REPORT

Foundation Investigation and Design Cyrville Road Underpass Rehabilitation Site No. 3-314 - Highway 417 Ottawa, Ontario

G.W.P. 4074-11-00
W.P. 4043-11-01

Submitted to:

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PART A

Foundation Investigation Report
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Site No. 3-314 - Highway 417
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by WSP Canada Group Limited (WSP) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with numerous bridge and structural culvert rehabilitations and/or replacements on Highway 417 between the Aviation Parkway and Ramsayville Road as well as the widening of Highway 417 from Ottawa Road (OR) 174 to Hunt Club Road in Ottawa, Ontario (Assignment number 4016-E-0008).

This report presents the results of the foundation investigation carried out to collect subsurface information required for the static and seismic assessment of the bridge foundations as input to the rehabilitation of the Cyrville Road Underpass, Site No. 3-314 located on Highway 417 in Ottawa, Ontario (G.W.P. 4074-11-00 and W.P. 4043-11-01). The rehabilitation of the existing 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 (RFP), dated May 2016, and subsequent addenda. Golder's scope of work for foundation engineering services associated with the Highway 417 Cyrville Road Underpass is contained in Table 17.8.3 of WSP's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Quality Control Plan for foundation engineering services for this project, dated March 13, 2017.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 General

The Cyrville Road Underpass is located approximately 600 m north west of Innes Road in Ottawa, Ontario. At this location, Highway 417 is a divided highway with two travel lanes in each direction separated by a median.

The existing bridge was constructed in 1974 and is a two-span structure, about 102 m long with span lengths of about 51.2 m. It is a concrete post-tensioned rectangular voided structure with four rectangular voids. The overall structure width is about 13 m including barrier walls, and sidewalks. The centre pier consists of a single circular column supported by a spread footing founded on bedrock. The bridge abutments sit on “perched” pile caps supported on battered steel HP12×74 (HP310x110) piles bearing on bedrock. The existing approach embankments are about 7 m to 8 m high relative to the highway profile. The embankment side slopes were constructed at about 2H:1V (horizontal:vertical). Based on visual observation at the time of the site investigation and subsequent discussion with WSP, no signs of foundation settlement or significant embankment erosion were observed, and the existing embankment slopes appear to be performing satisfactorily.

2.2 Regional Geology

As delineated in *The Physiography of Southern Ontario*[1], this section of Highway 417 lies on the boundary of the minor physiographic regions known as the Ottawa Valley Clay Plain and the Russell and Prescott Sand 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 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

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[1] Chapman, L. J. and Putnam, D. F., 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey. Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000. Ontario Ministry of Natural Resources.

bedrock[2]. The Russell and Prescott Sand Plains are generally characterized by a sand mantle about 3 to 5 m thick overlying an extensive deposit of sensitive marine clay deposited within the Champlain Sea basin, underlain by glacial till and shale bedrock.

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 bedrock of Carlsbad Formation.

The site falls within the Western Québec (WQ) seismic zone according to the Geological Survey of Canada. The WQ zone constitutes a large area which encompasses the urban areas of Montreal, Ottawa-Hull and Cornwall. Within the WQ zone recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montreal-Maniwaki axis. The two major earthquakes in the WQ zone includes the 1935 Témiscaming event which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2, and the 1944 Cornwall-Massena event which had a magnitude of 5.6.

3.0 INVESTIGATION PROCEDURES

3.1 Current Investigation (2018)

The subsurface investigation for the bridge rehabilitation was carried out on June 7, 2018. During that time, one borehole (18-2401) was advanced at the Highway 417 level near the existing pier. The borehole location is shown on Drawing 1.

The borehole was advanced using 108 mm inside diameter (200 mm outside diameter) continuous flight hollow stem augers on a track mounted drill rig, supplied and operated by George Downing Estate Drilling of Grenville-sur-la-rouge, Québec. The borehole was advanced to a depth of about 8.8 m below the existing ground surface.

Samples of the overburden were obtained at 0.8 m intervals of depth using 50 mm outside diameter split-spoon samplers in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586, latest edition).

Upon reaching refusal to auger advancement in Borehole 18-2401, the borehole was advanced about 6.1 m into the bedrock surface using rotary diamond drilling techniques while retrieving NQ sized core. A water truck was on site to supply the drill rig with water for coring the bedrock. Traffic control required to allow the water truck and support vehicles to park on the site was supplied by Beacon Lite Ltd. of Ottawa, Ontario.

A monitoring well was installed in Borehole 18-2401 to monitor the groundwater level at the site. The monitoring wells consisted of 30 mm outside diameter PVC tubing with a 3.0 m long screen. The groundwater level was measured in the monitoring well on July 26, 2018. The monitoring wells were decommissioned after taking the final water level measurement by removal of the piping and reinstatement by grouting in accordance with O. Reg. 903.

The remainder of the borehole was 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 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 to Golder's laboratories in Ottawa for testing. Index and classification tests consisting of water content determinations and grain size distribution analyses were carried out on selected soil specimens

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[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.

at the Golder Ottawa laboratory. Unconfined compressive strength tests were carried out on selected rock core samples in the Golder Ottawa laboratory. The laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate.

One soil sample from Borehole 18-2401 was submitted to Eurofins Environment Testing for chemical analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack).

In addition to the borehole investigation, shear wave velocity profiling at the site was completed using the Multichannel Analysis of Surface Waves (MASW) technique and was conducted on June 26, 2018, by personnel from the Golder Associates' 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 source. The source locations were offset at distances of 5, 10, and 15 m off the end and collinear with the geophone array.

The borehole elevation was surveyed by Golder using a Trimble R8 GPS unit. The borehole location, including MTM NAD83 northing and easting coordinates, ground surface elevation referenced to geodetic datum, and drilled depth are summarized in the following table and are shown on Drawing 1.

Table 1: Summary of Borehole Location

Borehole Number	Borehole Location	MTM NAD83 Zone 9 Northing (m)	MTM NAD83 Zone 9 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
18-2401	Pier	5031331.6	374189.2	66.4	8.8

3.2 Previous Investigation (1972)

A previous investigation was carried out in 1972 by the MTO (then the Ministry of Transportation and Communications, Ontario) for the design of the existing bridge. The results of that investigation are contained in the report titled "*Foundation Investigation Report for The Proposed Underpass Structure at the Crossing of Hwy. #417, (E.B. and W.B. Lanes) and Realigned Cyrville Rd., Twp. Of Gloucester, Reg. Mun. of Ottawa-Carleton, District No. 9 (Ottawa), W.O. 72-11109 – W.P. 13-68-04*" dated November 7, 1972 (GEOCRES No. 31G05-114).

As part of the current assignment, this previously collected subsurface information pertinent to the site was reviewed and compiled.

Twelve boreholes, were put down at the site as part of the original investigation along the then-proposed bridge alignment. The approximate borehole and ground surface elevations are shown on the Record of Borehole sheets included in Appendix B and are also shown on Drawing 1. The locations and ground surface elevation of the previous boreholes should be considered approximate since the locations were referenced to an imperial borehole location plan rather than metric MTM coordinates. The detailed subsurface soils and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the borehole records from the 1972 investigation.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The Record of Borehole 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 sheets and on Figure B1 and Figure B2 in Appendix B. The Record of Borehole sheets and laboratory test results from the previous investigation are presented in Appendix C. The results of basic chemical analysis completed on select soil samples are provided in Appendix D. The borehole locations and the interpreted stratigraphic profile projected along the Cyrville Road Underpass are shown on Drawings 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 types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

The MASW test results and report are presented in Appendix E 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.

In general, the subsurface conditions at the site consist of a layer of fill overlying shale bedrock.

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

4.2 Current Investigation

In general, the subsurface conditions at Borehole 18-2401 for the current investigation consists of asphaltic concrete overlying granular fill overlying shale bedrock.

4.2.1 Pavement Structure and Fill

At the location of borehole 18-2401, the pavement structure consists of about 0.2 m of asphaltic concrete overlying a layer of sandy gravel fill. The fill extends to a depth of about 2.3 m.

Standard Penetration Tests carried out within the granular fill gave SPT “N” values of greater than 50 blows per 0.3 m of penetration, indicating a very dense state of compactness.

The results of grain size distribution testing carried out on one sample of the granular fill are provided on Figure B1 in Appendix B. The results of natural water content testing carried out on of one sample of the granular fill was about 2 percent.

4.3 Previous Investigation

In general, the subsurface conditions in the boreholes for the previous investigation consist of fill overlying silty sand to sandy silt overlying glacial till overlying shale bedrock. It should be noted that these boreholes were advanced prior to the construction of the Cyrville Road underpass, and the subsurface conditions would likely have changed since the previous investigation.

4.3.1 Fill and Topsoil

A layer of fill was encountered at the ground surface in Borehole 12 and generally consists of sand and gravel. This layer extends to about 1.2 m below the existing ground surface.

A 300 mm thick layer of topsoil was encountered at the ground surface in Boreholes 10 and 11.

4.3.2 Silty Sand to Sand to Silty Sand to Sandy Silt

A discontinuous deposit of silty sand to sand, trace to no gravel and clay and silty sand to sandy silt, trace to some clay was encountered in Boreholes 1 to 11. This layer extends to depths ranging from about 1.5 to 2.6 m below the existing ground surface (Elevations ranging from about 66.0 m to 65.0 m).

Standard Penetration Tests carried out within the silty sand, sand, and sandy silt deposits gave SPT “N” values ranging from 5 to 66 blows per 0.3 m of penetration, indicating a loose to very dense state of compactness.

The results of grain size distribution testing carried out as part of the previous investigation are provided in Appendix C.

4.3.3 Clayey Silt (Till)

A deposit of glacial till was encountered below the silty sand, sand, and sandy silt deposits in Boreholes 1 through 12. The glacial till generally consists of a clayey silt matrix with sand and gravel, and shale fragments. The glacial till extends to depths ranging from about 2.3 to 3.8 m below the existing ground surface (Elevation ranging from about 65.0 m to 63.7).

Standard Penetration Tests carried out within the glacial till gave SPT “N” values ranging from 24 to greater than 100 blows per 0.3 m of penetration, indicating a compact to very dense state of compactness.

The results of grain size distribution testing carried out as part of the previous investigation are provided in Appendix C.

The results of Atterberg limit testing on four samples of the clayey silt from the previous investigation gave a plasticity index value ranging from about 4 to 8 percent and a liquid limit value ranging from about 18 to 22 percent. The measured natural water content of four samples of the clayey silt were about 8 and 11 percent.

4.4 Bedrock

Bedrock was encountered beneath the fill in Borehole 18-2401 from the current investigation, and beneath the glacial till in Boreholes 1, 4, 5, 11, and 12 from the previous investigation at depths ranging from about 2.3 to 3.8 m below the existing ground surface (Elevations ranging from about 64.8 m to 63.7 m). In borehole 18-2401, the bedrock was cored about 6.1 m using NQ-sized coring equipment.

Refusal to sampler advancement was encountered in Boreholes 6, 7, 8, 9, and 10 at depths ranging from about 2.6 to 3.4 m below the existing ground surface (Elevations ranging from about 64.8 to 64.1 m).

The following table summarizes the bedrock surface or refusal depths and elevations as encountered at the borehole locations.

Table 2: Summary of Bedrock Surface and Auger Refusal Depths and Elevations

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock/Refusal (m)	Bedrock Surface/ Refusal Elevation (m)
18-2401	66.4	2.3	64.1
1	67.7	3.4	64.4
4	67.8	3.4	64.5
5	67.9	3.4	64.6

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock/Refusal (m)	Bedrock Surface/ Refusal Elevation (m)
6	67.2	3.1[1]	64.1[1]
7	67.9	2.9[1]	65.0[1]
8	67.6	2.8[1]	64.8[1]
9	67.4	2.6[1]	64.8[1]
10	67.7	3.4[1]	64.3[1]
11	67.5	3.8	63.7
12	67.1	2.3	64.8

Note 1: Refusal to sampler advancement.

The bedrock encountered in these boreholes consist of highly weathered to fresh, thinly laminated, grey to black shale of the Carlsbad Formation. The Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from about 0 to 52 percent, indicating a poor to fair quality rock.

Results of unconfined compressive strength testing carried out on one bedrock core sample are presented in Figure B2. The results gave a value of about 20 MPa indicating a weak bedrock.

4.5 Groundwater Conditions

A monitoring well was installed in Borehole 18-2401 to monitor the groundwater level at the site. The water level was measured in the monitoring well July 26, 2018 and the results are summarized in the following table.

Table 3: Summary of Measured Groundwater Levels

Borehole	Ground Surface Elevation (m)	Screened Interval Material	Water Level Depth (m)	Water Level Elevation (m)	Date of Reading
18-2401	66.4	Bedrock	1.2	65.2	July 26, 2018

It is expected that this water level will be subject to fluctuations both seasonally and as a result of precipitation events.

4.6 Corrosion and Cement Type

One soil sample from Borehole 18-2401 was submitted to Eurofins Environment 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 attached in Appendix D and are summarized in the table below.

Table 4: Summary of Corrosivity of Sample

Borehole / Sample No.	Sample Depth (m)	Sample Type	Chloride (%)	pH	Electrical Conductivity (mS/cm)	Resistivity (ohm-cm)	Sulphate (%)
18-2401 / 3	1.52 – 1.65	Soil	0.030	8.1	0.78	1280	0.07

5.0 CLOSURE

This report was prepared by Ms. Sarah Ghadbane, P.Eng., and reviewed by Mr. Michael Snow P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Fintan Heffernan, P.Eng., a Senior Consultant with Golder and the Designated MTO Foundations Contact, conducted an independent quality control review of this report.

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PART B

Foundation Design Report
Cyrville Road Underpass Rehabilitation
Site No. 3-314 - Highway 417
Ottawa, Ontario
GWP 4074-11-00
W.P. 4043-11-01

6.0 FOUNDATION ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations as input to the rehabilitation of the Cyrville Road Underpass, Site No. 3-314 located on Highway 417 in Ottawa, Ontario (G.W.P. 4074-11-00 and W.P. 4043-11-01). The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation as well as the available Geocres information for the site.

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 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.

6.1 General

Golder Associates Ltd. (Golder) has been retained by WSP Canada Group Limited (WSP) on behalf of the MTO to provide recommendations on the foundation aspects for the design of the rehabilitation of the Cyrville Road Underpass, Site No. 3-314 located on Highway 417 approximately 800 m northwest of Innes Road in Ottawa, Ontario.

6.2 Existing Foundations

The existing bridge is a two-span structure, about 102 m long with span lengths of about 51.2 m. It is a concrete post-tensioned rectangular voided structure with four rectangular voids. The overall structure width is about 13 m including barrier walls, and sidewalks. The centre pier consists of three circular columns supported by spread footings founded on bedrock. The bridge abutments sit on “perched” pile caps supported on battered steel HP12×74 (HP310x110) piles bearing on bedrock. The underside of pile cap is at about Elevation 68.1 metres on the east side and at about Elevation 66.9 metres on the west side.

The existing approach embankments are about 7 m to 8 m high relative to the highway profile. The embankment side slopes were constructed at about 2H:1V. Based on visual observation at the time of the site investigation and subsequent discussion with WSP, no signs of foundation settlement or significant embankment erosion were observed, and the existing embankment slopes appear to be performing satisfactorily.

It is understood that the bridge is to be designed in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC).

6.3 Seismic Design

6.3.1 Seismic 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 GSC has developed a new set of seismic hazard maps (referred to as the 5th generation seismic hazard maps) that were made available for public use in December 2015.

In accordance with Section 4.4.2 of the CHBDC, and as specified by the MTO, the proposed 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 within the Highway 417 median 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 E) and indicate that the average shear wave velocity in the upper 30 m of the subsurface stratigraphy beneath the anticipated foundation depths at the two MASW locations were 1,101 m/s and 1,196 m/s. However, Table 4.1 of the CHBDC indicates that Site Class A and B are not to be used if there is more than 3 metres of softer materials between the rock and the underside of the bridge foundations (i.e. footings or pile caps), which is the case at this site.

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 underlain by very localized horizons of soil which are considered to have a small probability to undergo liquefaction under the design earthquake event. This is not considered to have a material impact on the dynamic response of the site, and as such a Site Class C designation is considered appropriate for design.

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 45.37 N and longitude 75.57 W), the following are the reference Site Class C (reference) peak seismic hazard values based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca).

Table 5: Site Class C Spectral Values for Subject Site

	Value at Given Probability of Exceedance in 50 Years		
Seismic Hazard Values	10% (475-year)	5% (975-year)	2% (2,475-year)
PGA	0.106 g	0.171 g	0.295 g
T <= 0.2 s	0.168 g	0.267 g	0.461 g
T = 0.5 s	0.091 g	0.143 g	0.246 g
T = 1.0 s	0.045 g	0.071 g	0.122 g
T = 2.0 s	0.021 g	0.0033 g	0.057 g
T = 5.0 s	0.005 g	0.008 g	0.015 g
T => 10.0 s	0.002 g	0.003 g	0.005 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.

Based on the regular geometry of the bridge (since its skew angle is less than 20 degrees), it is understood that the structure will be designed using a "force-based approach" as defined in the CHBDC.

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 liquefaction susceptibility of soils was evaluated by comparing the penetration resistance required to trigger liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required.

The methodology used to assess liquefaction potential at the site is consistent with the “simplified” approach 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.

The liquefaction analysis was carried out using the data collected at the selected boreholes. The design groundwater level was determined based on the groundwater measurement in borehole 18-2401 in the model. 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 assessment using the “simplified” approach indicate that the site soils have a low potential for liquefaction and may be considered non-liquefiable for design.

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 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 the following Sections. 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. 13-68-04, Dwg Nos. 3-314-1 to 3-314-3), the abutments are supported on “perched” foundations on steel HP 12x74 (HP 310x110) piles end bearing on bedrock. The piles are configured in two rows with the front row piles battered towards Highway 417 at 1H:4V and the back row piles battered away from Highway 417 at 1H:12V.

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 uniaxial compressive strength of the rock at this site and the rock quality, for the HP 12x74 (HP 310x110) piles at the abutments the axial factored ultimate geotechnical resistance (ULS) will be 2,500 kN. The factored ULS geotechnical resistance may be greater than the structural capacity of the pile, which could govern design and should be checked by the structural design engineer. The factored serviceability geotechnical resistance does 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. A consequence factor of 1.0 was applied.

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. 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 Cyrville 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 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 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 behaviour (such that k_h is a function of deflection).

Table 6: Constants of Horizontal Subgrade Reaction

Location	Elevation (m)	Soil Type	n_h (MN/m^3)
Abutments	73.7 – 65.2	Inferred Compact to Dense Sand/Gravel (Emb. Fill)	13
	65.2 – 64.1	Dense Sandy Gravel Fill (Below Water Table)	11
	64.1 – 63.7	Compact Glacial Till (Below Water Table)	4.4
	63.7	Bedrock	-

Notes: ^{3} PCL = Pile Cap Level

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

The factored ULS geotechnical resistance of pile formations at the abutments was developed using the Broms[3] (1964) approach with values of 200 kN at the west abutment and 300 kN at the east abutment for HP 12 x 74 (HP 310 x 110).

6.4.3 Shallow Foundations (Piers)

The original structural design drawings (W.P. 13-68-04, Dwg No. 3-314-1 to 3-314-3) indicate that the pier is supported on a spread footing, measuring about 5.3 m square in plan and bearing directly on sound bedrock (i.e. RQD > 50%). The pier columns have a circular cross-section with a diameter of about 1.68 metres. It is understood that the footings have a minimum embedment depth of 1.8 m for frost protection purposes, per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

6.4.3.1 Factored Geotechnical Bearing Resistance

At the spread footings founded directly on the bedrock for the pier, the static factored geotechnical resistance at Ultimate Limit States (ULS) was calculated based on the compressive strength and condition of the bedrock using the methodology outlined in the Canadian Foundation Engineering Manual (2006), and may be considered to be 1,000 kPa for the structural assessment. Serviceability Limit States (SLS) resistances do not apply to the footings on the bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored geotechnical bearing resistance at ULS. A consequence factor of 0.9 was applied.

The factored geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the CHBDC.

6.4.3.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the cast-in-place concrete footings and the bedrock surface should be calculated in accordance with Section 6.10.5 of the CHBDC. An unfactored coefficient of friction, tan φ' = 0.70 can be used for the interface between the concrete footing and bedrock.

6.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls and any associated wing walls (if required) will 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. Seismic (earthquake) loading must also be taken into account in the design.

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 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).

---

[3] Broms, Bengt. B., M.ASCE, 1964. *Lateral Resistance of Piles in Cohesionless Soils*, Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers.

Table 7: 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.

Table 8: Seismic Lateral Earth Pressure Coefficients

Wall Type	Design Earthquake	Site PGA	Coefficient Value (K_{AE})
Yielding Wall	**475 Year**	0.106	0.35
	975 Year	0.171	0.37
	2,475 Year	0.295	0.42
Non-Yielding Wall	**475 Year**	0.106	0.40
	975 Year	0.171	0.44
	2,475 Year	0.295	0.56

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 27, 44, and 76 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 18-2401 was submitted to Eurofins Environment 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 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 high potential for corrosion of exposed ferrous metal which should be considered in the design.

7.0 CLOSURE

This report was prepared by Ms. Sarah Ghadbane, P.Eng., and was reviewed by Mr. Michael Snow, P.Eng., a senior geotechnical engineer and Principal of Golder. Mr. Fintan Heffernan, P.Eng., Designated MTO Foundations Contact, conducted an independent quality control review of this report.

Golder Associates Ltd.

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WAM/SG/MSS/mvrd
https://golderassociates.sharepoint.com/sites/11263g/shared documents/01_foundations/6 - reports/1240 cyrville/03_final/1662565-1240-001-r-rev0-cyrville road-fidr-2006_19.docx

MINISTRY OF TRANSPORTATION, ONTARIO

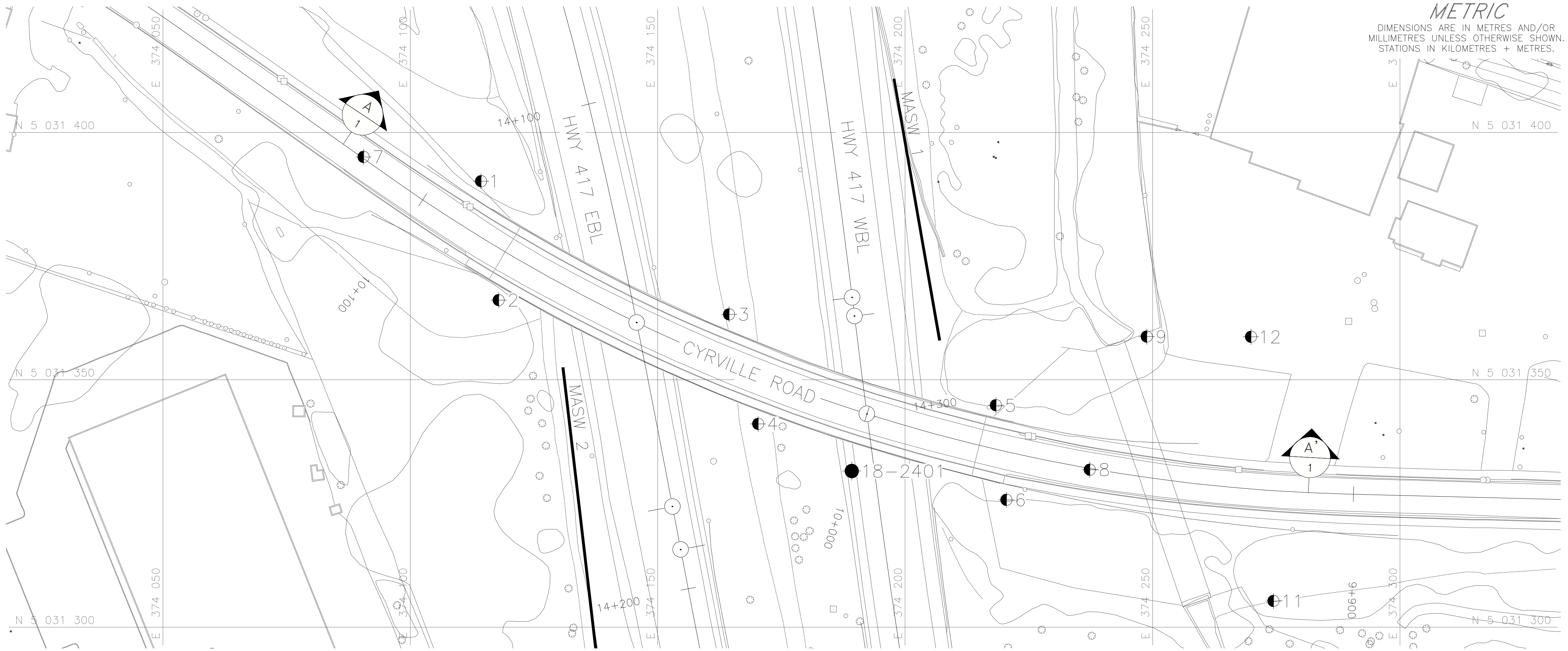
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 4074-11-00

SHEET

CYRVILLE ROAD OVERPASS
HIGHWAY 417
BOREHOLE LOCATIONS AND SOIL STRATA
LAT. 45.418672 LONG. -75.613657

GOLDER



PLAN
SCALE
10 0 10 20 m

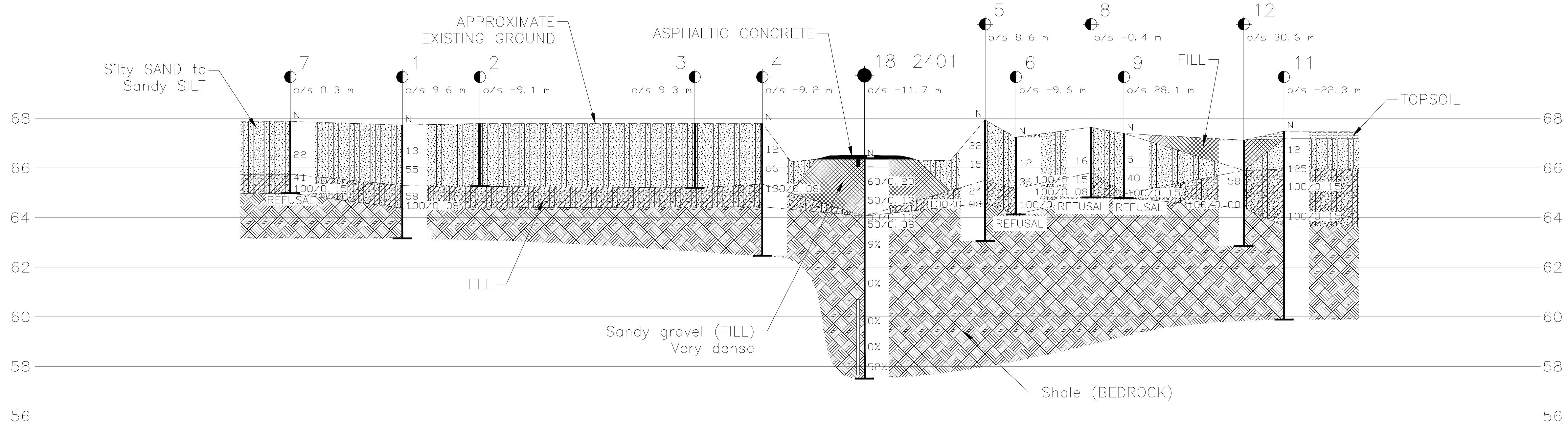
KEY PLAN
SCALE
1 0 1 2 km

LEGEND

●	Borehole – Current Investigation
◐	Borehole – Previous Investigation (Geocres No. 31G05-114)
▮	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 XXX XX, XXXX

BOREHOLE CO-ORDINATES (MTM ZONE 9)

No.	ELEVATION	NORTHING	EASTING
1	67.7	5031390.1	374114.3
2	67.8	5031366.1	374117.9
3	67.8	5031363.2	374164.4
4	67.8	5031341.1	374170.3
5	67.9	5031344.9	374218.4
6	67.2	5031325.7	374220.5
7	67.9	5031395.0	374090.6
8	67.6	5031331.8	374237.3
9	67.4	5031358.7	374248.8
11	67.5	5031305.3	374274.5
12	67.1	5031358.7	374269.9
18-2401	66.4	5031331.6	374189.2



PROFILE A–A'
SCALE
10 0 10 20 m
2 0 2 4

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.

REFERENCE
Base plans provided in digital format by WSP, drawing file nos. XA1-NAD 83.dwg and XB1-NAD 83 (CSRS).dwg, received APR. 19, 2017.

S.L. GHADBANE 100184126
F.J. HEFFERNAN

NO.	DATE	BY	REVISION

Geocres No. 31G5-304

HWY. 417		PROJECT NO. 1662565	DIST. EASTERN
SUBM'D. SG	CHKD. SG	DATE: 6/21/2019	SITE: 3-314
DRAWN: JM	CHKD. MSS	APPD. FJH	DWG. 1

PLOT DATE: June 21, 2019
FILENAME: N:\Active\Spatial_IM\MTO\Hwy417Rehab&Widening\99_PROJ\1662565_MTO_Hwy417Replacements\40_PROD\0016_CyrvilleRoadOverpass\1662565-0016-BG-0001.dwg

APPENDIX A

Borehole and Drillhole Records, Current Investigation

Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Records of Borehole 18-2401

Figure A1 – Borehole Core Photograph

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
$\sigma_1, \sigma_2, \sigma_3$	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 $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
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

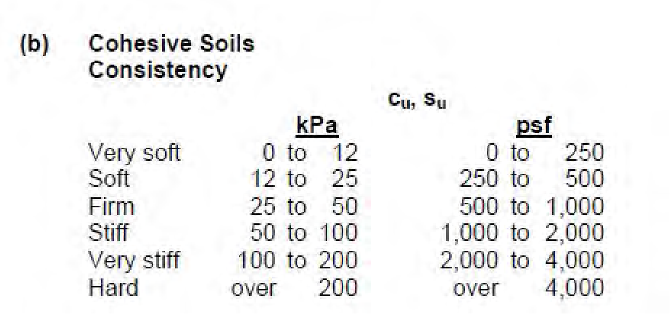
Piezo-Cone Penetration Test (CPT)

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	 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	Sand and Gravel
	With (cohesive)	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
FLT	Fault
SH	Shear
VN	Vein
FR	Fracture
SY	Stylolite
BD	Bedding
FO	Foliation
CO	Contact
AXJ	Axial Joint
KV	Karstic Void
MB	Mechanical Break

PL	Planar
CU	Curved
UN	Undulating
IR	Irregular
K	Slickensided
PO	Polished
SM	Smooth
SR	Slightly Rough
RO	Rough
VR	Very Rough

GOLDER

Foundation Design

RECORD OF BOREHOLE No 18-2401

SHEET 1 OF 1

METRIC

PROJECT 1662565-1240
G.W.P. 4074-11-00
DIST Eastern HWY 417
DATUM Geodetic

LOCATION N 5031331.6; E 374189.2 NAD 83 MTM ZONE 9 (LAT. 45.418480; LONG. -75.613380)
BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NW Casing
DATE June 7-8, 2018

ORIGINATED BY DG
COMPILED BY ZS
CHECKED BY SG

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
66.4	GROUND SURFACE				
0.0	ASPHALTIC CONCRETE				
0.2	(SW) Sandy gravel (FILL) Very dense Grey Moist	1	GRAB	-	72 23 3 2
		2	SS	60/0.20	
		3	SS	50/0.13	
64.1 2.3	Weathered Shale (BEDROCK)	4	SS	50/0.13	
		5	SS	50/0.08	
63.7 2.7	Shale (BEDROCK) Bedrock cored from depths of 2.7 m to 8.8 m For bedrock coring details refer to Record of Drillhole 18-2401	1	RC	REC 100%	RQD = 9%
		2	RC	REC 81%	RQD = 0%
		3	RC	REC 67%	RQD = 0%
		4	RC	REC 100%	RQD = 0%
58.5 7.9	Shale (BEDROCK)	5	RC	REC 100%	RQD = 52%
57.6 8.8	END OF BOREHOLE NOTES: 1. Water level in well screen at a depth of 1.2 metres below ground surface (Elev. 65.2 m), measured on July 26, 2018				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (66, 65, 64, 63, 62, 61, 60, 59, 58); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTO\HWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 6/21/19 ZS

PROJECT: 1662565-1240

RECORD OF DRILLHOLE: 18-2401

SHEET 1 OF 1

LOCATION: N 5031331.6 ;E 374189.2

DRILLING DATE: June 7-8, 2018

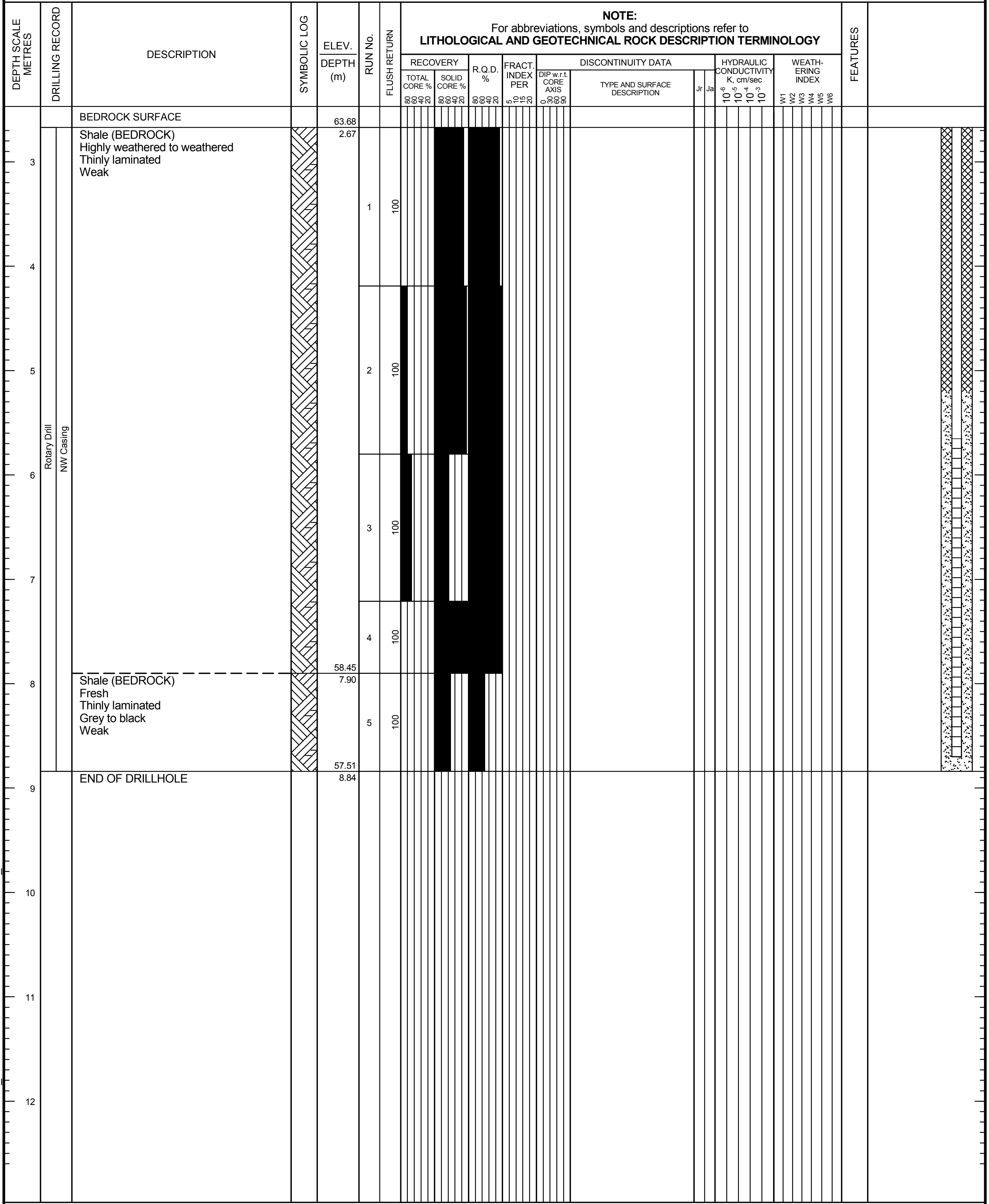
DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Downing Drilling

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



DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN
BEDROCK SURFACE	63.68		
Shale (BEDROCK) Highly weathered to weathered Thinly laminated Weak	2.67	1	100
		2	100
		3	100
	58.45 7.90	4	100
Shale (BEDROCK) Fresh Thinly laminated Grey to black Weak	57.51 8.84	5	100
END OF DRILLHOLE			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: DG
CHECKED: SG

GTA-RCK 031 N:\ACTIVE\SPATIAL_IM\MTO\HWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-MISS.GDT 6/21/19 ZS

BH 18-2401 (Wet)
Cored Length of 2.67 to 8.84 metres
Core Box 1 to 3 of 3

2.67 m Top of Bedrock

8.84 m End of Borehole

Geotechnical Investigation

Cyrville Road Underpass

Ottawa, Ontario

Project No.	1662565
Drawn:	WAM
Date:	8/27/2018
Checked:	SG
Review:	MSS

Figure A1

APPENDIX B

Laboratory Testing, Current Investigation

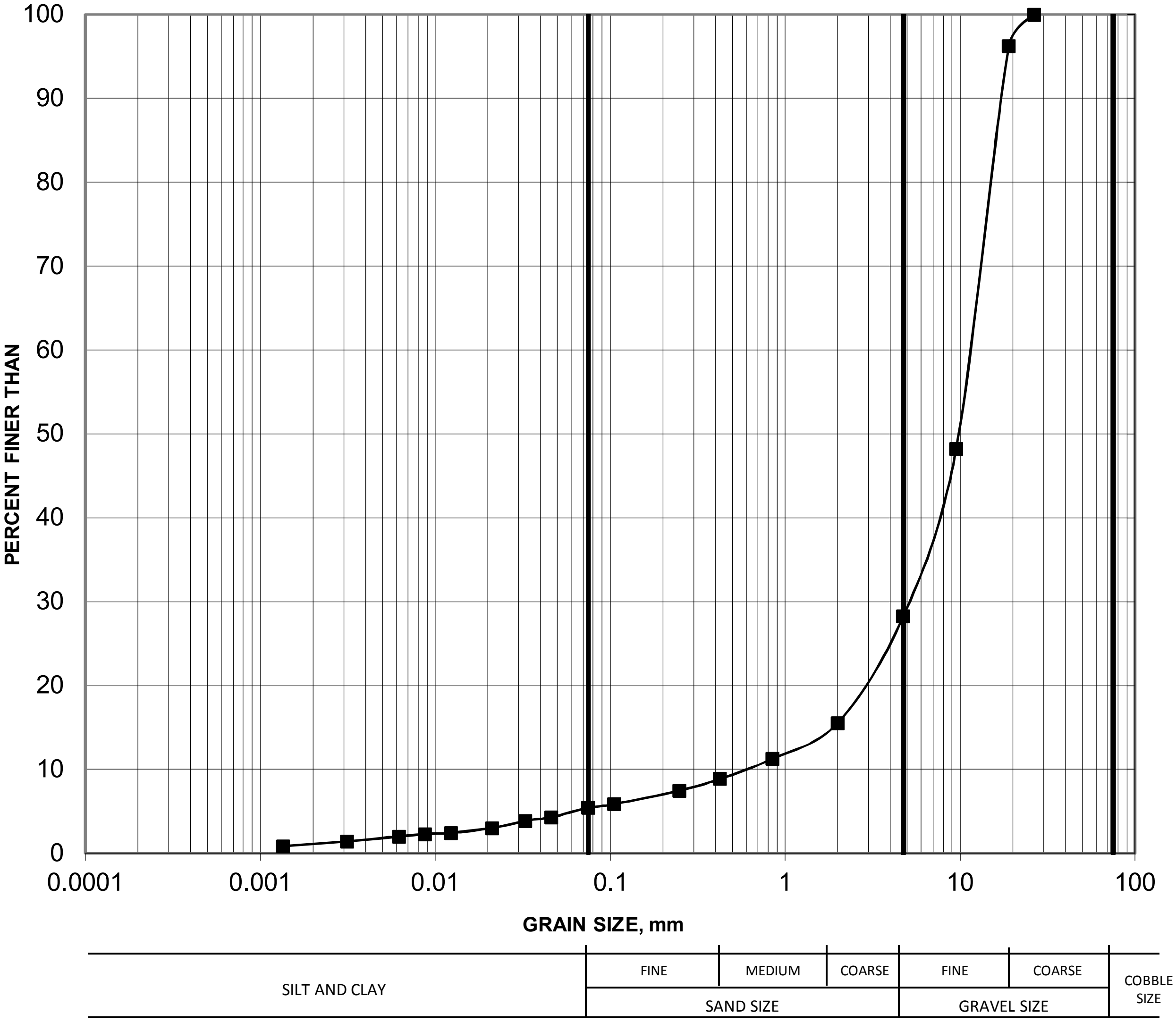
Figure B1 – Grain Size Distribution Test Results – Sandy Gravel (FILL)

Figure B2 – Results of Unconfined Compressive Strength Testing

GRAIN SIZE DISTRIBUTION

FIGURE B1

SANDY GRAVEL (FILL)



Borehole	Sample	Depth (m)
■ 18-2401	1	0.15-0.46

Project: 1662565/1240

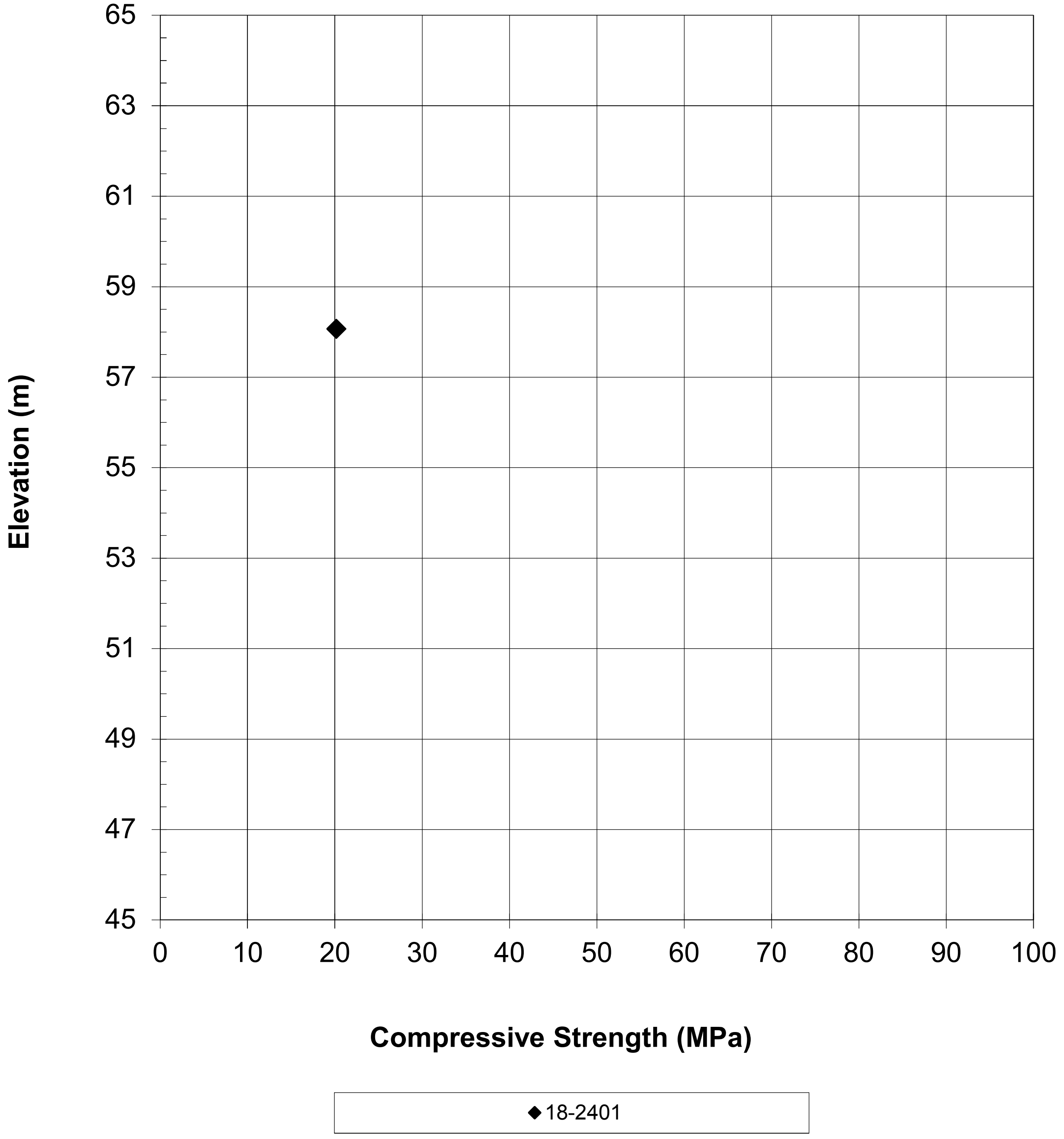
Golder Associates

Created by: MI

Checked by: CW

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH UNCONFINED COMPRESSION TESTS

FIGURE B2



Project: 1662565 /1240

Created by: CW

Checked by: MI

APPENDIX C

Borehole and Drillhole Records and Laboratory Test Results (Previous Investigation, GEOCRES 31G5-114)

Record of Previous Boreholes BH 1 to BH 12

Laboratory Test Results

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

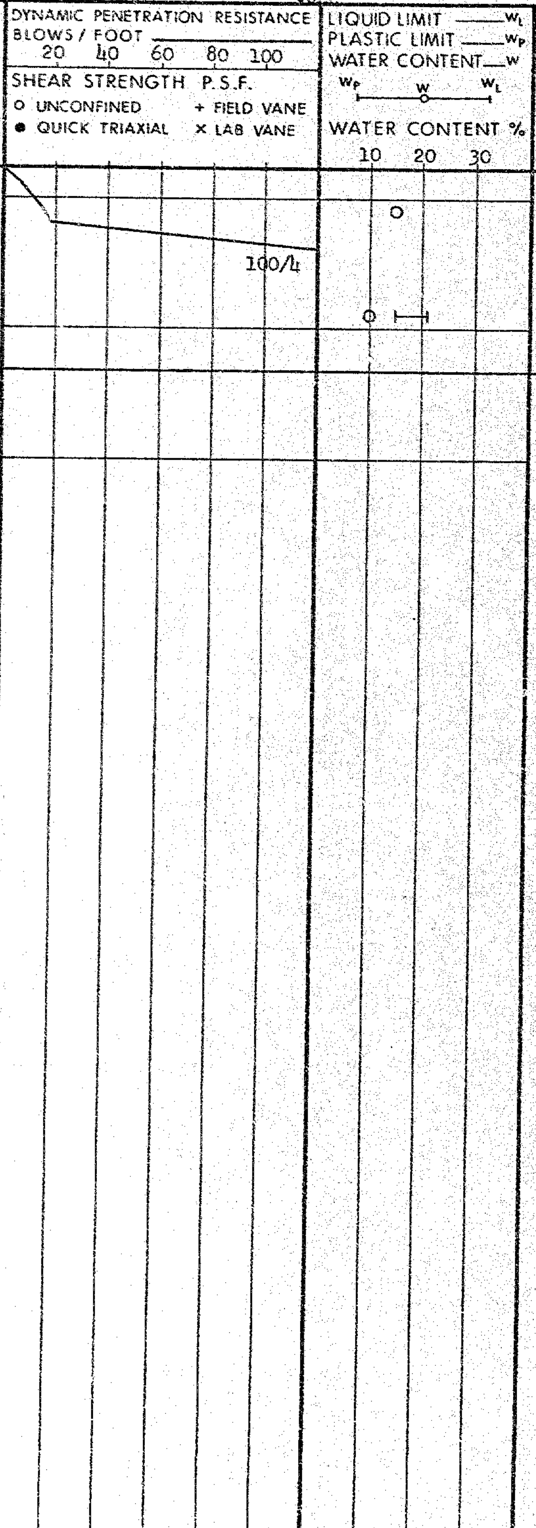
RECORD OF BOREHOLE Nº 1

JOB 72-11109 -
W.P. 13-68-04
DATUM Geodetic

LOCATION Co-ords. 16,506,449 N; 1,227 332 E.
BORING DATE Sept. 21, 1972
BOREHOLE TYPE Cont. Flight Auger-BX Casing, BXL Rock Core Cone

ORIGINATED BY SAA
COMPILED BY SAA
CHECKED BY

SOIL PROFILE			SAMPLES				REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	GR.SA.SI.CL.
222.2	Ground Level						
0.0	Silty sand with trace						
	of clay & gravel.		1	SS	13	220	4 64 21 11
	Grey		2	SS	55		▼ 217.2
214.2	Compact to Very Dense						
8.0	Glacial Till, fragments		3	SS	58		
211.2	of shale. Hard		~~4~~	~~SS~~	~~100~~ 3"		
11.0	Bedrock - Shale		5	BXL	80%	210	
207.2	Sound Grey		6	BXL	100%		
15.0	End of Borehole						
						200	



20
15 ⌖ 5 % STRAIN AT FAILURE
10

OFFICE REPORT SOIL EXPLORATION

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RECORD OF BOREHOLE Nº 2

JOB 72-11109

W.P. 13-68-04

DATUM Geodetic

LOCATION Co-ords. 16,506,372 N; 1,227,342 E.

BORING DATE Sept. 21, 1972

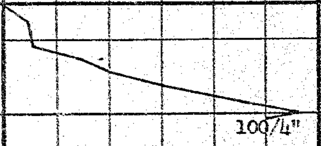
BOREHOLE TYPE Dynamic Cone Test

ORIGINATED BY SAA

COMPILED BY SAA

CHECKED BY

SOIL PROFILE			SAMPLES							
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —— W_L PLASTIC LIMIT —— W_P WATER CONTENT —— W W_P —○— W —— W_L WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
222.4	Ground Surface									
0.0	Probably Silty Sand					220				
214.1 8.3	End of Cone Test					210	100/4"			



20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT O[illegible] SOIL EXPLORATION

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RECORD OF BOREHOLE Nº 3

JOB 72-11109

W.P. 13-63-04

DATUM Geodetic

LOCATION Co-ords. 16,506,363 N; 1,227,495 E.

BORING DATE Sept. 21, 1972

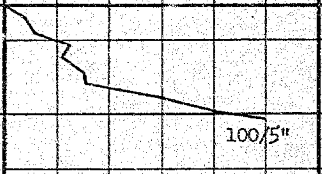
BOREHOLE TYPE Dynamic Cone Test

ORIGINATED BY SAA

COMPILED BY SAA

CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W WATER CONTENT %	BULK DENSITY γ	REMARKS
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE			P.C.F.	GR SA SI CL
222.4	Ground Level									
0.0	Probably Silty Sand					220				
213.9										
8.5	End of Cone Test						100/5"			
						210				



20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT SOIL EXPLORATION

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DESIGN SERVICES BRANCH

FOUNDATION OFFICE

RECORD OF BOREHOLE Nº 4

JOB 72-11109

W.P. 13-68-04

DATUM Geodetic

LOCATION Co-ords. 16,506,291 N; 1,227,514 E.

BORING DATE Sept. 21, 1972

BOREHOLE TYPE Cont. Flight Auger-BX Casing, BXL Rock Core

ORIGINATED BY SAA

COMPILED BY SAA

CHECKED BY

SOIL PROFILE			SAMPLES			REMARKS
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	GR.SA.SI.CL.
222.4	Ground Surface					
0.0	Silty sand to sandy silt, trace to some clay. Grey		1	SS	12	0 30 57 13
			2	SS	66	▼217.9
214.4	Compact to Very Dense		3	SS	100/3"	29 49 17 5
8.0 211.4	Glacial till with shale fragments					
11.0	weathered		4	BXL	50%	
12.0	Bedrock Shale		5	BXL	80%	
204.9	Sound Grey		6	BXL	90%	
17.5	End of Borehole					

ELEV. SCALE

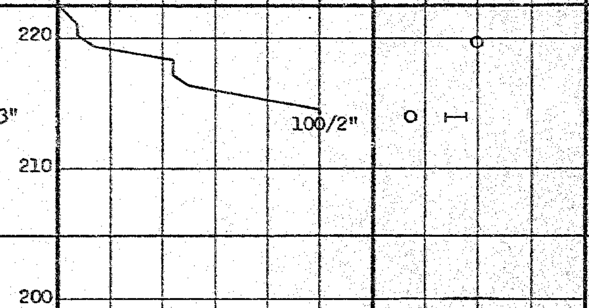
DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT
20 40 60 80 100

SHEAR STRENGTH P.S.F.
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

LIQUID LIMIT —— W_L
PLASTIC LIMIT —— W_P
WATER CONTENT —— W

WATER CONTENT %
10 20 30

BULK DENSITY γ P.C.F.



20
15 ⌖ 5 % STRAIN AT FAILURE
10

OFFICE REPORT SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATION OFFICE

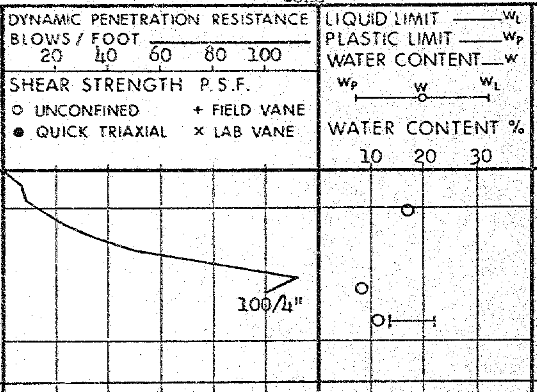
RECORD OF BOREHOLE Nº 5

JOB 72-11109
W.P. 13-68-04
DATUM Geodetic

LOCATION Co-ords. 16,506,303 N; 1,227,670 E.
BORING DATE Sept. 21, 1972
BOREHOLE TYPE Cont. Flight Auger-BX Casing, BXL Rock Core
Cone

ORIGINATED BY SAA
COMPILED BY SAA
CHECKED BY

SOIL PROFILE			SAMPLES				REMARKS
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	GR.SA.SI.CL.
222.9	Ground Surface						
0.0	Silty sand to sandy silt, trace of clay.						▼ 220.9
			1	SS	22	220	0 22 68 10
			2	SS	15		
214.9	Compact						
8.0	Glacial Till, frgaments of shale. Very Stiff		3	SS	24		21 47 23 9
211.9			4	SS	100/3"		
11.0	weathered		5	BXL	60%	210	
13.0	Shale Bedrock						
206.9	Sound Grey		6	BXL	90%		
16.0	End of Borehole						
						200	



BULK DENSITY γ P.C.F.

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT O SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATION OFFICE

RECORD OF BOREHOLE Nº 6

JOB 72-11109

W.P. 13-68-04

DATUM Geodetic

LOCATION Co-ords. 16,506,241 N; 1,227,676 E.

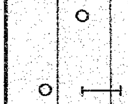
BORING DATE Sept. 22, 1972

BOREHOLE TYPE Cont. Flight Auger; Cone Test

ORIGINATED BY SAA

COMPILED BY SAA

CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE SLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —— W_L PLASTIC LIMIT —— W_P WATER CONTENT —— W W_P W W_L WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE				
220.6	Ground Level									
0.0	Silty sand with trace of clay		1	SS	12	220	 145			0 75 15 10 ▼ 215.6
213.8	Compact to dense		2	SS	36					
6.8 210.4	Glacial Till, shale fragments. Hard		3	SS	100/3"					
10.2	End of Borehole Probably Bedrock					210				

20
15 ⊕ 5 % STRAIN AT FAILURE
10

OFFICE REPORT SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH

FOUNDATION OFFICE

RECORD OF BOREHOLE Nº 7

JOB 72-11109

W.P. 13-68-04

DATUM Geodetic

LOCATION Co-ords. 16,506,466 N; 1,227,254 E.

BORING DATE Sept. 20/72

BOREHOLE TYPE Cont. Flight Auger

ORIGINATED BY SAA

COMPILED BY SAA

CHECKED BY

SOIL PROFILE			SAMPLES							
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W W_P W W_L WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
222.7	Ground Level									
0.0	Silty sand,trace of clay(occ.seams of clayey silt up to 2" thick)Compact to Dense		1	SS	22	220		o		0 62 28 10 ▼ 216.7
215.7 7.0	Glacial Till,fragments of shale. Hard		2	SS	41					
			3	SS	100/6"					
213.2 9.5	End of Borehole									
						210				

OFFICE REPORT ON SOIL EXPLORATION

20
15 ◇ 5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATION OFFICE

RECORD OF BOREHOLE Nº 8

JOB 72-11109
LOCATION Co-ords. 16,506,261 N; 1,227,731 E.
ORIGINATED BY SAA
W.P. 13-68-04
BORING DATE Sept. 22, 1972
COMPILED BY SAA
DATUM Geodetic
BOREHOLE TYPE Cont. Fligh. Auger
CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE				GR.SA.SI.CL.
221.9	Ground Level									
0.0	Silty sand, trace of clay.					220				
			1	SS	16			○		0 57 35 8
215.9	Compact									▽ 215.9
6.0	Glacial Till, fragments of shale.		2	SS	100/6"					
212.3	Hard		3	SS	100/3"					
9.3	End of Borehole Probable Bedrock					210				

OFFICE REPORT ON SOIL EXPLORATION

20
15 ◊ 5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

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RECORD OF BOREHOLE Nº 9

JOB 72-11109
W.P. 13-68-04
DATUM Geodetic

LOCATION Co-ords. 16,506,346 N; 1,227 774 E.
BORING DATE September 22, 1972
BOREHOLE TYPE Cont. Flight Auger

ORIGINATED BY SAA
COMPILED BY SAA
CHECKED BY

SOIL PROFILE			SAMPLES							
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —— w_L PLASTIC LIMIT —— w_P WATER CONTENT —— w; w_P w w_L; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
221.1	Ground Level									
0.0	Silty sand, trace of gravel. Brown					220				
			1	SS	5					▼ 216.0 WL in open BH
	Loose to Dense		2	SS	40					
213.1										
212.6	Glacial Till. Hard		3	SS	[illegible]/6"					
8.5	End of Borehole Probable Bedrock					210				

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

DESIGN SERVICES BRANCH

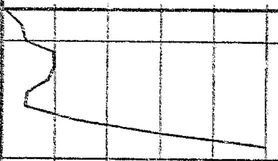
FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 10

JOB 72-11109
W.P. 13-68-04
DATUM Geodetic

LOCATION Co-ords. 16,506,686 N; 1,227,754 E.
BORING DATE Nov. 1, 1972
BOREHOLE TYPE Flight Auger and Cone Test

ORIGINATED BY JC
COMPILED BY JC
CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT —— w_L PLASTIC LIMIT —— w_P WATER CONTENT —— w w_P w w_L WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE				
222.0	Ground Level									
	Topsoil									
1.0	Silty sand to sand with a trace of gravel.		1	SS	14	220				
			2	SS	12					▼216.7
213.8 8.2	Compact to Dense		3	SS	45					
211.0	Glacial Till Hard		4	SS	65/10"					
11.0	End of Borehole Probably Bedrock					210				

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 11

JOB 72-11109

W.P. 13-68-04

DATUM Geodetic

LOCATION Co-ords. 16,506,172 N; 1,227,855 E.

BORING DATE Nov. 1, 1972

BOREHOLE TYPE Flight Auger, BXL Rock Core and Cone Test

ORIGINATED BY JC

COMPILED BY JC

CHECKED BY

SOIL PROFILE			SAMPLES							
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F. ○ 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.
221.4	Ground Level									
	Topsoil									
1.0	Silty sand to sand with a trace of gravel		1	SS	12	220				▽ 217.9
216.4	Compact		2	SS	125					
5.0	Clayey silt with sand & gravel (Glac.Till) (shale fragments throughout)		3	SS	100/3"					
208.9	Hard		4	SS	100/3"	210				
12.5	Shale Bedrock (Fractured and weathered)		5	BXL RC	20%					
198.4			6	BXL RC	60%	200				
196.5	Sound		7	BXL	95%					
24.9	End of Borehole									
						190				

20
15 ⌖ 5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 12

JOB 72-11109

W.P. 13-68-04

DATUM Geodetic

LOCATION Co-ords. 16,506,345 N; 1,227,842 E.

BORING DATE Oct. 31, 1972

BOREHOLE TYPE Flight Auger and BXL Rock Core

ORIGINATED BY JC

COMPILED BY JC

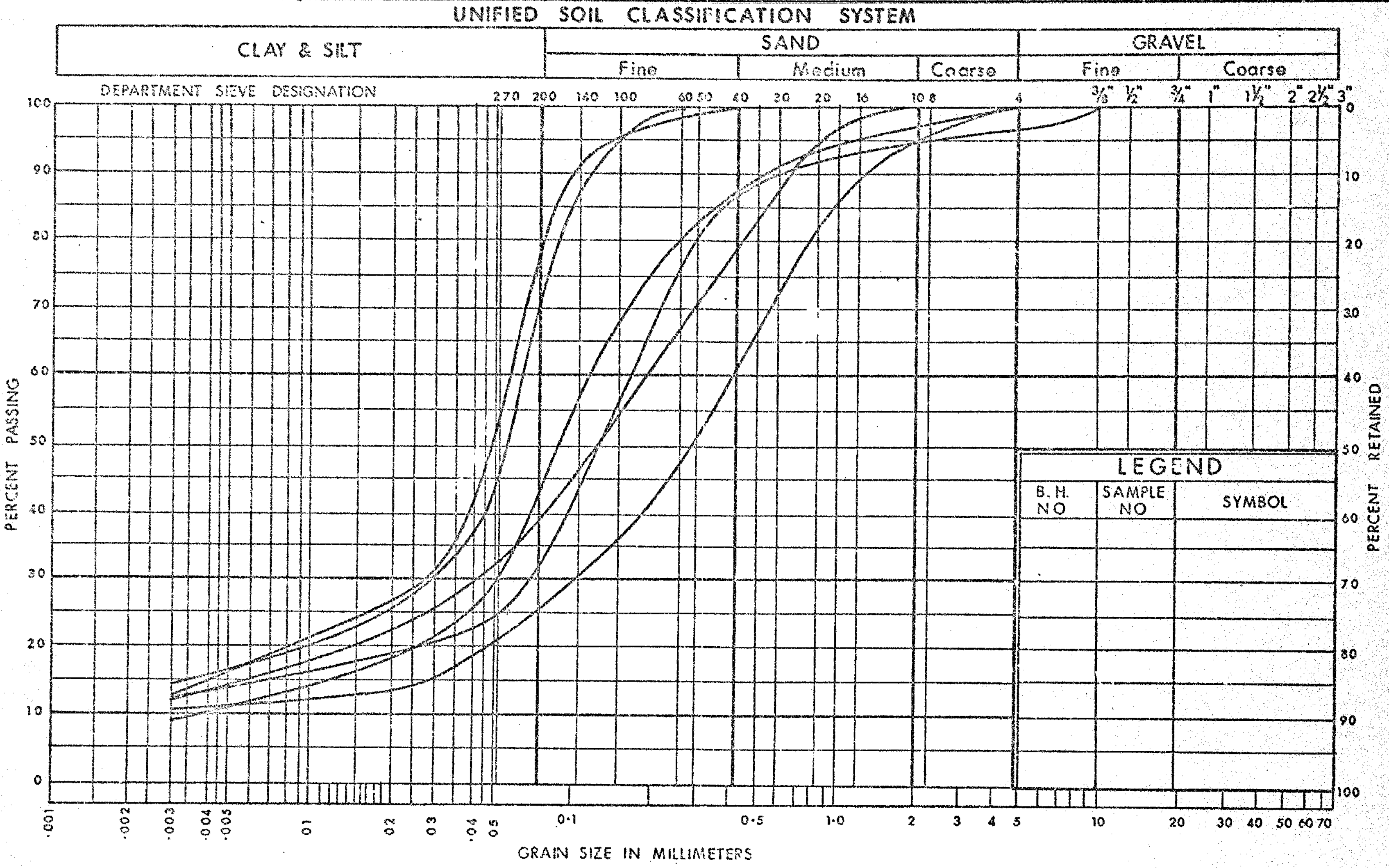
CHECKED BY

SOIL PROFILE			SAMPLES							
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W; W_P W W_L; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
220.2	Ground Level									
0.0 216.2	Granular Road Fill (sand and gravel) Compact					220				
4.0 212.7	Clayey silt with sand & gravel(Glac.Till) Hard		1	SS	58					▼ 216.0 WL in open BH
211.2	Weathered		2	SS	100/0"					
9.0 206.2	Shale Bedrock Numerous fissures throughout. Sound		3 4	RC BXL RC BXL	88% 84%	210				
14.0	End of Borehole									
						200				

OFFICE REPORT OF SOIL EXPLORATION

20
15 ✧ 5 % STRAIN AT FAILURE
10

FORM OD-MT-17
REV. AUG. 1971



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS

ONTARIO

DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION

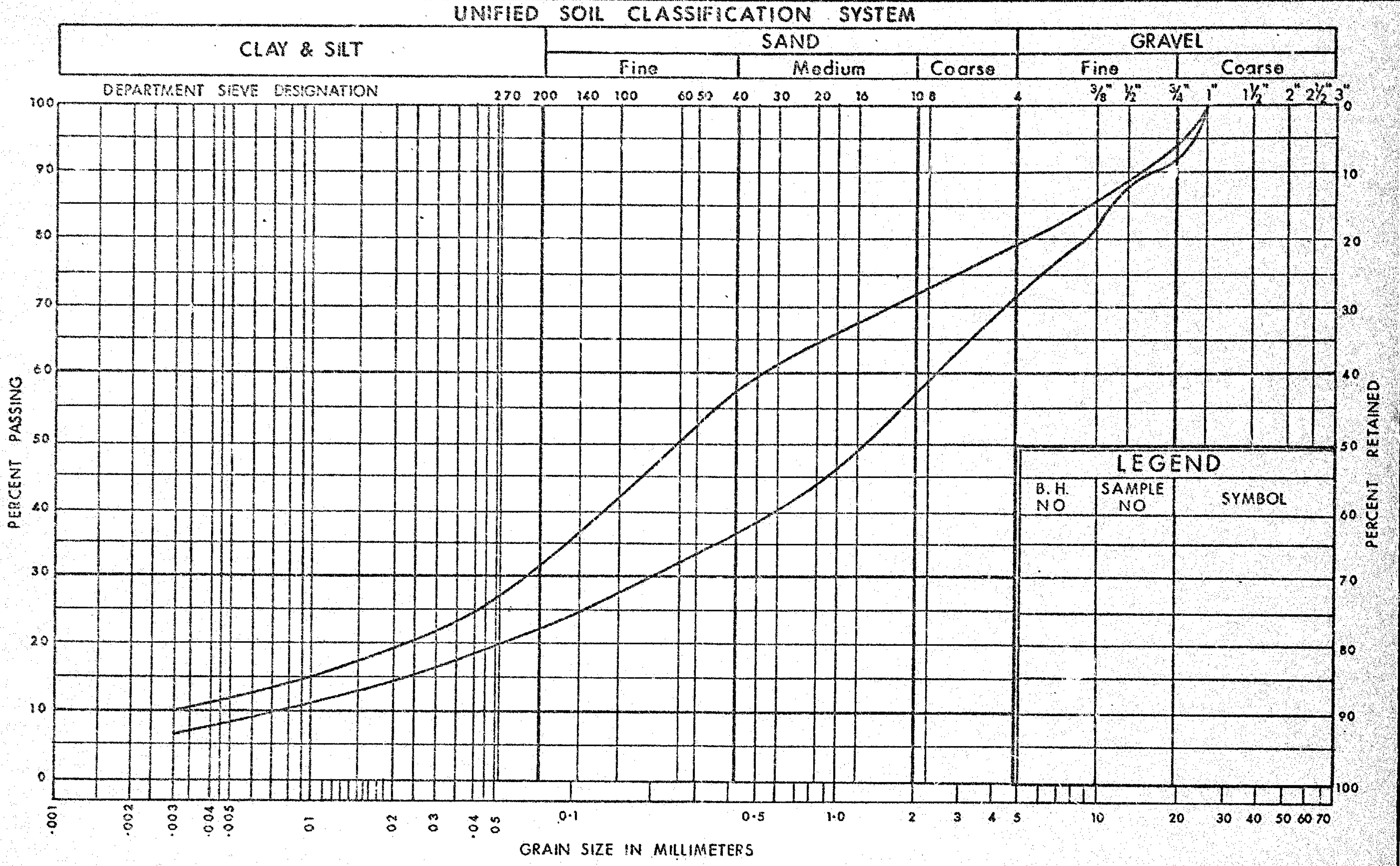
SILTY SAND TO SANDY SILT

W.P. No. 13 - 68 - 04

JOB No. 72 - 11109

FIG. 1

FORM OD-MT-17
REV. AUG. 1971

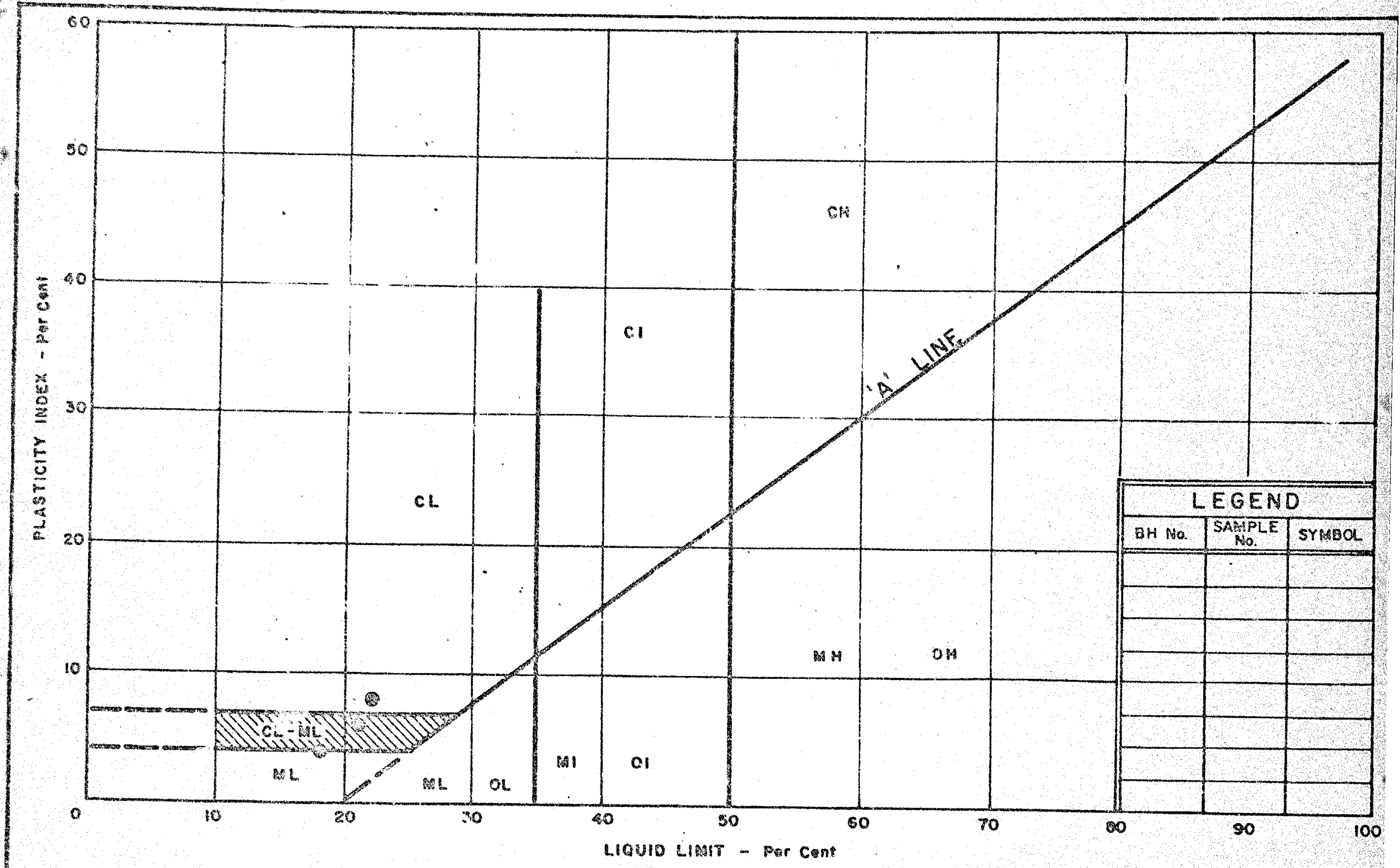


DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES
BRANCH
ONTARIO

GRAIN SIZE DISTRIBUTION
GLACIAL TILL

W.P. No. 13-68-04
JOB No. 72-11109
FIG. 2

FORM OB-MT-252
REVISED JUL 1966



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART
GLACIAL TILL

W.P. No. 13-68-04
JOB No. 72-11109

FIG. 3

APPENDIX D

Basic Chemical Analysis – Eurofins Report Number 1814197

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: 1814197
Date Submitted: 2018-08-09
Date Reported: 2018-08-14
Project: 1662565/1240
COC #: 834574

				Lab I.D.	1379724
				Sample Matrix	Soil
				Sample Type	
				Sampling Date	2018-08-09
				Sample I.D.	18-2401Sa3/5-5.42'
Group	**Analyte**	**MRL**	**Units**	**Guideline**	
Anions	Cl	0.002	%		0.030
	SO4	0.01	%		0.07
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.78
	pH	2.00			8.11
	Resistivity	1	ohm-cm		1280

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 E

Results of MASW Testing

TECHNICAL MEMORANDUM

DATE July 16, 2018

Project No. 1662565/1240

TO Susan Trickey, Golder Associates Ltd.

FROM Stephane Sol, Christopher Phillips

EMAIL ssol@golder.com, cphillips@golder.com

CHBDC SEISMIC SITE CLASS TESTING RESULTS
CYRVILLE 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 Cyrville Road and Highway 417 in Ottawa. The geophysical testing was performed by Golder Associates Ltd. (Golder) personnel on June 26, 2018.

Figure 1: MASW Location Site Map. MASW Lines in red – Line 1 (Northbound) and Line 2 (Southbound).

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 June 26, 2018, by personnel from the Golder Mississauga office. 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. A seismic refraction survey was also conducted along both lines to be able to determine the depth to bedrock as well as to estimate the shear wave velocity of the overburden. An example of active seismic records collected at each line are shown in Figures 2 and 3, below.

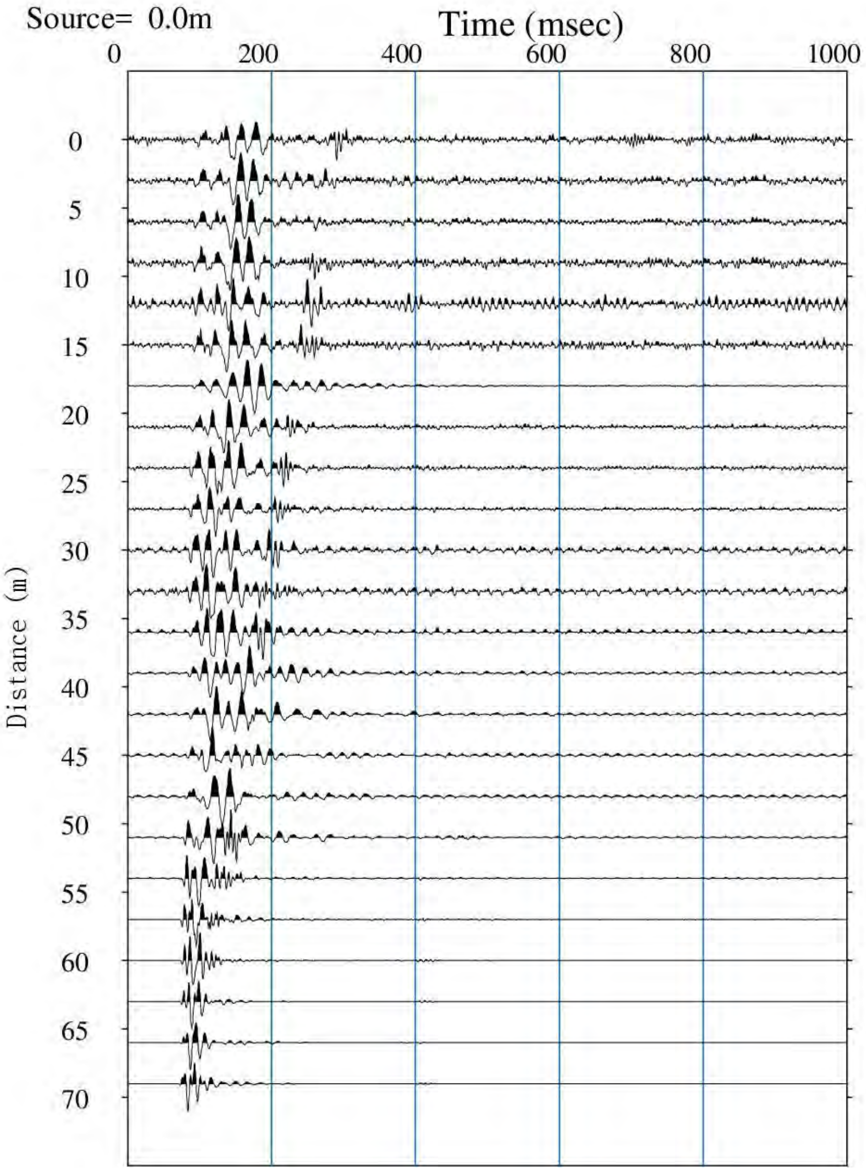


Figure 2: Typical seismic record collected at the site of MASW Line 1.

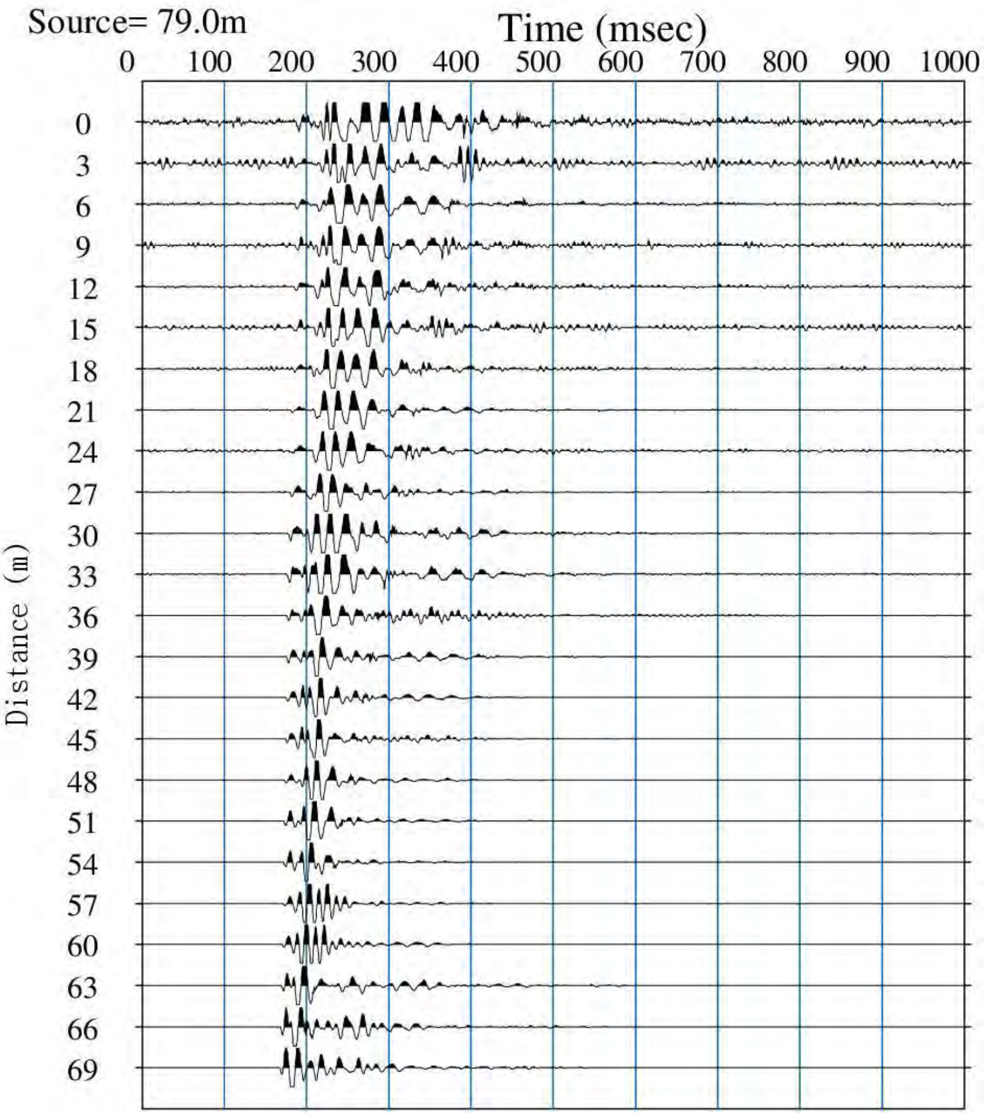


Figure 3: Typical seismic record collected at the site of MASW Line 2.

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 (9-31 Hz). The active survey of Line 2 provided a dispersion curve with a suitable frequency range (9-81 Hz). The minimum measured surface wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 9 Hz at Lines 1 and 2.

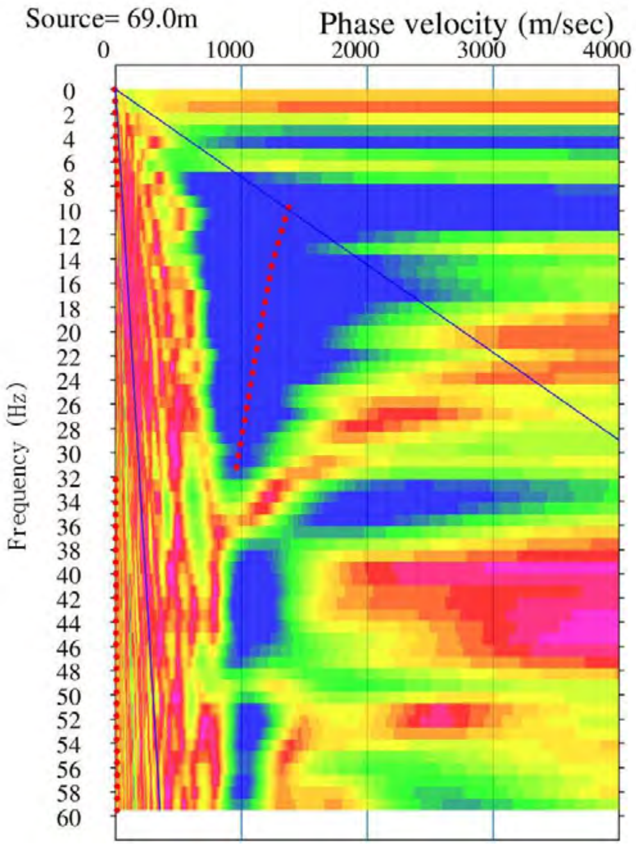


Figure 4: Active MASW Dispersion Curve Picks (red dots) along MASW Line 1

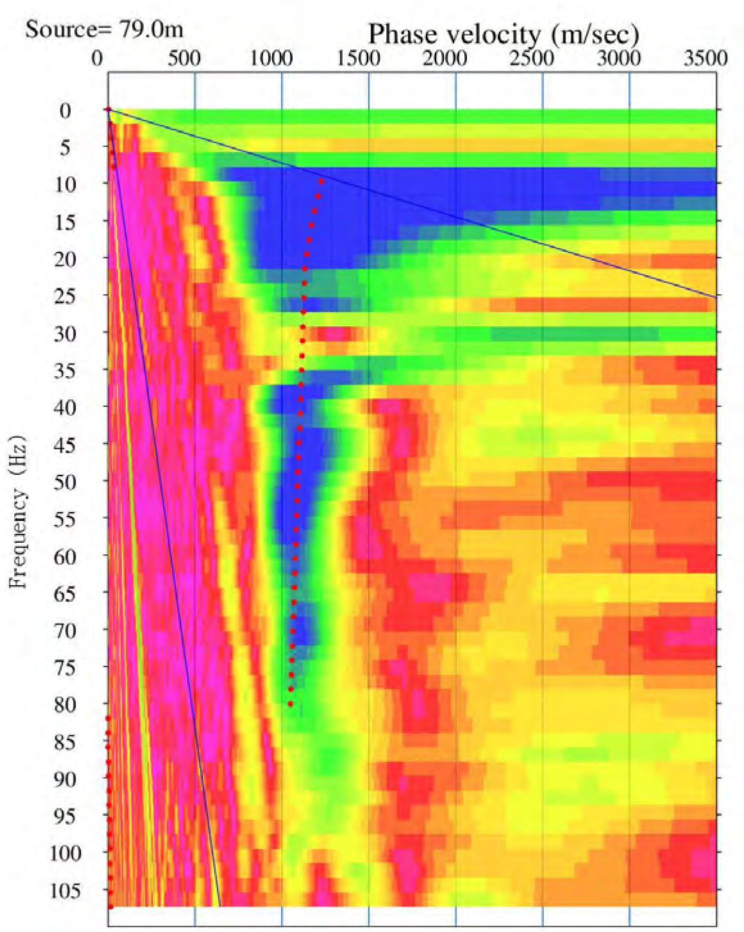


Figure 5: Active MASW Dispersion Curve Picks (red dots) along MASW Line 2

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 1% along both lines.

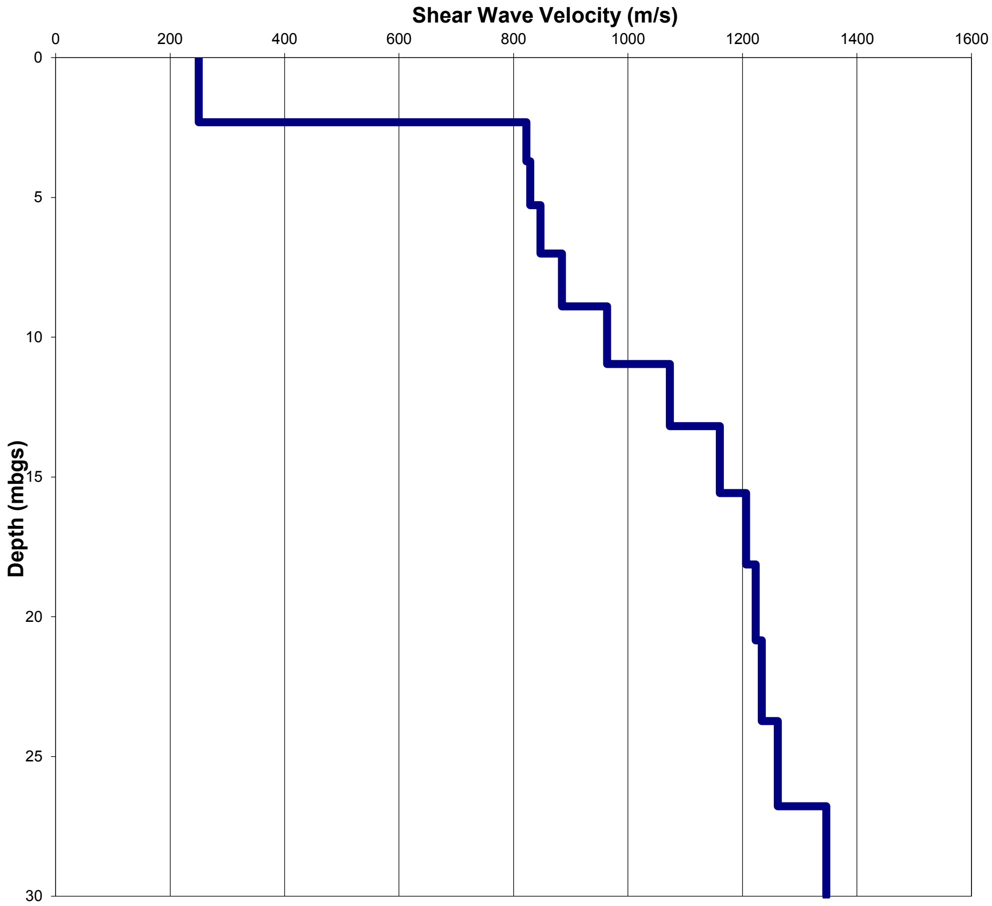


Figure 6: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 1

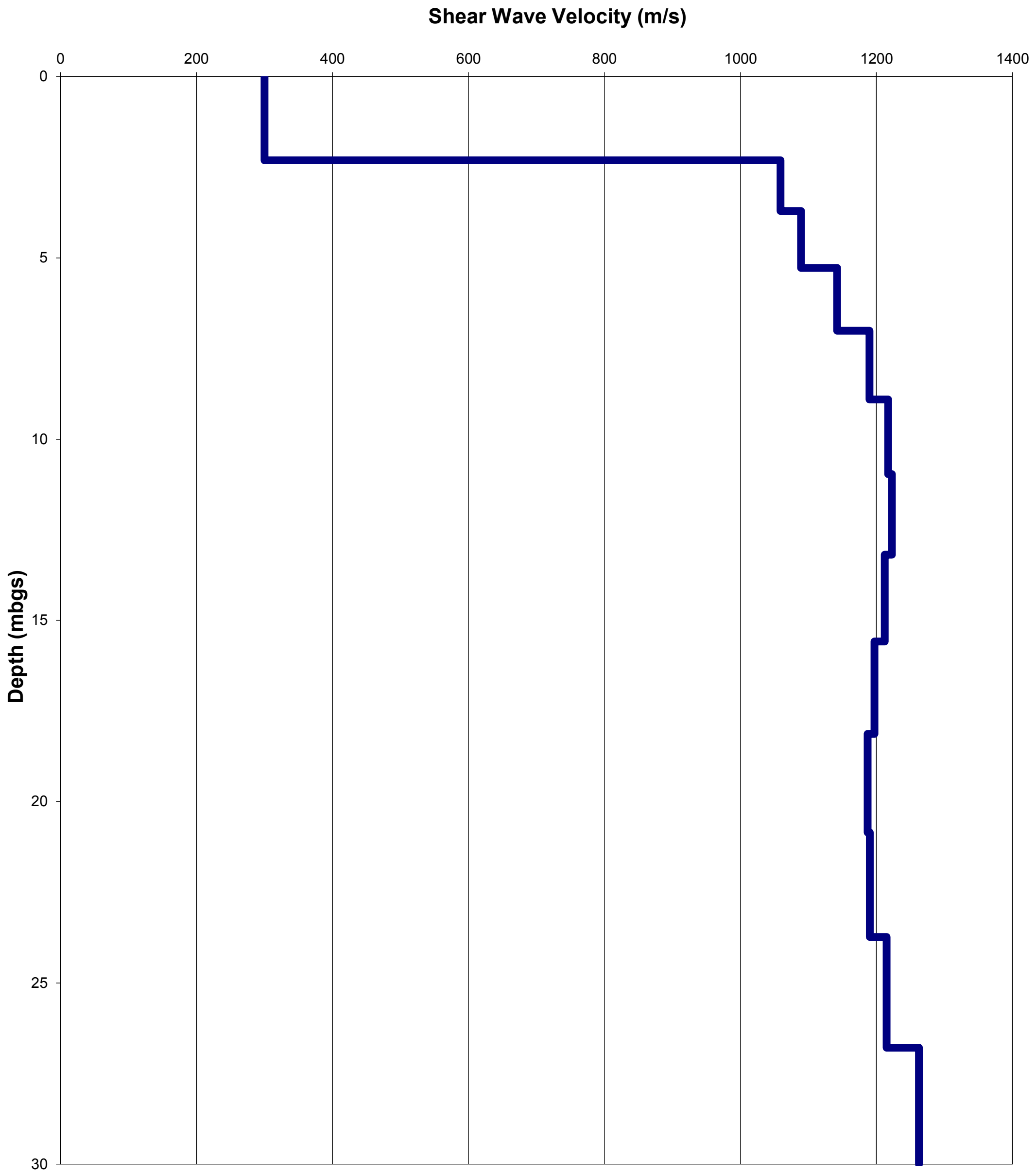


Figure 7: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 2

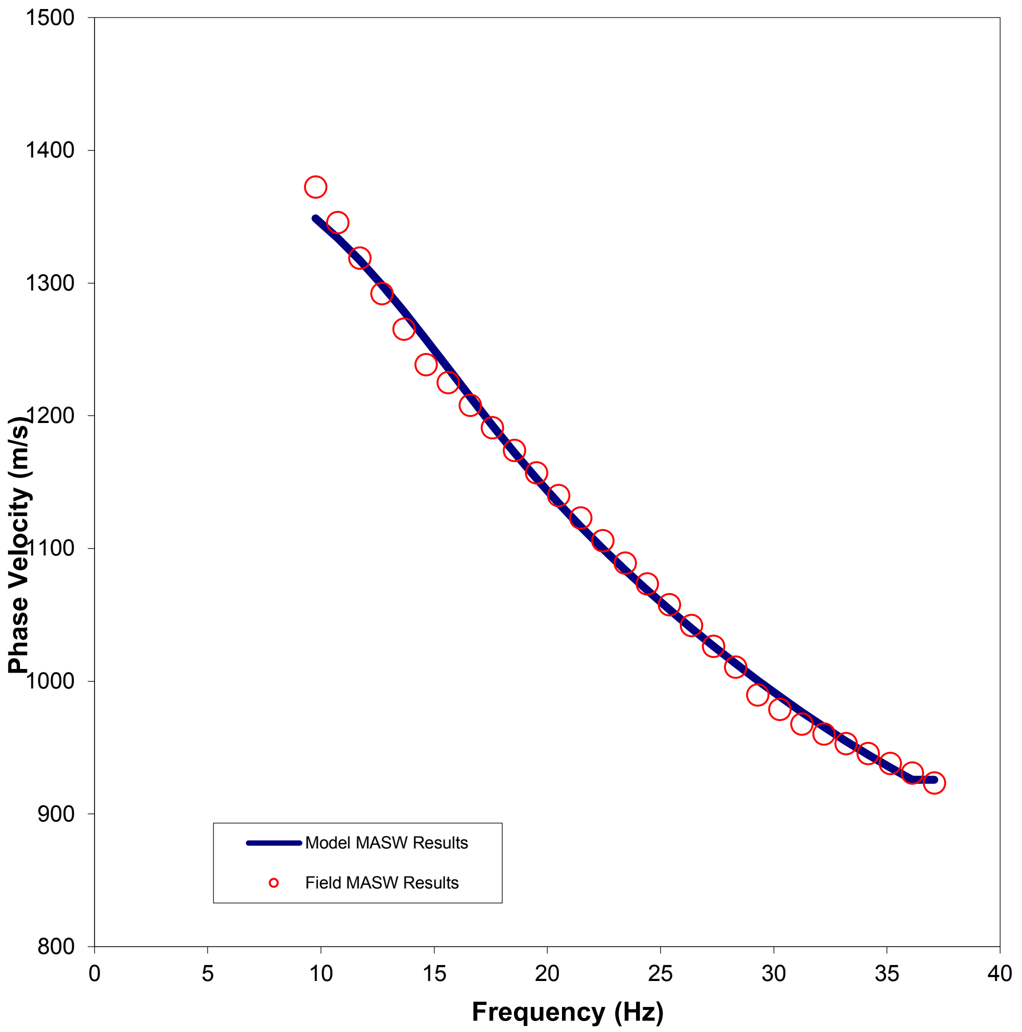


Figure 8: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 1

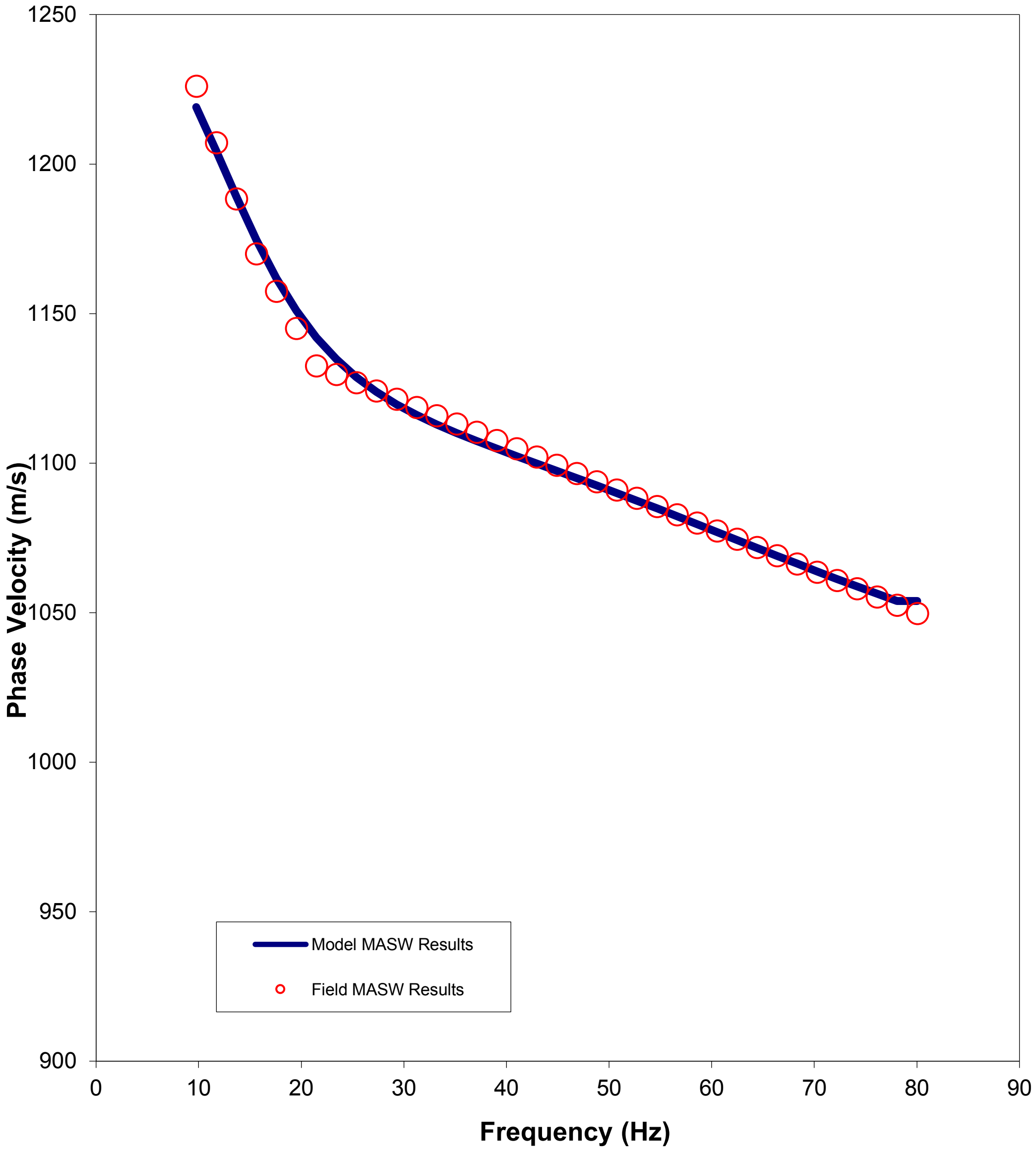


Figure 9: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 2

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 863 m/s (Table 1). The average shear-wave velocity along MASW Line 2 in the south was found to be 969 m/s (Table 2).

Table 1: 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	250	0.004286
1.07	2.31	1.24	250	0.004945
2.31	3.71	1.40	823	0.001703
3.71	5.27	1.57	829	0.001889
5.27	7.01	1.73	847	0.002044
7.01	8.90	1.90	885	0.002143
8.90	10.96	2.06	964	0.002138
10.96	13.19	2.23	1073	0.002074
13.19	15.58	2.39	1160	0.002060
15.58	18.13	2.55	1206	0.002118
18.13	20.85	2.72	1223	0.002224
20.85	23.74	2.88	1234	0.002338
23.74	26.79	3.05	1261	0.002417
26.79	30.00	3.21	1346	0.002387
			Vs Average to 30 mbgs (m/s)	**863**

Table 2: 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	300	0.003571
1.07	2.31	1.24	300	0.004121
2.31	3.71	1.40	1059	0.001323
3.71	5.27	1.57	1089	0.001438
5.27	7.01	1.73	1142	0.001515
7.01	8.90	1.90	1190	0.001593
8.90	10.96	2.06	1217	0.001693
10.96	13.19	2.23	1223	0.001820
13.19	15.58	2.39	1212	0.001972
15.58	18.13	2.55	1197	0.002134
18.13	20.85	2.72	1187	0.002291
20.85	23.74	2.88	1190	0.002424
23.74	26.79	3.05	1215	0.002511
26.79	30.00	3.21	1262	0.002546
			Vs Average to 30 mbgs (m/s)	**969**

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.

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SS/CRP/jl

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APPENDIX F

Site Photographs (provided by WSP)

South Elevation - Looking towards HWY 417 WB

North Elevation - Looking towards Cyrville Road EB

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