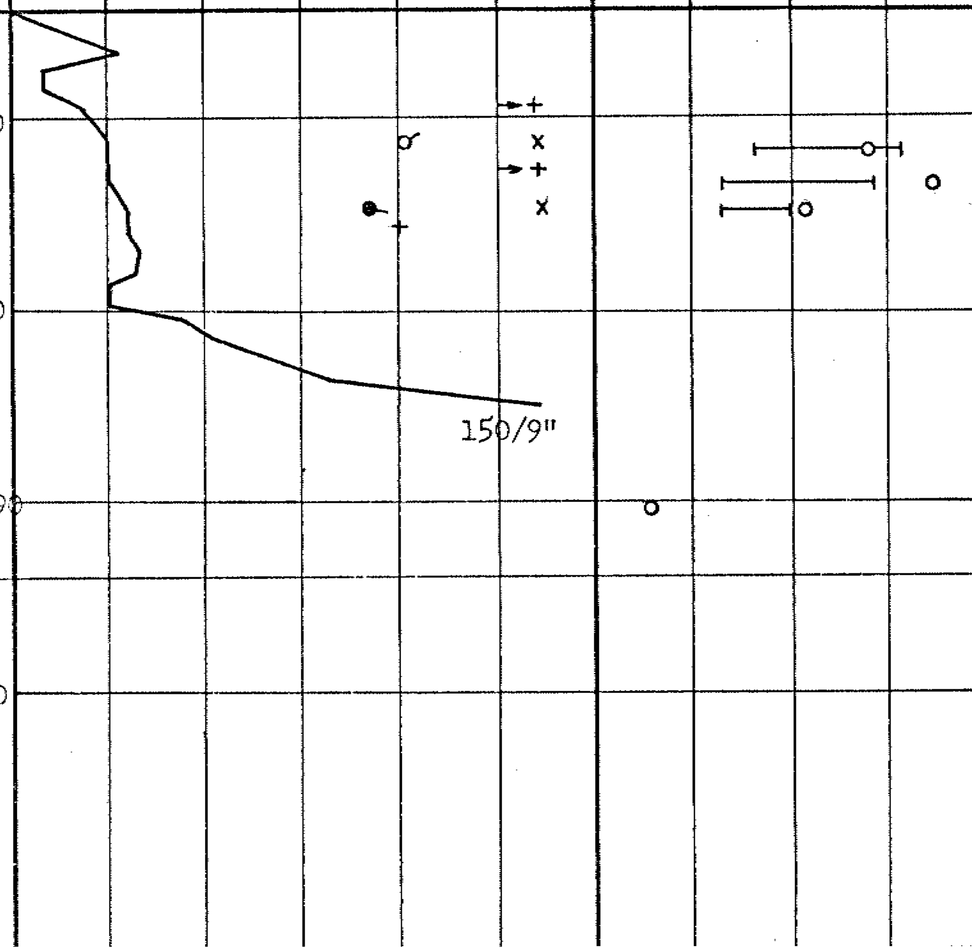
SOIL PROFILE ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
215.7	Ground Level							214.7 in open BH Apr. 25/72
0.0	Silty clay, trace of sand and gravel. Stiff to Very Stiff Grey		1	SS	15	210		
			2	TW	PH		113	0 2 48 50
								0 2 52 46
			3	TW	PH		111.5	0 1 85 14
			4	TW	PH			
200.7 15.0	Het.mix.of silt,sand & gravel, trace of clay Glacial Till Dense to Very Dense Grey		5	SS	32	200		
			6	SS	57			
			7	SS	100/3"	190		31 37 27 5
186.7	Probable Bedrock							
29.0	End of Borehole					180		

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 400 800 1200 1600 2000

LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w

WATER CONTENT % 15 30 45



FORM OB-MT-126 (REV. 1971)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 5A

FOUNDATION SECTION

JOB 71-11125

LOCATION Co-ords. 16,498,738 N; 1,232,754 E.

ORIGINATED BY SAA

W.P. 10-69-08

BORING DATE April 25, 1972

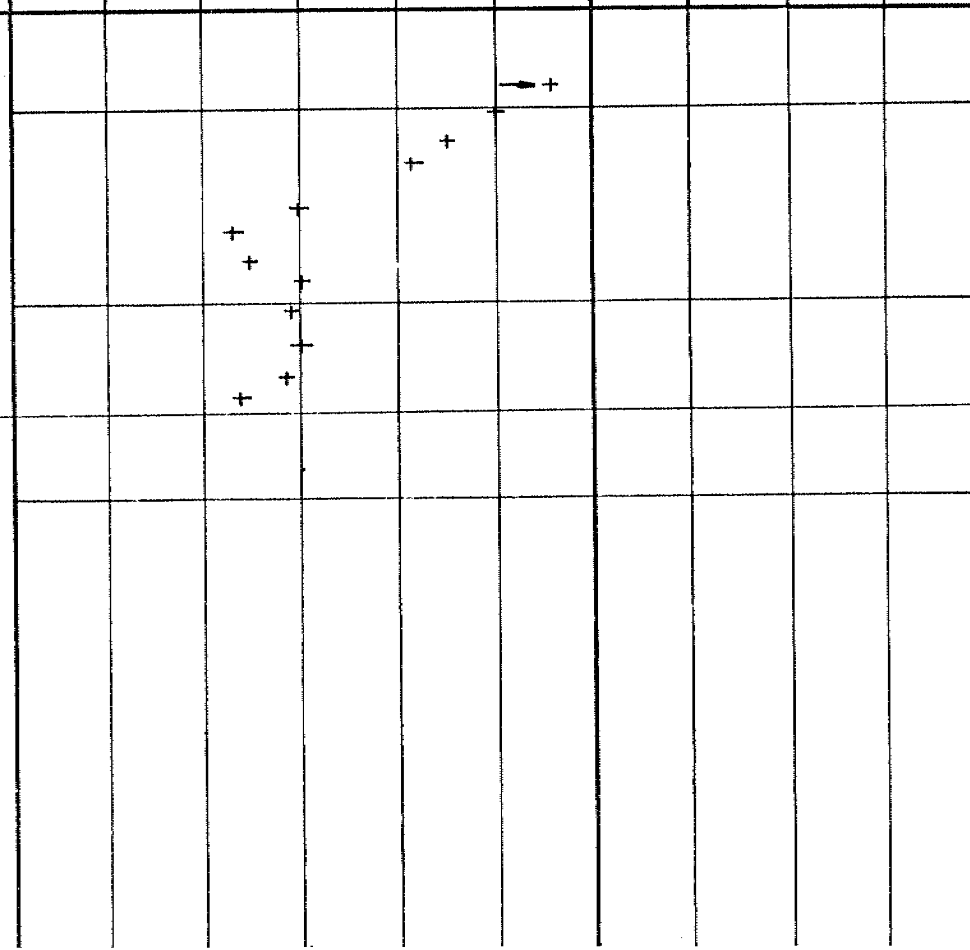
COMPILED BY SAA

DATUM Geodetic

BOREHOLE TYPE Vane Hole Only

CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE) 400 800 1200 1600 2000	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE				
215.0	Ground Level									
	Silty clay, trace of sand and gravel. Firm to Very Stiff Grey					210 200				
194.0 21.0	Het.mix.of silt,sand & gravel,trace of sand.					190				



APPENDIX C

Results of Chemical Analysis

Eurofins Environment Testing Report No. 1811817

eurofins Environment Testing

Certificate of Analysis

Client: Golder Associates Ltd. (Ottawa)
1931 Robertson Road
Ottawa, ON
K2H 5B7
Attention: Mr. Alex Meacoe
PO#:
Invoice to: Golder Associates Ltd. (Ottawa)

Report Number: 1811817
Date Submitted: 2018-07-09
Date Reported: 2018-07-13
Project: 1662565/1230
COC #: 188737

Group	Analyte	MRL	Units	Guideline	Lab I.D. 1372546 / Sample Matrix Soil / Sample Type / Sampling Date 2018-07-09 / Sample I.D. BH 18-2301 SA 6/15-17
Anions	Cl	0.002	%		0.031
	SO4	0.01	%		0.02
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.36
	pH	2.00			8.15
	Resistivity	1	ohm-cm		2780

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX D

Multichannel Analysis of Surface Waves (MASW) Test Result

TECHNICAL MEMORANDUM

DATE June 06, 2019 **Project No.** 1662565 / 1230

TO Susan Trickey
Golder Associates Ltd.

FROM Stephane Sol / Christopher Phillips **EMAIL ssol@golder.com ; cphillips@golder.com**

CHBDC SEISMIC SITE CLASS TESTING RESULTS
WALKLEY ROAD AND HIGHWAY 417, OTTAWA, ONTARIO

This technical memorandum presents the results of two Multichannel Analysis of Surface Waves (MASW) tests performed for the purpose of the Canadian Highway Bridge Design Code (CHBDC 2014) Seismic Site Classification (Figure 1). The tests are located on each side of the interchange between Walkley Road and Highway 417 in Ottawa. The geophysical testing was performed by Golder Associates Ltd. (Golder) personnel on July 17, 2018.

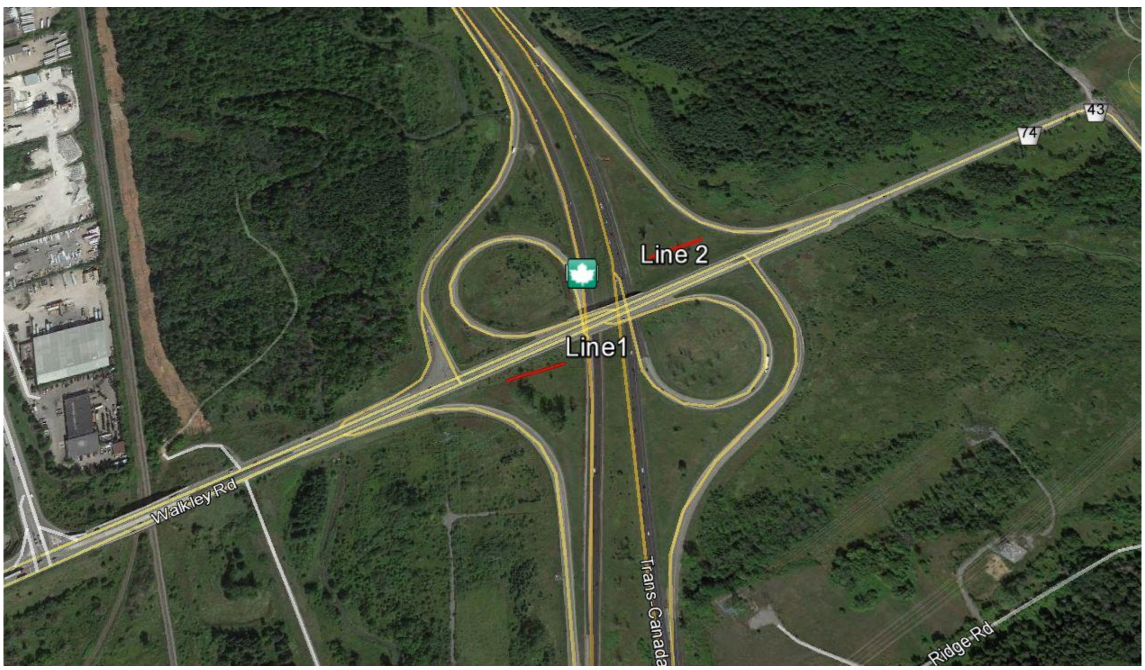


Figure 1: MASW Location Site Map (MASW Lines in red – Line 1 (South) and Line 2 (North)

Methodology

The MASW method measures variations in surface-wave velocity with increasing distance and wavelength and can be used to infer the rock/soil types, stratigraphy and soil conditions.

A typical MASW survey requires a seismic source, to generate surface waves, and a minimum of two geophone receivers, to measure the ground response at some distance from the source. Surface waves are a special type of seismic wave whose propagation is confined to the near surface medium.

The depth of penetration of a surface wave into a medium is directly proportional to its wavelength. In a non-homogeneous medium, surface waves are dispersive, i.e., each wavelength has a characteristic velocity owing to the subsurface heterogeneities within the depth interval that particular wavelength of surface wave propagates through. The relationship between surface-wave velocity and wavelength is used to obtain the shear-wave velocity and attenuation profile of the medium with increasing depth.

The seismic source used can be either active or passive, depending on the application and location of the survey. Examples of active sources include explosives, weight-drops, sledge hammer and vibrating pads. Examples of passive sources are road traffic, micro-tremors, and water-wave action (in near-shore environments).

The geophone receivers measure the wave-train associated with the surface wave travelling from a seismic source at different distances from the source.

The participation of surface waves with different wavelengths can be determined from the wave-train by transforming the wave-train results into the frequency domain. The surface-wave velocity profile with respect to wavelength (called the ‘dispersion curve’) is determined by the delay in wave propagation measured between the geophone receivers. The dispersion curve is then matched to a theoretical dispersion curve using an iterative forward-modelling procedure. The result is a shear-wave velocity profile of the tested medium with depth, which can be used to estimate the dynamic shear-modulus of the medium as a function of depth.

Field Work

The MASW field work was conducted on July 17, 2018, by personnel from the Golder Mississauga office. Two MASW were collected and their location is indicated in Table 1. For each MASW line, a series of 24 low frequency (4.5 Hz) geophones were laid out at 3 m intervals. Both active and passive readings were recorded along the MASW lines. For the active investigation, a seismic drop of 45 kg and a 9.9 kg sledge hammer were used as seismic sources. Active seismic records were collected with seismic sources located 5, 10, and 15 m from and collinear to the geophone array. An example of active seismic records collected at each line are shown in Figures 2 and 3, below.

Table 1: Location of MASW Lines

Location	MTM NAD83 Zone 9 Northing (m)	MTM NAD83 Zone 9 Easting (m)
Line 1 - Start	5029022	375773
Line 1 - End	5028999	375709
Line 2 - Start	5029171	375870
Line 2 - End	5029196	375933

The horizontal datum is MTM NAD83 Zone 9.

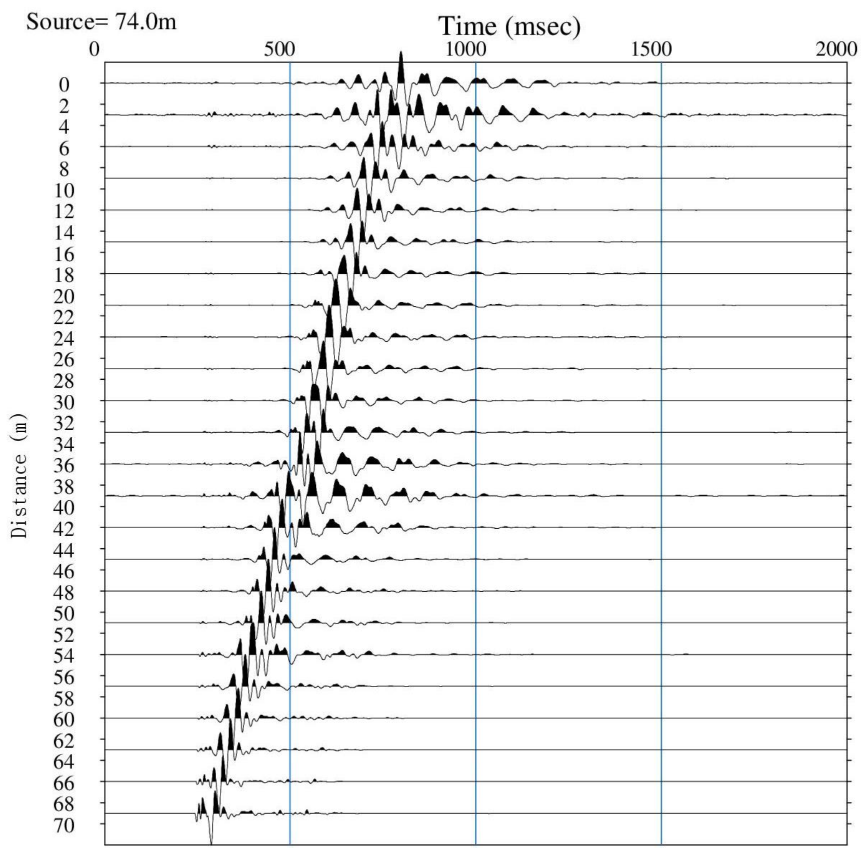


Figure 2: Typical seismic record collected at the site of MASW Line 1.

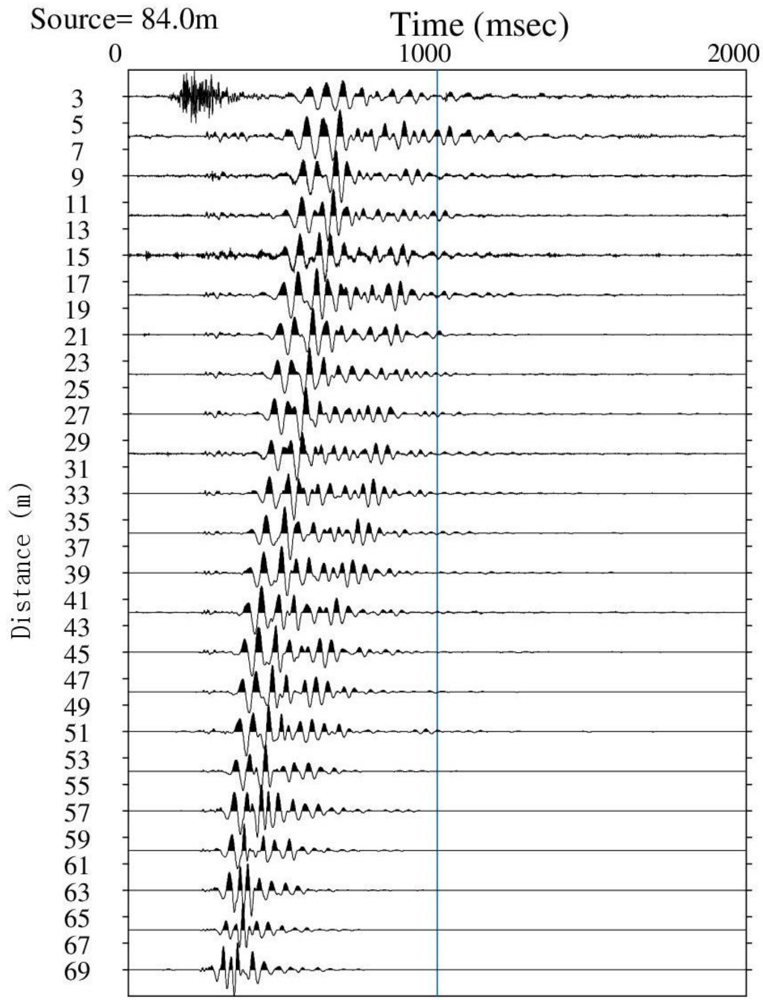


Figure 3: Typical seismic record collected at the site of MASW Line 2.

Data Processing

Processing of the MASW test results consisted of the following main steps:

1) Transformation of the time domain data into the frequency domain using a Fast-Fourier Transform (FFT) for each source location;
2) Calculation of the phase for each frequency component;
3) Linear regression to calculate phase velocity for each frequency component;
4) Filtering of the calculated phase velocities based on the Pearson correlation coefficient (r2) between the data and the linear regression best fit line used to calculate phase velocity;

5) Generation of the dispersion curve by combining calculated phase velocities for each shot location of a single MASW test; and,

6) Generation of the stiffness profile, through forward iterative modelling and matching of model data to the field collected dispersion curve.

Processing of the MASW data was completed using the SeisImager/SW software package (Geometrics Inc.). The calculated phase velocities for a seismic shot point were combined and the dispersion curve generated by choosing the minimum phase velocity calculated for each frequency component as shown on Figure 4 for Line 1 and Figure 5 for Line 2. Shear wave velocity profiles were generated through inverse modelling to best fit the calculated dispersion curves. The active survey of Line 1 provided a dispersion curve with a suitable frequency range (6-40 Hz). The active survey of Line 2 provided a dispersion curve with a suitable frequency range (9-27 Hz). The minimum measured surface wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 6 Hz at Line 1 and 9 Hz at Line 2.

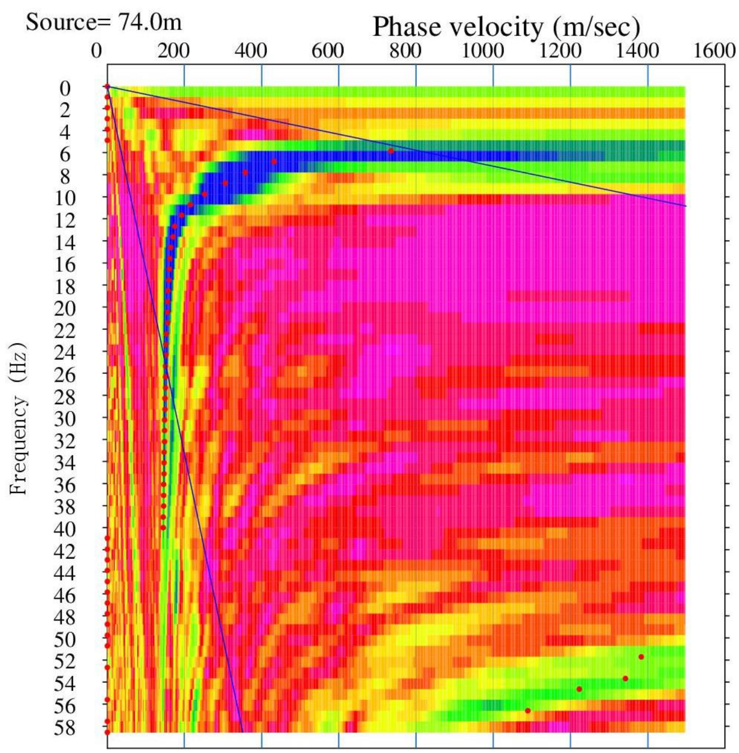


Figure 4: Active MASW Dispersion Curve Picks (red dots) along MASW Line 1

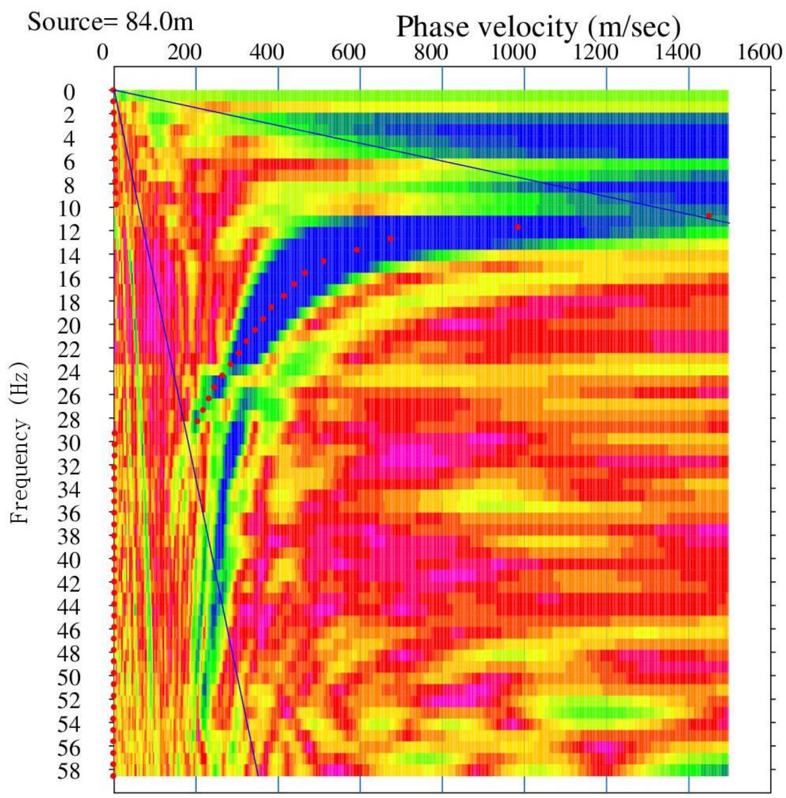


Figure 5: Active MASW Dispersion Curve Picks (red dots) along MASW Line 2

Results

The MASW test results are presented in Figures 6 and 7, which present the calculated shear wave velocity profile derived from the field testing along MASW Lines 1 and 2, respectively. The results along MASW Line 1 have been calculated using a weight-drop located 5 m from the last geophone. The results along MASW Line 2 have also been calculated using a weight-drop located 5 m from the last geophone. The field collected dispersion curves are compared with the model generated dispersion curves on Figures 8 and 9 for MASW Lines 1 and 2, respectively. There is a satisfactory correlation between the field collected and model calculated dispersion curves, with a root mean squared error of less than 5% along both lines.

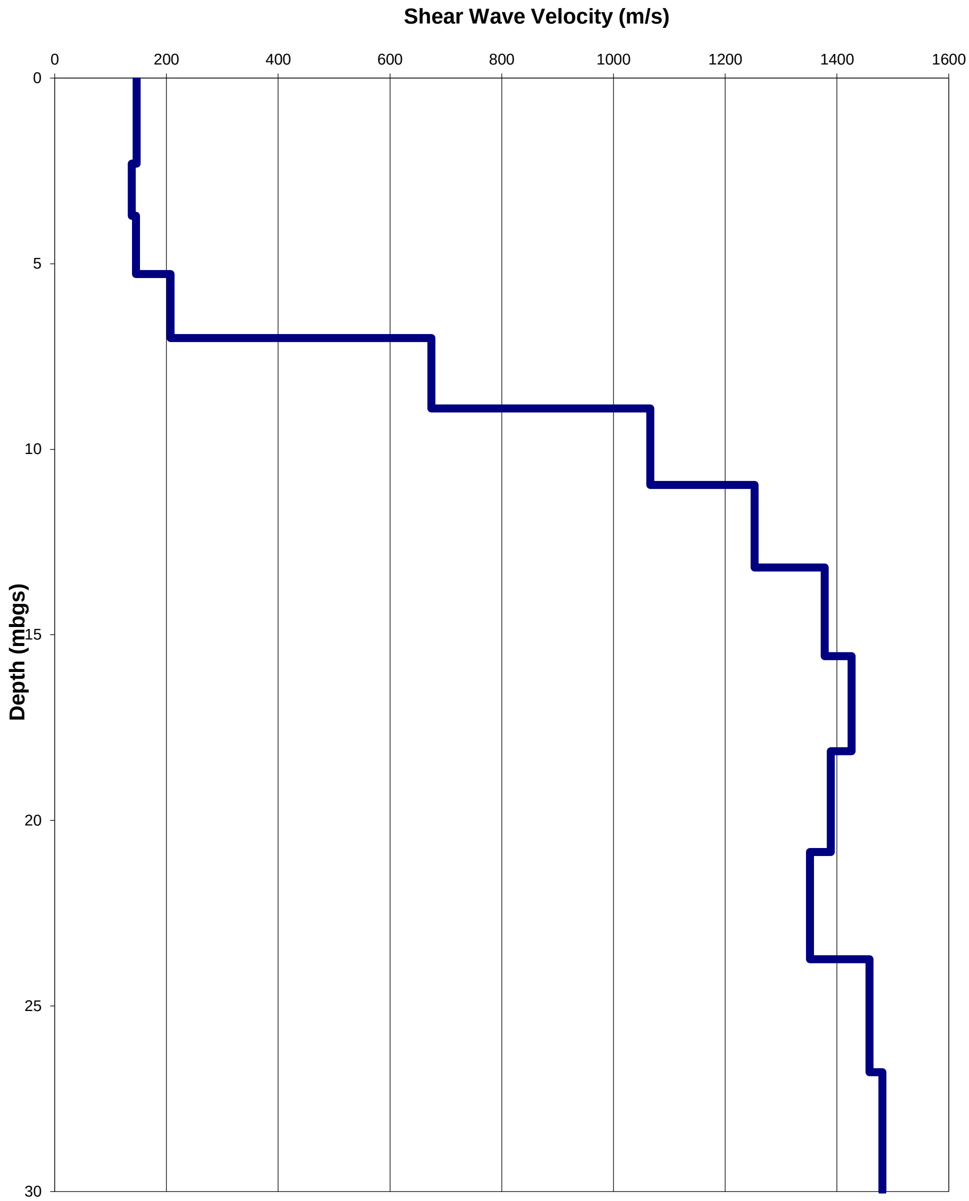


Figure 6: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 1

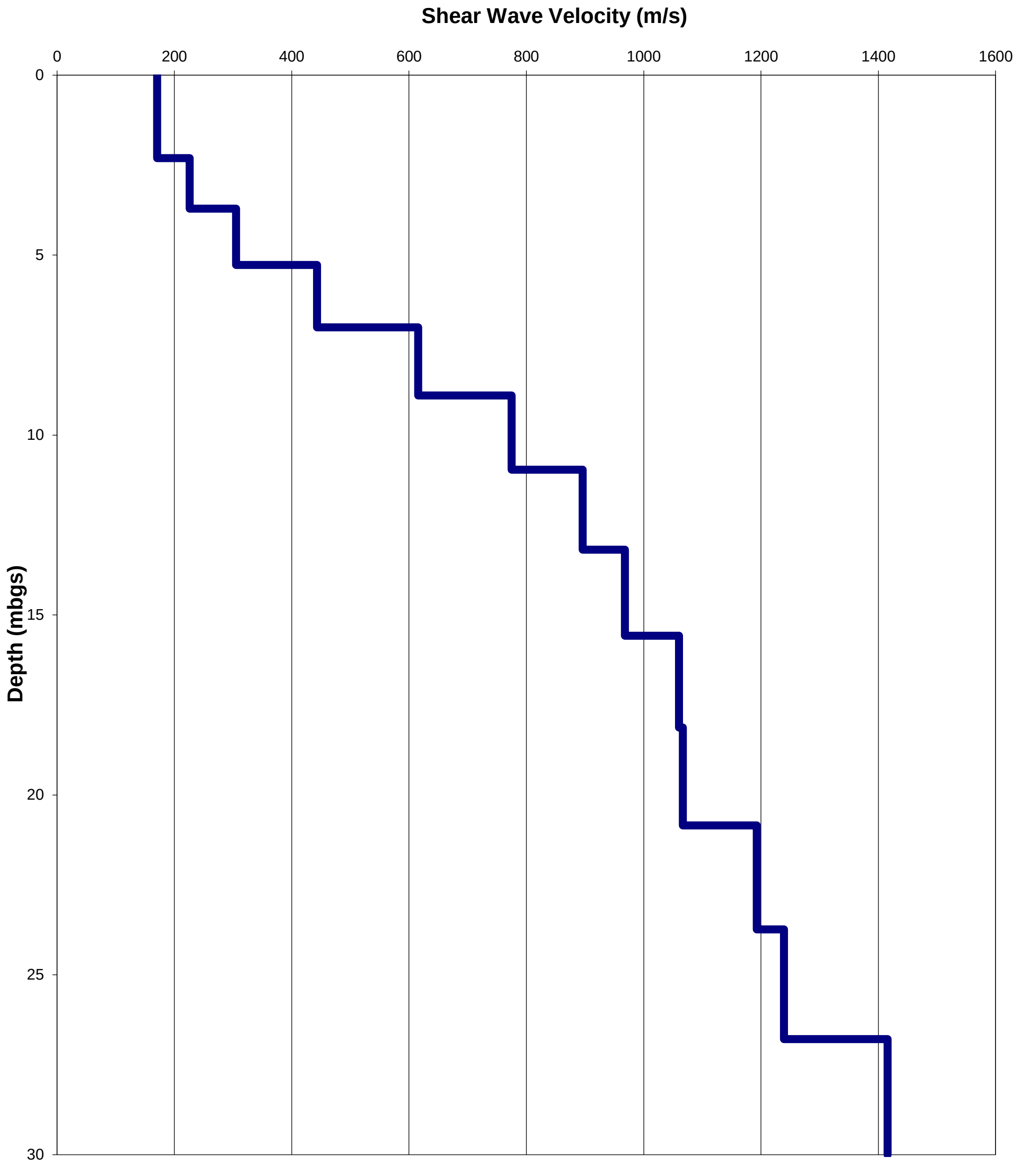


Figure 7: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 2

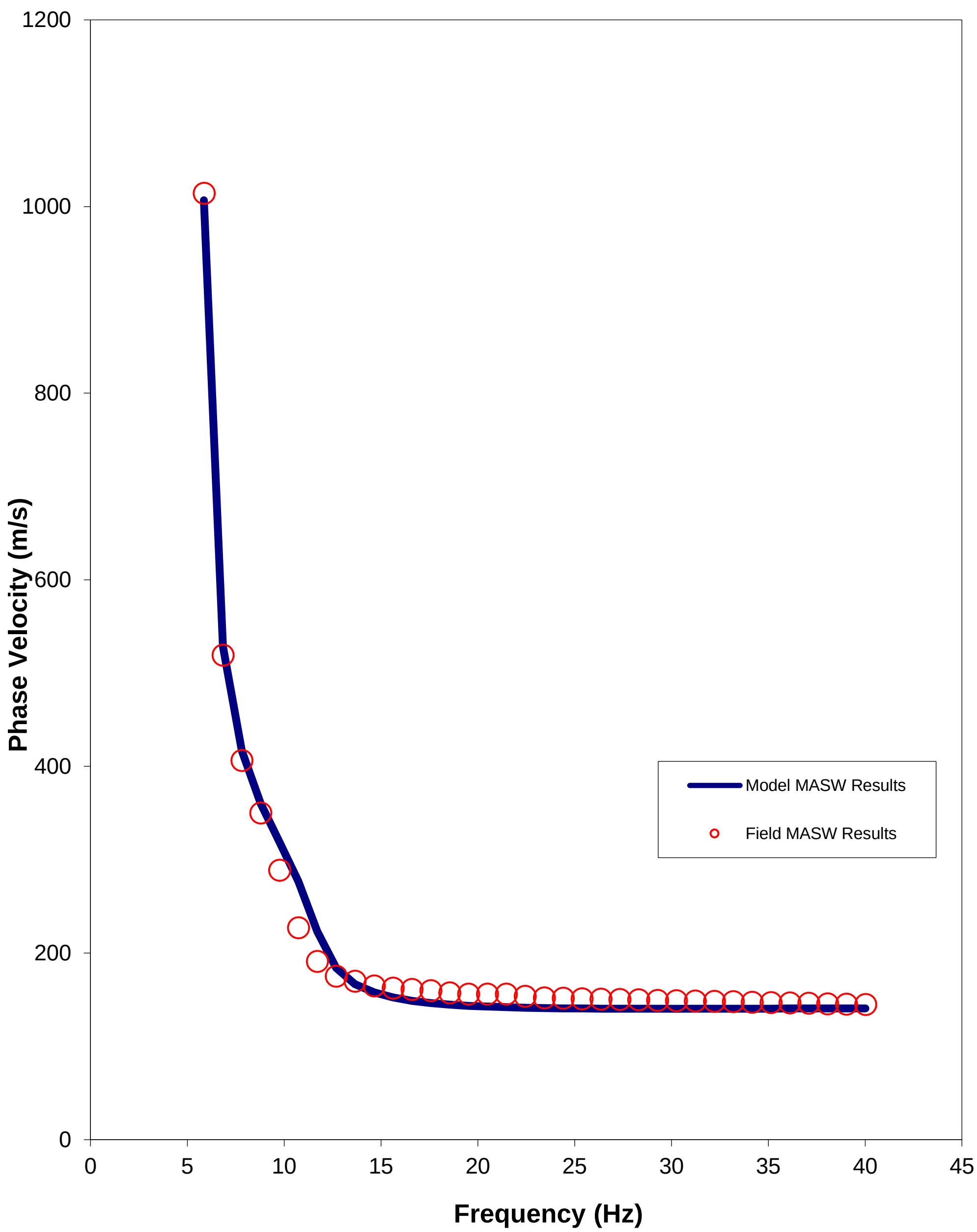


Figure 8: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 1

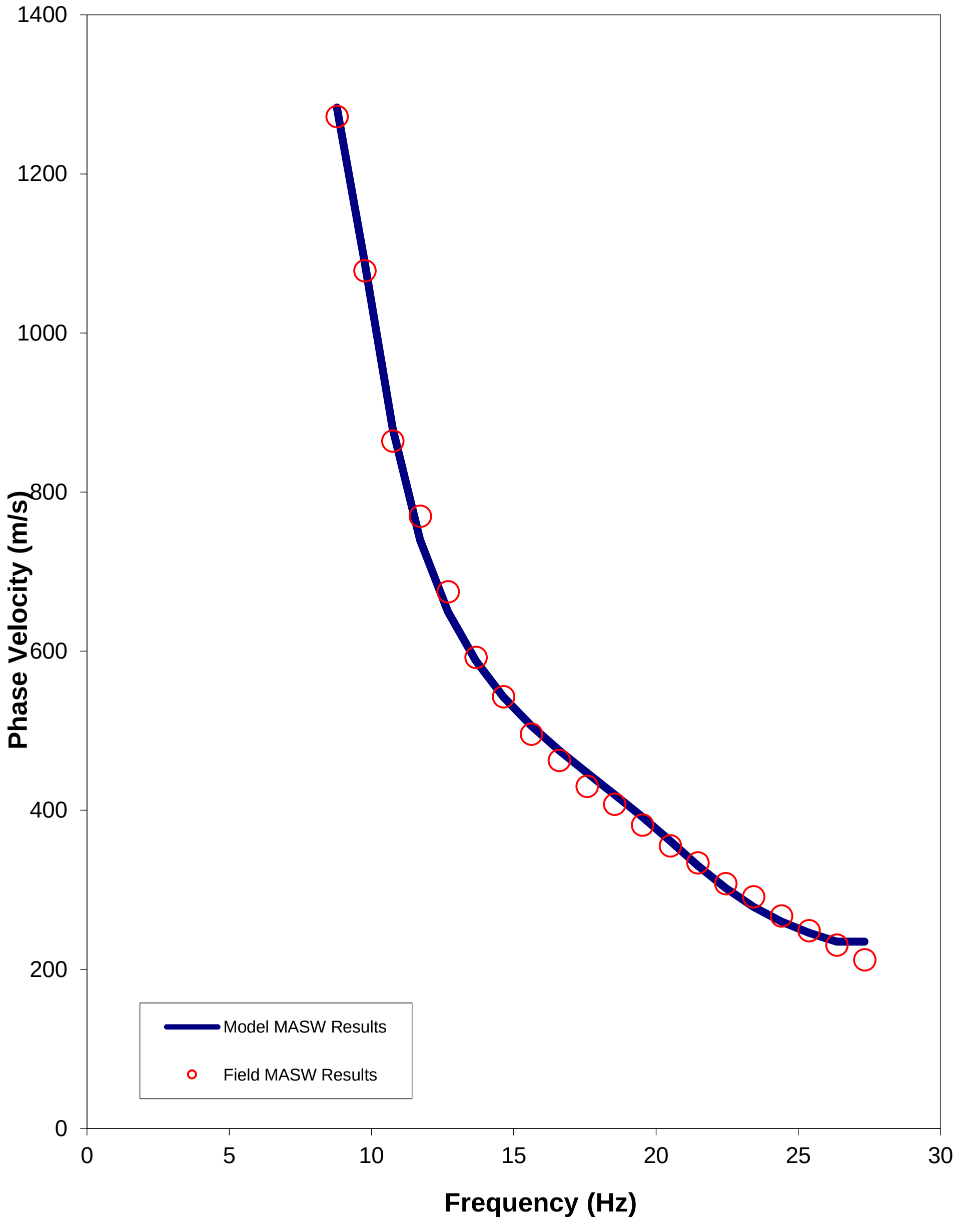


Figure 9: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 2

To calculate the average shear-wave velocity as required by the CHBDC 2014, the results were modelled to 30 metres below ground surface. The average shear-wave velocity along MASW Line 1 in the north was found to be 473 m/s (Table 1). The average shear-wave velocity along MASW Line 2 in the south was found to be 582 m/s (Table 2).

Table 2: Shear-Wave Velocity Profile along MASW Line 1

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.07	1.07	147	0.007296
1.07	2.31	1.24	147	0.008419
2.31	3.71	1.40	138	0.010161
3.71	5.27	1.57	146	0.010762
5.27	7.01	1.73	207	0.008358
7.01	8.90	1.90	674	0.002812
8.90	10.96	2.06	1066	0.001933
10.96	13.19	2.23	1253	0.001777
13.19	15.58	2.39	1378	0.001734
15.58	18.13	2.55	1426	0.001791
18.13	20.85	2.72	1389	0.001959
20.85	23.74	2.88	1351	0.002135
23.74	26.79	3.05	1458	0.002091
26.79	30.00	3.21	1481	0.002170
			Vs Average to 30 mbgs (m/s)	**473**

Table 3: Shear-Wave Velocity Profile along MASW Line 2

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.07	1.07	171	0.006282
1.07	2.31	1.24	171	0.007249
2.31	3.71	1.40	226	0.006205
3.71	5.27	1.57	305	0.005131
5.27	7.01	1.73	443	0.003906
7.01	8.90	1.90	616	0.003079
8.90	10.96	2.06	775	0.002659
10.96	13.19	2.23	896	0.002483
13.19	15.58	2.39	968	0.002469
15.58	18.13	2.55	1060	0.002409
18.13	20.85	2.72	1067	0.002549
20.85	23.74	2.88	1193	0.002418
23.74	26.79	3.05	1239	0.002460
26.79	30.00	3.21	1416	0.002270
Vs Average to 30 mbgs (m/s)				**582**

The CHBDC 2014 requires special site specific evaluation if certain soil types are encountered on the site, so the site classification stated here should be reviewed, and modified if necessary, according to borehole stratigraphy, standard penetration resistance results, and undrained shear strength measurements, if available for this site.

Limitations

This technical memorandum is based on data and information collected by Golder Associates Ltd. and is based solely on the conditions of the properties at the time of the work, supplemented by historical information and data obtained by Golder Associates Ltd. as described in this memo.

Golder Associates Ltd. has relied in good faith on all information provided and does not accept responsibility for any deficiency, misstatements, or inaccuracies contained in the reports as a result of omissions, misinterpretation, or fraudulent acts of the persons contacted or errors or omissions in the reviewed documentation.

The services performed, as described in this memo, were conducted in a manner consistent with that level of care and skill normally exercised by other members of the engineering and science professions currently practicing under similar conditions, subject to the time limits and financial and physical constraints applicable to the services.

Any use which a third party makes of this memo, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this memo.

The findings and conclusions of this memo are valid only as of the date of this memo. If new information is discovered in future work, including excavations, borings, or other studies, Golder Associates Ltd. should be requested to re-evaluate the conclusions of this memo, and to provide amendments as required.

Closure

We trust that this technical memorandum meets your needs at the present time. If you have any questions or require clarification, please contact the undersigned at your convenience.

Stephane Sol, Ph.D., P. Geo
Senior Geophysicist

Christopher Phillips, M.SC., P. Geo
Senior Geophysicist, Principal

SS/CP/jl

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