



FINAL REPORT

Foundation Investigation and Design Catherine Street High Fill Embankment Highway 417 at Bronson Avenue Ottawa, Ontario *G.W.P. No. 4173-15-00*

Submitted to:

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PART A

Foundation Investigation
Catherine Street High Fill Embankment
Highway 417 at Bronson Avenue
Ottawa, Ontario

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 the detailed design of numerous bridge replacements, overhead signs, noise barrier walls, temporary roadway protection systems, replacement of storm sewers (including trenchless crossings) and a high fill embankment on Highway 417 between Island Park Drive and Kent Street in Ottawa, Ontario (Assignment number 4016-E-0001).

This report presents the results of the foundation investigation carried out for the construction of the proposed high fill embankment to be located at 458 Catherine Street, following the demolition of the existing A1 Mini Storage Facility currently on the site.

The terms of reference and scope of work for the foundation investigation are outlined in the MTO's Request for Proposal (RFP), dated April 2016, and subsequent addenda. Golder's scope of work for foundation engineering services associated with the 458 Catherine Street site 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 May 13, 2017.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 Site Description

The proposed high fill embankment is to be located at 458 Catherine Street at the intersection of Bronson Avenue and Catherine Street in the City of Ottawa. The site is bounded by Highway 417 to the south, Bronson Avenue to the west, Catherine Street to the north, and the Highway 417, W-N/S offramp to the east. The location of the site is shown on the Key Plan on Drawing 1. Site photographs showing the general conditions at the site are presented in Appendix F.

The A1 Mini Storage facility is currently located within the proposed footprint of the new embankment. The facility consists of several buildings, paved parking areas and driving lanes. There is an existing hydro line that runs from the facility to the west along Catherine Street, and several catch basins and storm sewer lines are present in the paved parking and loading areas east and west of the buildings. It is understood that the overall construction project includes the demolition and removal of the facility and all associated works. Also prior to construction of the new embankment the site will serve as a staging area for the construction of the new overpass structures.

At this location, the adjacent Highway 417 is a divided highway with four travel lanes in each direction separated by a concrete barrier wall.

The available construction drawings indicated that the existing concrete cantilevered retaining wall supporting the north side of the Highway 417 WBL is between 6 and 8 m in height relative to the elevation of the existing parking lot. The foundation for the retaining wall varies along the length of the wall, from strip footings dowelled into bedrock near the existing bridge location, to spread footings supported on the native sand materials, to piled foundations at the east end of the wall (see reference drawing D5001-1 in Appendix C). The top of footing elevation ranges from approximately 68.4 m near the existing overpass to 67.4 m at the east end of the wall. The retaining wall consists of a cast-in-place stem wall with a coping overlay. It is understood that the existing retaining wall will be buried behind the proposed embankment as part of the proposed construction plan.

2.2 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, the proposed site 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 bedrock².

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 limestone of the Verulam Formation³. The limestone is described as interbedded bioclastic, sublithographic to fine crystalline, very thin to medium bedded, with shale interbeds up to 8 centimetres thick. Bedrock outcrops are mapped north and south of the Highway 417 / Bronson Avenue Interchange.

The site lies between two faults striking southeast to northwest. The more prominent fault, the Gloucester fault crosses Highway 417 at the approximate location of Preston Street⁴. The second fault crosses Highway 417 some 400 m to the east of Bronson Avenue. Bedding which is normally sub-horizontal often dips steeply adjacent to and within fault zones.

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 that have recently occurred in the WQ zone are 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 fieldwork for the subsurface investigation was carried out between November 9 and November 27, 2018 and included advancing a total of five boreholes designated 18-301 to 18-305, inclusive.

Boreholes 18-301 and 18-302 were advanced in the parking lot west of the existing A1 Mini Storage Facility. Borehole 18-303 was advanced on the south portion of the site, between the storage facility and the highway, while Boreholes 18-304 and 18-305 were advanced in the east loading and parking areas.

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

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.

³ Williams, D.A. Rae, A.M., and Wolf, R.R. 1984: *Paleozoic Geology of the Ottawa Area, Southern Ontario*, Ontario Geological Survey, Map P.2716. Geological Series-Preliminary Map, scale 1:50,000. Geology 1982

⁴ MacDonald, G. and Harrison, J.E. 1976: *Generalized Bedrock Geology, Ottawa-Hull, Ontario and Quebec*, Geological Survey of Canada, Map 1508A, scale 1:125,000. Geology 1967.

Table 1 below further outlines the location of the boreholes with respect to the existing facility.

Borehole 18-303 was advanced at the crest of the embankment using portable rotary drilling equipment employing a third weight hammer lifted manually and dropped from the SPT height. Where a third weight hammer was used, the N values presented on the Record of Boreholes are “uncorrected” and should be interpreted in consideration of their reduced penetration energy. The remaining boreholes were advanced with a CME75 truck mounted drill rig. All drilling equipment was supplied and operated by CCC Geotechnical & Environmental Drilling Ltd. of Ottawa, Ontario.

Soil samples from boreholes advanced with truck-mounted drilling equipment were obtained at vertical intervals of about 0.76 m, using 50 mm outside diameter split spoon samplers, in accordance with the Standard Penetration Test (SPT) procedure ASTM D1586. Soil samples from the portable drilling equipment were obtained in continuous vertical increments of about 0.6 m.

A monitoring well was installed in Borehole 18-303, to observe the stabilised groundwater level at the site. The monitoring well consisted of 32 mm outside diameter PVC tubing with a 1.1 m long screen. The groundwater levels were measured in the monitoring well on December 10, 2018 and then the well was decommissioned according to Ontario MOE Regulation 903 (O.Reg 903) by a licenced well technician.

The boreholes were backfilled with a mixture of bentonite and soil cuttings and capped with asphaltic concrete cold patch. The site conditions were restored following completion of the field work.

The field work was supervised on a full-time basis by members of Golder’s staff who located the boreholes in the field, directed the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil samples. The soil samples were identified in the field, placed in appropriate containers, and transported to Golder’s laboratory in Ottawa for further examination and testing. Index and classification tests consisting of water content determinations, Atterberg Limits testing, organic content and grain size distribution analyses were carried out on selected soil samples. The laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate.

In addition to the borehole investigation at the current site, shear wave velocity profiling was completed at the nearby Highway 417 Overpass Structures at Rochester Street (Sites 3-56/1 and 3-56/2), located approximately 550 m west of Bronson Avenue. Due to the close proximity and similar nature of the surficial and bedrock geology of the sites, the profiles developed at Rochester Street (Sites 3-56/1 and 3-56/2) are considered relevant to the current site.

The shear wave velocity profiling was carried out in the grassy area next to the eastbound E-N/S off-ramp of the Highway 417 / Rochester Street Interchange, just west of Rochester Street, using the Multichannel Analysis of Surface Waves (MASW) technique. The MASW profiling was carried out on October 18, 2017, by personnel from Golder’s Mississauga and Ottawa offices. A series of low frequency (4.5 Hz) geophones were laid out at 2 m intervals. A 9.9 kg sledgehammer and 34 kg drop weight were used as the seismic source. The source locations were offset at various distances beyond the end and collinear with the geophone array.

The borehole locations and elevations were surveyed by Golder using a Trimble R8 GPS unit referenced to the NAD83 CSRS CBNv6-2010.0 MTM Zone 9 geodetic datum. The borehole locations, including northing and easting coordinates, ground surface elevations, and drilled depths are summarized in Table 1.

Table 1: Borehole Summary 2018 Investigation

Borehole	Location	NAD83 CSRS CBNv6-2010.0 MTM Zone 9		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m)	Easting (m)		
18-301	North Side of Western Parking Lot	5029802.9	367298.3	68.7	1.9
18-302	Western Parking Lot Adjacent to Storage Building	5029799.6	367324.6	68.5	2.8
18-303	South of Storage Building Adjacent to Highway 417	5029814.2	367376.5	68.6	4.3
18-304	North of Storage Building	5029855.8	367422.1	67.3	4.3
18-305	Eastern Loading Area Adjacent to Storage Building	5029834.9	367416.3	67.9	6.0

3.2 Previous Investigations

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

3.2.1 2018 - Replacement of Sites 3-60/1 and 3-60/2

A foundation investigation was carried out for the rapid bridge replacement of the Highway 417 Overpass Structures at Bronson Avenue (Sites 3-60/1 3-60/2) by Golder. The subsurface information and results of the investigation are contained in the report titled:

- *Foundation Investigation and Design Highway 417 Overpass Structures at Bronson Avenue Rapid Bridge Replacement Sites: 3-60/1 & 3-60/2, Ottawa, Ontario, dated June 2019, (Report No. 1655214-1130).*

A total of 24 coreholes and four boreholes were advanced along Bronson Avenue and eleven boreholes within the Highway 417 approach embankments and proposed staging area. In particular, Boreholes 17-134, 18-1303 and 18-1307 were advanced along the north side of Highway 417 in the general vicinity of the high fill area. In general, at the borehole locations advanced along Bronson Avenue (i.e., Boreholes 17-134 and 18-1303) the subsurface conditions consist of Portland concrete cement (PCC) sidewalk at surface, overlying granular fill overlying PCC footing and/or limestone bedrock. At the embankment borehole (Borehole 18-1307), the subsurface conditions consist of asphaltic concrete and PCC overlying sandy fill materials and glacial till.

The Record of Borehole sheets from the investigation at Sites 3-60/1 & 3-60/2 pertinent to the current site are provided for reference in Appendix C. The borehole locations and ground surface elevations for these boreholes are shown on Drawing 1.

The subsurface conditions within these boreholes are referenced in Section 4.0 below only with respect to the bedrock conditions.

3.2.2 1961 and 1962 - Original Investigation

Two previous investigations were carried out by H.Q Golder and Associates Ltd for the design of the existing overpass structures and retaining walls in 1961 and 1962, respectively. The subsurface information and results of the original investigations are contained in the reports titled:

- Site Investigation, Proposed Queensway-Bronson Overpass, Bridge No. 18, Ottawa, Ontario", dated August 1961, (Report No. 6105, GEOCREs No. 31G05-043).
- Site Investigation, Proposed Retaining Walls, Ottawa Queensway, Ottawa, Ontario", dated January 1962, (Report No. 6146, GEOCREs No. 31G05-049).

A total of six boreholes (each with an adjacent dynamic penetration test DPT) and one additional DPT were advanced at the site as part of the original investigation along the then proposed bridge alignment over Bronson Avenue. A total of 18 boreholes were advanced along the Highway as part of the original geotechnical investigation for the then proposed retaining wall structures. The Record of Borehole sheets from the previous investigations pertaining to the current site (i.e., Boreholes 1, 2, B1, B3, B13, B14 and B16) are provided for reference in Appendix C. The approximate borehole locations and ground surface elevations for these boreholes are shown on Drawing 1.

The locations of the boreholes from the original 1961 and 1962 investigations should be considered approximate since the locations were referenced to an imperial borehole location plan rather than metric MTM coordinates. Further, the boreholes from the original investigations were advanced prior to construction of the bridge and the ground surface conditions shown may not be representative of the post-construction subsurface conditions, particularly with respect to the composition and thickness of overburden and fill. It is also unknown if the surface of the bedrock as encountered during these investigations was altered during construction of the overpass structure. Therefore, the stratigraphy encountered in these boreholes was not included in the Description of Subsurface Conditions below or the stratigraphic profile shown on Drawing 1.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 General

The subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ testing from the current investigation are provided on the Record of Borehole sheets in Appendix A. The results of the laboratory testing carried out during the current investigation are presented on the Record of Borehole sheets as well as on Figures B1 to B4 in Appendix B. The borehole locations and the interpreted stratigraphic profile projected along the site are provided on Drawing 1.

The MASW test results and report from Sites 3-56/1 and 3-56/2 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 for those sites.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic sections from Drawing 1, are inferred from observations of drilling progress and noncontinuous 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.

4.2 Site Stratigraphy Overview

In general, the subsurface conditions at the borehole locations consists of asphaltic concrete pavement, about 0.1 to 0.3 m thick, underlain by sand and gravel to silty sand fill extending to depths ranging from 1.5 and 2.3 m below ground surface (i.e., to Elevations 65.0 to 67.2 m). At Boreholes 18-304 and 18-305, peat deposits about 0.4 and 1.5 m in thickness, respectively, underlies the fill. A sand and silt to silt deposit was encountered beneath the fill in Borehole 18-303 and the peat layer in 18-304 that extends to depths of 3.7 and 3.8 m, respectively (i.e., Elevations 64.9 and 63.5 m). Silty clay was encountered below the peat in Borehole 18-305, with a bottom depth of about 4.6 m (Elevation of 63.3 m). A non-cohesive glacial till layer was encountered underlying the fill in Borehole 18-301 and 18-302, the sand and silt in Boreholes 18-303 and 18-304 and the silty clay in Borehole 18-305, extending down to auger refusal between 1.9 and 6.0 m in depth (i.e. Elevations 61.9 to 66.8 m).

The groundwater level was measured at a depth of 2.4 m, corresponding to Elevation 66.1 m, in Borehole 18-303.

A more detailed description of the overburden soil deposits, and groundwater conditions encountered is provided in the following sections.

4.3 Pavement Material

The boreholes for this investigation were advanced through the paved parking lot and loading area. The thickness of the asphaltic concrete ranges from 60 to 300 mm.

4.4 Fill

Sand and Gravel to Sand

Fill consisting predominantly of sand with varying amounts of gravel was encountered below the asphaltic concrete in all the boreholes. The top of the fill was encountered at elevations ranging from 67.2 to 68.6 m and the thickness of the fill ranges from 0.2 to 0.5 m.

Silty Sand to Sand

Silty sand to sand fill, with varying amounts of gravel and containing cobbles, was encountered beneath the upper sandy fill at each of the boreholes. The top of this fill was encountered at elevations ranging from 66.8 to 68.3 m and the thickness of the fill ranges from 0.4 to 2.0 m. The SPT N values ranged from 4 to greater than 100 indicating a loose to very dense state of packing, but more typically loose to compact.

The moisture content of three samples ranged from 16 to 23 percent. The results of a grain size analysis test carried out on one sample of the fill are provided on Figure B1 in Appendix B.

Sandy Silt

Fill consisting of sandy silt with trace amounts of gravel and containing cobbles was encountered beneath the sand to silty sand fill in Borehole 18-301. The top of the sandy silt fill was encountered at Elevation 67.9 m and the fill is about 0.7 m in thickness. An SPT N value of 13 was measured in this layer, indicating a compact condition.

4.5 Peat

A deposit of peat was encountered underlying the fill in Boreholes 18-304 and 18-305. The top of the peat was encountered at elevations ranging from 65.0 to 65.6 m and the peat is 0.4 and 1.5 m in thickness respectively. The SPT N values ranged from the weight of the hammer to 1, indicating a very loose state of packing.

The moisture content of two samples tested was 366 and 485 percent. The organic content of one sample of the peat layer was determined to be approximately 71 percent.

4.6 Silt to Sand and Silt

Deposits of silt and sand and silt were encountered beneath the fill in Borehole 18-303 and beneath the peat layer in Borehole 18-304, respectively. The surfaces of the silty deposits were encountered at elevations of 66.3 and 64.6 m and the deposits are 1.4 and 1.1 m in thickness at Boreholes 18-303 and 18-304, respectively. An SPT N value of 11, in the silt at Borehole 18-304, indicates the silt has a compact state of packing.

Strong hydrocarbon odour was noted in the sample of silt obtained from Borehole 18-304. Further details with regards to the material handling, reuse and/or disposal of environmentally impacted materials are provided in WSP's 2018 Phase II ESA and 2019 Earth Management Plan Reports, which are provided under separate cover.

The moisture content of the two samples tested were 18 and 23 percent. The results of grain size analysis testing carried out on two samples of silty deposits are provided on Figure B2 in Appendix B.

4.7 Silty Clay

Silty clay was encountered beneath the peat material in Borehole 18-305. The top of this layer was encountered at elevation 64.1 m and the thickness of this layer is 0.8 m. The silty clay is grey in colour, unweathered and a SPT N-value of 'weight of hammer' indicates a firm consistency.

The moisture content of one sample tested was 44 percent. The results of an Atterberg Limits test completed on one sample of this material indicated a liquid limit value of 49 percent, a plastic limit of 20 percent and a corresponding plasticity index of 29 percent. The Atterberg Limits analysis results are provided on Figure B4 in Appendix B and indicate a silty clay with intermediate plasticity (CI).

4.8 Glacial Till

A non-cohesive glacial till deposit consisting of a heterogeneous mixture of sand, gravel and silt and containing cobbles was encountered beneath the fill in Boreholes 18-301 and 18-302, the sand and silt in Boreholes 18-303 and 18-304 and beneath the silty clay deposit in Borehole 18-305. The top of this layer was encountered at elevations ranging from 63.3 to 67.2 m. The till was not fully penetrated but was proven to the depth of auger refusal, with proven thicknesses ranging from 0.4 to 1.4 m. The SPT N values ranged from 9 to 83, indicating a loose to very dense state of packing.

The moisture content of four samples tested ranged from 6 to 13 percent. The results of a grain size analysis test carried out on a single sample of this material are provided on Figure B3 in Appendix B.

4.9 Auger Refusal and Bedrock

Bedrock was not proven as part of the current investigation. Auger refusal was encountered in all the boreholes, except Borehole 18-303, at elevations ranging from 61.9 to 66.8 m.

Bedrock was proven by coring using NQ sized equipment during Golder's 2017 and 2018 investigations for the adjacent Highway 417 Overpasses at Bronson Avenue (Sites 3-60/1 and 3-60/2) in Boreholes 17-134 and 18-1303. The bedrock encountered was limestone with shale partings and interbeds. Table 2 summarizes the depth to and the elevation of the bedrock surface as encountered at the borehole locations from Sites 3-60/1 and 3-60/2.

Table 2: Summary of Bedrock Surface Depths and Elevation from Sites 3-60/1 and 3-60/2

Borehole	Borehole Location	Existing Ground Surface Elevation (m)	Depth to the Bedrock Surface (m)	Bedrock Surface Elevation (m)
17-134	Bronson Avenue Northeast of east abutment	69.3	1.9	67.4
18-1303	Bronson Avenue Northeast of east abutment	69.5	2.5	67.0

4.10 Groundwater Conditions

A groundwater monitoring well was installed in Borehole 18-303 as part of the current investigation to allow for measurement of the groundwater level across the site. Due to the proximity of the monitoring well from the investigation carried out for Sites 3-60/1 3-60/2, the groundwater measurement from Borehole 17-134 has been included in this report.

Table 3 summarizes the depths and elevations of the groundwater level measured in the monitoring wells.

Table 3: Summary of Groundwater Conditions

Borehole	Ground Surface Elevation (m)	Screened Interval Material	Groundwater Level		Date
			Depth (m)	Elevation (m)	
18-303	68.6	Sand and Silt	2.4	66.2	December 10, 2018
17-134	69.3	Bedrock	1.5	67.8	August 23, 2017

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

5.0 CLOSURE

This report was prepared by Mr. Kenton Power, P.Eng. It was reviewed by Mr. Bill Cavers, P.Eng., a Senior Geotechnical Engineer and Associate of Golder. Mr. Fintan Heffernan, P.Eng. a Senior Consultant with Golder and the Designated MTO Foundations Contact for this project, carried out an independent quality control review of this report.

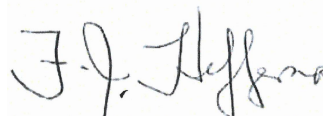
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KCP/WC/FJH/hdw

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PART B

Foundation Design
Catherine Street High Fill Embankment
Highway 417 at Bronson Avenue
Ottawa, Ontario
GWP 4173-15-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the construction of a new high fill embankment as part of the replacement of the Highway 417 Overpass Structure at Bronson Avenue in Ottawa, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation, the General Arrangement (GA) drawing provided by WSP and the available GEOCRETS 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 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 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.

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

6.2 Existing Conditions

The site is located at 458 Catherine Street, at the intersection of Bronson Avenue and Catherine Street. At the time of this report, the A1 Mini Storage facility is located within the proposed footprint of the new embankment. The facility consists of several buildings, paved parking and driving lanes. There is an existing hydro line that runs from the facility to the west along Catherine Street. Several catch basins and storm sewer lines are also present in the paved parking and loading areas to the east and west of the building. It is understood that the overall construction project includes the demolition and removal of the facility and all associated works. Also, the site will serve as a staging area during construction of the replacement overpass structures prior to constructing the new embankment.

The available construction drawings indicate that the existing concrete cantilevered retaining wall supporting the north side of the Highway 417 WBL is between 6 and 8 m in height relative to the elevation of the existing parking lot. The foundation for the retaining wall varies along the length of the wall, from strip footings dowelled into bedrock near the existing bridge location, to spread footings supported on the native sand materials, to piled foundations at the east end of the wall (see reference drawing D5001-1 in Appendix C). The top of footing elevation ranges from approximately 68.4 m near the existing overpass to 67.4 m at the east end of the wall. The retaining wall consists of a cast-in-place stem wall with a coping overlay. It is understood that the proposed plan is to bury the existing retaining wall within the new high fill embankment.

6.3 Proposed Construction

Based on the GA drawing and profiles provided by WSP on July 10, 2019, the new embankment fill will have an approximate maximum height between 6 and 7 m. The existing elevations, proposed grade raises and finished elevations post-construction for the proposed embankment are outlined in Table 4. A dry pond is also being incorporated into the proposed construction. The dry pond is to be constructed between Catherine Street and the toe of the new embankment. The location of the footprint of the proposed dry pond is indicated on Drawing 1.

Table 4: Proposed and Existing Grades

Station	Proposed Top of Embankment Fill (m)	Existing Ground Surface Elevation at Embankment Toe (m)	Approximate Height of Proposed Embankment (m)
227+870	75.2	68.8	6.4
227+900	75.0	68.5	6.5
227+940	74.7	68.5	6.2
227+960	74.5	68.7	5.8

6.4 Seismic Design

6.4.1 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.

6.4.2 Seismic Site Classification

In addition to the borehole investigation at the current site, shear wave velocity profiling was completed at the nearby Highway 417 Overpass Structures at Rochester Street (Sites 3-56/1 and 3-56/2), located approximately 580 m west of the Bronson Avenue. Due to the close proximity and similar nature of the surficial and bedrock geology of the sites, the profiles developed for the Sites 3-56/1 and 3-56/2 are considered relevant to the current site.

The shear wave velocity profiling was carried out in the grassy area next to the eastbound E-N/S off-ramp of the Highway 417 / Rochester Street Interchange, just west of Rochester Street, using the Multichannel Analysis of Surface Waves (MASW) technique. Profiling was carried out on October 18, 2017, by personnel from the Golder's Mississauga and Ottawa offices.

The shear wave velocities measured at the site are presented in a technical memorandum in Appendix E and indicate that the average shear wave velocity in the upper 30 m of the subsurface stratigraphy at the MASW location was 734 m/s.

Based on the results of the MASW testing at the Rochester Street site an average shear wave velocity of 800 m/s was calculated for the soil and bedrock profile at the current site, corresponding to Site Class B in accordance with Table 4.1 of the CHBDC. However, as noted in Table 4.1 of the CHBDC Site Classes A and B, hard rock and rock, are not to be used if there is more than 3 m of softer materials between the rock and the underside of the foundations in this case the underside of the embankment. Therefore, a Site Class C designation is appropriate for the design of the proposed embankment.

6.4.3 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 75.70 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

Parameter	2% Exceedance in 50 years (2,475 return period)
PGA	0.280 g
$T \leq 0.2$ s	0.439 g
$T = 0.5$ s	0.237 g
$T = 1.0$ s	0.118 g
$T = 2.0$ s	0.056 g
$T = 5.0$ s	0.015 g
$T \geq 10.0$ s	0.005 g

6.5 Embankment Design

6.5.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the existing overpass structures and foundation systems 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” for these sites. Accordingly, the appropriate corresponding ULS and SLS consequence factor, ψ of 1.0, and geotechnical resistance factors from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in the following sections.

6.6 Subgrade Preparation

Any surficial topsoil, organic matter, and softened/loosened soils or fill containing deleterious material should be stripped from within the limits of the footprint of the new embankment, including from any existing embankment side slopes. All subgrade soils should be proof rolled prior to fill placement.

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS.PROV 804.

6.6.1 Demolition and Decommissioning

It is anticipated that the demolition and decommissioning component of the work proposed in the current construction plan will include the removal of the existing buildings, including foundation and existing pavements. It is also anticipated that the decommissioning and removal of some/all of the buried services will also be required. It is considered that the existing building foundations may be left in place and backfilled, from a foundations design perspective, provided any environmental requirements are met.

Excavations created through the demolition and decommissioning process should be backfilled with engineered fill consisting of OPSS Select Subgrade (SSM). Backfill shall be placed in accordance with OPSS.PROV 206 and compacted according to OPSS.PROV 501.

It is understood that the retaining wall is to remain in place and will be buried within the new high fill embankment. It is recommended that the upper portion of the wall, to a depth of at least 1.8 m below grade (i.e. frost penetration depth), be removed prior to completing the construction of the embankment fill.

6.6.2 Peat Removal

As noted in Section 4.0, an organic peat deposit upwards of 1.5 m thick was encountered in Boreholes 18-304 and 18-305 at elevations ranging from 65.0 to 65.6 m. There are two options for managing the peat: it may be left in place below the new embankment or removed and replaced with granular fill.

It is considered that the peat may be left in place since the material will be beneath the side slope and should not impact the roadway. The embankment fill will settle more than 25 mm due to compression of the peat, but the peat is fibrous, relatively permeable, and the settlements should occur during construction, prior to paving. Long term settlements may occur due to decomposition of the peat but given that the peat is not located directly beneath the roadway platform, it is anticipated that the decomposition of the peat will not increase the pavement maintenance requirements. As indicated in Section 6.6.3, the results of the stability analyses indicate that an embankment founded on the peat will have adequate factors of safety against global instability.

If removal and replacement of the peat is considered, the peat should be removed in accordance with OPSS.PROV 209 and 902. The potential need for temporary protection (excavation support) systems would need to be considered for removal of the buried organic layer. Further details for temporary protection systems are provided below in Section 7.4

6.6.3 Assessment of Global Stability

The global stability for the proposed grade raise constructed using conventional granular fill with 3H:1V side slopes as outlined in WPS's March 2019 GA, was evaluated using GeoStudio 2018 Slope/W software for limit equilibrium analysis. Input parameters for the analysis are based on the in-situ SPT N values and the results of laboratory testing. It has been assumed that demolition and decommissioning of the existing facility during site preparation activities as outlined above will occur prior to construction of the embankment. It has further been assumed that the peat will be left in place.

The following additional parameters were used in the analysis.

- The embankment is to be constructed with a horizontal backslope.
- A seismic horizontal loading of 0.14g, equal to ½ of the site adjusted PGA value (0.280g) was used for seismic analysis, (see Section 6.4.3)
- Groundwater level of 66.2 m based on the well reading in Borehole 18-303

Table 6: Geotechnical Design Parameters for Stability Analysis

Property	Existing / New Embankment Fill	Native Silty clay	Native Glacial Till
Unit Weight, kN/m ³	20	17.3	22
Effective Friction Angle, ϕ' , °	30	29	33

With appropriate subgrade preparation and proper placement of earth or granular soils, the 6 to 7 m high embankment, with a 3H:1V side slope, will have a factor of safety greater than 1.54 against deep seated slope

instability and a factor of safety greater than 1.1 for seismic. The results do however indicate that some shallow sloughing (with factors of safety less than 1.1) could occur of the embankment side slopes during seismic loading. That sloughing would not however impair the embankment in the short-term and is mainly a maintenance/repair issue. The potential for sloughing could be reduced by providing well vegetated side slopes, as mentioned above in Section 6.6. The results of the slope stability analysis are provided in Figures D1 to D4 in Appendix D.

6.6.4 Settlement

An assessment of settlement that would result from construction of the proposed grade raise using conventional granular fill with 3H:1V side slope was carried out using Rocscience's Settle3D modelling software. The Young's Modulus Input parameters for the analysis are based on the in-situ SPT N values and the results of laboratory testing.

The following design geotechnical parameters have been used in the analysis.

Table 7: Geotechnical Design Parameters for Settlement Analysis

Property	New Embankment Fill	Native Silty clay	Native Sand and Silt	Native Glacial Till
Unit Weight, kN/m ³	20	17.3	22	22
Young's Modulus, MPa	45	10	18	75

Settlement Analysis

- The predicted settlement values reflect both the maximum embankment height after the grade raise and the areal distribution of the fill.
- The analysis assumes that subgrade preparations outlined above have been completed.
- The magnitude of total settlement beneath the proposed new high fill embankment has been estimated to range from 25 to 30 mm outside of the peat area and is considered immediate settlement and is expected to take place during construction of the embankment.
- The magnitude of settlement of the new embankment fill will depend on the type of fill placed and, on the method, and sequence of placement and compaction, but is expected to range from about 0.5 to 1.0 percent of the thickness of the embankment fill.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Excavations

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities.

Excavations to depths of up to 4.0 m below the existing grade through the existing fills to remove the peat material and between 1 and 2 m for the construction of the dry pond may be required. The groundwater level was measured at the site at about Elevation 66.2 m within fill materials (i.e., above the top of the peat layer).

The fill soils at this site would be generally classified as Type 3 soils (compact to loose fill material above groundwater level) in accordance with the OHSA. Accordingly, excavations should be made with side slopes no steeper than 1H:1V. Any fill which extends below the water table or the peat material would be classified as

Type 4 soil and unsupported excavations in these materials should be sloped no steeper than 3H:1V. As indicated in OHSAA, if an excavation contains more than one type of soil, the soil type for the excavation shall be classified as the type with the highest number among the soil types present within the excavation.

7.2 Frost Protection

As per Ontario Provincial Standard Drawing (OPSD) 3090.101 (Foundation Frost Penetration Depths for Southern Ontario), the frost penetration depth at the site is 1.8 m below the existing ground surface.

7.3 Embankment Construction

The embankment construction should be carried out in accordance with OPSS.PROV 206 with embankment fill consisting of SSM material in compliance with OPSS.PROV 1010.

New embankment fill should be placed and compacted OPSS.PROV 501. Benching the existing embankment side slopes should be carried out to “key in” the new fill materials in areas where the new embankment is constructed adjacent to existing embankments in accordance with OPSD 208.010.

7.4 Temporary Protection (Excavation Shoring)

If the required safe side slopes for the open cut excavations cannot be accommodated, excavation shoring will be required to facilitate the excavations.

The design of the shoring system will be entirely the responsibility of the contractor. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems), and the lateral movement should meet Performance Level 2 provided that any existing adjacent utilities / retaining wall footings can tolerate this magnitude of deformation. Highway traffic loading should be included as a surcharge load on the shoring system. The shoring design should also account for loading from construction equipment, in addition to the highway traffic loadings, and which may be higher than the highway traffic loads; the contractor’s shoring designer should confirm all load requirements.

Sheet pile systems are considered feasible at this site though increased difficulty with the installation of protection systems should be anticipated due to the presence of cobbles within the native glacial till.

7.5 Groundwater and Surface Water Control

The groundwater level at the site is typically at or just below base of the fill material. Excavations for peat material, demolition and decommissioning of the existing facility or for the construction of the dry pond will likely involve groundwater and surface water control. It should be possible to handle groundwater and surface water inflows by pumping from well filtered sumps established in the floor of the excavations.

However, the selection and design of temporary unwatering/dewatering system is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the system in accordance with Special Provision (SP) FOUN0003 (Dewatering Structure Excavations) which amends OPSS 902 (Construction Specification for Excavating and Backfilling – Structures).

In accordance with SP FOUN0003, the temporary dewatering system shall be designed and carried out in accordance with OPSS.PROV 517 (Dewatering) with amendments as per SP 517F01 (Dewatering System). Given the groundwater and soil conditions at this site, dewatering is expected to be of low complexity, and it is therefore not a requirement to carry out a preconstruction survey or to require a dewatering design engineer for the dewatering system as per Table A of SP 517F01.

8.0 CLOSURE

This report was prepared by Mr. Kenton Power, P.Eng. It was reviewed by Mr. Bill Cavers, P.Eng., a Senior geotechnical engineer and Associate of Golder. Mr. Fintan Heffernan, P.Eng. a Senior Consultant with Golder and the Designated MTO Foundations Contact for this project, carried out an independent quality control review of this report.

Golder Associates Ltd.



A handwritten signature in blue ink, appearing to read "Kenton Power".

Kenton Power, P.Eng.
Geotechnical Engineer

A handwritten signature in blue ink, appearing to read "Bill Cavers".

William Cavers, P.Eng
Associate - Senior Geotechnical Engineer



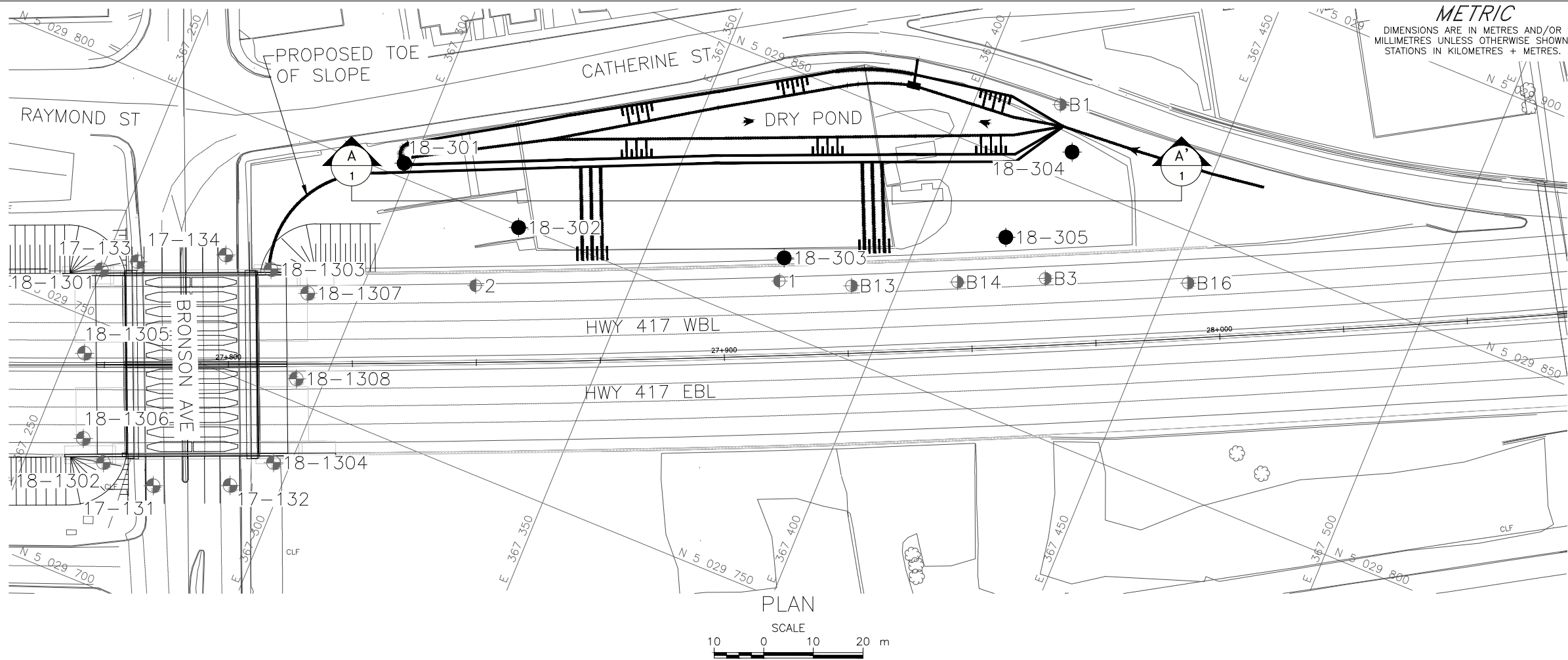
A handwritten signature in blue ink, appearing to read "F. J. Heffernan".

Fintan Heffernan, P.Eng.
MTO Designated Foundations Contact

KCP/WC/FJH/hdw

[https://golderassociates.sharepoint.com/sites/18579g/foundations/6-reports/1300 catherine high fill/final/1655214-1300-r-reva-final catherine high fill-2019-08-28.docx](https://golderassociates.sharepoint.com/sites/18579g/foundations/6-reports/1300%20catherine%20high%20fill/final/1655214-1300-r-reva-final%20catherine%20high%20fill-2019-08-28.docx)

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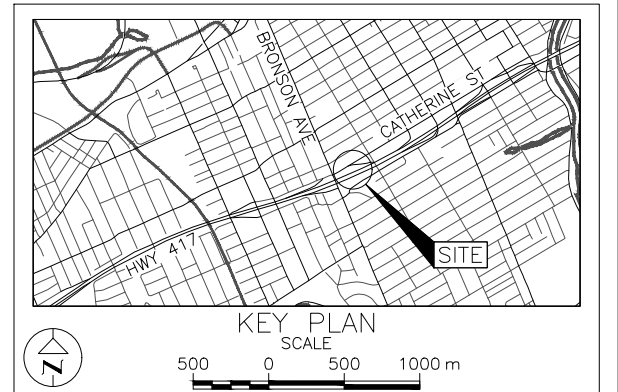


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

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GWP No. 4173-15-00

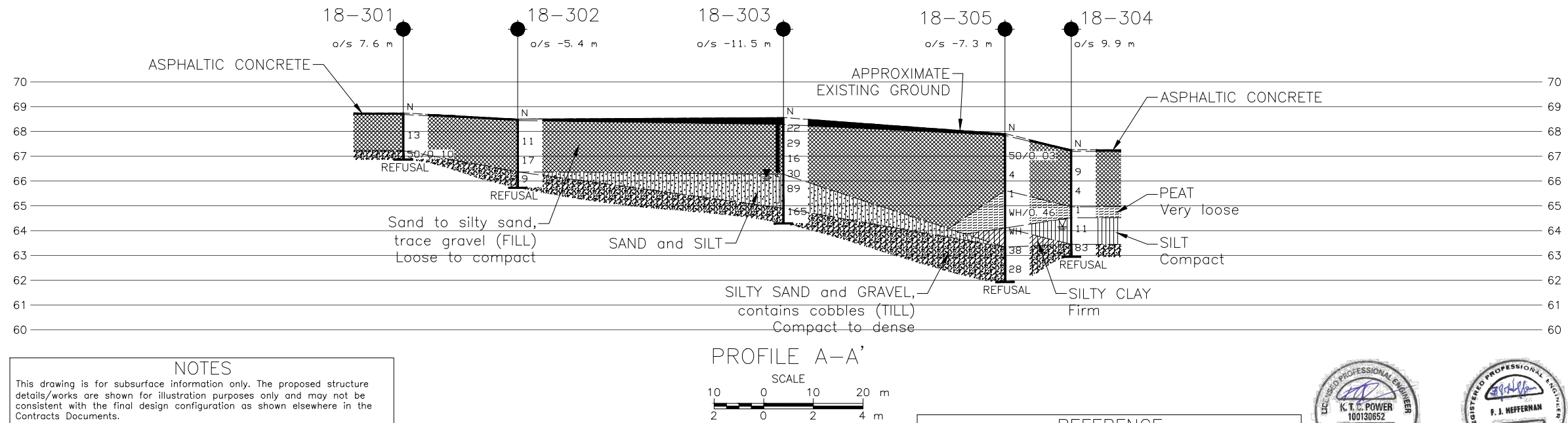
CATHERINE STREET HIGH FILL EMBANKMENT
HIGHWAY 417 AT BRONSON AVENUE
BOREHOLE LOCATIONS AND SOIL STRATA
LAT. 45.405541 LONG. -75.700970

SHEET



- LEGEND**
- Borehole - Current Investigation
 - ⊙ Borehole - Previous Investigation (Sites 3-60/1 and 3-60/2)
 - ⊙ Borehole - Previous Investigation (Geocres No. 31G05-043)
 - ⊙ Borehole - Previous Investigation (Geocres No. 31G05-049)
 - ⊙ Seal
 - ⊙ Piezometer
 - N Standard Penetration Test Value
 - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
 - ≡ WL in piezometer, measured on December 10, 2018
 - ≡ WL upon completion of drilling

BOREHOLE CO-ORDINATES NAD83 (CSRS)/MTM ZONE 9			
No.	ELEVATION	NORTHING	EASTING
18-301	68.7	5029802.9	367298.3
18-302	68.5	5029799.6	367324.6
18-303	68.6	5029814.2	367376.4
18-304	67.3	5029855.8	367422.1
18-305	67.9	5029834.9	367416.3
17-131	70.0	5029723.5	367276.0
17-132	70.1	5029729.5	367290.4
17-133	69.4	5029764.2	367256.1
17-134	69.3	5029772.1	367272.0
18-1301	71.6	5029759.9	367250.0
18-1302	72.3	5029724.1	367265.0
18-1303	69.5	5029772.9	367281.5
18-1304	69.7	5029737.0	367296.7
B1	67.6	5029863.8	367416.3
B3	68.1	5029830.2	367426.8
B13	68.6	5029814.1	367391.1
B14	68.4	5029822.8	367410.7
B16	68.2	5029840.2	367453.8
1	68.6	5029809.5	367377.4
2	68.6	5029785.4	367321.0



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 Canada Group Limited, drawing file no. S3416024-307-001GA.dwg, received MAR. 22, 2019 and Linear Pond Plan & Sections.dwg, received JULY 10, 2019.



NO.	DATE	BY	REVISION
Geocres No. 31G5-309			
HWY. 417		PROJECT NO. 1655214/1300	
SUBM'D. RK	CHKD. RK	DATE: 9/6/2019	SITE:
DRAWN: JM	CHKD. KCP	APPD. FJH	DWG. 1

APPENDIX A

Record of Boreholes, Current Investigation

Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Records of Boreholes 18-301 to 18-305

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
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, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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	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 10	Trace	Trace sand
10 to 20	Some	Some sand
20 to 35	(ey) or (y)	Sandy
over 35	And	Sand and Gravel

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

<u>Description</u>	<u>Bedding Plane Spacing</u>
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

<u>Description</u>	<u>Spacing</u>
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

<u>Term</u>	<u>Size*</u>
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, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

PROJECT 1655214-1300		RECORD OF BOREHOLE No 18-301		SHEET 1 OF 1		METRIC	
G.W.P. 4173-15-00		LOCATION N 5029802.9; E 367298.3 NAD 83 MTM ZONE 9 (LAT. 45.405370; LONG. -75.701620)		ORIGINATED BY KM			
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY ZS			
DATUM Geodetic		DATE November 9, 2018		CHECKED BY KP			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _P W W _L				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)				
68.7	GROUND SURFACE						20	40	60	80	100		25	50	75		
0.0	ASPHALTIC CONCRETE																
68.3	(SP/GP) Sand and gravel (FILL)																
0.4	Brown Moist																
67.9	(SP/SM) Sand to silty sand, trace gravel (FILL)																
0.8	Compact Brown Moist																
	(ml) Sandy silt, trace gravel, contains cobbles (FILL)		1	SS	13												
67.2	Compact Brown to grey Moist																
1.5	(SM/GM) SILTY SAND and GRAVEL, contains cobbles (TILL)		2	SS	50/0.10												
66.8	Very dense Grey Moist																
1.9	END OF BOREHOLE AUGER REFUSAL on inferred bedrock																

GTA-MTO 001 \GOLDER\GDS\GAL\OTTA\ACTIVE\SPATIAL_IMINTO\HWY417\REHAB&WIDENING\02_DATA\GINT1655214.GPJ GAL-GTA.GDT 19-9-6 JM

PROJECT		1655214-1300		RECORD OF BOREHOLE No 18-302		SHEET 1 OF 1		METRIC									
G.W.P.		4173-15-00		LOCATION		N 5029799.6; E 367324.6 NAD 83 MTM ZONE 9 (LAT. 45.405340; LONG. -75.701280)		ORIGINATED BY									
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY									
DATUM		Geodetic		DATE		November 9, 2018		CHECKED BY									
								KP									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
68.5	GROUND SURFACE																
0.0	ASPHALTIC CONCRETE																
68.1	(SP/GP) Sand and gravel (FILL) Brown Moist																
0.5	(SP/SM) Sand to silty sand, contains cobbles (FILL) Compact Brown Moist		1	SS	11												
66.4																	
2.1	(SM/GM) SILTY SAND and GRAVEL, contains cobbles (TILL) Loose Grey Moist		2	SS	17												
65.7																	
2.8	END OF BOREHOLE AUGER REFUSAL on inferred bedrock		3	SS	9												

PROJECT		1655214-1300		RECORD OF BOREHOLE No 18-303		SHEET 1 OF 1		METRIC				
G.W.P.		4173-15-00		LOCATION		N 5029814.2; E 367376.5 NAD 83 MTM ZONE 9 (LAT. 45.405470; LONG. -75.700620)		ORIGINATED BY				
DIST		Eastern HWY 417		BOREHOLE TYPE		Portable Drill/Rotary Drill, BW Casing		COMPILED BY				
DATUM		Geodetic		DATE		November 27, 2018		CHECKED BY				
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W _p W W _L		
68.6	GROUND SURFACE											
0.0	ASPHALTIC CONCRETE											
68.3												
0.3	(SP) Sand, some gravel (FILL) Grey Moist		1	SS	22							
67.8												
0.8	(SM) Silty sand, trace gravel, contains organic matter (FILL) Dark brown Moist to wet		2	SS	29							
			3	SS	16							
66.3												
2.3	(SM/ML) SAND and SILT Brown to grey Moist to wet		4	SS	30							
			5	SS	89							
64.9												
3.7	(SM/GM) SILTY SAND and GRAVEL, contains cobbles Grey Moist		6	SS	165							
64.3												
4.3	END OF BOREHOLE											
NOTES: 1. Manual third weight hammer used for all split spoon samples. "N" values are not representative of ASTM D1586 SPT N values and should be interpreted in consideration of their reduced energy. 2. Water level in well screen at a depth of 2.4 m below ground surface (Elev. 66.2 m), measured on December 10, 2018.												

GTA-MTO 001 \GOLDER\GDS\GALOTTAWA\ACTIVE\SPATIAL_IMMITO\HWY417\REHAB&WIDENING\02_DATA\GINT\1655214.GPJ GAL-GTA.GDT 19-9-6 JM



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

PROJECT 1655214-1300		RECORD OF BOREHOLE No 18-305		SHEET 1 OF 1		METRIC	
G.W.P. 4173-15-00		LOCATION N 5029834.9; E 367416.3 NAD 83 MTM ZONE 9 (LAT. 45.405650; LONG. -75.700110)		ORIGINATED BY KM			
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY ZS			
DATUM Geodetic		DATE November 9, 2018		CHECKED BY KP			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _P	W	W _L		GR	SA	SI	CL	
								20	40	60	80	100	WATER CONTENT (%)								
						○ UNCONFINED + FIELD VANE															
						● QUICK TRIAXIAL × REMOULDED															
67.9	GROUND SURFACE																				
0.0	ASPHALTIC CONCRETE																				
67.6	(SP/GP) Sand and gravel (FILL)																				
0.3	Brown Moist (SP/SW) Gravelly sand, contains cobbles (FILL) Very dense to loose Brown Moist		1	SS	80/0.18																
			2	SS	4																
65.6																					
2.3	(PT) PEAT, fibrous, contains wood fibers Very loose Dark brown Moist		3	SS	1													485.4			
			4	SS	WH													365.8			
64.1																					
3.8	(CI) SILTY CLAY, contains sand seams Firm Grey Wet		5	SS	WH																
63.3																					
4.6	(SM/GM) SILTY SAND and GRAVEL, contains cobbles (TILL) Compact to dense Grey Wet		6	SS	38																
			7	SS	28																
61.9																					
6.0	END OF BOREHOLE AUGER REFUSAL on inferred bedrock																				

APPENDIX B

Laboratory Test Results

Figure B1 – Grain Size Distribution Test Results – Gravelly Silty Sand (Fill)

Figure B2 – Grain Size Distribution Test Results – Sand and Silt to Silt

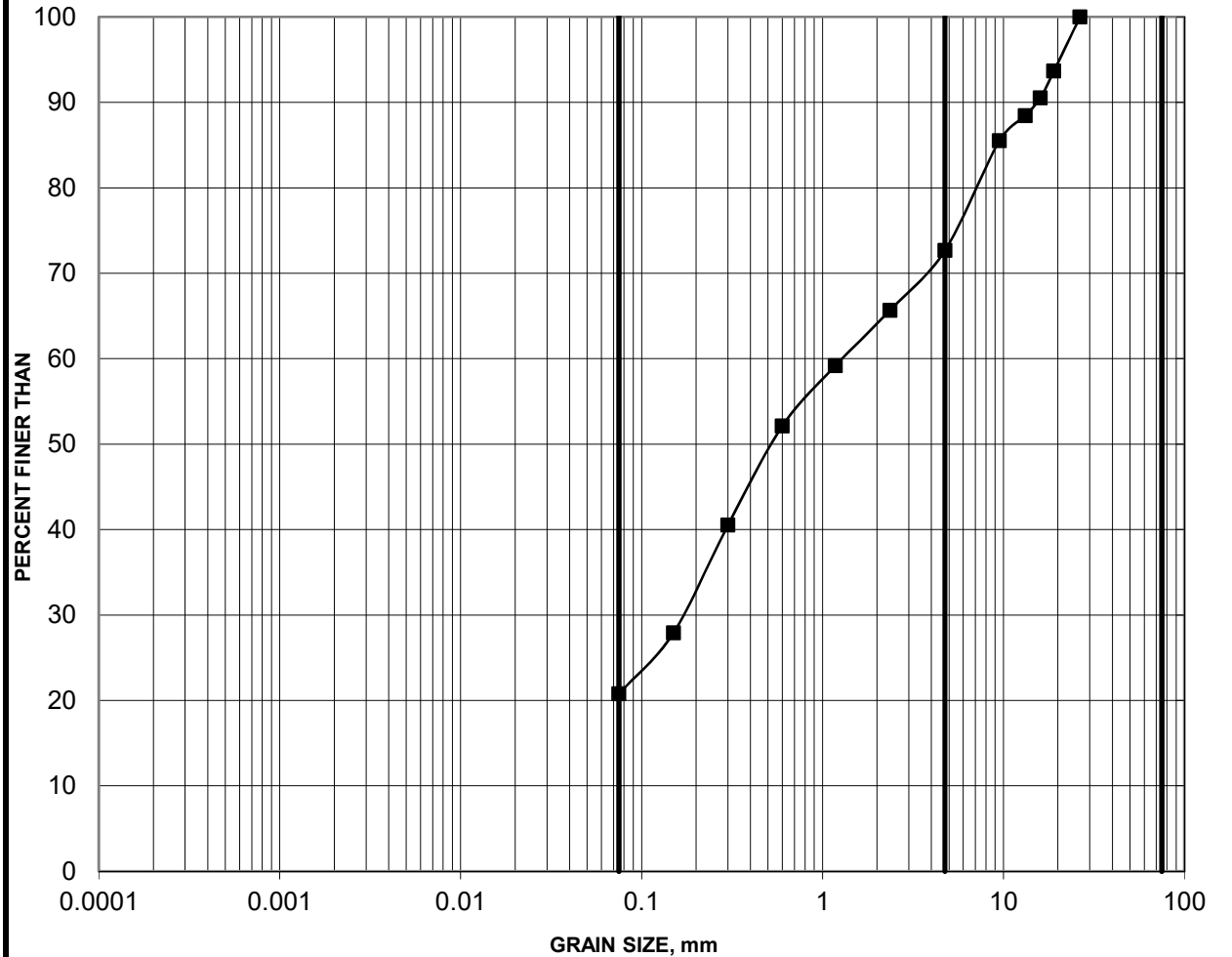
Figure B3 – Grain Size Distribution Test Results – Silty Sand and Gravel (Till)

Figure B4 – Plasticity Chart – Silty Clay

GRAIN SIZE DISTRIBUTION

FIGURE B1

GRAVELLY SILTY SAND (FILL)

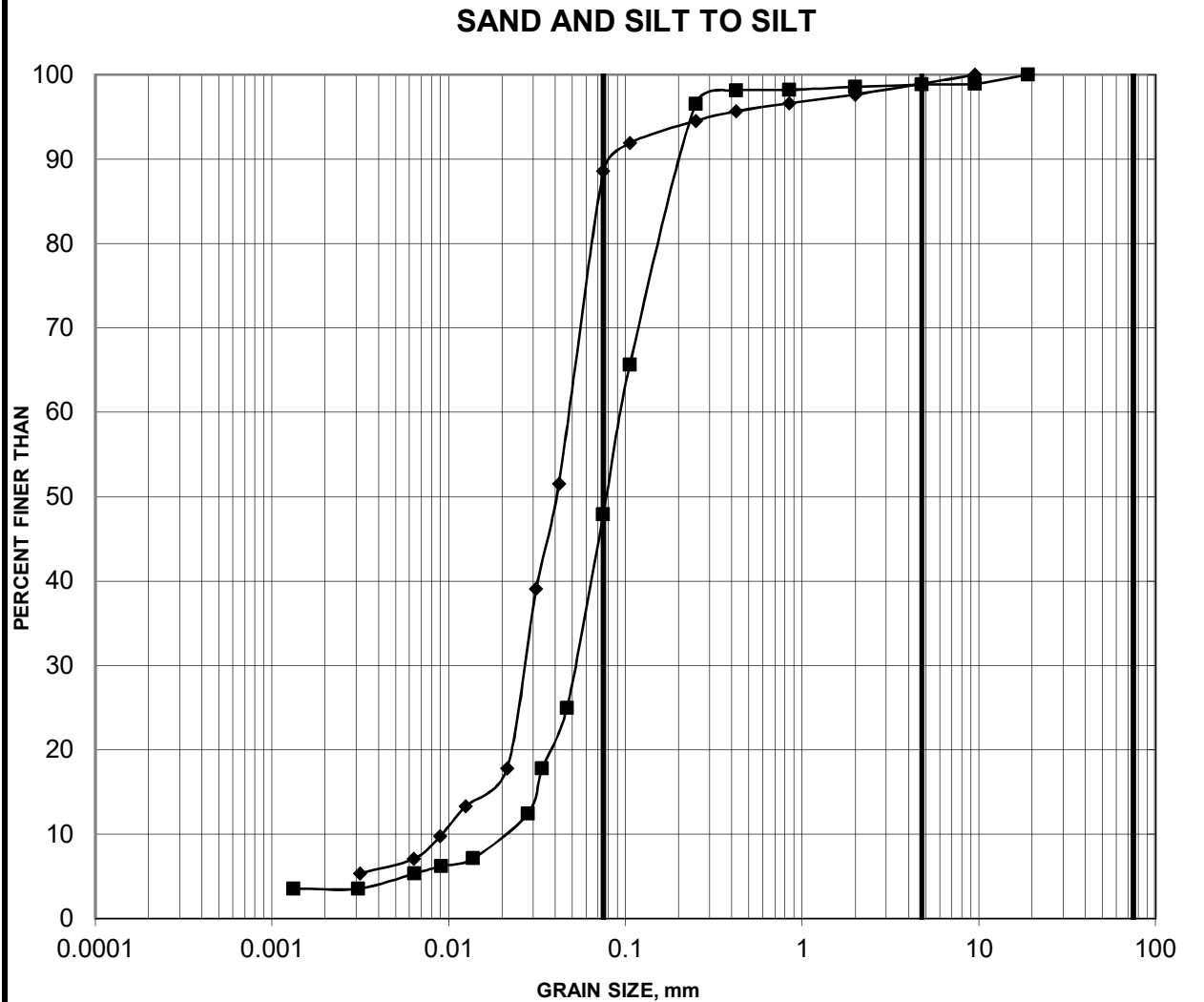


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
—■— 18-304	1	0.76-1.37

GRAIN SIZE DISTRIBUTION

FIGURE B2



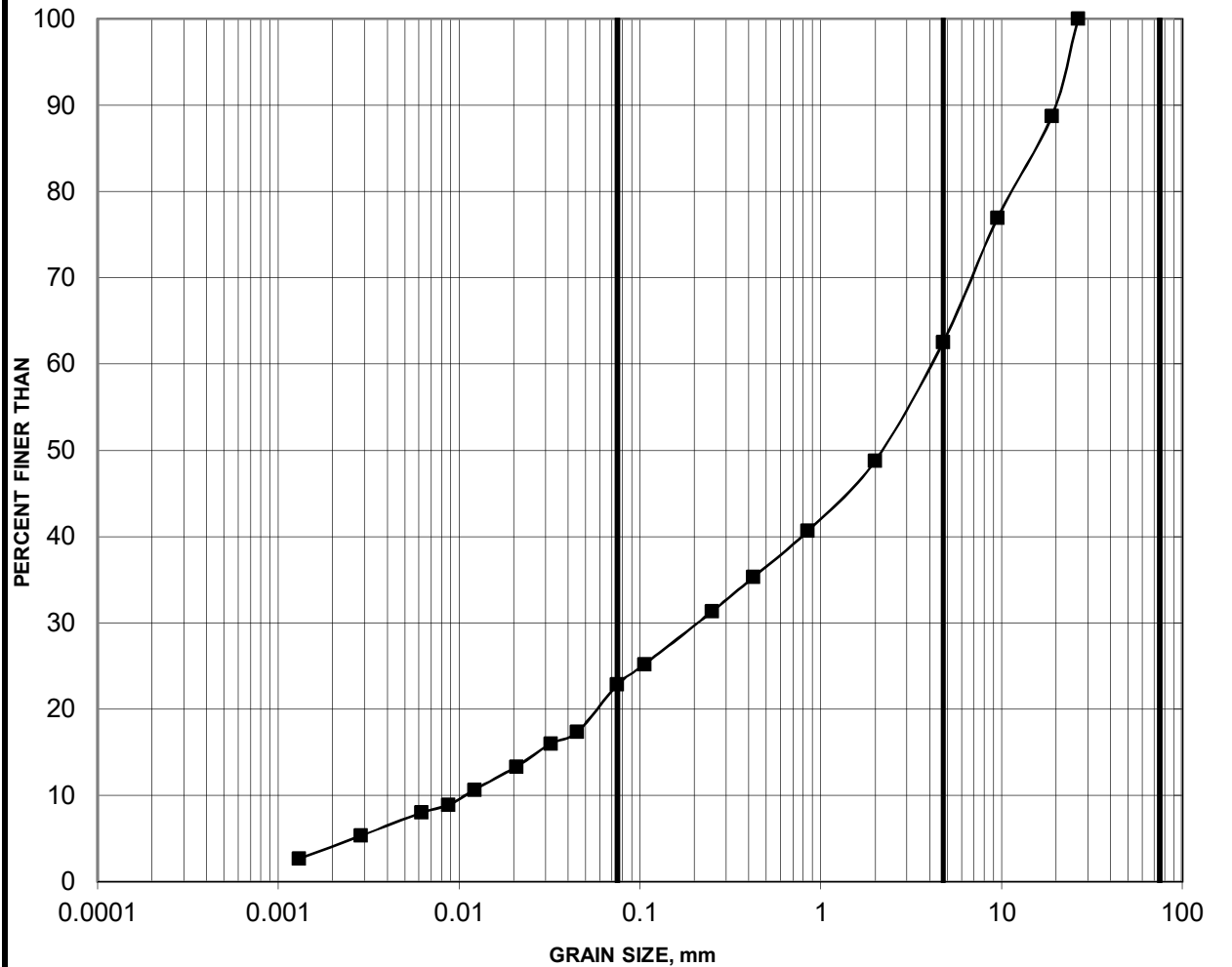
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
18-303	5	2.74-3.35
18-304	4	3.05-3.69

GRAIN SIZE DISTRIBUTION

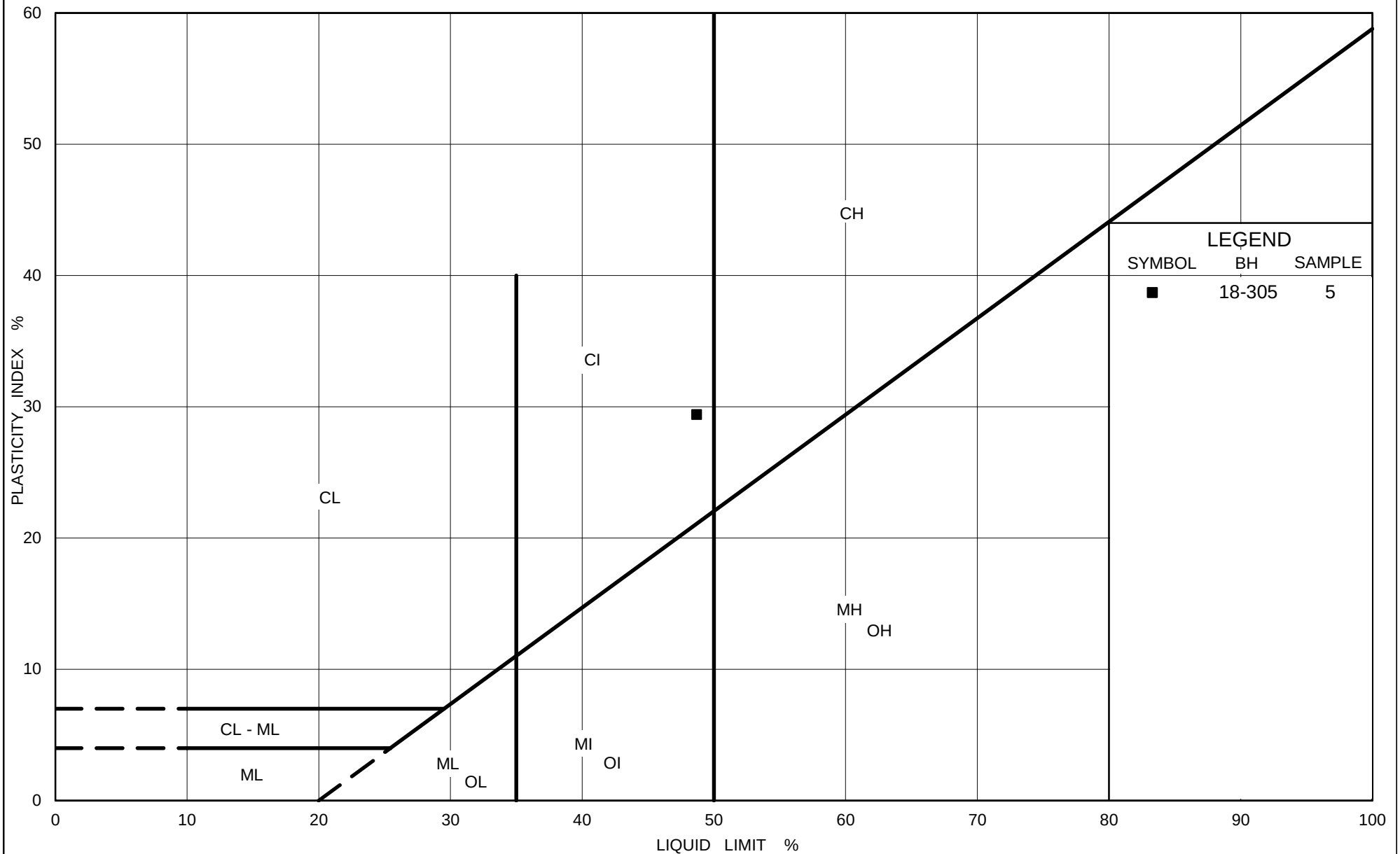
FIGURE B3

SILTY SAND AND GRAVEL (TILL)



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

Borehole	Sample	Depth (m)
—■— 18-304	5	3.81-4.30



Ministry of Transportation

Ontario

PLASTICITY CHART (CI) SILTY CLAY

FIG No. B4

Project No. 1655214/1300

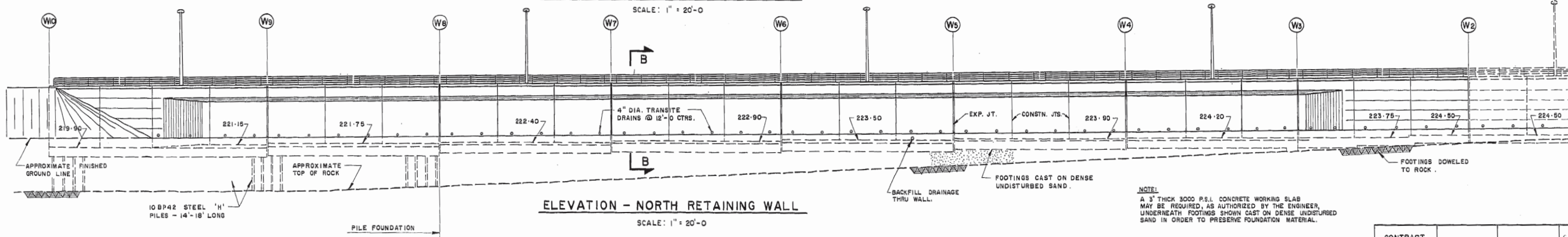
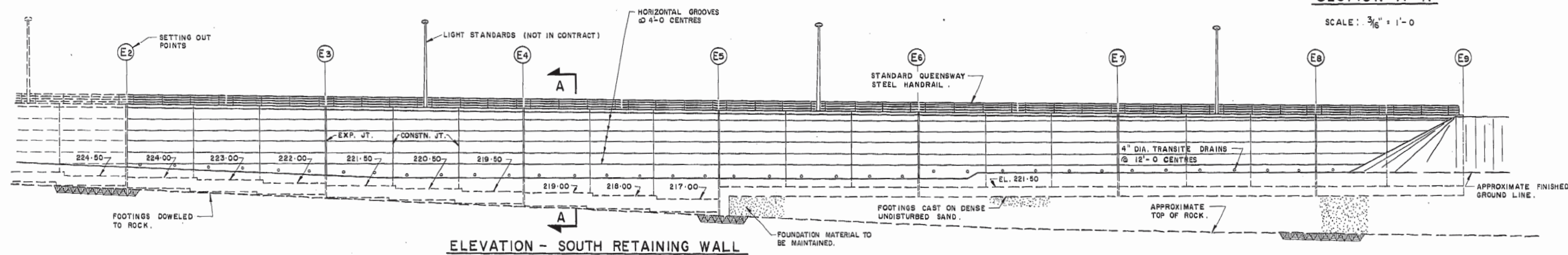
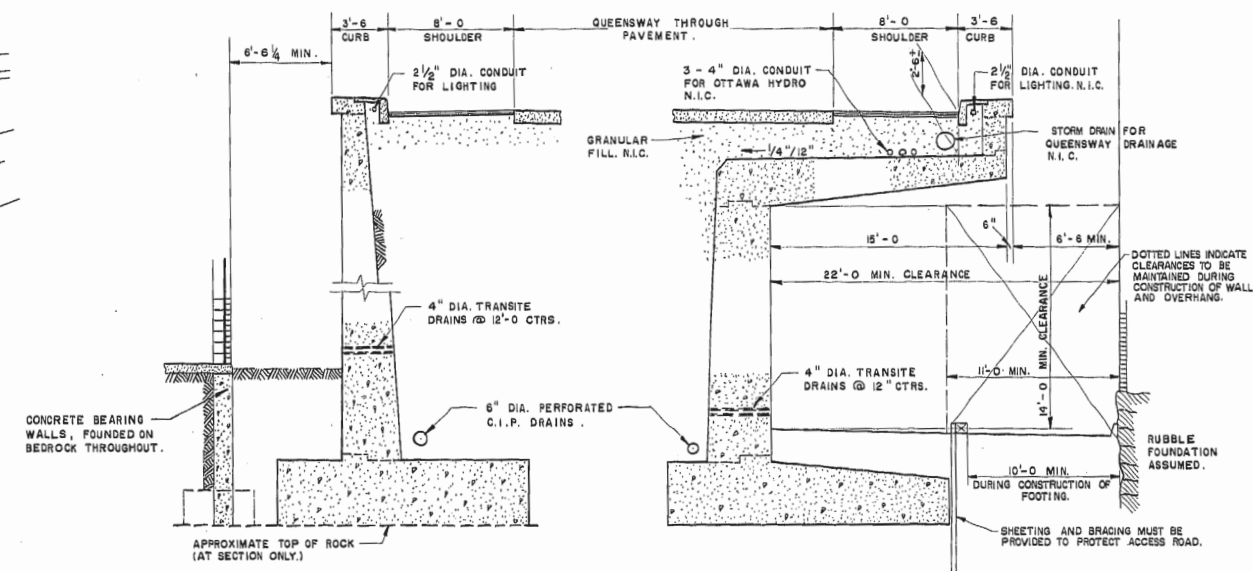
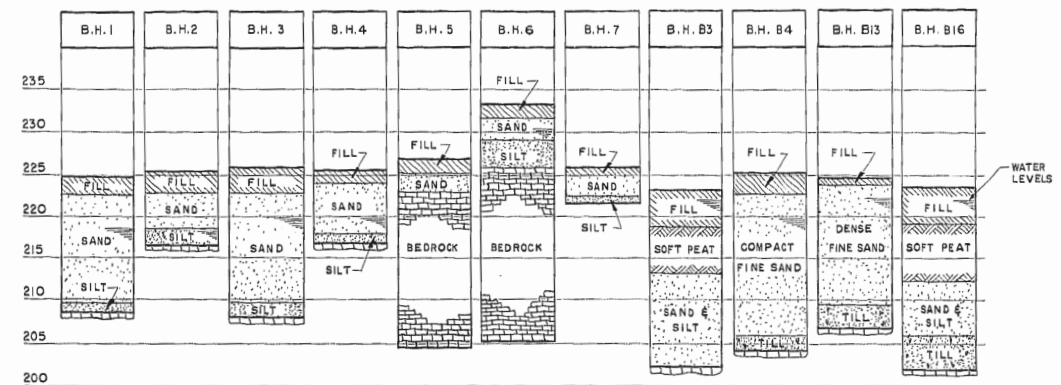
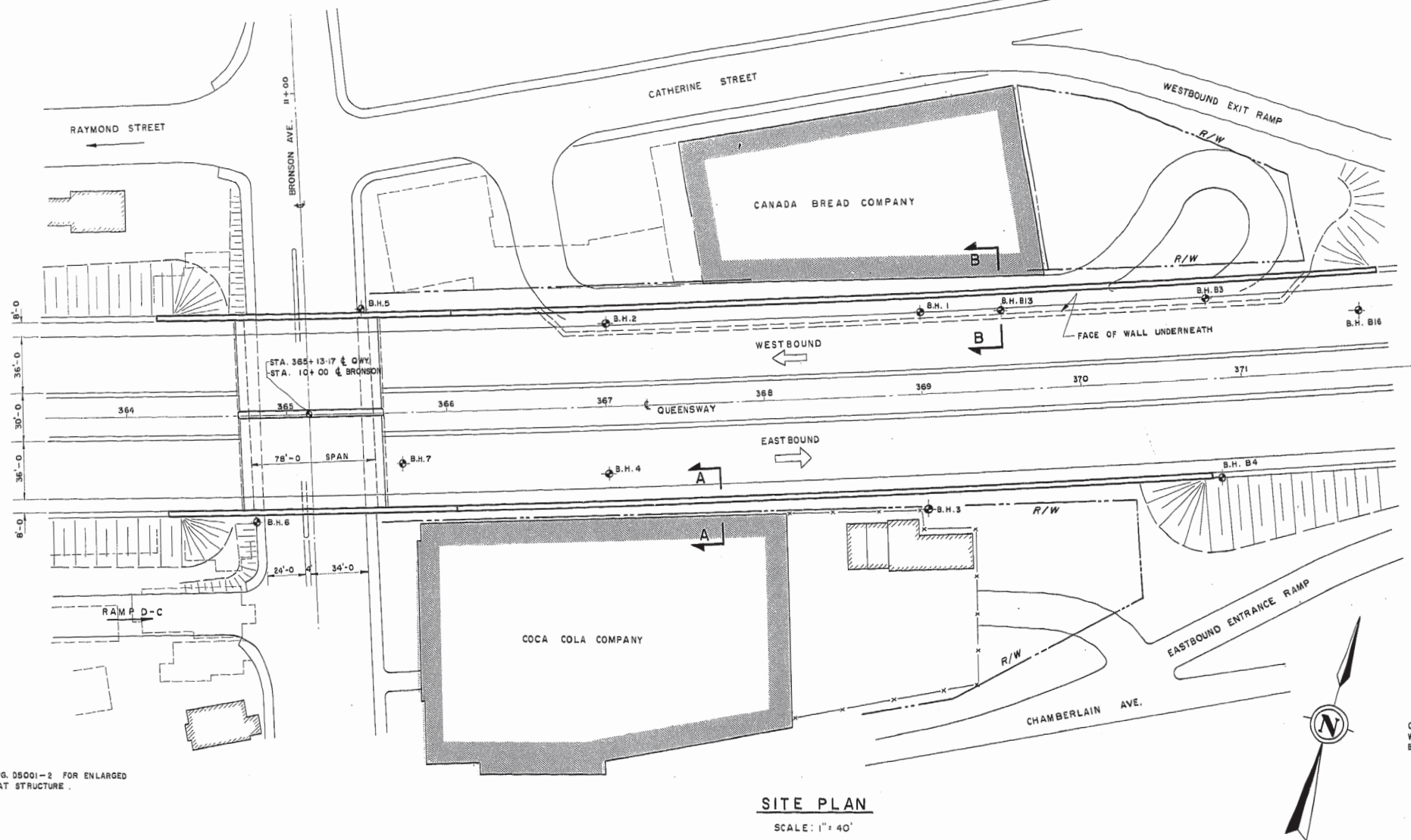
Compiled By : MI Checked By : CW

APPENDIX C

Drawing D5001-1 – Bridge N° 18 at Bronson Avenue, General Arrangement
**Previous Investigation, Highway 417 Overpass Structures at
Bronson Avenue (2018)**

Records of Boreholes 17-134, 18-1303 and 18-1307
**Previous Investigations, GEOCREs Nos. 31G05-043 and
31G05-049 (1961 and 1962)**

Records of Boreholes 1, 2, B1, B3, B13, B14 and B16
Laboratory Test Results



REVISED AS-CONSTRUCTED		No. _____		Revisions		By _____		Date _____	
DEPARTMENT OF HIGHWAYS OF ONTARIO									
OTTAWA QUEENSWAY LIMITED-ACCESS HIGHWAY									
OTTAWA CANADA									
BRIDGE NO 18 AT BRONSON AVE.									
GENERAL ARRANGEMENT									
DE LEUW CATHAR & CO. OF CANADA LIMITED					DEPT. OF HIGHWAYS OF ONTARIO				
Consulting Engineers									
Designed by: G.S.S.					Date: FEB. 1962				
Drawn by: P.T.-D.K.					Scale: AS SHOWN				
Checked by: G.S.S.					Sheet 1 of 26				
CONTRACT NUMBERS					WORKS PROJECT No. 944-59 DISTRICT No. 9				
D5001-1					D5001-1				

PROJECT		1655214-1130		RECORD OF BOREHOLE No 17-134		SHEET 1 OF 2		METRIC								
G.W.P.		4173-15-00		LOCATION		N 5029772.1; E 367272.0 NAD 83 MTM ZONE 9 (LAT. 45.405108; LONG. -75.701958)		ORIGINATED BY PAH								
DIST		Eastern HWY 417		BOREHOLE TYPE		Portable Drill/NQ Core		COMPILED BY ZS								
DATUM		Geodetic		DATE		August 1 and 3, 2017		CHECKED BY KCP								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
69.3	GROUND SURFACE						20	40	60	80	100					
0.0	ASPHALTIC CONCRETE															
69.0	PORTLAND CEMENT CONCRETE															
68.8	(SP) Gravelly sand (FILL) Grey		1	SS	51											
0.5	(SP) Sand, trace gravel (FILL) Very dense to compact Brown Moist		2	SS	15											
67.6	(GM) Sandy gravel, some silt (FILL) Grey brown		3	SS	150/0.15											
1.9	Limestone (BEDROCK)		C1	RC	REC 100%											
	Bedrock cored from depths 1.9 m to 5.8 m		C2	RC	REC 100%											
	For bedrock coring details refer to Record of Drillhole 17-134		C3	RC	REC 100%											
63.5	END OF BOREHOLE															
5.8	NOTES: 1. Water level in well screen at a depth of 1.5 m below ground surface (Elev. 67.8 m), measured on August 23, 2017.															

PROJECT: 1655214-1130

RECORD OF DRILLHOLE: 17-134

SHEET 2 OF 2

LOCATION: N 5029772.1 ;E 367272.0

DRILLING DATE: August 1 and 3, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: CCC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
						FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS °	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec	WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr		Ja	W1	W2	W3	W4		W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
2	Rotary Drill NQ Core	Continued from Record of Borehole 17-134		67.45																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												

DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: PAH

CHECKED: KCP

GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1655214.GPJ GAL-MISS.GDT 6/6/19 JM

PROJECT		1655214-1130		RECORD OF BOREHOLE No 18-1303		SHEET 1 OF 2		METRIC									
G.W.P.		4173-15-00		LOCATION		N 5029772.9; E 367281.5 NAD 83 MTM ZONE 9 (LAT. 45.405100; LONG. -75.701830)		ORIGINATED BY									
DIST		Eastern HWY 417		BOREHOLE TYPE		Portable Rotary Drill, BW Casing		COMPILED BY									
DATUM		Geodetic		DATE		November 18 and 19, 2018		CHECKED BY									
								KCP									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
69.5	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALTIC CONCRETE																
69.3	(SW) Gravelly sand, angular (FILL)		1	SS	25												
0.3	Grey																
	(SW) Sand, fine to coarse, trace gravel (FILL)		2	SS	167/0.28												
68.4	Loose to compact																
	Brown																
	Moist																
1.1	CONCRETE (FOOTING)		1	RC	-												
			2A	RC	-												
67.0	Limestone (BEDROCK)		2B	RC	-												
2.5	Bedrock cored from depths 2.5 m to 4.5 m																
	For bedrock coring details refer to Record of Drillhole 18-1303		3	RC	REC 100%												RQD = 93%
			4	RC	REC 100%												RQD = 100%
65.0	END OF BOREHOLE																
4.5	NOTES:																
	1. Manual third weight hammer used for all split spoon samples. "N" values are not representative of ASTM D1586 SPT N values and should be interpreted in consideration of their reduced energy.																
	2. Water level in open borehole at 1.7 m depth below ground surface (Elev. 67.8 m), upon completion of drilling.																

PROJECT: 1655214-1130

RECORD OF DRILLHOLE: 18-1303

SHEET 2 OF 2

LOCATION: N 5029772.9 ;E 367281.5

DRILLING DATE: November 18 and 19, 2018

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable

DRILLING CONTRACTOR: CCC Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES	
						RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS °	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec		WEATH- ERING INDEX						
						TOTAL CORE % 000000	SOLID CORE % 000000				TYPE AND SURFACE DESCRIPTION	Jr	Ja	10 ⁻⁹ 10 ⁻⁸ 10 ⁻⁷ 10 ⁻⁶	10 ⁻⁹ 10 ⁻⁸ 10 ⁻⁷ 10 ⁻⁶	W1 W2 W3 W4 W5 W6					
		Continued from Record of Borehole 18-1303		68.41																	
		CONCRETE (FOOTING)		1.10	1																
2																					
					2A																
				66.97																	
3	Rotary Drill BW Casing	Limestone (BEDROCK) Fresh Thinly to medium bedded Greyto dark grey Fine to medium grey Non-porous Slightly to moderately fossiliferous Medium strong		2.54	2B																
					3																
4					4																
		END OF DRILLHOLE		65.01 4.50																	
5																					
6																					
7																					
8																					
9																					
10																					
11																					

DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: RI

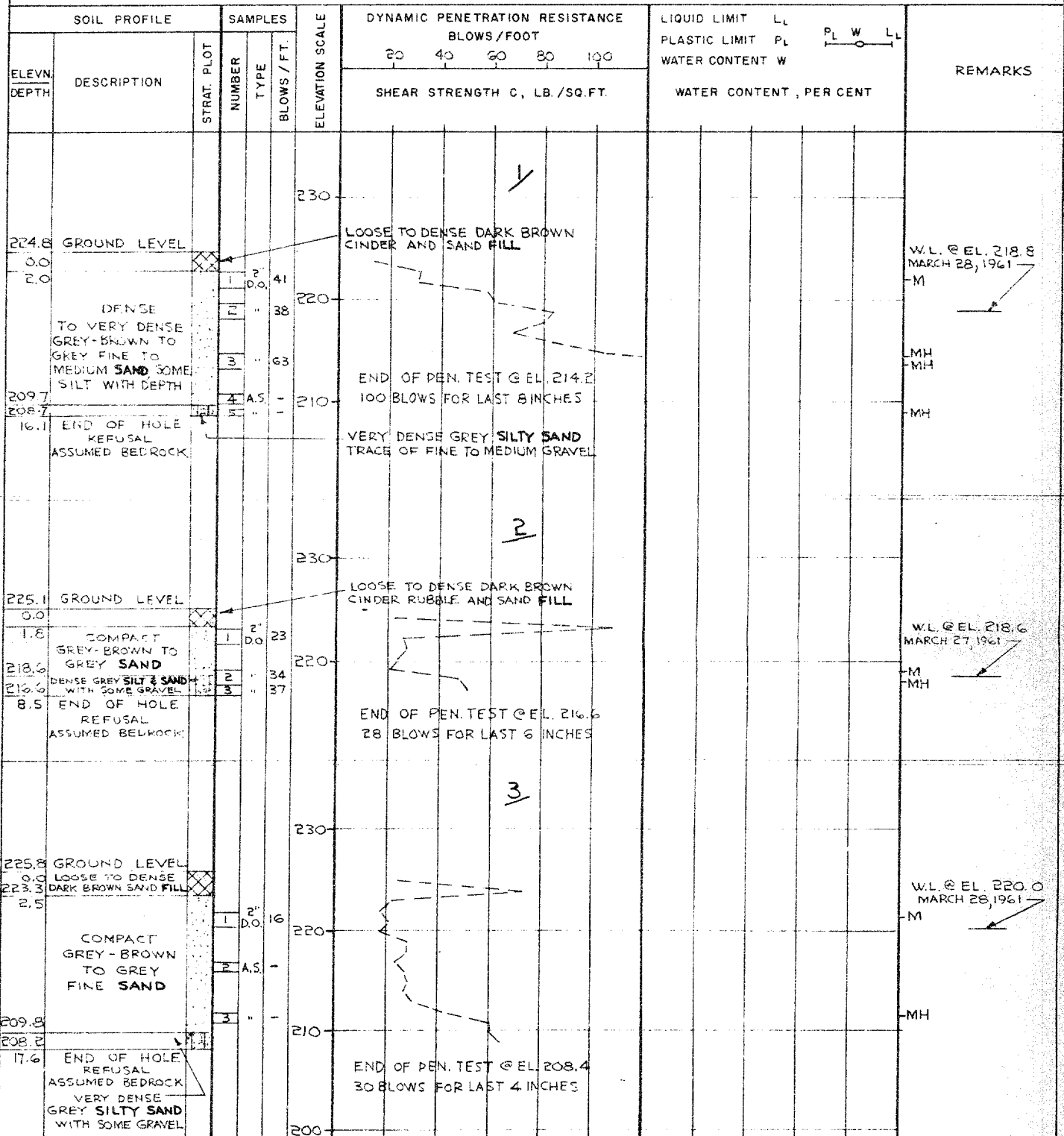
CHECKED: KCP

GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1655214.GPJ GAL-MISS.GDT 6/6/19 JM

PROJECT		1655214-1130		RECORD OF BOREHOLE No 18-1307		SHEET 1 OF 1		METRIC									
G.W.P.		4173-15-00		LOCATION		N 5029771.2; E 367290.2 NAD 83 MTM ZONE 9 (LAT. 45.405090; LONG. -75.701720)		ORIGINATED BY									
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY									
DATUM		Geodetic		DATE		October 15-16, 2018		CHECKED BY									
								KCP									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
75.5	GROUND SURFACE																
0.0	ASPHALTIC CONCRETE																
0.1	CONCRETE																
74.9																	
0.6	(SM) Sand, some silt, some gravel (FILL) Very dense Grey Moist		1	GS	-												
			2	SS	59												
			3	SS	52												
72.9			4	SS	85												
2.6	(SP-SM) Sand, some silt (FILL) Very dense to compact Brown																
			5	SS	59												
			6	SS	42												
			7	SS	61												
			8	SS	36												
			9	SS	15												
68.8																	
6.7	(SM) Silty sand, contains organic matter and ash (FILL) Compact Dark brown to black Moist																
68.2																	
7.3	(SM) Gravelly Silty SAND (TILL) Compact Grey Moist to wet		10	SS	13												
67.8																	
7.7	END OF BOREHOLE SAMPLER REFUSAL																
	NOTES: 1. Borehole dry upon completion of drilling.																

RECORD OF BOREHOLES 1, 2, 3

LOCATION SEE FIGURE 1 BORING DATE MARCH 27, 28, 1961 DATUM GEODETIC
 BOREHOLE TYPE POWER AUGER & WASH BORINGS BOREHOLE DIAMETER 4" & BX CASING
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
 CHECKED J.B.

RECORD OF BOREHOLE B 1

LOCATION SEE FIGURE 1

BORING DATE NOV. 7, 1961

DATUM GEODETIC

BOREHOLE TYPE POWER AUGER BORING

BOREHOLE DIAMETER 4.5"

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT					LIQUID LIMIT L _L PLASTIC LIMIT P _L WATER CONTENT W			LAB. TESTING	STANDPIPE INSTALLATION
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FT.		SHEAR STRENGTH C, LB./SQ.FT.					WATER CONTENT, PER CENT				
							20	40	60	80	100					
221.5	GROUND LEVEL															
0.0	LOOSE DARK BROWN SILTY SAND FILL		1	DP	8											
219.3			2	"	4											
2.2	SOFT DARK BROWN PEAT															
215.2			3	"	PM											
6.3	LOOSE GREY LAYERED ORGANIC FINE SAND AND SILT															
213.5			4	"	19											
8.0	COMPACT GREY SAND AND GRAVEL WITH SOME SILT															
			5	"	28											
207.7																
13.8	END OF HOLE REFUSAL BOULDER OR BEDROCK															

END OF PEN. TEST @ EL. 207.7
136 BLOWS FOR LAST 9"

VERTICAL SCALE

1 INCH TO 5 FEET

GOLDER & ASSOCIATES

DRAWN A.T. & J.A.

CHECKED *rrr*

RECORD OF BOREHOLE B 3

LOCATION SEE FIGURE 1 BORING DATE NOV. 8, 1961 DATUM GEODETIC
BOREHOLE TYPE POWER AUGER BORING BOREHOLE DIAMETER 4.5"
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT					LIQUID LIMIT L_L PLASTIC LIMIT P_L WATER CONTENT W		LAB. TESTING	STANDPIPE INSTALLATION	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER		TYPE	BLOWS / FT.	20	40	60	80	100			WATER CONTENT, PER CENT
223.2 0.0	GROUND LEVEL		1	DO	12									
218.8 4.4	LOOSE TO COMPACT DARK BROWN CINDER AND SAND FILL		2	"	4	220								
	SOFT DARK BROWN PEAT		3	"	PM									
213.7 9.5	LOOSE GREY		4	"	PM	215								
212.3 10.9	ORGANIC SANDY SILT		5	"	PM									
	LOOSE TO COMPACT GREY SILTY VERY FINE SAND		6	"	21	210								
			7	"	21									
202.7 20.5	END OF HOLE REFUSAL PROBABLY BEDROCK		8	AS	-	205								
					200									

VERTICAL SCALE
1 INCH TO 5 FEET

GOLDER & ASSOCIATES

DRAWN AT: J.A.
CHECKED: J.A.

RECORD OF BOREHOLE 813

LOCATION SEE FIGURE 1

BORING DATE NOV. 30 & DEC. 1, 1961

DATUM GEODETIC

BOREHOLE TYPE POWER AUGER BORING

BOREHOLE DIAMETER 4.5"

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT					LIQUID LIMIT L_L PLASTIC LIMIT P_L WATER CONTENT W			LAB. TESTING	STANDPIPE INSTALLATION
ELEVATION DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FT.		SHEAR STRENGTH C , LB. /SQ.FT.					WATER CONTENT, PER CENT				
							20	40	60	80	100					
224.8	GROUND LEVEL					225										STANDPIPE GROUND LEVEL
0.0	COMPACT BLACK CINDER FILL		1	2"	42											
1.0			2	"	60											BENTONITE PLUG
	DENSE TO VERY DENSE GREY - BROWN TO GREY FINE SAND TRACE OF SILT		3	"	55	220										
			4	"	63											
			5	"	93	215										SAND FILL
			6	"	69											
219.7			7	"	36	210										
15.1	DENSE TO VERY DENSE GREY SILTY SAND WITH GRAVEL															
206.9	END OF HOLE REFUSAL PROBABLY BEDROCK					205										W.L. IN STANDPIPE @ EL. 221.3 - DEC. 8, 1961

VERTICAL SCALE
1 INCH TO 5 FEET

GOLDER & ASSOCIATES

DRAWN A.T. & J.A.
CHECKED *JLG*

RECORD OF BOREHOLE B14

LOCATION SEE FIGURE 1

BORING DATE DEC 1 1961

DATUM GEODETIC

BOREHOLE TYPE

POWER AUGER BORING

BOREHOLE DIAMETER 4.5"

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT L_L				LAB. TESTING	STANDPIPE INSTALLATION
ELEV./DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FT.		BLOWS / FOOT					PLASTIC LIMIT P_L					
							20	40	60	80	100	WATER CONTENT W					
							SHEAR STRENGTH C , LB. / SQ. FT.					WATER CONTENT, PER CENT					
221.4	GROUND LEVEL					225											
221.4	LOOSE TO COMPACT DARK BROWN CINDER AND SAND FILL		1	DO	17		END OF PEN. TEST @ EL. 222.9										
220.5	SOFT DARK BROWN PEAT		2	"	6		REFUSAL ON TIMBER										
219.0	LOOSE GREY ORGANIC SANDY SILT		3	"	38	220											
219.0			4	"	34												
219.0			5	"	43	215											
219.0	DENSE GREY FINE SAND TO SILTY FINE SAND		6	"	24	210											
207.4																	
205.7	DENSE GREY SILTY SAND WITH GRAVEL					205											
18.7	END OF HOLE																

STANDPIPE
GROUND LEVEL
BENTONITE
PLUG

MH

SAND FILL

W.L. IN STANDPIPE
@ EL. 221.2 - DEC. 16,
1961

 VERTICAL SCALE
 1 INCH TO 5 FEET

GOLDER & ASSOCIATES

 DRAWN A.T. & J.A.
 CHECKED *JA*

RECORD OF BOREHOLE B16

LOCATION SEE FIGURE 1

BORING DATE DEC 4-5, 1961

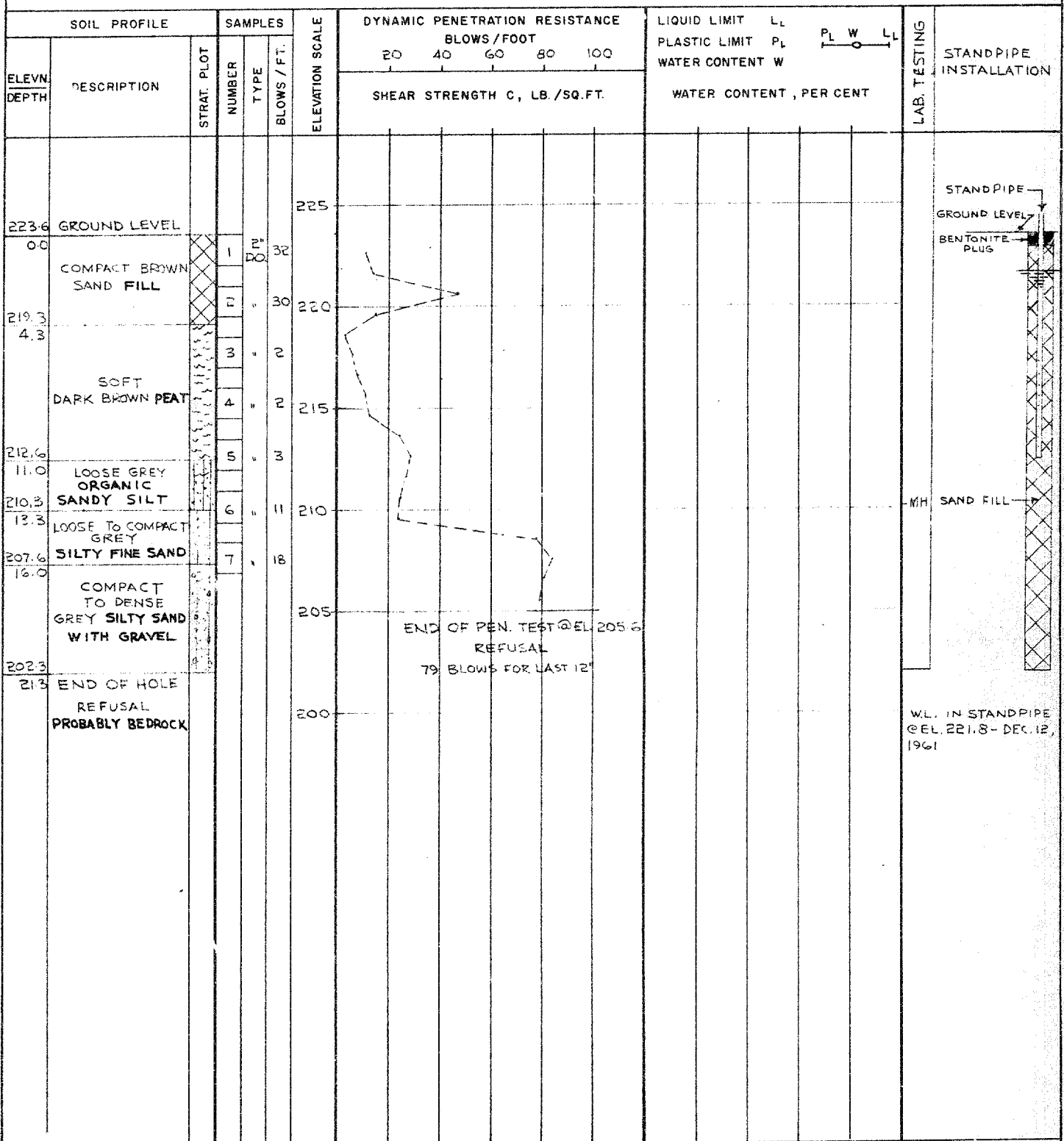
DATUM GEODETIC

BOREHOLE TYPE POWER AUGER BORING

BOREHOLE DIAMETER 4.5"

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

VERTICAL SCALE
1 INCH TO 5 FEET

GOLDER & ASSOCIATES

DRAWN AT E.J.A.
CHECKED *jes*

APPENDIX D

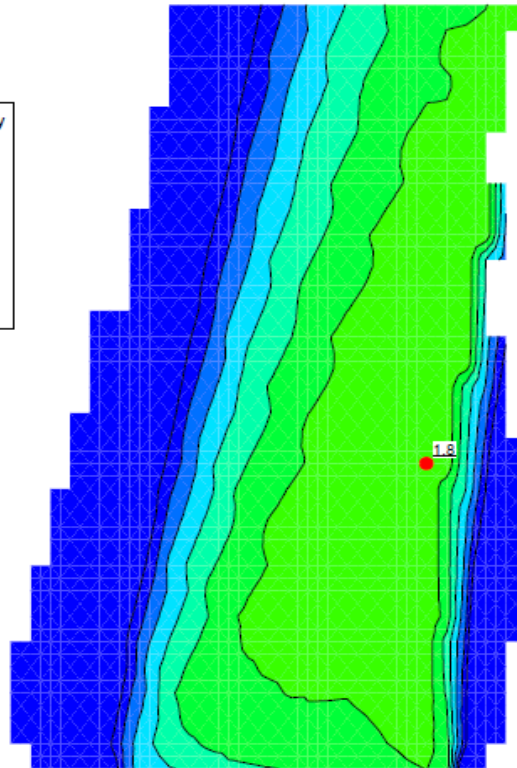
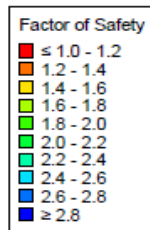
Results of Slope Stability Analysis

Figure D1 – Static Condition – Section 1

Figure D2 – Seismic Condition – Section 1

Figure D3 – Static Condition – Section 2

Figure D4 – Seismic Condition – Section 2

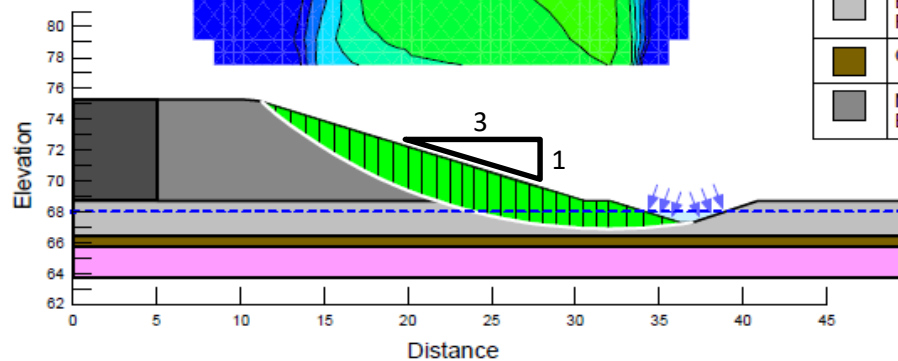


File Name: 2019_07_17_1655214-1300_CathStreetHF_BH18-302_RK.gsz
 Title: 358 Catherine Street HF
 Name: 1.1 Static Drained Global
 Method: Morgenstem-Price
 Direction of movement: Left to Right
 Horz Seismic Load: 0

Geometry Obtained From
 Cross-Section for Stn 227+870 Provided By
 WSP on July 10, 2019

Minimum Slip Surface Depth of 1.5 m

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Existing Embankment Fill	Mohr-Coulomb	20	0	30
	Existing Grade Fill	Mohr-Coulomb	20	0	30
	Glacial Till	Mohr-Coulomb	22	0	33
	New Embankment Fill	Mohr-Coulomb	20	0	30



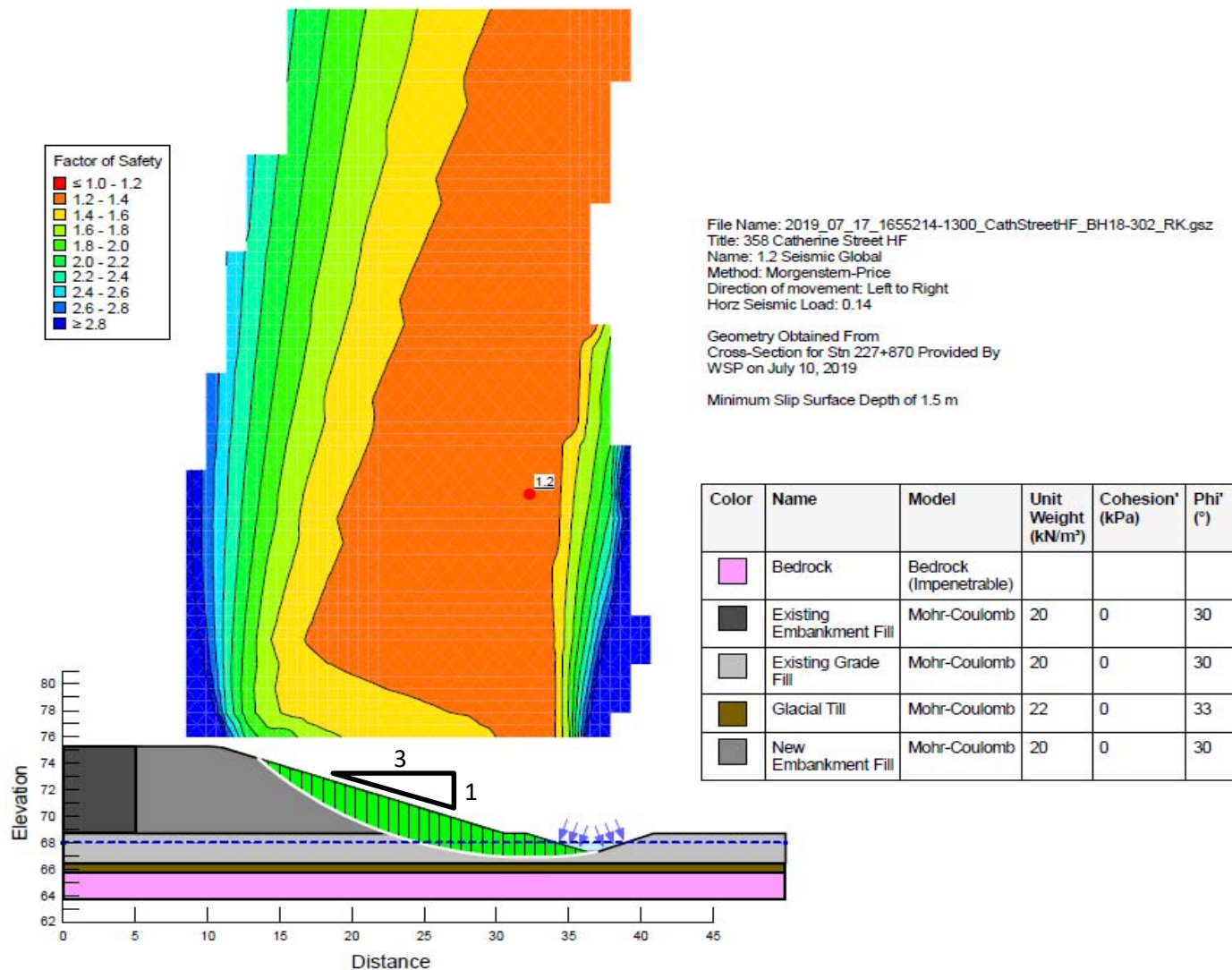
Foundation Investigation and Design

**Catherine High Fill Embankment Slope Stability - Static Analysis,
 Borehole 18-302 Stratigraphy Along STN 227+870**

Ottawa, Ontario

Project No. 1655214-1300
 Drawn: RK
 Date: 2019-07-18
 Checked: KP
 Review: WC

Figure D1



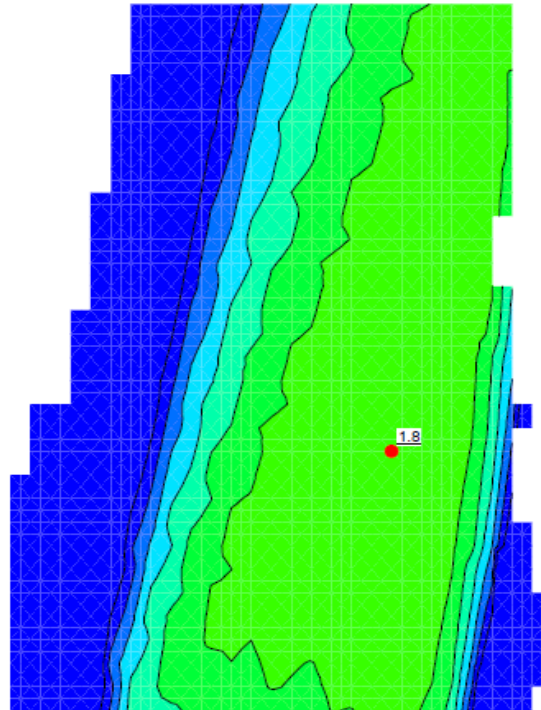
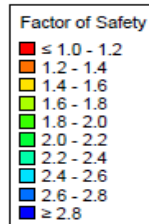
Foundation Investigation and Design

Catherine High Fill Embankment Slope Stability - Seismic Analysis, Borehole 18-302 Stratigraphy Along STN 227+870

Ottawa, Ontario

Project No.	1655214-1300
Drawn:	RK
Date:	2019-07-18
Checked:	KP
Review:	WC

Figure D2

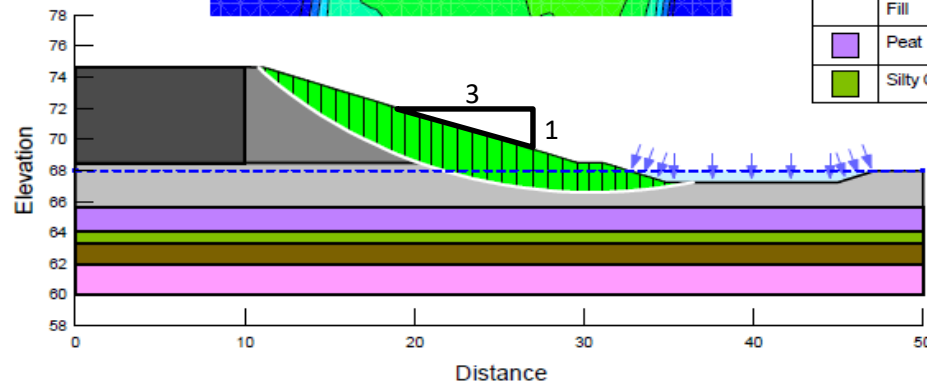


File Name: 2019_07_17_1655214-1300_CathStreetHF_BH18-305_RK.gsz
 Title: 358 Catherine Street HF
 Name: 1.1 Static Drained Global
 Method: Morgenstern-Price
 Direction of movement: Left to Right
 Horz Seismic Load: 0

Geometry Obtained From
 Cross-Section for Stn 227+940 Provided By
 WSP on July 10, 2019

Minimum Slip Surface Depth of 1.5 m

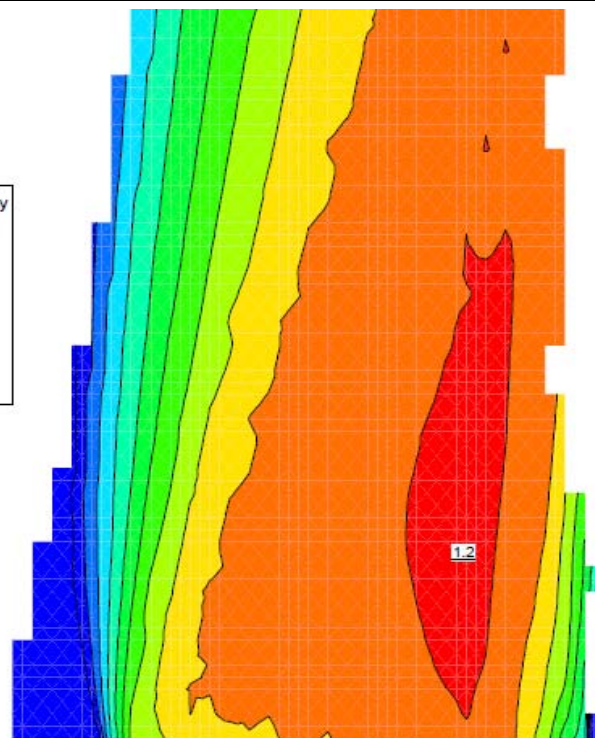
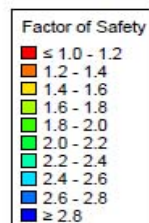
Color	Name	Model	Unit Weight (kN/m ³)	Cohesion* (kPa)	Phi* (°)
	Bedrock	Bedrock (Impenetrable)			
	Existing Embankment Fill	Mohr-Coulomb	20	0	30
	Existing Grade Fill	Mohr-Coulomb	20	0	30
	Glacial Till	Mohr-Coulomb	22	0	33
	New Embankment Fill	Mohr-Coulomb	20	0	30
	Peat	Mohr-Coulomb	11	10	35
	Silty Clay	Mohr-Coulomb	17.5	7.4	29



Foundation Investigation and Design
Catherine High Fill Embankment Slope Stability - Static Analysis,
Borehole 18-305 Stratigraphy Along STN 227+940
Ottawa, Ontario

Project No.	1655214-1300
Drawn:	RK
Date:	2019-07-18
Checked:	KP
Review:	WC

Figure D3

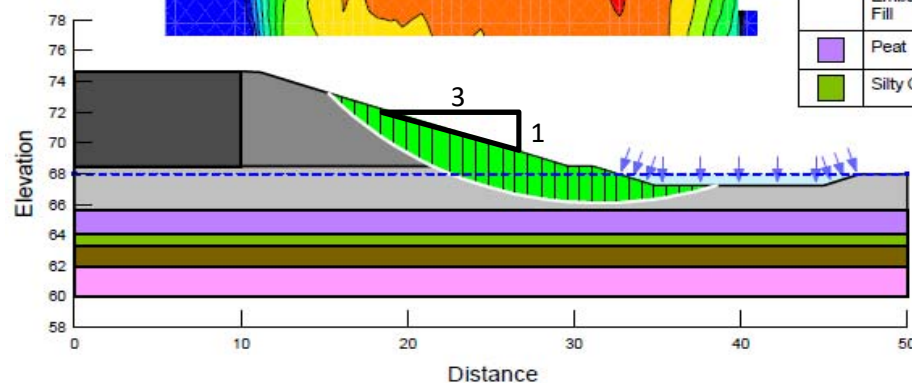


File Name: 2019_07_17_1655214-1300_CathStreetHF_BH18-305_RK.gsz
 Title: 358 Catherine Street HF
 Name: 1.2 Seismic
 Method: Morgenstern-Price
 Direction of movement: Left to Right
 Horz Seismic Load: 0.14

Geometry Obtained From
 Cross-Section for Stn 227+940 Provided By
 WSP on July 10, 2019

Minimum Slip Surface Depth of 1.5 m

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Existing Embankment Fill	Mohr-Coulomb	20	0	30
	Existing Grade Fill	Mohr-Coulomb	20	0	30
	Glacial Till	Mohr-Coulomb	22	0	33
	New Embankment Fill	Mohr-Coulomb	20	0	30
	Peat	Mohr-Coulomb	11	10	35
	Silty Clay	Mohr-Coulomb	17.5	7.4	29



Foundation Investigation and Design

Catherine High Fill Embankment Slope Stability - Seismic
 Analysis, Borehole 18-305 Stratigraphy Along STN 227+940

Ottawa, Ontario

Project No.	1655214-1300
Drawn:	RK
Date:	2019-07-18
Checked:	KP
Review:	WC

Figure D4

APPENDIX E

MASW Test Results and Report

Sites 3-56/1 and 3-56/2 Highway 417 Overpasses at Rochester Street

DATE December 8, 2017**PROJECT No.** 1655214/1500**TO** Susan Trickey
Golder Associates Ltd.**FROM** Stephane Sol
Christopher Phillips**EMAIL** ssol@golder.com
cphillips@golder.com**CHBDC SEISMIC SITE CLASS TESTING RESULTS – HWY417 (ROCHESTER ST EXIT)
OTTAWA, ONTARIO**

This technical memorandum presents the results of one Multichannel Analysis of Surface Waves (MASW) test performed for the purpose of the Canadian Highway Bridge Design Code (CHBDC 2014) Seismic Site Classification for a site located near the HWY417 off ramp to Rochester Street just east of Preston Street in Ottawa, Ontario (Figure 1). The MASW line was located on a grassy area on north of the off ramp. The geophysical testing was performed by Golder personnel on October 18, 2017.

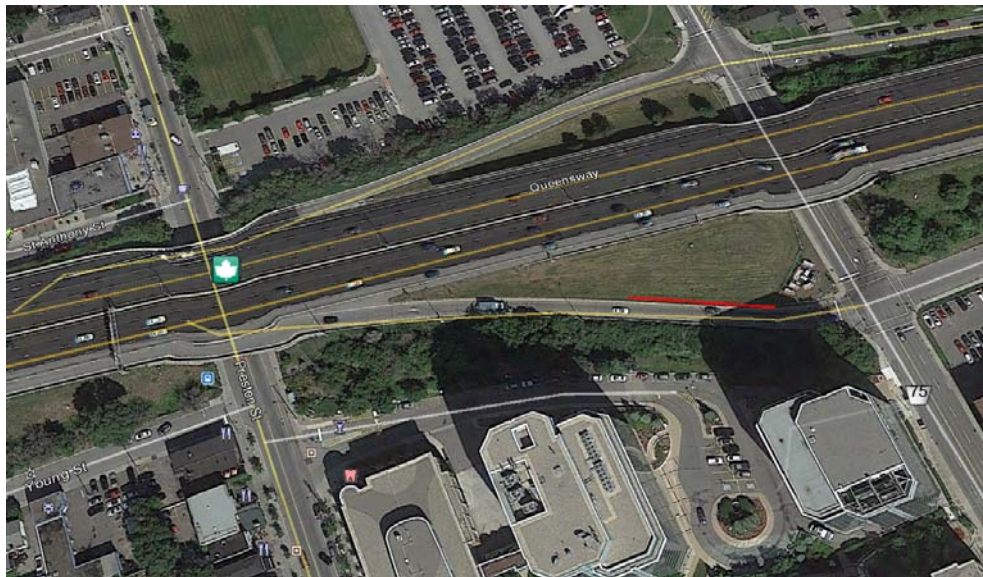


Figure 1: MASW Location Site Map (MASW Line in red)

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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 October 18, 2017, by personnel from the Golder Mississauga and Ottawa offices. For the MASW line, a series of 24 low frequency (4.5 Hz) geophones were laid out at 2 metre intervals. Both active and passive readings were recorded along the MASW line. For the active investigation, a seismic drop of 34 kg and a 9.9 kg sledge hammer were used as seismic sources. Active seismic records were collected with seismic sources located 5, 10, 15, and 20 metres from and collinear to the geophone array. An example of active seismic records collected is shown in Figure 2 below.

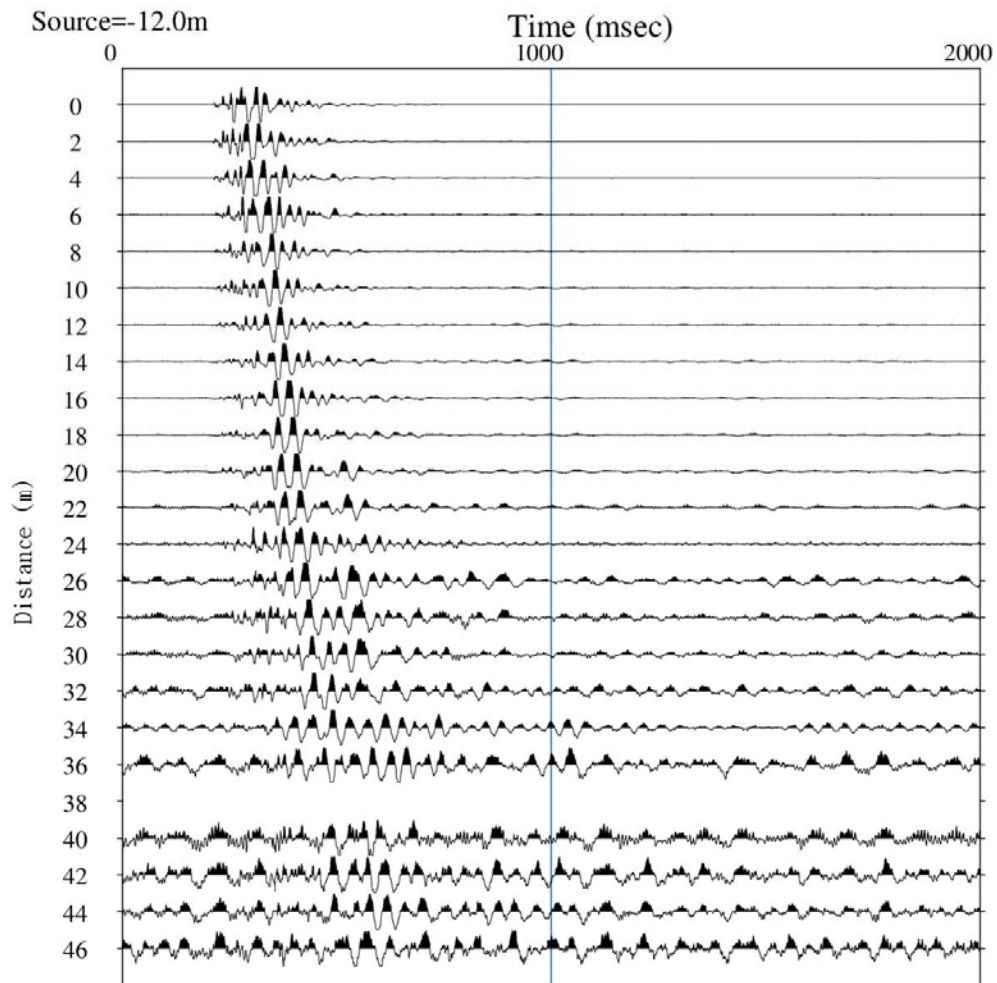


Figure 2: Typical seismic record collected along MASW Line 1

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 (r^2) 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 3 and 4. Shear wave velocity profiles were generated through inverse modelling to best fit the calculated dispersion curves. The active survey provided a dispersion curve with a suitable frequency range (14 -33 Hz). The minimum measured surface wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 14 Hz.

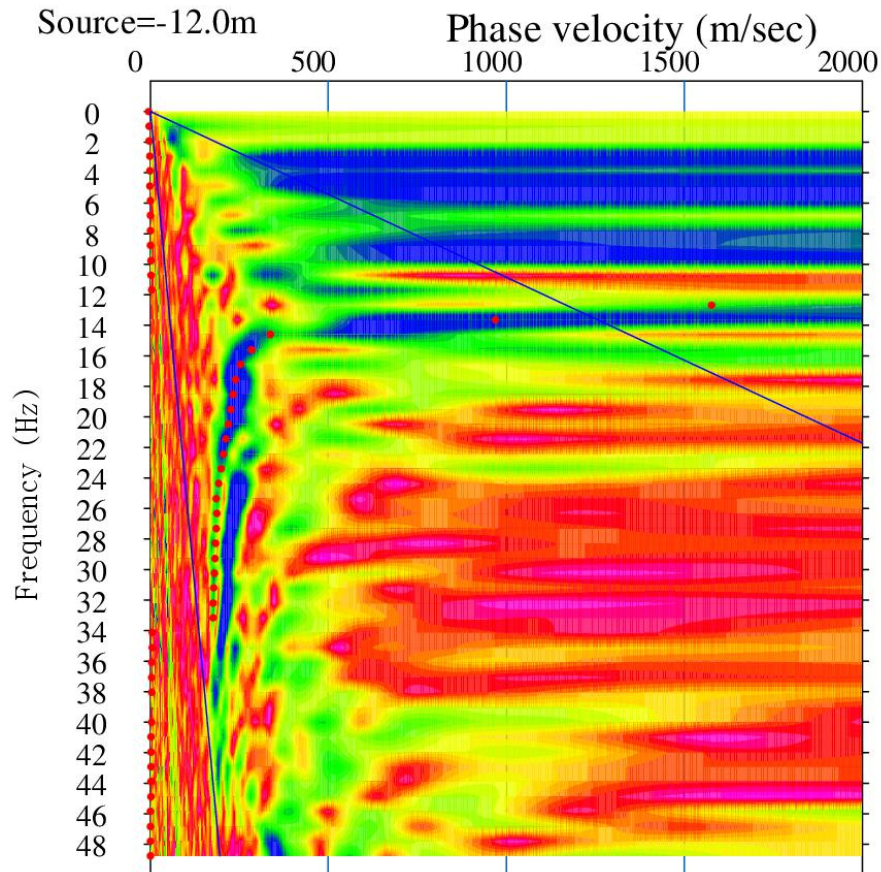


Figure 3: Active MASW Dispersion Curve Picks (red dots) along MASW Line 1

Results

The MASW test results are presented in Figure 4, which presents the calculated shear wave velocity profile derived from the field testing. The results along MASW Line 1 have been calculated using a weight-drop located at 15 m from the last geophone. The field collected dispersion curves are compared with the model generated dispersion curves on Figure 5. There is a satisfactory correlation between the field collected and model calculated dispersion curves, with a root mean squared error of less than 11%.

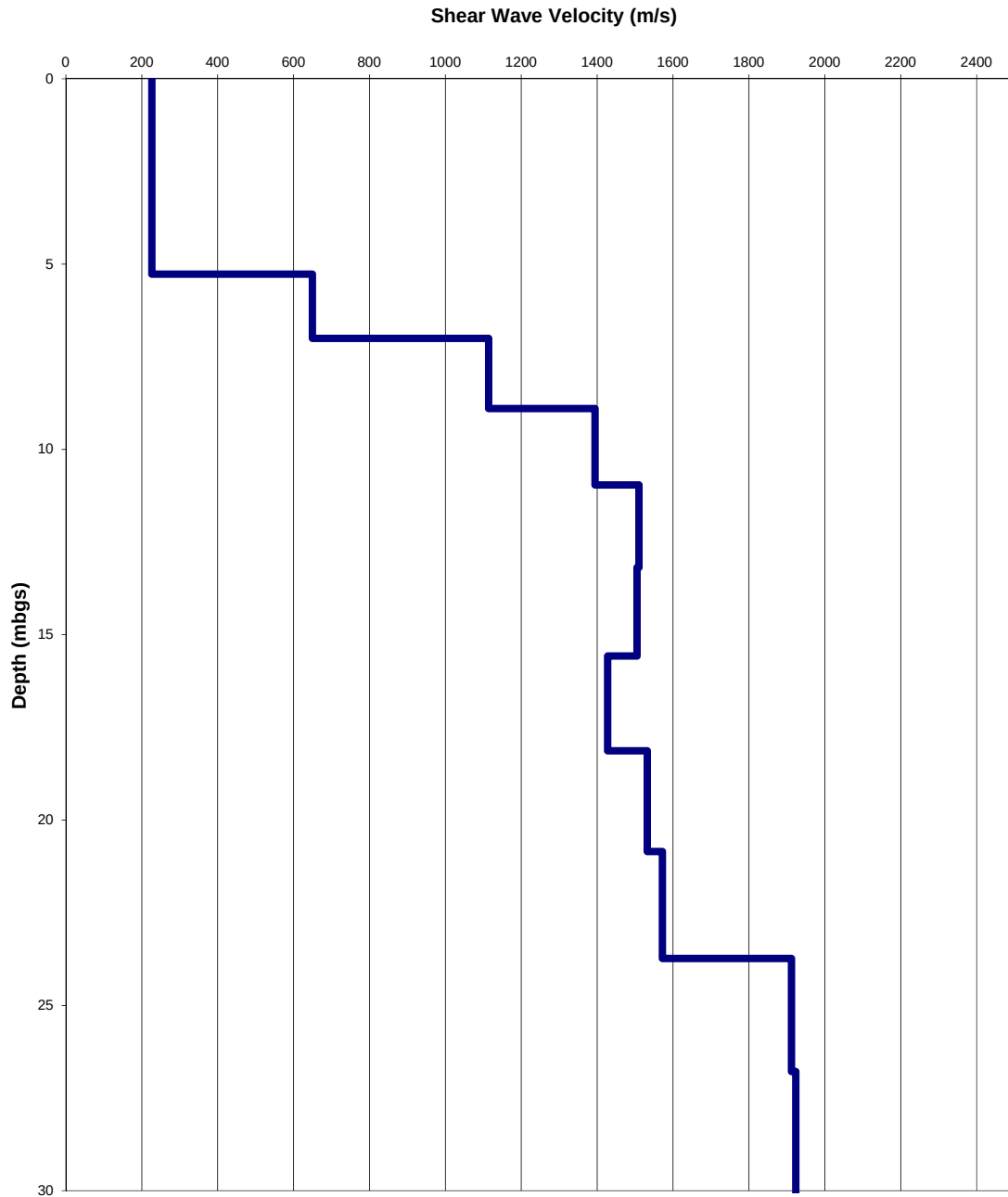


Figure 4: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 1

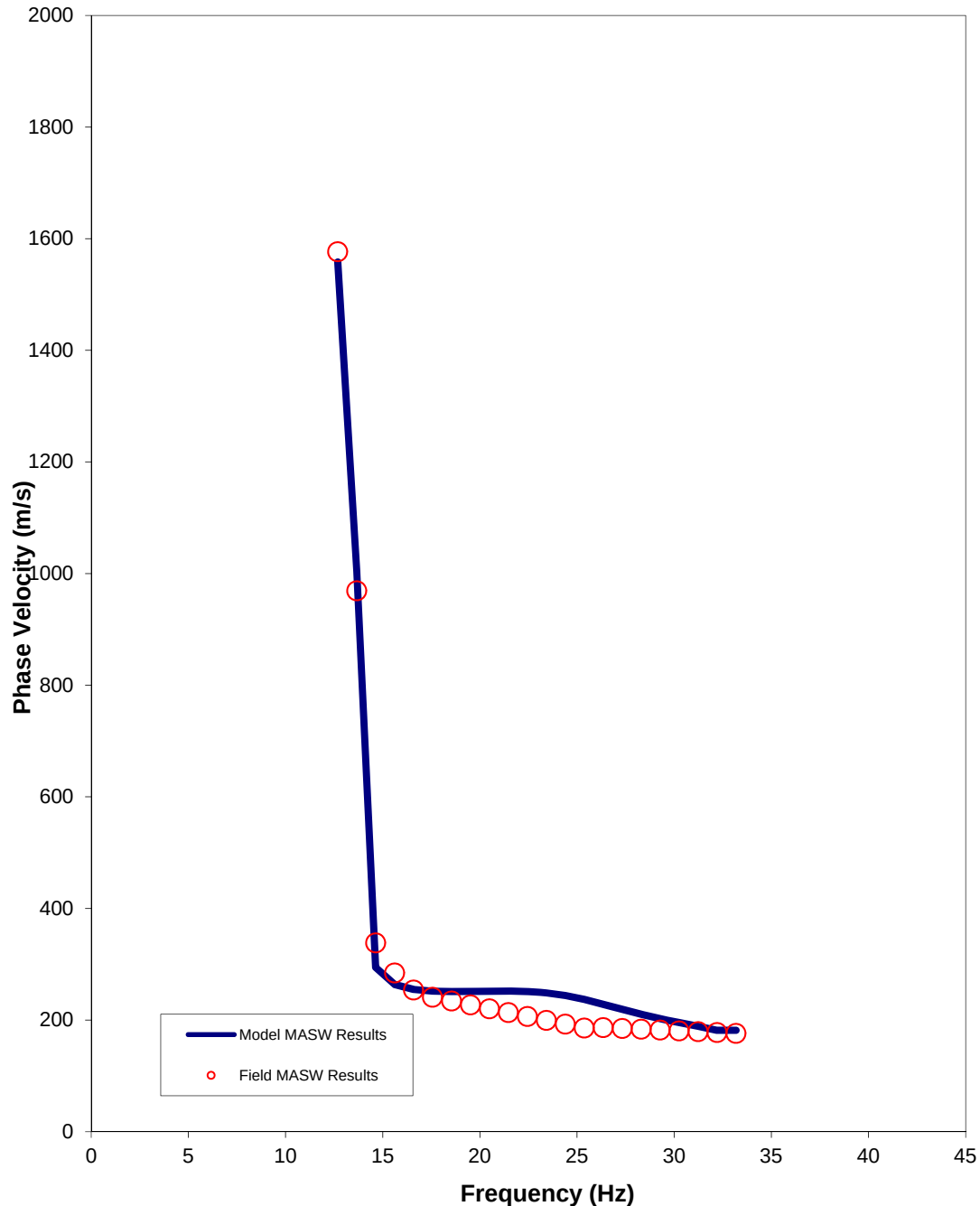


Figure 5: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 1

To calculate the average shear-wave velocity as required by the Canadian Highway Bridge Design Code (CHBDC, 2014), the results were modelled to 30 metres below ground surface. The average shear-wave velocity along MASW Line 1 was found to be 734 m/s (Table 1).

The Canadian Highway Bridge Design Code (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.

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	226	0.004732
1.07	2.31	1.24	226	0.005460
2.31	3.71	1.40	226	0.006188
3.71	5.27	1.57	226	0.006916
5.27	7.01	1.73	650	0.002664
7.01	8.90	1.90	1114	0.001701
8.90	10.96	2.06	1395	0.001478
10.96	13.19	2.23	1510	0.001473
13.19	15.58	2.39	1505	0.001588
15.58	18.13	2.55	1428	0.001789
18.13	20.85	2.72	1532	0.001775
20.85	23.74	2.88	1572	0.001835
23.74	26.79	3.05	1912	0.001595
26.79	30.00	3.21	1924	0.001671
Vs Average to 30 mbgs (m/s)			734	

Limitations

This technical memorandum, which specifically includes all tables, figures and attachments, 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.

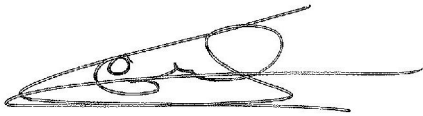
Any use which a third party makes of this memo, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on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

GOLDER ASSOCIATES LTD.



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



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

SS/CRP/mvrd

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preston2017_8dec2017.docx

APPENDIX F

Site Photographs



**Photograph 1: Looking east towards proposed location of the Catherine Street High Fill Embankment;
2019-03-13**



**Photograph 2: Looking south towards existing parking lot and Highway 417 WBL retaining
wall; 2019-03-13**



Photograph 3: Looking along Catherine Street south towards existing facility; 2019-03-13



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