REPORT

Geotechnical Investigation

Proposed Site Redevelopment
Westgate Mall Phase 1
Ottawa, Ontario

Submitted to:

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by RioCan Holdings Inc. (RioCan) to conduct a geotechnical investigation in order to provide geotechnical input to the detailed design of the proposed Phase 1 redevelopment of the Westgate Mall site that is located at the corner of Carling Avenue and Merivale Road in Ottawa, Ontario. A Site Location Plan is attached as Figure 1. It is understood that Phase 1 of the site (located at southeast quadrant of the site) is to be redeveloped with the development consisting of a combined residential and commercial 22 storey tower and 5 storey podium with two levels of underground parking as well as an asphalt surfaced parking. The investigation and reporting was carried out in general accordance with the scope of work provided in our proposal no. P18106595 dated August 3, 2018.

The purpose of this investigation was to assess the general subsurface and groundwater conditions within the study area by means of a limited number of boreholes and associated laboratory testing. Based on an interpretation of the factual information obtained during the current investigation, along with the existing subsurface information available for the site from previous investigations, a general description of the soil and groundwater conditions is presented. These interpreted subsurface conditions and available project details were used to prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The reader is referred to the ‘Important Information and Limitations of This Report’ which follows the text but forms an integral part of this document.

2.0 DESCRIPTION OF PROJECT AND SITE

Currently, the site is occupied by the Westgate Mall which is an “L” shaped commercial retail building located on the north and west sides of the property and a stand-alone restaurant building located at the south east corner of the site. Parking areas and drive aisles occupy the space around these two buildings on the remaining portions of the property. The property is bordered to the north by a Hydro Ottawa easement and then by Highway 417, to the west by a commercial building surrounded by parking areas, to the south by Carling Avenue, and to the east by Merivale Road. A Hydro Ottawa easement is also present within a portion of the eastern side of the site. High tension hydro-electric cables and towers are present within the aforementioned easements.

Based on the preliminary information provided by RioCan, the proposed residential intensification at the Westgate Mall site is planned to consist of a series of five high-rise residential towers to be built in four phases (designated as Phases I, II, III and IV).

At this time, only Phase 1 of the redevelopment plans, which consists of a single building to be located in the southeast corner of the site (as shown on Figure 1), is being considered for construction. The Phase 1 development area is currently occupied by a parking lot and a single-storey restaurant.

The preliminary plans and information provided by RioCan indicate that the Phase 1 building will consist of a 22 storey tower and 5 storey podium with two levels of underground parking as well as an asphalt surfaced parking. The building will be approximately rectangular in shape. The ground floor and mezzanine levels of the podium will be rectangular in shape with two-storey high ceilings for most of its footprint (i.e., the mezzanine level will be mostly open to below). Levels 2 to 4 of the podium will be a smaller “L” shaped structure on top of the two-storey ground floor and mezzanine levels. The tower will be approximately rectangular in shape and will sit on top of the larger “L” shaped podium.

Seven existing boreholes from previous investigations (completed by Golder Associates) have been used to supplement the current investigation. The locations of these previous boreholes are shown on the attached Site Plan (Figure 1). The results of the previous investigations are contained in the following reports:

- Golder Report No. 15-22569-17001 titled: "*Preliminary Geotechnical Investigation, Proposed Site Redevelopment, Westgate Mall Site, Ottawa, Ontario*", and dated December 2015;
- Golder Report No. 782032 titled: "*Soil Investigation, Proposed Restaurant, Westgate Shopping Centre, Ottawa, Ontario*", and dated March 1978;
- Golder Report No. 762168 titled "*Hintonburg West Storm Sewer Stage 6 Cave Creek Storm Extension*", and dated October 1975;
- Golder Report No. 752031 titled: "*Subsurface Investigation Hintonburg West Storm Collector Stage 6 Ottawa, Ontario*", and dated August 1975; and,
- Golder Report No. 70794 titled: "*Subsurface Investigation Proposed Storm Water Sewer System Hintonburg Drainage Area Ottawa, Ontario*", and dated April 1971.

Based on the results of previous investigations and the published geology maps available from the Geologic Survey of Canada (GSC) for this area, the subsurface conditions at this site are expected to consist of a surficial layer of fill, overlying a relatively shallow deposit of silty clay, underlain by a thicker deposit of glacial till.
The glacial till is underlain by interbedded limestone and dolostone bedrock of the Gull River formation. Depth to bedrock within the footprint of the proposed structure varies between about 8 metres below the existing ground surface on the east side and 17 metres below the existing ground surface on the west side.

3.0 PROCEDURE

The fieldwork for this investigation was carried out between September 11 and 13, 2018. During that time, a total of 3 boreholes (numbered 18-01 to 18-03, inclusive) were advanced at the approximate locations shown on the attached Site Plan (Figure 1). The boreholes were advanced using a truck-mounted hollow-stem auger drill rig supplied and operated by George Downing Estate Drilling Limited of Hawkesbury, Ontario. The boreholes were advanced to depths ranging from between 6.7 to 13.2 metres below the existing ground surface. Practical refusal to auger advancement was encountered in boreholes 18-01 and 18-02 which were then extended into the bedrock using rotary diamond drilling techniques while retrieving NQ sized core. Within these boreholes, the drilled lengths in the bedrock were 3.1 and 3.2 metres, respectively (i.e., to total depths of 13.2 and 10.2 metres below the existing ground surface).

Standard penetration tests were carried out within the overburden at regular intervals of depth. Samples of the soils encountered were recovered using 35 millimetre diameter split-spoon sampling equipment. Grab samples of the existing pavement structure were also collected from selected boreholes.

The fieldwork was supervised by technicians from our staff who located the boreholes, directed the drilling and in-situ testing operations, logged the boreholes and samples, and took custody of the soil and bedrock samples retrieved. On completion of the drilling operations, the soil and bedrock samples were transported to our laboratory for further examination by the project engineer and for laboratory testing, which included natural water content, grain size distribution, and Atterberg limit tests on selected soil samples and unconfined compressive strength (UCS) testing on selected bedrock core samples.

Two samples of soil, one from each boreholes 18-01 and 18-03 was submitted to Eurofins Environment Testing for basic chemical analyses related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements.

Geophysical testing in the form of Multichannel Analysis of Surface Waves (MASW) testing and seismic refraction testing was performed at this site for analysis of seismic site class and to better delineate the bedrock surface across the site.

The borehole locations were selected in consultation with RioCan, marked in the field, and subsequently surveyed by Golder Associates personnel. The borehole coordinates and ground surface elevations were measured using a Trimble R8 GPS survey unit. The geodetic reference system used for the survey is the North American datum of 1983 (NAD83). The borehole coordinates are based on the Modified Transverse Mercator (MTM Zone 9) coordinate system. The elevations are referenced to Geodetic datum (CGVD28).

4.0 SUBSURFACE CONDITIONS

4.1 General

Information on the subsurface conditions is presented as follows:

- Record of Borehole and Drillhole Sheets from the current investigation are provided in Appendix A.
- Record of Borehole and Drillhole Sheets from previous investigations are provided in Appendix B.
- Photographs of the bedrock core and the results of the UCS testing are provided in Appendix C.
- Results of the basic chemical analyses are provided in Appendix D.
- Results of the water content and Atterberg limit testing are provided on the Record of Borehole Sheets.
- Results of the grain size distribution testing are provided on Figures 2, 3, and 4.

The Record of Borehole sheets describe the subsurface conditions at the borehole locations only.
The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling in some cases, observations of drilling progress as well as results of Standard Penetration Tests (SPTs) and, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface soil, bedrock and groundwater conditions will vary between and beyond the borehole locations.

Unless otherwise noted, the following sections present a more detailed overview of the subsurface conditions encountered in the boreholes advanced during the previous 2015 investigation (boreholes 15-09 and 15-10) and the current 2018 investigation within the Phase 1 study area only. It should be noted that the shallow subsurface conditions noted on the borehole logs from the previous investigations may have changed since the boreholes were drilled, as such only auger refusal/bedrock depths and hydraulic response tests from previous drilling are discussed herein.

4.2 Overview of Subsurface Conditions

In general, the subsurface stratigraphy within the area of the investigation consists of surficial fill materials (including fill associated with the parking lot pavement structure) overlying silty clay which is generally underlain by glacial till at depths of 3.7 to 4.9 metres. The two boreholes from the current investigation that penetrated through the till encountered limestone bedrock at depths ranging from 10.1 metres (Borehole 18-01) to 7.1 metres (Borehole 18-02). Available subsurface information from previous investigations indicates that the bedrock surface exists at depths of up to about 16.3 metres (Borehole 15-09).

4.3 Pavement Structure / Fill

Asphaltic concrete was encountered at all of the boreholes; the thickness of the asphaltic concrete, where encountered, is provided in the table below.

Borehole No.	Investigation	Asphalt Thickness (mm)
18-01	Current	80
18-02		30
18-03		120
15-09	Previous	180
15-10		100

Fill was encountered in each of the boreholes. The upper portion of the fill generally consists of grey, granular pavement structure comprised predominantly of varying amounts of sand and gravel. In some of the boreholes, the lower portion of the fill consists of grey brown to black silty clay. The presence of organic matter and wood pieces was occasionally observed in the silty clay fill. The depth to the bottom of the fill at each of the borehole locations is provided in the table below.

Borehole No.	Depth of Granular Pavement Structure below existing grades (m)	Depth of Silty Clay Fill (m)
18-01	0.53	2.29[2]
18-02	0.13	2.13[1]
18-03	0.76	0.91
15-09	0.76	1.63
15-10	0.56	2.44

[1] 0.3 metre thick sand and gravel fill layer present between the pavement structure and the silty clay fill

[2] 0.4 metre thick layer of sand present between the pavement structure and the silty clay fill

SPT “N” values measured within the fill ranged from 8 to 16 blows per 0.3 m of penetration. The SPT “N” values suggest that the pavement structure fill is compact while the silty clay fill is very stiff. The results of grain size distribution testing carried out on one sample of the pavement structure are presented on Figure 2.

4.4 Silty Clay to Clay

At all of the current 2018 and previous 2015 borehole locations within the Phase 1 development, the pavement structure and fill are underlain by a deposit of sensitive marine silty clay from the previous Champlain Sea that covered most of the Ottawa area.

In general, with the exception of borehole 15-10, the upper portion of the silty clay has been weathered to a grey brown crust. The weathered zone, where present, extends to depths of between about 2.4 and 3.7 metres below existing ground surface. SPT ‘N’ values ranging from 2 to 10 blows per 0.3 metres of penetration were obtained within the weathered crust portion of the silty clay deposit, indicating a stiff to very stiff consistency.

At borehole 18-03, there is a localized 0.5 metre thick layer of sand and silt below the weathered silty clay at a depth of 2.4 metres below existing ground surface. The results of moisture content testing on this sample of sandy silt gave a result of about 24 percent. The results of grain size distribution testing carried out on the same sample are presented on Figure 3.

Below the weathered crust in boreholes 18-01, 18-02, 15-09, below the silt layer at 18-03 and below the fill at borehole 15-10, the silty clay is grey in colour. The unweathered silty clay deposit extends to depths ranging from between about 3.7 and 4.9 metres below existing surface. The results of in situ vane shear tests completed within the grey silty clay measured undrained shear strength values ranging from between about 42 and 67 kilopascals corresponding to a firm to stiff consistency.

The results of moisture content testing on one sample of the silty clay gave a result of about 55 percent. The results of Atterberg limits testing on one sample of the grey silty clay gave a plasticity index of 44 percent and a liquid limit of 65 percent, indicating a high plasticity clay.

4.5 Glacial Till

At all of the previous and current boreholes, a deposit of glacial till was encountered beneath the silty clay. The glacial till typically consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sand and silt with a trace to some clay. At some locations, the till consists of clayey sand containing gravel, cobbles and boulders. Where fully penetrated in boreholes 18-01, 18-02 and previous borehole 15-09, the glacial till extends to depths ranging from between about 7.1 and 16.3 metres below existing ground surface. Where not fully penetrated in boreholes 18-03, 15-10, the glacial till was proven to extend to depths of 6.7 and 6.1 metres below the existing ground surface, respectively.

SPT “N” values within the glacial till layer gave ‘N’ values ranging from between about 5 blows per 0.3 metres of penetration to greater than 50 blows per 0.3 metres of penetration indicating a compact to a very dense state of packing; however, the higher blow counts could be indicative of boulders and cobbles in the till rather than the state of packing.

The results of natural moisture content testing carried out on three samples of the glacial till gave values ranging from between about 6 to 12 percent. The results of grain size distribution testing carried out on three samples of the glacial till are presented on Figure 4.

4.6 Bedrock

Boreholes 18-01, 18-02 and previous boreholes 15-09 and W-3 were extended through the glacial till deposit into the underlying bedrock using rotary diamond drilling techniques. The depths and elevations to bedrock surface are summarized below:

Borehole No.	Ground Surface Elevation (masl)	Depth to Bedrock (m)	Elevation of Bedrock (masl)
18-01	74.74	10.11	64.63
18-02	74.81	7.06	67.75
15-09	74.54	16.28	58.26
W-3	80.6	8.2	66.9
W-31	74.2	19.3	54.9
Sta. 3+20	74.4	16.1	58.3

The bedrock consists of limestone with shale interbeds. The boreholes listed above were extended about 3.1 to 11.5 metres into the bedrock. The recovered bedrock cores from these locations consist of fresh, thinly to medium bedded, dark grey, fine grained limestone bedrock with shale partings and occasional nodular sections.

The Total Core Recovery (TCR) of the cored bedrock ranged between 90 and 100 percent and the Rock Quality Designation (RQD) ranged from about 70 to 100 percent, indicating a fair to good quality rock.

The results of laboratory testing carried out on two samples of the cored bedrock from 18-01 and 18-02 measured Uniaxial Compressive Strengths (UCS) of about 122 and 123 MPa, respectively, indicating the sample of the rock tested is very strong. Results of the UCS test are presented in Appendix C.

4.7 Groundwater Conditions

No new monitoring wells were installed in the boreholes as part of the 2018 investigation. However, a monitoring well was installed in borehole 15-10 during the previous investigation. The groundwater levels observed in the monitoring well on October 14 and November 9, 2015 have been summarized in the following table:

Borehole	Geological Material Well Installed In	Groundwater Elevation on October 14, 2015 (m)	Groundwater Elevation on November 9, 2015 (m)
15-10	silty clay / glacial till	70.1	70.1

It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

4.8 Corrosion Testing

Two samples of soil, one each from boreholes 18-01 and 18-03 were submitted to Eurofins Environment Testing for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of this testing are provided in Appendix D and are summarized below.

Borehole / Sample Number	Sample Depth (m)	Chloride (%)	Sulphate (µg/g)	pH	Conductivity (mS/cm)	Resistivity (Ohm-cm)
18-01 SA7	5.3 – 5.9	0.023	40	8.3	0.57	1750
18-03 SA5	3.1 – 3.7	0.068	40	8.7	0.82	1220

5.0 DISCUSSION AND GEOTECHNICAL RECOMMENDATIONS

This section of the report provides engineering information related to the geotechnical design aspects of the project based on our interpretation of the available subsurface information and on our understanding of the project requirements. The discussion below focuses on the development of the Phase 1 building area. The subsurface conditions vary across the site as well as within the footprints of the proposed individual buildings planned as part of the site redevelopment.

The information in this portion of the report is provided for detailed design purposes in support of the design by the engineers and architects. The recommendations provided herein are consistent with the Ontario Building Code of 2012 (OBC 2012). Where comments are made on construction, they are provided only in order to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking any work at the site should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, equipment capabilities, costs, sequencing and the like.

This report addresses only the geotechnical aspects of the subsurface conditions at this site.
The geo-environmental (chemical) aspects, including the consequences of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources, are outside the terms of reference for this report. The results of a concurrent Phase II Environmental Site Assessment for this project is provided under separate cover.

5.1 Site Grading

It is understood that, as currently proposed, the design finished grades will generally remain unchanged.

The permissible grade raises could potentially need to be limited by the capacity of the silty clay deposit to support additional loading. As a general guideline, which can be applied to the overall site, it is considered that a permissible grade raise of 1 metre (above the existing ground surface level) could be used. Grade raises in excess of 1 metre should be reviewed on a location-by-location basis.

5.2 Foundation Design

The following sections focus on Phase 1 of the development, which will be constructed first.

Based on the conceptual design information provided to Golder, the structure proposed to be constructed as part of the Phase 1 site redevelopment is planned to have two underground parking levels. As such, the excavation for the Phase 1 tower is expected to extend to depths of about 7 to 10 metres below existing site grades.

The subsurface conditions present below the pavement structure and fill at this site generally consist of sensitive silty clay, underlain by glacial till over limestone bedrock. The lower portion of the silty clay is grey, unweathered, and firm to stiff. The silty clay is sensitive and highly compressible when subjected to new loads and the thickness of the clay varies across both the Phase 1 building footprint and across the site. Based on these conditions, the use of shallow spread footing foundations or a raft slab placed within or above the silty clay deposit is not considered feasible for the proposed high rise tower in Phase 1, but could be considered for low to mid-rise structures depending on loading conditions. Additional assessment would be required for the use of shallow foundations for low to mid-rise buildings on the silty clay. With the currently proposed two levels of underground parking, it is understood that consideration is being given to supporting the western portion of the 5-storey podium on shallow spread footings on the undisturbed, compact to dense glacial till. This is considered a feasible option, and guidance is provided in the following section.

5.2.1 Shallow Spread Footings

On the east side of the structure, the structure may be founded on spread footings supported on the underlying bedrock provided that they can be designed using the bearing resistance values provided below.

Spread footings founded on clean, sound and undisturbed bedrock are considered to be a feasible option. Although the quality of the rock observed in the boreholes and as defined by the RQD values is indicated to be good for the upper portion of the limestone bedrock at this site, it is common to find more fractured and weathered zones of rock near the bedrock surface (i.e., upper 1 to 1.5 metres). When they are encountered, these zones of more fractured rock should be removed. For spread footings placed on sound bedrock, a factored Ultimate Limit States (ULS) bearing resistance of 5,000 kilopascals can be used for design of the foundations. Serviceability Limit States (SLS) net bearing resistances do not generally apply to the design of foundations on the bedrock, provided the bedrock surface is properly cleaned of soil and highly weathered/fractured bedrock at the time of construction. The ULS bearing resistance for foundations on bedrock will need to be reduced within the vicinity of the existing collector sewer which crosses over the southeast corner of the site as outlined in Section 5.3 of this report.

For ULS sliding resistance of a cast-in-place footing placed on bedrock, an unfactored sliding friction coefficient of 0.70 can be used. In accordance with OBC 2012 requirements, a resistance factor of 0.8 should be applied to the sliding resistance between the footings and the underlying bedrock.

On the west side of the structure, the structure may be founded on spread footings supported on the underlying glacial till provided that they can be designed using the bearing resistance values provided below. Also, the design of the new structure will have to consider the differential settlements between the foundations supported on bedrock, and those supported on the more compressible glacial till. Structural separation maybe required between the foundations supported on bedrock, and those supported on glacial till.

Spread footings founded on the compact to dense glacial till (i.e., SPT ‘N’ values higher than about 25) below about Elevation 67.0 metres (on the west side of the Phase 1 building) are considered to be a feasible option. An SLS net bearing resistance of 250 kilopascals and a factored ULS bearing resistance of 400 kilopascals can be used for design of pad footings up to 5.0 metres in dimensions and for strip footings up to 2.0 metres in width placed on native and undisturbed glacial till below this elevation. The SLS values provided correspond to total and differential settlement values of 25 and 19 millimetres, respectively.

It should be noted that the expected settlements of spread footings placed directly on the underlying bedrock are very small, differential settlements of up to about 25 millimetres may occur between the spread footings placed on glacial till and those placed directly on the underlying bedrock.

For ULS sliding resistance of a cast-in-place footing placed on glacial till, an unfactored friction coefficient of 0.45 can be used. In accordance with OBC 2012 requirements, a resistance factor of 0.8 should be applied to the sliding resistance between the footings and the underlying glacial till.

5.2.2 Steel H-Pile or Steel Pipe (Tube) Pile Foundations

5.2.2.1 Founding Elevations

Should the above preliminary bearing resistance not be sufficient for the design of the west side of the structure, the proposed western portion of the structure may be supported on closed-ended steel pipe (tube) piles or steel H-piles driven to refusal either within the lower, very dense portion of the till deposits or on the underlying bedrock.

Based on the borehole results from this investigation and previous studies, the following table provides an overview of the expected elevations of the very dense glacial till, as well as the bedrock surface elevations within the vicinity of the Phase 1 building.

Approximate Location	Borehole Number (Report Number)	Approximate Elevation (m) of Surface of Very Dense Glacial Till	Approximate Bedrock Surface Elevation (m)
Northwest Corner of Phase 1 Tower	15-09 (1522569)	64.0	58.3
Southwest Corner of Phase 1 Tower	BH Sta. 3+20 (762168) BH W-31 (70794)	N/A 62.9	58.3 54.9
Middle of Phase 1 Tower	BH 18-01 (current study)	67.1	64.6

As an alternative to driven piles (i.e. H-piles and/or closed-ended pipe piles), the use of an open-ended drilled pile advanced into the bedrock could also be considered. This pile type requires a specialized contractor and is generally more expensive than driven piles, but the use of drilled piles greatly reduces the risk of pile deflections, pile damage and piles ‘hanging up’ in the glacial till. The drilled pipe piles should be advanced to a minimum embedment depth of 1.5 metres into the bedrock.

5.2.2.2 Axial Geotechnical Resistance

An HP 310x110 piles or 324 mm diameter closed-ended steel pipe piles driven to practical refusal within the very dense portions of the glacial till may be designed using factored axial geotechnical resistances at Ultimate Limit States (ULS) of 1,300 kN. The geotechnical reaction for an individual pile at SLS will not govern and may be higher than the factored geotechnical resistance at ULS; however, settlements of pile groups should be reviewed once the pile layout has been chosen. Higher capacities would be achievable if larger pile sizes are used.

For an HP 310x110 piles driven to found on the limestone bedrock, the factored axial geotechnical resistance at ULS may be taken as 2,000 kN. Serviceability Limit States (SLS) resistances do not apply to piles founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. It should be noted that pre-drilling may be required to advance the piles through the lower, very dense portions of the till if piles driven to bedrock are considered.

As an alternative to pre-drilling, the use of open-ended drilled pipe piles socketed into the bedrock may be considered. For an open-ended, concrete-filled, 245 mm diameter steel pipe pile having a minimum wall thickness of 12 mm and at least 1.5 metre penetration into the bedrock, an axial geotechnical resistance at ULS of 1,400 kN may be used. Serviceability Limit States (SLS) resistances would not govern for piles founded on or within the limestone bedrock.

The preliminary ULS pile capacities discussed herein have been based on semi-empirical analyses using laboratory and in-situ test data and incorporate a geotechnical resistance factor of 0.4. Higher resistance values (0.5 for Pile Driver Analyzer or 0.6 for static pile load test methods) can be used where field testing is completed which would allow the use of higher design pile capacities. Given the highly variable subsurface conditions at the site and the large number of piles that will be required for the proposed buildings (including future phases), consideration could be given to carrying out a test pile program to optimize pile design and to better define the depth to which pile refusal will be encountered, if piles are selected to support a portion of the Phase 1 podium. The test program could be completed prior to construction of the Phase 1 building (although this would require mobilization of pile driving equipment to the site prior to building construction and would be disruptive to existing retail operations) which would allow for optimization of the pile design for the Phase 1 building.

Pile installation should be in accordance with OPSS 903 *(Construction Specification for Deep Foundations).* For driven piles, the drawings should incorporate the appropriate note stating that the piles (both H-piles and pipe piles) should be equipped with pile points (e.g. Titus Standard H Point, or similar) and should be driven to bedrock. The pile points will provide additional protection to the pile tips against damage from boulders during driving, and they will also provide some penetration into the underlying sloping bedrock for piles that reach the bedrock. For piles driven to refusal on bedrock, and as described in OPSS 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

As a result of the two levels of underground basement, it is possible that some of the piles will be very short (i.e., less than 3 metres in length). The piles should be at least 3 metres in length to provide sufficient lateral confinement from the surrounding soils. In areas where the bedrock is less than 3 metres below the pile caps, the piles should be pre-drilled into the bedrock to provide at least 3 metres in length, or a spread footing placed directly onto the bedrock surface could be used at these locations.

5.3 Impacts to Existing Collector Sewer

Plans previously provided by David Schaeffer Engineering Ltd. as well as the City of Ottawa indicate that an approximately 2.1 metre (84 inches) diameter storm sewer pipe is located near the southeast corner of the property (and potentially encroaching onto the Phase 1 Site at this location). The top of the pipe is indicated to be at about elevation 57.9 metres and it is understood that this pipe was installed by tunneling (i.e. not open cut).

Based on the borehole data at this location, the top of the bedrock appears to be at approximately elevation 67.1 metres, which would provide about 9.2 metres of rock cover above the pipe. For design purposes, the proposed shallow foundations or piles for the new building should be designed/located to have a minimum setback of 5 metres from the side of the pipe to avoid additional stresses from the deep foundations being imposed onto the tunnel liner. In addition, the ULS capacity for shallow footings on the underlying bedrock should be reduced to 1,000 kPa within 10 metres of the existing collector sewer.

5.4 Frost Protection

All perimeter and exterior foundation elements or interior foundation elements (i.e., footings, pile caps, grade beams, etc.) in unheated areas should be provided with a minimum of 1.5 metres of earth cover for frost protection purposes. Isolated, unheated exterior foundation elements adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 metres of earth cover.

As an alternative to earth cover, consideration could be provided to the use of an insulation detail. Additional guidance on insulation details can be provided if required.

5.5 Seismic Design Considerations

For the proposed building, the seismic design provisions of the 2012 OBC depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or rock below founding level.

Based on the results of the MASW testing carried out at this site, this site can be assigned a Site Class of C for seismic design purposes in accordance with the 2012 OBC for a structure founded on or within the overburden.

For the east portion of the building that will be supported on spread footings placed directly on the bedrock surface and separated structurally from the west portion of the building that will be founded on glacial till, the shear wave velocity values measured within the bedrock using the MASW testing results gave a harmonic mean of 877 metres per second for a depth of 30 metres below the proposed foundations. Based on this value, a Site Class B can be used for the design of the east portion of the new building that will be entirely founded on the spread footings placed directly on the bedrock.

5.6 Garage Floor Slab

In preparation for the construction of the garage floor slab, all fill and, all loose, wet, and disturbed material should be removed from beneath the floor slab down to the undisturbed native soil or bedrock. Provision should be made for at least 250 millimetres of Ontario Provincial Standard Specification (OPSS) Granular A to form the base of the floor slab. Any bulk fill required to raise the grade up to the underside of the Granular A should consist of OPSS Granular B Type II. The underslab fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

The floor slabs should be structurally separate from the foundation walls and columns. Sawcut control joints should be provided at regular intervals and along column lines to minimize shrinkage cracking.

Provision should be made for drainage underneath the floor slab consisting of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in geotextile, which leads by gravity drainage to an adjacent storm sewer or sump pit from which the water is pumped.

5.7 Garage Excavation and Groundwater Control

It is understood that the two levels of underground garage parking will extend about 7 metres below the existing ground surface. Accordingly, excavation to these depths will be through surficial fill, native silty clay, into the underlying glacial till, and possibly into the bedrock at the very eastern end of the new building footprint. Measurements taken during the current investigation suggest that the groundwater level is generally at about 2 to 4 metres depth below ground surface, and either within the lower portion of the silty clay deposit, or the upper portion of the glacial till.

The proposed excavation will be below the measured groundwater level. The contractor is responsible for the design of a temporary groundwater control system, including assessing the appropriate type of pump(s) and its arrangement, and should be required to submit a detailed work plan for review. For any pumping that exceeds a rate of 50 m^3/day (50,000 L/day), but less than 400 m^3/day (400,000 L/day), a Ministry of Environment, Conservation and Parks (MECP) Environmental Activity and Sector Registration (EASR) is required, and must be supported by a water taking plan and a discharge plan. For pumping that exceeds 400 m^3/day (400,000 L/day), an MECP Permit To Take Water (PTTW) would be required. Based on the available groundwater information at the site, it is likely that a PTTW will be required for this project, but the category level will have to be defined upon further study.

No unusual problems are anticipated in excavating the overburden using conventional hydraulic excavating equipment, recognizing that cobbles and boulders could be present in the fill and glacial till.

In accordance with the Occupational Health and Safety Act (OHSA) of Ontario, the soil that will be encountered within the excavations (fill, silty clay, and glacial till) would be generally classified as Type 3 soils. Below the groundwater level, the glacial till soils would be classified as Type 4 soil. Provided that the groundwater level is lowered as the excavation progresses, trench excavations may be made with side slopes at 1 horizontal to 1 vertical, or flatter, otherwise excavations below the groundwater level in these deposits would likely require flatter side slopes (e.g., 3 horizontal to 1 vertical) to remain stable.

Where site conditions (such as the presence of soft or weak soils, proximity of existing structures and utilities, or space restrictions) do not allow for the above noted side slopes then suitable safety and support measures must be undertaken according to the requirements of the OSHA. These measures include installation of a suitable shoring system to create and maintain positive support to the sidewalls of the excavation. Guidelines on excavation shoring are provided in Section 5.8.

The silty clay and glacial till soils that will form the floor of the foundation excavations are highly sensitive to disturbance. Consideration should therefore be given to protecting the subgrade in foundation areas with a mud slab of lean concrete or a layer of compacted granular fill materials. The thickness of the mud slab and compacted granular fill working mat will depend on the size and weight of the equipment to be used at the bottom of the excavation. Any disturbed soil will need to be removed prior to placing the protective layer. That mud slab/granular fill materials should be placed immediately following inspection and approval of the subgrade. The period of time between exposure of the subgrade and covering with the protective layer should be limited to as brief as possible and, in the interim, no construction traffic should be permitted on the subgrade. The excavation should also be made using an excavator bucket without teeth; i.e., a smooth blade should be used.

5.8 Temporary Building Excavation Shoring

The excavation for the proposed Phase 1 tower will extend about 7 metres below the existing ground surface and will be close to the east and south property limits and, as such, vertical (or near vertical) excavation walls may be required. The contractor is fully responsible for the detailed design and performance of the temporary shoring systems. However, this section of the report provides some general guidelines on possible concepts for the shoring to be used by the designers for assessing the possible impacts of the shoring design and site works as well as to evaluate, at the design stage, the potential for impacts of this shoring on the adjacent properties. Temporary shoring can be used in combination with open cuts above the top of shoring, however, the earth pressure distribution must take into account the effects of the soil pressures from the upper open cut section.

The shoring method(s) chosen to support the excavation sides must take into account the soil and bedrock stratigraphy, the permissible movement of the shoring, the groundwater conditions, the methods adopted to manage the groundwater and construct the shoring systems, the potential ground movements associated with the excavation and construction of the shoring system, and their impact on adjacent structures and utilities.

It is understood that the excavation floor level will generally be about 7 metres in depth below the existing ground surface elevation, with some deeper excavations required locally. The City of Ottawa right-of-ways for Merivale Road and Carling Avenue, which contain below grade services, are located adjacent to the east and south sides, respectively, of the proposed excavation for the Phase 1 tower. As such, any services located in close proximity to and/or within the zone of influence of the shoring system could be affected by ground movements behind the shoring. Details on the utilities in these areas should be confirmed during the detailed design studies to better tailor the shoring guidelines provided herein.

For preliminary design purposes, a soldier pile and timber lagging system is considered a suitable shoring method that may be considered for the proposed 7 metres deep excavation at the site. Due to the presence of very dense till with boulders at shallow depth on the east side of the site, the soldier piles may require predrilling to provide sufficient embedment for toe fixity. The shoring system must be provided with appropriate lateral support.

Where foundations or settlement sensitive infrastructure, such as buried utilities, are present within the zone of influence of the shoring system, the deflections may need to be greatly limited and interlocked steel sheet piling or a secant pile wall with pre-stressed tie backs may be required. Steel sheet pile systems would not be suitable where very dense till is present at shallow depth. Soldier pile and lagging walls are considered suitable for the sides of the excavations (provided that settlement-sensitive structures or utilities are not present in the zone of influence of the walls) where the objective is to maintain an essentially vertical excavation wall and the movements above and behind the wall need only be sufficiently limited so that relatively flexible features (such as roadways or sidewalks) will not be adversely affected.

For all of the above systems, some form of lateral support to the wall is required for excavation depths greater than about 3 to 4 metres. Lateral restraint could be provided by means of tie-backs consisting of grouted soil or bedrock anchors. However, the use of rock/ground anchor tie-backs would require the permission of the adjacent property owners (including the City, who owns the adjacent roadways) since the anchors would be installed beneath their properties. The presence of utilities beneath the adjacent streets, which could interfere with the tie-backs, should also be considered. Alternatively, interior struts can be considered, connected either to the opposite side of the excavation (if not too distant) or to raker piles and/or footings within the excavation.

5.9 Ground Movements

During the excavation for the underground levels of the proposed buildings, lateral deformation and vertical settlement of the adjacent ground will occur as a result of installation and deflection of the retaining/shoring system and dewatering activities. The ground movements induced could affect the stability or performance of buildings or underground utilities adjacent to the excavation. Therefore, the magnitude and extent of ground movement and potential impacts on surrounding infrastructure should be assessed prior to construction to confirm movements will be in tolerable limits and monitored during construction.

5.10 Foundation Wall Backfill

Foundation/basement walls should be backfilled with free draining non-frost susceptible granular fill meeting the requirements of OPSS Granular B Type I materials. The backfill should be compacted to 95 percent of the material’s standard Proctor maximum dry density using suitable compaction equipment. To reduce compaction induced stresses, only light compaction rollers or plate tampers should be used within 1.0 metre of the wall. In any areas where the temporary shoring wall serves as the outside form for the foundation wall, vertical drainage must be installed against the shoring wall. The drainage channels could consist of filtered drainage wick such as Miradrain (or proven equivalent).

Water flow from either the granular backfill or drainage channels should be collected by means of a perforated drain line located at the base of the wall. This drain line should be provided with a granular surround and should lead to a sump pit from which water can be pumped.

Beneath hard surfacing (e.g., pavements or sidewalks/walkways), the granular backfill for the foundation wall should be placed to form a frost taper at 3 horizontal to 1 vertical to a depth of 1.8 metres (i.e., the frost depth). The purpose of this frost taper is to limit the severity of differential heaving that could occur between areas backfilled with non-frost susceptible engineered fill and the adjacent areas underlain by the existing frost susceptible soils.

5.11 Lateral Earth Pressures for Design

The lateral earth pressures acting on the garage/foundation walls will depend on the existing soil conditions, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The details on the wall backfill drainage are provided in Section 5.12 of this report.

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

Where the wall support and structure allow lateral yielding, (e.g., for unrestrained retaining walls), active earth pressures may be used in the design of the wall. Where the support does not allow lateral yielding, (i.e., for the proposed basement walls) at-rest earth pressures should be assumed for design.

If a shored excavation (in overburden) is used as part of the formwork for the wall, the lateral earth pressures for foundation walls are based on the existing retained soils (i.e., fill and silty clay) and the following parameters (unfactored) may be used:

Soil	**Unit Weight (kN/m^3)**	**Coefficients of static lateral earth pressure**	
		Active, Ka	**At rest, Ko**
Existing Fill	21	0.33	0.50
Silty Clay	17	0.36	0.53
Glacial Till	22	0.31	0.47

If the garage/foundation wall is backfilled with granular free draining fill either in a zone with width equal to at least 50 percent of the height of the wall or within the wedge-shaped zone defined by a line drawn at 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing/pile cap/grade beam, the following parameters (unfactored) may be used:

Material	**Unit Weight (kN/m^3)**	**Coefficients of static lateral earth pressure**	
		Active, Ka	**At rest, Ko**
Granular A or Granular B Type II	22	0.27	0.43
Granular B Type I	22	0.31	0.47

Seismic loading will result in increased lateral earth pressures acting on the walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.

The horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the design PGA (i.e., k_h = 0.32). For structures which allow lateral yielding, k_h is taken as 0.5 times the design PGA (i.e., k_h = 0.16).

The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the K_{AE} obtained using the k_h values described above and assumed no vertical acceleration and wall to soil friction. These seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

Wall Type	Site PGA (2475-year Earthquake)	K_{AE}	
		Granular A/Granular B Type II	Granular B Type I
Yielding Wall	0.32g	0.39	0.43
Non-Yielding Wall		0.53	0.59

The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution).

A minimum surcharge pressure of 12 kilopascals due to traffic and compaction induced pressure should be included in the total lateral earth pressures for the structural design of the wall.

The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_a) \gamma (H-d) + q$$

Where: $\sigma_h(d)$ = Lateral earth pressure at depth, d, (kPa);

K_o = Coefficient of static earth pressure;

γ = Unit weight of the backfill soil (kN/m^3); as given previously;

d = Depth below the top of the wall (m);

K_{AE} = Seismic active earth pressure coefficient;

q = Surcharge to account for traffic and compaction pressure, where applicable; and,

H = Total height of the wall (m).

All of the lateral earth pressure equations are given in an unfactored format and will need to be factored for Ultimate Limit States design purposes.

5.12 Permanent Drainage

The measured groundwater depth at the site is variable, but it is generally considered to be between about 2 to 4 metres below existing site grades. To manage the long term groundwater levels and the interaction with the proposed development, a drainage system diverting collected groundwater inflow to the sewer system is recommended. It is recommended that a hydrogeological assessment be completed to provide input toward the volumes of water anticipated to be diverted to the municipal sewer system.

The subfloor drainage system (i.e., below the lowest garage level) may consist of a network of robust sub-drain pipes conveying collected groundwater to a sump or sumps from which the groundwater can be pumped to a municipal sewer. The drainage system would consist of interconnected perforated drain pipes (bedded and backfilled with free draining granular soils) installed around the perimeter and within the building footprint. The capacity of the subfloor drainage system should be modified during construction as required.

Drainage, such as a composite synthetic drainage system or equivalent, should be provided to the exterior walls. The composite drain must withstand the design horizontal earth pressures used for basement wall design, and should be connected to the basement level underslab drainage system. The drainage system collector pipes should drain to a sump for collection and discharge to a sewer.

5.13 Site Servicing

At least 150 millimetres of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface occurs during construction, it may be necessary to place a sub-bedding layer consisting of 300 millimetres of compacted OPSS Granular B Type II beneath the Granular A. The bedding material should, in all cases, extend to the spring line of the pipe and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials and native soils could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from the spring line of the pipe to at least 300 millimetres above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 millimetres. The cover material should be compacted to at least 95 percent of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the existing inorganic fill, weathered silty clay, and glacial till as trench backfill. Where the trench will be covered with hard surfaced areas, the type of material placed in the frost zone (between subgrade level and 1.8 metres depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.14 Pavement Design

In preparation for pavement construction, all topsoil, unsuitable fill, disturbed, or otherwise deleterious materials (i.e., those materials containing organic material) should be removed from the pavement areas. Some of the existing fill could remain provided that it is free of organic matter, and that the subgrade be subjected to a proof roll with a loaded tandem truck to reveal weak or soft areas prior to the construction of the new pavement structure. Soft or weak areas should be removed and repaired with acceptable earth borrow or OPSS Select Subgrade Material (SSM).

Pavement areas requiring grade raising to proposed subgrade level should be brought to grade using acceptable (compactable and inorganic) earth borrow or OPSS SSM. These materials should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the materials standard Proctor maximum dry density using suitable compaction equipment.

The surface of the pavement subgrade should be crowned or sloped to promote drainage of the pavement granular structure towards perimeter swales or subdrains placed at the subgrade level

The following light duty pavement design is recommended for the parking lot for this project, the following heavy duty pavement design is recommended for the bus lane for this project:

Material		Light Duty Pavement Thickness of Pavement Elements (mm)	Heavy Duty Pavement Thickness of Pavement Elements (mm)
Bituminous Concrete OPSS 1150	Superpave 12.5 mm	60	40
	Superpave 19.0 mm	-	50
Granular Material OPSS 1010	Granular A Base	150	150
	Granular B, Type II Subbase	300	450
	Prepared and Approved Subgrade		

The granular base and subbase materials should be uniformly compacted as per OPSS 310, Method A. The asphaltic concrete should be compacted in accordance with the procedures outlined in OPSS 310.

The asphaltic cement should consist of PG 58-34 and the design of the mixes should be based on a Traffic Category B.

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., grade raise fill has been adequately compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.

Where the new pavements will connect to existing pavements, the new pavement structures should be continued at least to the limits of construction, with any longitudinal transitions and/or tapers occurring thereafter. At these locations, the longitudinal transitions should be constructed by cutting the existing pavement structure vertically to the bottom of the existing subbase. The new granular layers should then be tapered up or down, as required, at a slope of 5 horizontal to 1 vertical to match the existing pavement structure. The asphaltic concrete does not need to be tapered between the new construction and the existing pavement. However, the asphaltic concrete of the existing pavement should be milled back an additional 300 millimetres to a depth of about 60 millimetres in areas where its thickness is greater than 100 millimetres, or matching the proposed surface course of the new asphaltic concrete. A tack coat should be provided and the new surface course asphaltic concrete placed over the milled surface to form the new pavement joint. Where the existing pavement is less than 100 millimetres, then a butt joint on a vertical saw cut surface is acceptable. A tack coat should be placed on the vertical saw cut surface. The tack coat should be in accordance with the City SP F-3107.

5.15 Corrosion and Cement Type

Two samples of soil, one from each boreholes 18-01 and 18-03 were submitted to EXOVA Laboratories Ltd. for chemical analysis related to potential corrosion of exposed buried steel and concrete elements (corrosion and sulphate attack). The results of this testing are provided in Appendix D. The results indicate that concrete made with Type GU Portland cement should be acceptable for concrete substructures.

The results also indicate a very high potential for corrosion of buried ferrous elements, which should be considered in the design of substructures and pile foundations.

6.0 ADDITIONAL CONSIDERATIONS

At the time of writing this report, only conceptual details related to the Phase 1 building were available. This information suggests this building will consist of a 20+ storey tower with two garage levels to be located at the southeast corner of the property. Golder Associates should review the final drawings and specifications for this project prior to tendering to confirm that the guidelines in this report have been adequately interpreted.

The construction activities could impact the existing adjacent structures and buildings. Appropriate damage assessments (pre and post condition surveys for example) should be carried out as necessary.

During construction, sufficient foundation inspections, subgrade inspections, in-situ density tests, materials testing, pile and rock anchor installation monitoring should be carried out to confirm that the conditions exposed are consistent with those encountered in the boreholes, and to monitor conformance to the pertinent project specifications. Concrete testing should be carried out in a CCIL certified laboratory.

The soils at this site are sensitive to disturbance from ponded water, construction traffic and frost. All bearing surfaces must be inspected by Golder prior to filling or concreting to ensure that strata having adequate bearing capacity have been reached and that the bearing surfaces have been properly prepared.

7.0 CLOSURE

We trust that this report provides sufficient geotechnical engineering information to facilitate the design of this project. If you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact this office.

Golder Associates Ltd.

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Geotechnical Engineer

Nicolas Leblanc, P.Eng.
Senior Geotechnical Engineer

CRG/SG/mvrd
https://golderassociates.sharepoint.com/sites/30869g/deliverables/geotechnical report/final report/18106595-001-r-rev0-westgate mall phase 1 geotechnical report-november 2018.docx

IMPORTANT INFORMATION AND LIMITATIONS
OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, **RioCan Holdings Inc.**. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS
OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

KEY MAP

SCALE 1:25,000

LEGEND

⊕	BOREHOLE LOCATION, CURRENT INVESTIGATION
⊕	APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION (1522569)
⊕	APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION (782032)
⊕	APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION (762168)
⊕	APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION (752031)
⊕	APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION (70794)
▭	PHASE I SITE
▭	PROPOSED PHASE I BUILDING FOOTPRINT

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

1. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2016
2. SERVICE LAYER CREDITS: SOURCES: ESRI, HERE, GARMIN, USGS, INTERMAP, INCREMENT P, NRCAN, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), ESRI KOREA, ESRI (THAILAND), NGCC, © OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY
3. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28

0 5 10 20

1:600 METRES

CLIENT

RIOCAN MANAGEMENT INC.

PROJECT

DETAILED DESIGN GEOTECHNICAL INVESTIGATION PROPOSED SITE REDEVELOPMENT □ PHASE I WESTGATE MALL SITE,

TITLE

SITE PLAN

CONSULTANT

GOLDER

YYYY-MM-DD	2018-11-01
DESIGNED	SG
PREPARED	BR
REVIEWED	SG
APPROVED	NRL

PROJECT NO.	CONTROL	REV.	FIGURE
18106595	0001	0	1

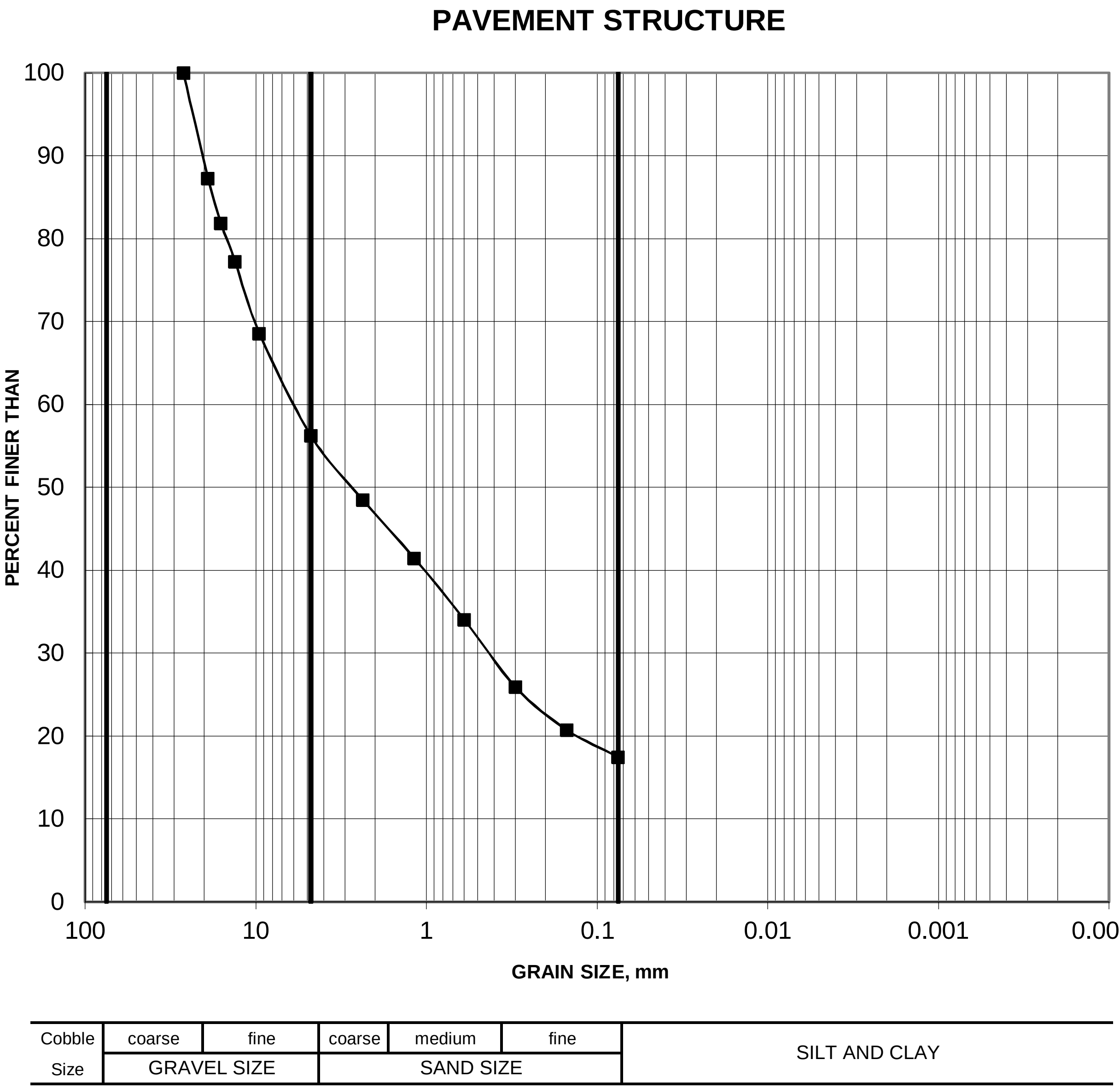
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM:

25mm

0

Path: N:\Active\Spatial_IM\RioCan_Management_Inc\Westgate\99_PROJ\18106595_RioCAN_DetDesGeoTInv\40_PROD\0001_BG_Geotech\18106595-0001-BG-0001.mxd

GRAIN SIZE DISTRIBUTION	FIGURE 2



Borehole	Sample	Depth (m)
—■— 18-03	1	0.15-0.46

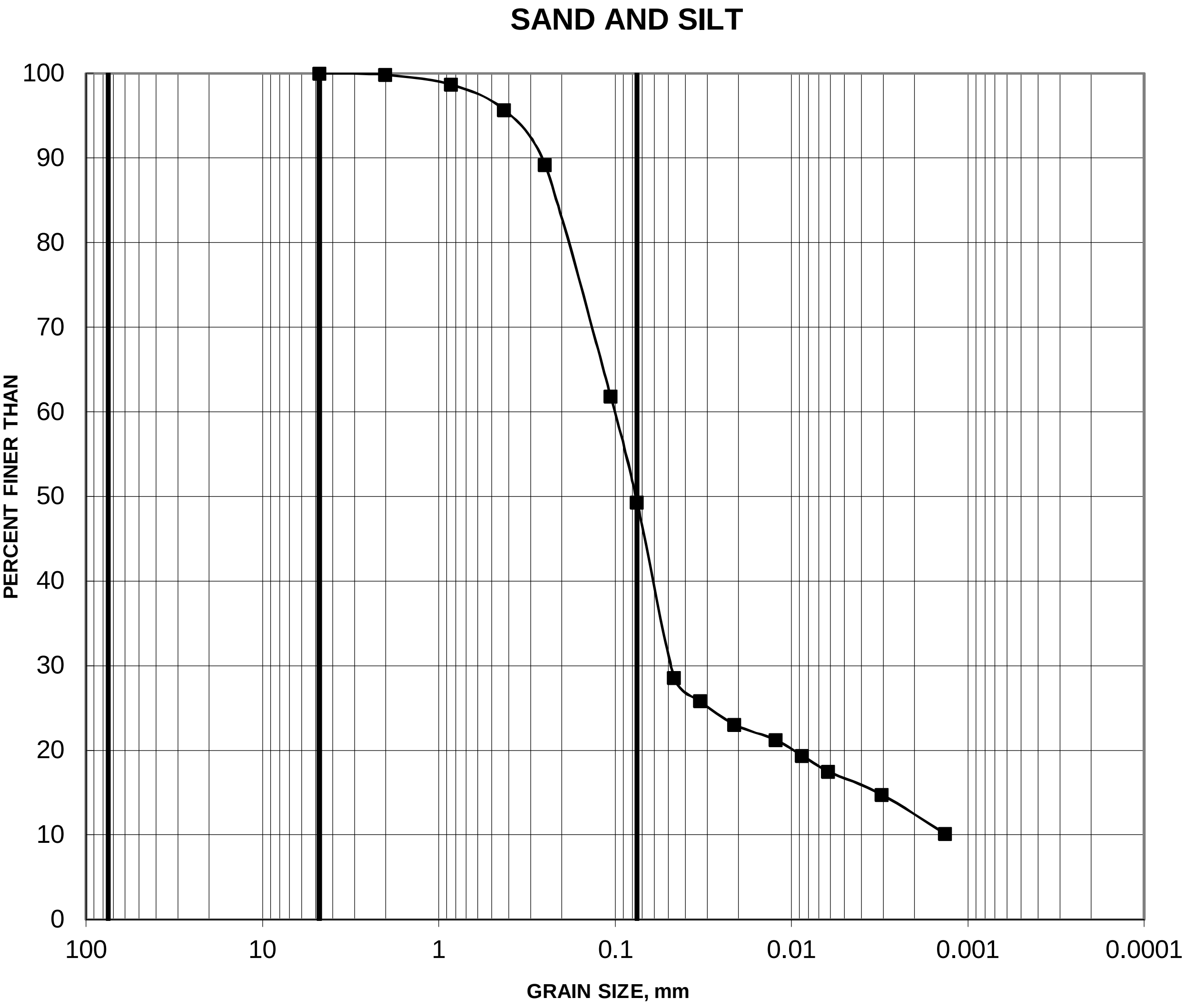
Project: 18106595

Golder Associates

Created by: MI
Checked by: CW

GRAIN SIZE DISTRIBUTION

FIGURE 3



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
■ 18-03	4	2.29-2.90

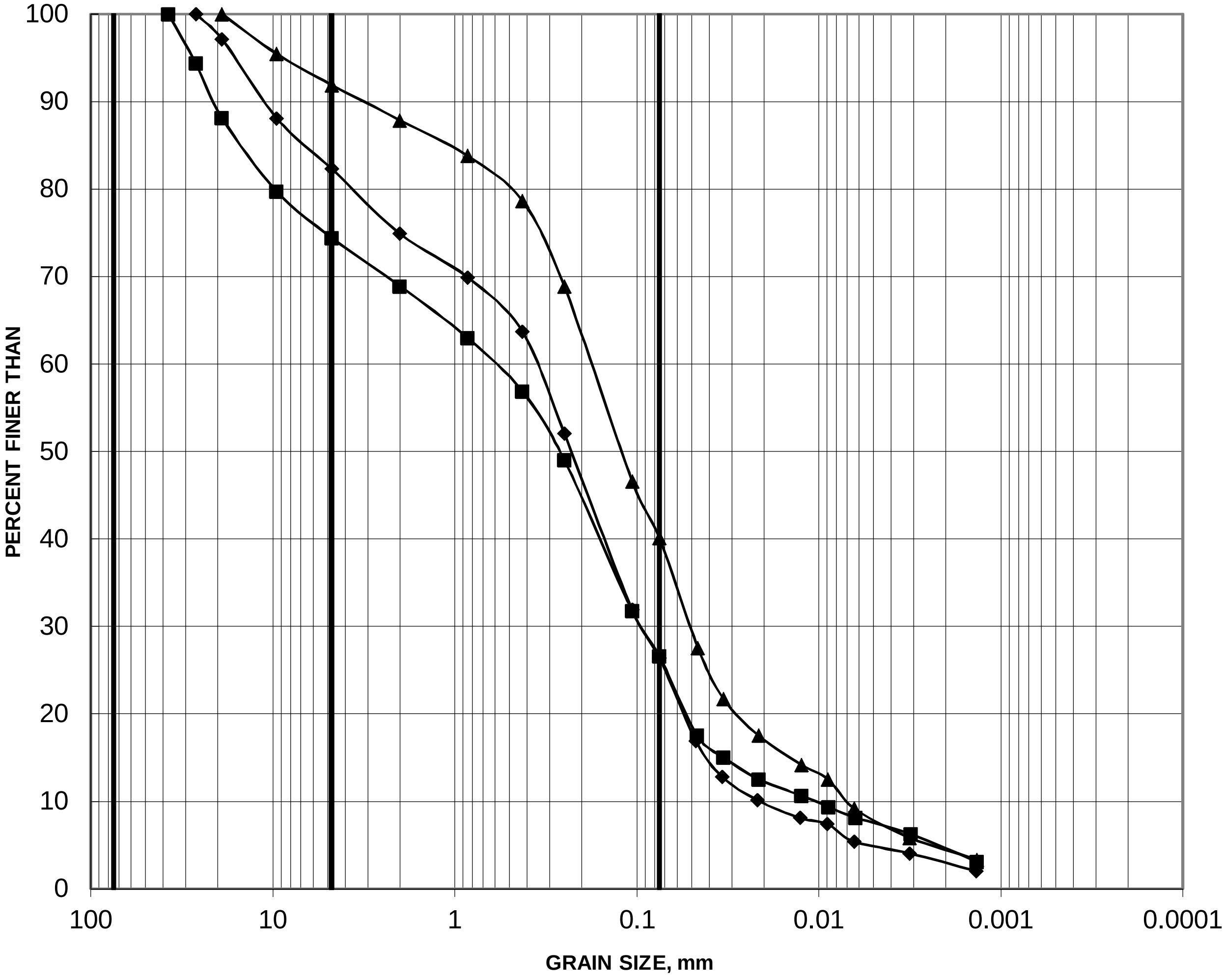
Project: 18106595

Golder Associates

Created by: MI
Checked by: CW

GRAIN SIZE DISTRIBUTION	FIGURE 4

GLACIAL TILL



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
─■─ 18-01	9	6.86-7.47
─◆─ 18-01	12	9.14-9.75
─▲─ 18-02	8	5.33-5.94

Project: 18106595

Golder Associates

Created by: MI
Checked by: CW

APPENDIX A

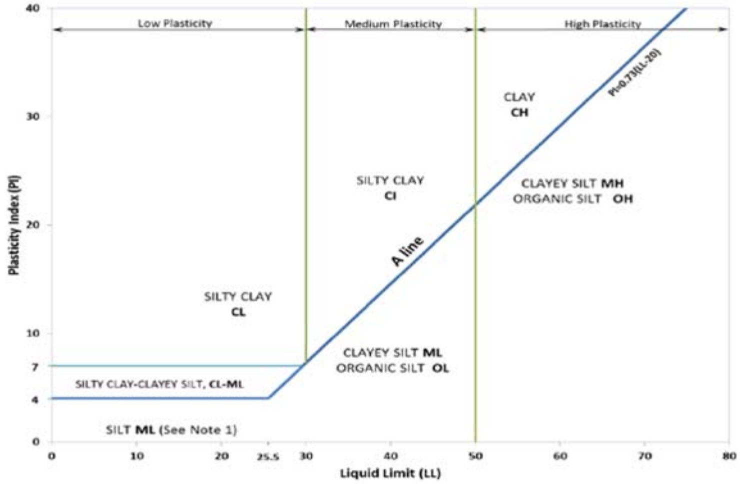
Borehole Logs – Current Investigation

METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators					Organic Content	USCS Group Symbol	Primary Name
				Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)			
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% **(see Note 2)**	CL	SILTY CLAY
			Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat							75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.
Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.
For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.
For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.
A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BORHEOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

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.) split-spoon sampler for a distance of 300 mm (12 in.).

Cone Penetration Test (CPT)
An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); 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

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
FS	Foil sample
GS	Grab Sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size
TP	Thin-walled, piston – note size
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , 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, Gs)
DS	direct shear test
GS	specific gravity
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 (FV)	field vane (LV-laboratory vane test)
γ	unit weight

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

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 - 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects.
2. Definition of compactness terms are based on SPT-'N' ranges as provided in Terzaghi, Peck and Mesri (1996) and correspond to typical average N_{60} values. Many factors affect the recorded SPT-'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), groundwater conditions, and grainsize. As such, the recorded SPT-'N' value(s) should be considered only an approximate guide to the compactness term. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

2 shear strength = (compressive strength)/2

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

PROJECT: 18106595-1000

RECORD OF BOREHOLE: 18-01

SHEET 1 OF 3

LOCATION: N 5027715.7 ;E 364816.5

BORING DATE: September 11, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | SAMPLES | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80) | SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ (20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}) | WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

ELEV. DEPTH (m)	DESCRIPTION
74.74	GROUND SURFACE
0.00	ASPHALTIC CONCRETE
0.08	FILL - (GW) sandy GRAVEL, angular; grey brown (PAVEMENT STRUCTURE)
74.21 0.53	FILL - (SP) SAND, medium to fine, trace gravel; brown; non-cohesive, moist
73.83 0.91	FILL - (CL) SILTY CLAY, some sand and gravel; grey brown, contains organic matter; non-cohesive, moist, loose
72.45 2.29	(CI/CH) SILTY CLAY to CLAY; grey brown, contains organic matter (rootlets) (WEATHERED CRUST); cohesive, w>PL, very stiff
71.69 3.05	(CI/CH) SILTY CLAY to CLAY; grey brown, contains clayey silt and sandy silt seams; cohesive, w>PL, stiff
70.78 3.96	(SM) gravelly SILTY SAND; grey to grey brown, contains cobbles and boulders (GLACIAL TILL); non-cohesive, wet, very dense to compact

NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	GRAB	-	
2	SS	8	
3	SS	6	
4	SS	7	
5	SS	1	
6	SS	58	
7	SS	45	
8	SS	27	
9	SS	26	MH
10	SS	>50	
11	SS	69	
12	SS	53	MH
13	SS	>50	

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: PAH
CHECKED: CRG

PROJECT: 18106595-1000

RECORD OF BOREHOLE: 18-01

SHEET 2 OF 3

LOCATION: N 5027715.7 ;E 364816.5

BORING DATE: September 11, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	SOIL PROFILE STRATA PLOT	SOIL PROFILE ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10		Borehole continued on RECORD OF DRILLHOLE 18-01	▨	64.63 10.11							
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: PAH

CHECKED: CRG

PROJECT: 18106595-1000

RECORD OF DRILLHOLE: 18-01

SHEET 3 OF 3

LOCATION: N 5027715.7 ;E 364816.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 11, 2018

DRILL RIG: CME 75

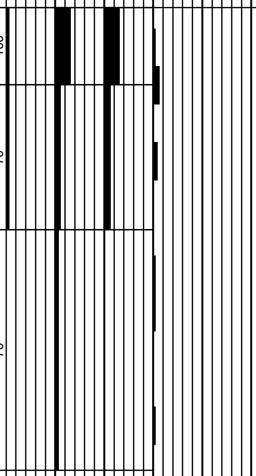
DRILLING CONTRACTOR: Downing Drilling

DATUM: CGVD28

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.
		BEDROCK SURFACE		64.63														
	Rotary Drill NQ3 Core	Fresh, thinly to medium bedded, light to medium grey, fine grained, strong LIMESTONE, with black shale partings		10.11	1	100												
11					2	70												
12					3	70												
13				61.57 13.17														
		End of Drillhole																
14																		
15																		
16																		
17																		
18																		
19																		
20																		



DEPTH SCALE
1 : 50

GOLDER

LOGGED: PAH
CHECKED: CRG

MIS-RCK 004 18106595.GPJ GAL-MISS.GDT 11/1/18 ZS

PROJECT: 18106595-1000

RECORD OF BOREHOLE: 18-02

SHEET 1 OF 2

LOCATION: N 5027756.6 ;E 364838.0

BORING DATE: September 12, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES: 0 – 10

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 20 40 60 80

PIEZOMETER OR STANDPIPE INSTALLATION

SOIL PROFILE

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	74.81
ASPHALTIC CONCRETE	0.03
FILL - (SW) gravelly SAND; grey (PAVEMENT STRUCTURE)	0.13
FILL - (SW/GW) SAND and GRAVEL, angular, some non-plastic fines; brown; non-cohesive	74.40 0.41
FILL - (CL) SILTY CLAY, some gravel; grey brown, contains organic matter and wood pieces; non-cohesive, moist, compact	
(CI/CH) SILTY CLAY to CLAY; grey brown, fissured, contains organic matter (rootlets) (WEATHERED CRUST); cohesive, w~PL, very stiff to stiff	72.68 2.13
(CI/CH) SILTY CLAY to CLAY; grey brown, contains sandy silt seams; cohesive, w>PL, firm	71.76 3.05
(SM/ML) SAND and SILT, some gravel; grey brown to grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive, wet, dense to compact	70.90 3.91
Borehole continued on RECORD OF DRILLHOLE 18-02	67.75 7.06

SAMPLES

NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	GRAB	-	
2	SS	10	
3	SS	11	
4	SS	7	
5	SS	2	
6	SS	30	
7	SS	43	
8	SS	15	MH
9	SS	21	
10	SS	>50	

∇

DEPTH SCALE

1 : 50

GOLDER

LOGGED: PAH

CHECKED: CRG

PROJECT: 18106595-1000

RECORD OF DRILLHOLE: 18-02

SHEET 2 OF 2

LOCATION: N 5027756.6 ;E 364838.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 12, 2018

DRILL RIG: CME 75

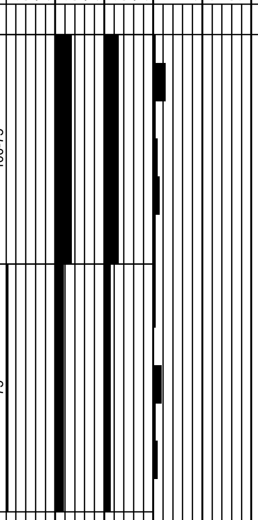
DRILLING CONTRACTOR: Downing Drilling

DATUM: CGVD28

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		BEDROCK SURFACE		67.75															
	Rotary Drill NQ3 Core	Fresh, thinly to medium bedded, grey, fine grained, strong LIMESTONE, with black shale partings		7.06	1	100-75													
				64.59 10.22	2	75													
		End of Drillhole																	WL in open borehole at 6.10 m depth upon completion of drilling

Depth scale: 8, 9, 10, 11, 12, 13, 14, 15, 16, 17



DEPTH SCALE

1 : 50

GOLDER

LOGGED: PAH

CHECKED: CRG

PROJECT: 18106595

RECORD OF BOREHOLE: 18-03

SHEET 1 OF 1

LOCATION: N 5027755.0 ;E 364770.9

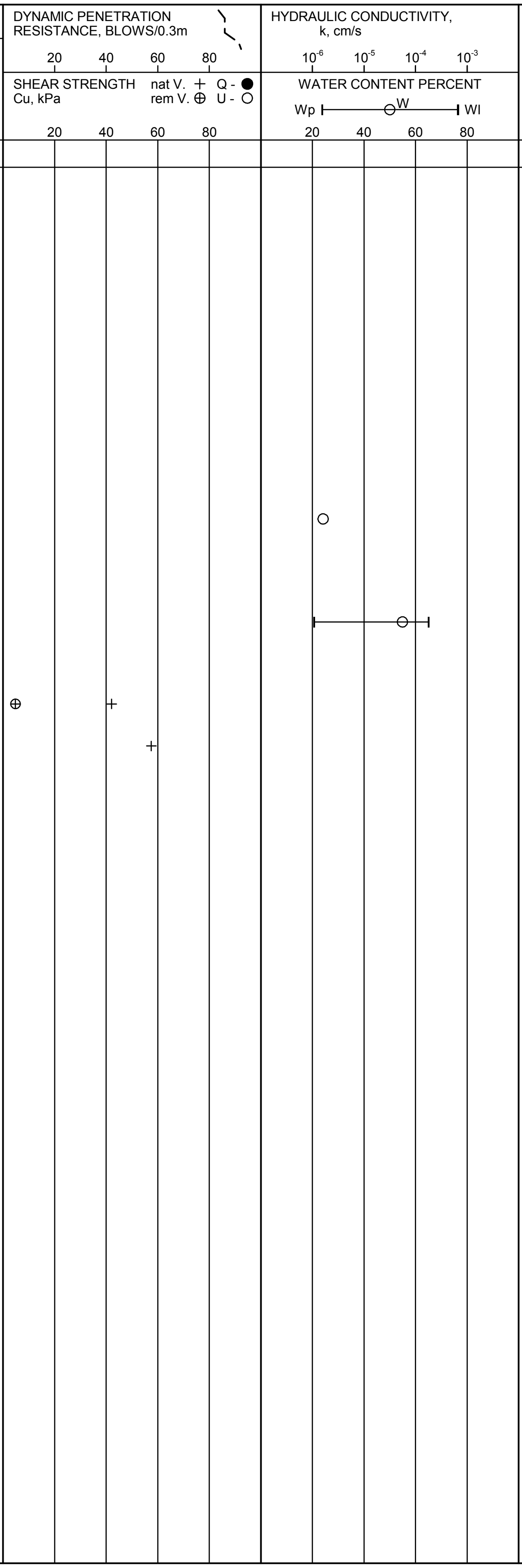
BORING DATE:

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	74.13					
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	0.00					
		FILL - (SM/GM) SILTY SAND and GRAVEL, angular and subrounded; brown (PAVEMENT STRUCTURE)	0.12	1	GRAB	-	M	
		FILL - (CL) SILTY CLAY; grey brown, contains sand seams; cohesive	73.37 0.76					
1		(CI/CH) SILTY CLAY to CLAY; grey brown, fissured (WEATHERED CRUST); cohesive, w>PL, very stiff	0.91	2	SS	8		
2				3	SS	10		
		(SM/ML) SAND and SILT; grey brown, contains shells; non-cohesive, wet, very loose	71.69 2.44	4	SS	2	MH	
3		(CI/CH) SILTY CLAY to CLAY; grey brown, slightly fissured; cohesive, w>PL, firm to stiff	71.23 2.90	5	SS	1		
4								
5		(SM/ML) SAND and SILT, some gravel to gravelly; grey, contains cobbles and boulders (GLACIAL TILL); wet, compact to loose	69.25 4.88	6	SS	3		∇
6				7	SS	17		
				8	SS	4		
7		End of Borehole	67.42 6.71					WL in open borehole at 5.00 m depth upon completion of drilling
8								
9								
10								



DEPTH SCALE

1 : 50

GOLDER

LOGGED:

CHECKED: CRG

APPENDIX B

Borehole Logs - Previous Investigation

PROJECT: 1522569-17000

RECORD OF BOREHOLE: 15-09

SHEET 1 OF 3

LOCATION: N 5026180.8 ;E 442535.7

BORING DATE: October 13, 2015

DATUM: Geodetic

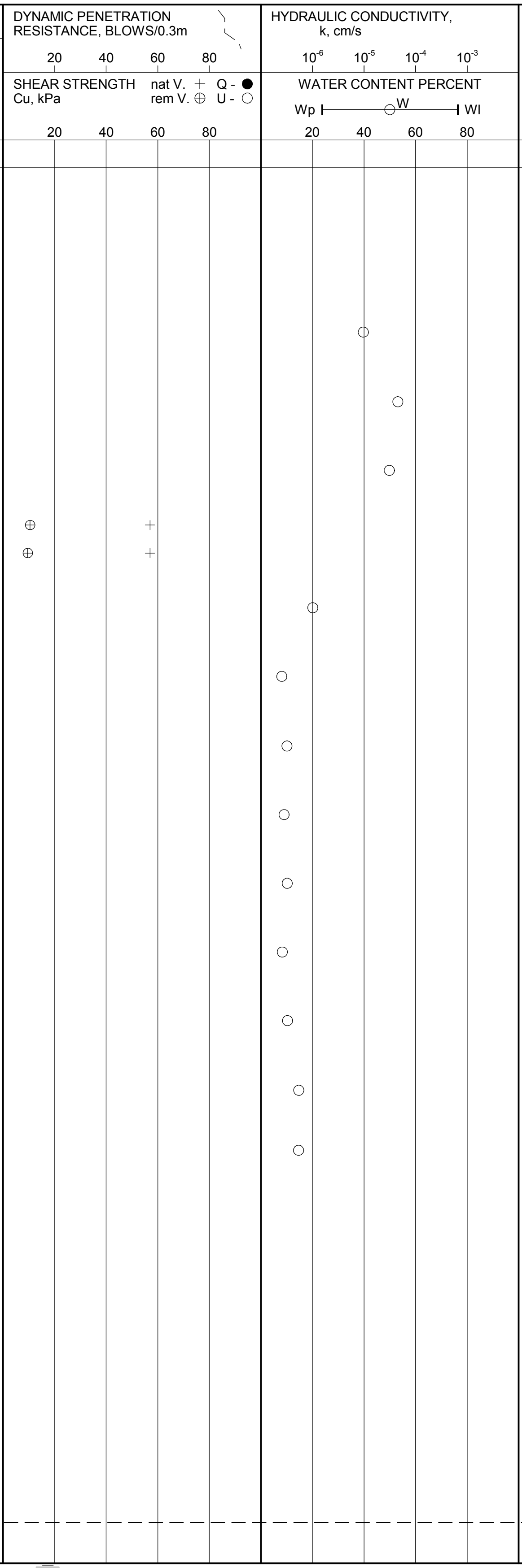
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)
	GROUND SURFACE	74.54
Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	0.00
	FILL - (SW) gravelly SAND, some silt; brown, (PAVEMENT STRUCTURE); non-cohesive, moist, compact	0.18
	FILL - (CL) SILTY CLAY, some sand and gravel; grey brown, with red oxidation; cohesive, w>PL	73.78 0.76
	(CI/CH) SILTY CLAY; grey brown, (Weathered Crust); cohesive, w>PL, stiff to very stiff	72.91 1.63
	(CI/CH) SILTY CLAY; grey; cohesive, w>PL, stiff	70.88 3.66
	(SC) gravelly CLAYEY SAND; grey, (GLACIAL TILL); cohesive, w>PL, stiff to very stiff	69.97 4.57
	(ML) sandy SILT; some gravel; grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive, wet, loose	68.44 6.10
	(SM) SILTY SAND, some gravel; grey, contains cobbles and boulders, (GLACIAL TILL); non-cohesive, wet, compact	66.92 7.62
	(SM) gravelly SILTY SAND, some low to none plastic fines; grey, contains cobbles and boulders, (GLACIAL TILL); non-cohesive, wet, dense	65.55 8.99
	(SW) SAND, some gravel, trace low to none plastic fines; grey, contains cobbles and boulders, (GLACIAL TILL); non-cohesive, wet, compact to dense	64.63 9.91
Rotary Drill NQ Core	(SM) SILTY SAND, some gravel; grey, contains cobbles and boulders, (GLACIAL TILL); non-cohesive, wet	62.96 11.58

CONTINUED NEXT PAGE

NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	SS	16	
2	SS	12	
3	SS	8	
4	SS	11	
5	SS	2	
6	SS	12	
7	SS	9	M
8	SS	8	
9	SS	5	
10	SS	25	
11	SS	25	
12	SS	37	M
13	SS	11	
14	SS	>30	
C1	NQ RC	DD	
C2	NQ RC	DD	
C3	NQ	DD	



PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE
1 : 75

LOGGED: AT
CHECKED: NRL

PROJECT: 1522569-17000

RECORD OF BOREHOLE: 15-09

SHEET 2 OF 3

LOCATION: N 5026180.8 ;E 442535.7

BORING DATE: October 13, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} / WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---								
15	Rotary Drill NQ Core	(SM) SILTY SAND, some gravel; grey, contains cobbles and boulders, (GLACIAL TILL); non-cohesive, wet		C3	RC NQ RC	DD				
16			58.26 16.28							
17		FRESH, thinly to medium bedded, dark grey, strong to very strong, fine grained LIMESTONE BEDROCK, with shale partings and occassional nodular sections		C4	NQ RC	DD			UCS= 04 MPa	
18				C5	NQ RC	DD				
19				C6	NQ RC	DD				
20			54.20							
		End of Borehole	20.34							
21										
22										
23										
24										
25										
26										
27										
28										
29										
30										

MIS-BHS 001 1522569-17001.GPJ GAL-MIS.GDT 12/01/15 JEM

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: AT

CHECKED: NRL

PROJECT: 1522569-17000

RECORD OF DRILLHOLE: 15-09

SHEET 3 OF 3

LOCATION: N 5026180.8 ;E 442535.7

DRILLING DATE: October 13, 2015

DATUM: Geodetic

DRILL RIG:

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR:

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.
		BEDROCK SURFACE		62.96 11.58															
12	Rotary Drill NQ Core	(SM) SILTY SAND, some gravel; grey, contains cobbles and bouders, (GLACIAL TILL); non-cohesive, wet			C1	100													
13																			
14					C2	100													
15					C3	100													
16					C4A	100													
		FRESH, thinly to medium bedded, dark grey, strong to very strong, fine grained LIMESTONE BEDROCK, with shale partings and occassional nodular sections		58.26 16.28															
17					C4B	100													
18					C5	100													
19					C6	100													
20																			
		End of Drillhole		54.20 20.34															
21																			
22																			
23																			
24																			
25																			
26																			

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: AT

CHECKED: NRL

PROJECT: 1522569-17000

RECORD OF BOREHOLE: 15-10

SHEET 1 OF 1

LOCATION: N 5026186.1 ;E 442607.2

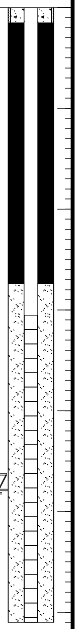
BORING DATE: October 8, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} / WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		74.86							
0	Geoprobe 76 mm Diam. (Direct Push)	ASPHALTIC CONCRETE		0.10							Flush Mount Protective Casing
		FILL - (SW/GW) SAND and GRAVEL; brown, (PAVEMENT STRUCTURE)		74.30 0.56							
1		FILL - (CL) SILTY CLAY; grey to black; cohesive, w>PL		73.64 1.22	1	50 DT	PH				
2		FILL - (CL) SILTY CLAY, trace gravel; grey brown; cohesive, w>PL		72.42 2.44	2	50 DT	PH				Bentonite Seal
3		(CI/CH) SILTY CLAY; grey; cohesive, w>PL		71.20 3.66	3	50 DT	PH				Silica Sand
4		(SM) SILTY SAND, some gravel; grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive; moist		68.76 6.10	4	50 DT	PH				32 mm Diam. PVC #10 Slot Screen ∇
5					5	50 DT	PH				
6		End of Borehole									W.L. in Screen at Elev. 70.06 m on October 14, 2015 W.L. in Screen at Elev. 70.08 m on November 9, 2015
7											
8											
9											
10											
11											
12											
13											
14											
15											



DEPTH SCALE

1 : 75

Golder Associates

LOGGED: AT

CHECKED: NRL

Form G.A.-D-1.

Project No. 782032

RECORD OF BOREHOLES 1 & 2

LOCATION See Figure 2 BORING DATE MAR. 9 & 10, 1978 DATUM LOCAL

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

SHEAR STRENGTH Cu., LB./SQ. FT. 500 1000 1500 2000 NAT. V. - + Q. - ● REM.V. - ⊕ U. - O

COEFFICIENT OF PERMEABILITY, k., CM./SEC. 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT Wp W WL

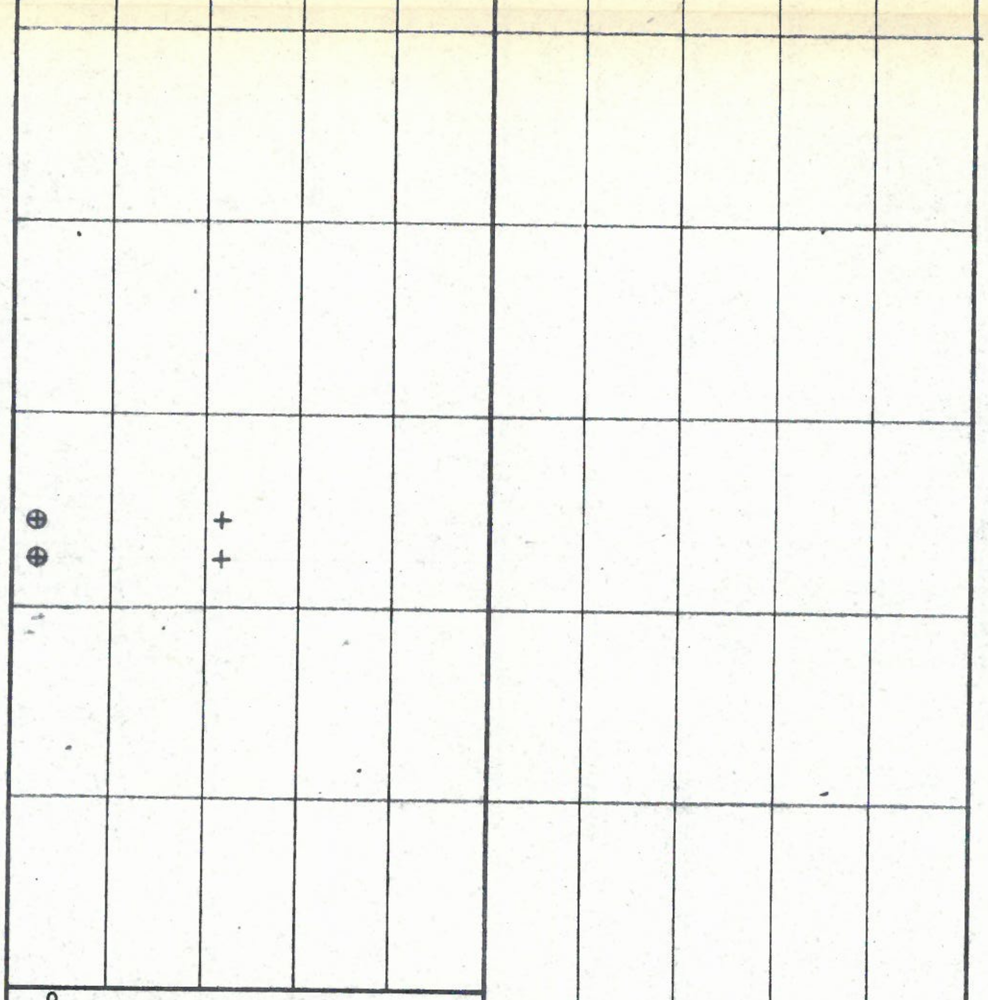
ADDITIONAL LAB. TESTING

BH. 1

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE	PIEZOMETER OR STANDPIPE INSTALLATION
	97.8	GROUND SURFACE						
POWER AUGER 8" DIAM. (HOLLOW STEM)	0.0	ASPHALT	■					
	0.4	BROWN SAND AND GRAVEL (FILL)	▨					
	95.0 2.8	STIFF GREY BROWN SANDY SILTY CLAY, SOME GRAVEL AND ORGANIC MATTER, OCCASIONAL COBBLE (FILL)	▨	1	2" D.O	10	95	
	90.7 7.1	VERY STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)	🟩	2	"	4	90	OPEN HOLE DRY TO ELEV. 81.3, MAR. 10, 1978
				3	"	1		
	85.3 12.5	VERY LOOSE GREY SANDY SILT TILL	🟨	4	"	1	85	
	84.3 13.5	VERY DENSE GREY SILTY SAND, SOME GRAVEL, TRACE CLAY (TILL)	🟨			84		
				5	"	61		
	81.3 16.5	END OF HOLE					80	

BH. 2

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.	ELEVATION SCALE	PIEZOMETER OR STANDPIPE INSTALLATION
							100	
	98.2	GROUND SURFACE						
POWER AUGER 8" DIAM. (HOLLOW STEM)	0.0	ASPHALT	■					
	0.4	BROWN SAND AND GRAVEL (FILL)	▨					
	95.5 2.7	STIFF GREY BROWN SILTY CLAY, SOME SAND, GRAVEL, COBBLES AND ORGANIC MATTER (FILL)	▨				95	
	92.7 5.5	VERY STIFF TO STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)	🟩	1	2" D.O	8		OPEN HOLE DRY TO ELEV. 85.7 MAR. 10, 1978
				2	"	4	90	
	86.2 12.0	GREY GLACIAL TILL	🟨					
	12.5	END OF HOLE					85	
							80	



15 0 5 10 Percent axial strain at failure

VERTICAL SCALE 1 IN. TO 5 FT.

Golder Associates

DRAWN DN

CHECKED

Form G.A.-D-I.

Project No. 782032

RECORD OF BOREHOLES 3 & 4

LOCATION See Figure 2

BORING DATE MAR. 10, 1978

DATUM LOCAL

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

DYNAMIC PENETRATION RESISTANCE, BLOWS/FT. 20 40 60 80

SHEAR STRENGTH Cu., LB./SQ.FT. 500 1000 1500 2000

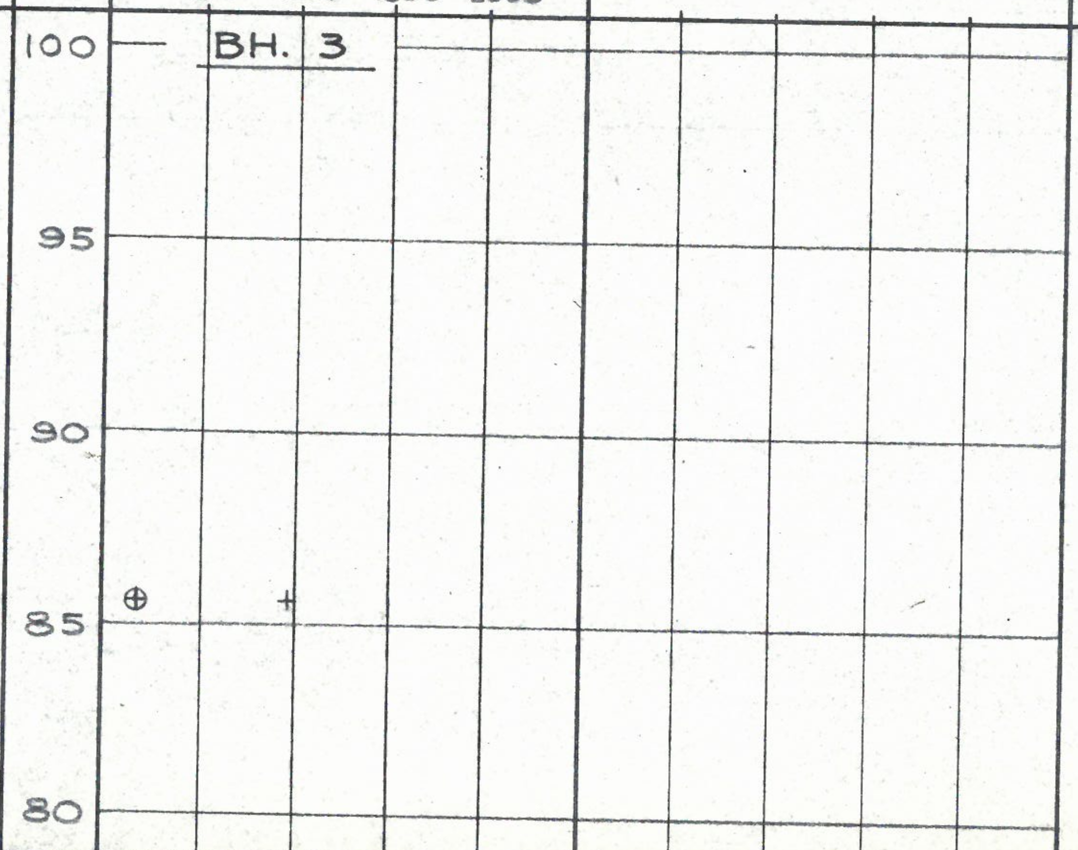
NAT. V. - + Q. - ●
REM.V. - ⊕ U. - ○

COEFFICIENT OF PERMEABILITY, k., CM./SEC. Ix10 Ix10 Ix10 Ix10

WATER CONTENT, PERCENT Wp W WL

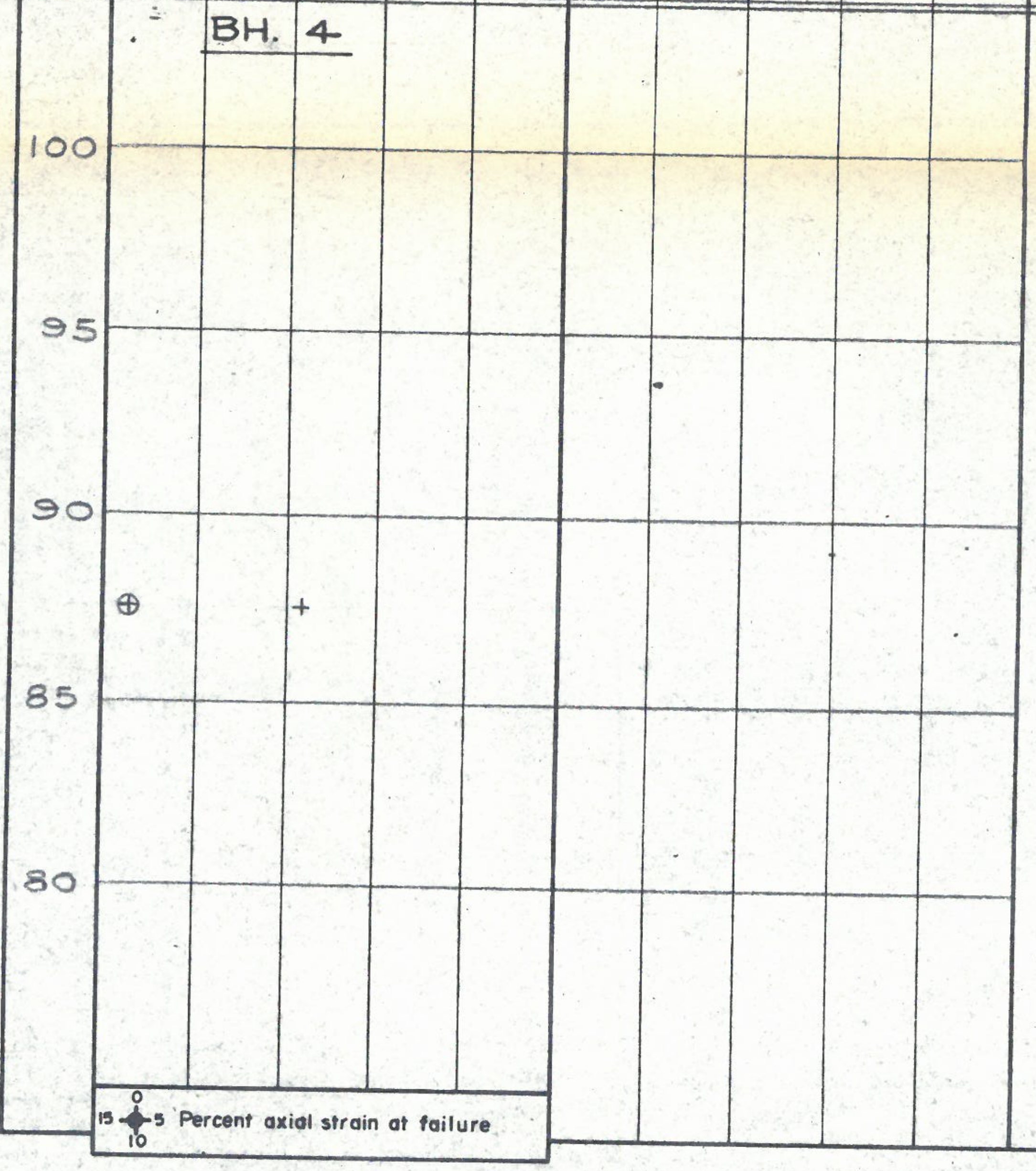
BH. 3

BORING METHOD	ELEV'N. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FT.	PIEZOMETER OR STANDPIPE INSTALLATION
	98.1	GROUND SURFACE				
POWER AUGER 8" DIAM. (HOLLOW STEM)	0.0	ASPHALT				
	0.4	BROWN SAND AND GRAVEL (FILL)				
	95.2 2.9	STIFF GREY BROWN SILTY CLAY, SOME ORGANIC MATTER TRACE GRAVEL AND COBBLES (FILL)	1	2" D.O.	6	OPEN HOLE DRY TO ELEV. 85.1 MAR. 10, 1978
			2	"	6	
	89.2 8.9	VERY STIFF TO STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)	3	"	2	
	85.6 12.5	GREY GLACIAL TILL				
	13.0	END OF HOLE				



BH. 4

BORING METHOD	ELEV'N. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FT.	PIEZOMETER OR STANDPIPE INSTALLATION
	98.6	GROUND SURFACE				
POWER AUGER 8" DIAM. (HOLLOW STEM)	0.0	ASPHALT				
	0.4	BROWN SAND AND GRAVEL (FILL)				
	95.8 2.8	STIFF GREY BROWN SILTY CLAY, SOME ORGANIC MATTER TRACE GRAVEL (FILL)	1	2" D.O.	6	OPEN HOLE DRY TO ELEV. 86.6 MAR. 10, 1978
	92.6 6.0	VERY STIFF TO STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)	2	"	8	
			3	"	3	
	87.1 11.5	GREY GLACIAL TILL				
	12.0	END OF HOLE				



15 0 5 10 Percent axial strain at failure

VERTICAL SCALE
1 IN. TO 5 FT.

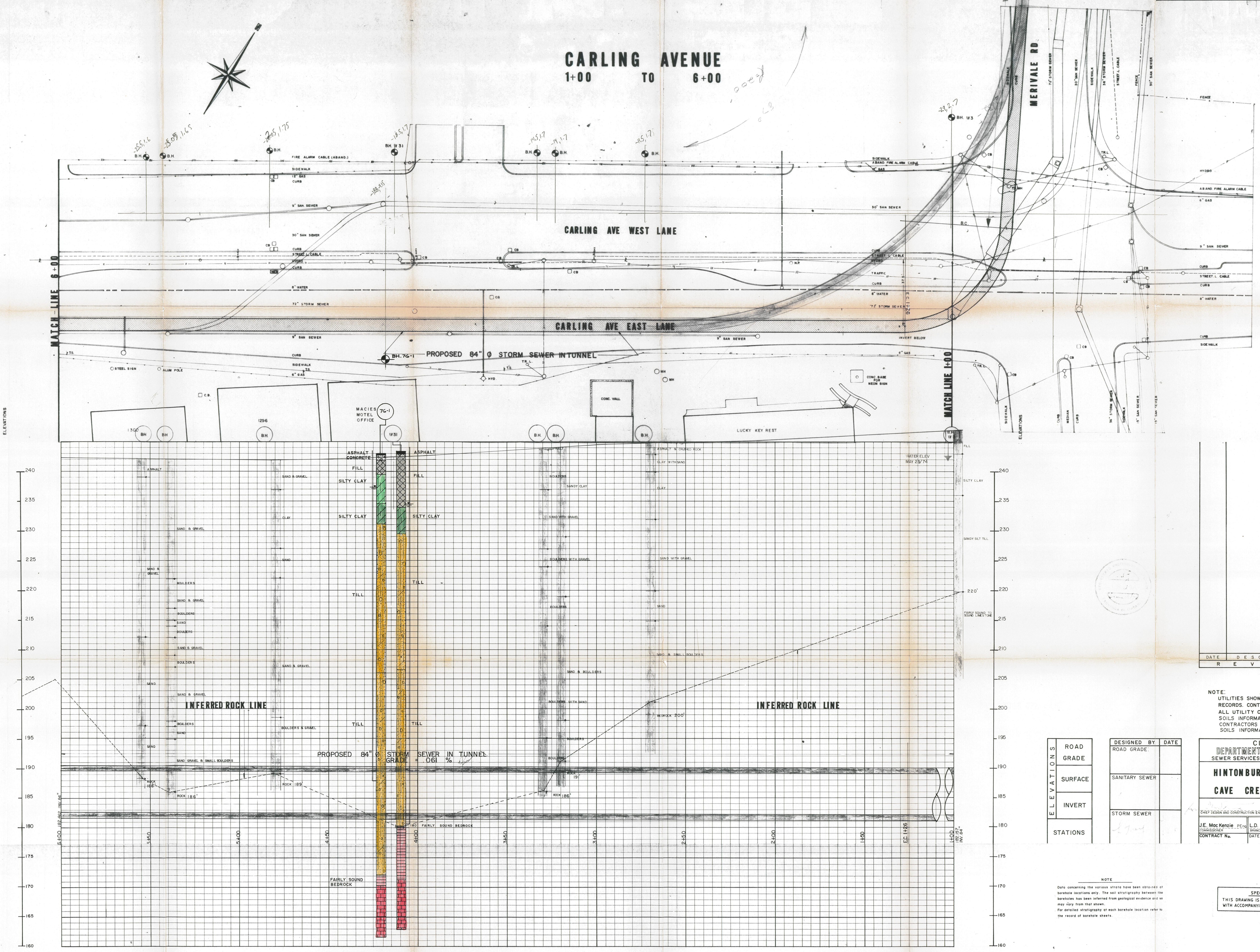
Golder Associates

DRAWN DN

CHECKED

BORING PLAN AND SOIL PROFILE

FIGURE 1



DATE	DESCRIPTION	DRAWN BY	APPROVED BY

REVISIONS

NOTE:
UTILITIES SHOWN ARE TAKEN FROM BEST AVAILABLE RECORDS. CONTRACTOR IS REQUESTED TO CHECK WITH ALL UTILITY COMPANIES BEFORE DIGGING.
SOILS INFORMATION SHOWN IS NOT GUARANTEED AND CONTRACTORS ARE ADVISED TO COLLECT ADDITIONAL SOILS INFORMATION AS DEEMED NECESSARY.

ELEVATIONS	
ROAD GRADE	
SURFACE	
INVERT	
STATIONS	

DESIGNED BY	DATE
ROAD GRADE:	
SANITARY SEWER	
STORM SEWER	

CITY OF OTTAWA
DEPARTMENT OF PHYSICAL ENVIRONMENT
SEWER SERVICES AND POLLUTION CONTROL BRANCH
1+00 to 6+00
HINTONBURGH WEST STORM SEWER
STAGE VI
CAVE CREEK STORM EXTENSION

CHIEF DESIGN AND CONSTRUCTION ENGINEER

J.E. Mac Kenzie P.Eng
COMMISSIONER

L.D. Smith P.Eng
BRANCH DIRECTOR

CONTRACT No.

DATE. OCT 75

DESIGNED BY. L. GRAY

DRAWN BY. VAN VELTHUIZEN

CHECKED BY.

SURVEY BOOKS: 3691

SCALES:
HOR. 1"=20'
VERT. 1"=6'

PLAN No.: 1362

SHEET 7 of 18

NOTE

Data concerning the various strata have been obtained at borehole locations only. The soil stratigraphy between the boreholes has been inferred from geological evidence and so may vary from that shown.
For detailed stratigraphy at each borehole location refer to the record of borehole sheets.

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

Date OCT. 25, 1976

Golder Associates

Drawn D.N.
Chkd.
Appd.

Forr G.A.-D-1.

Project No. 752031

RECORD OF BOREHOLE W-31

752031

LOCATION See Figure 1

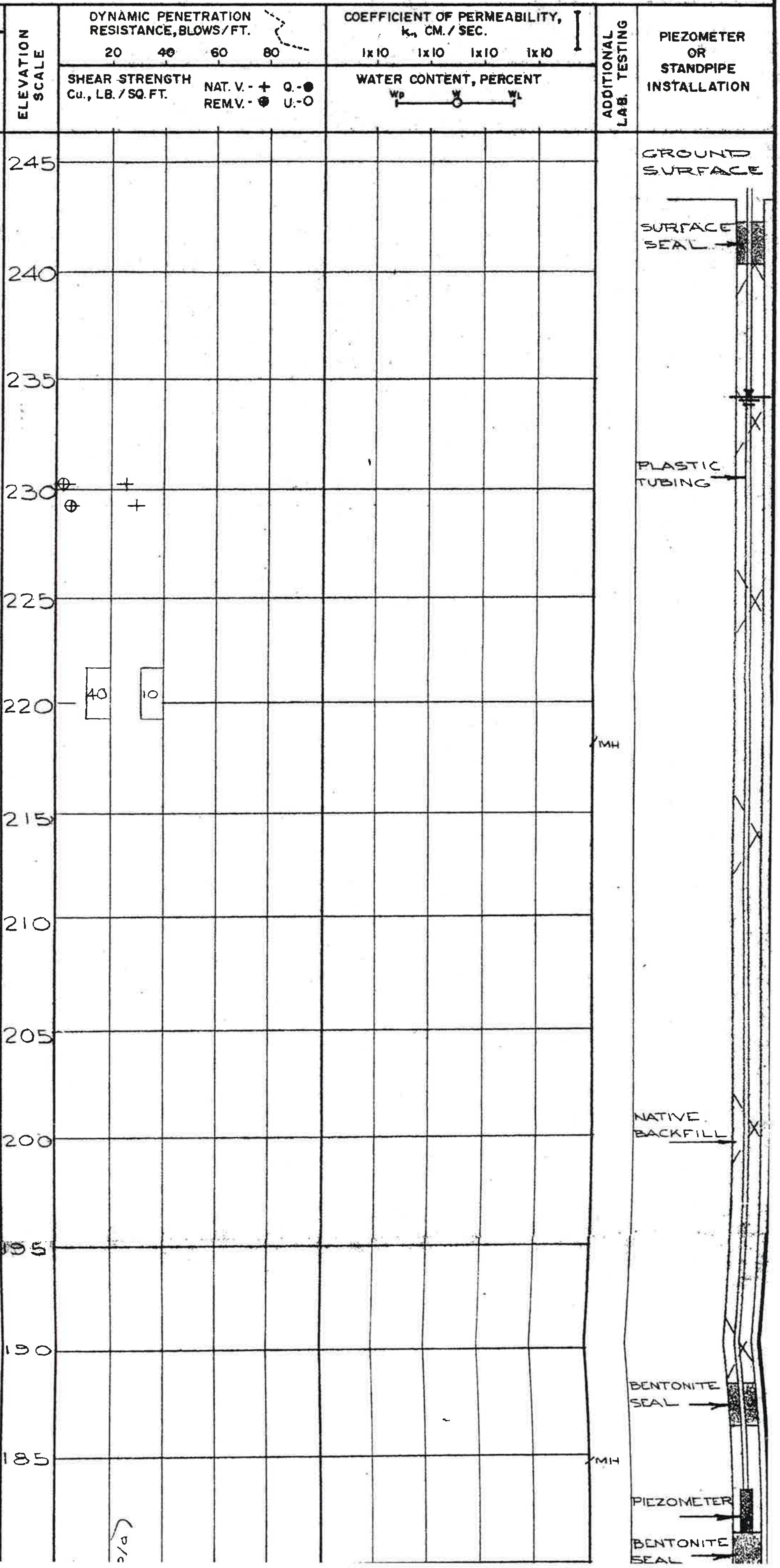
BORING DATE APRIL 11 TO 14, 1975

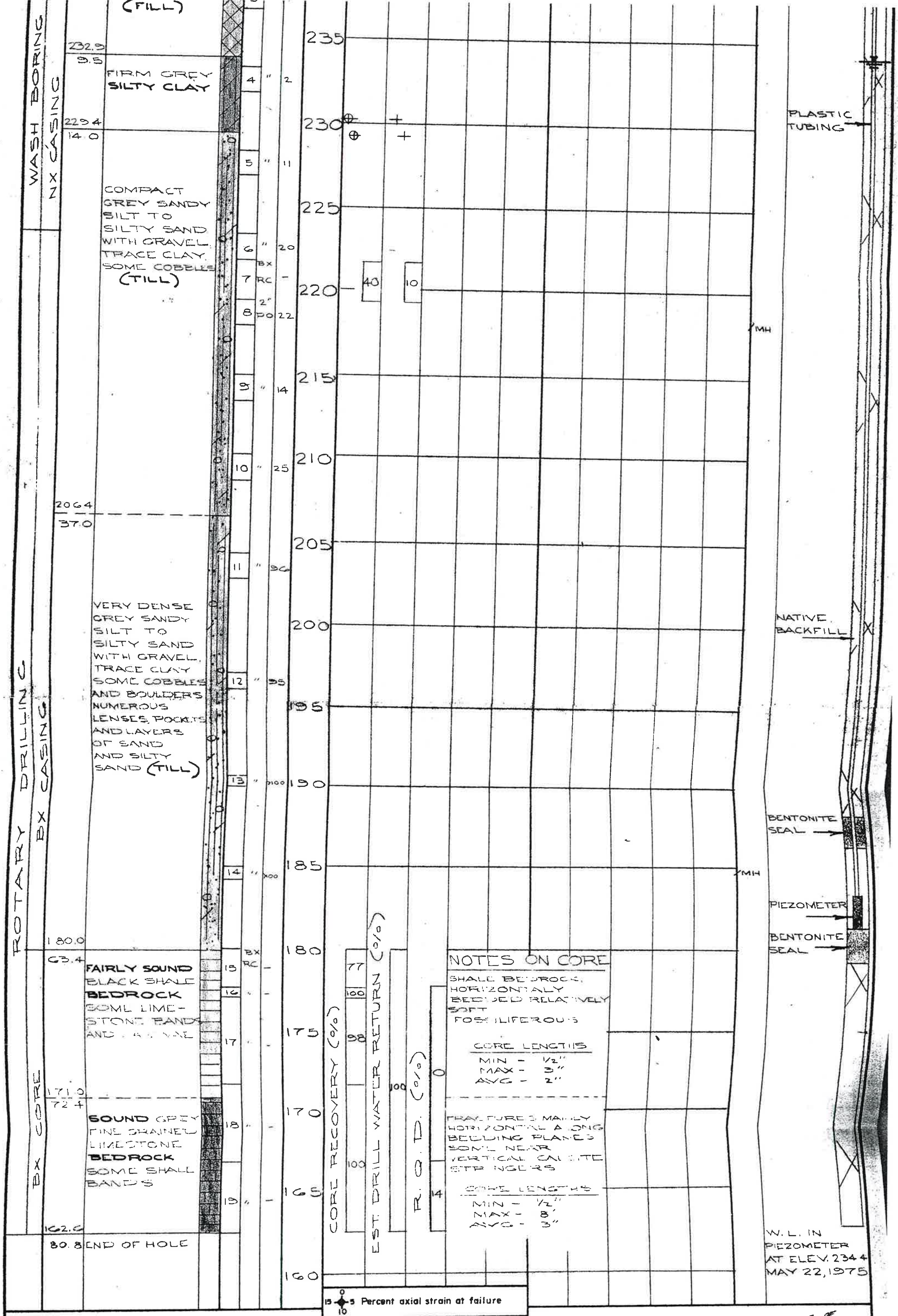
DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FT.
	243.4	GROUND SURFACE				
WASH BORING NX CASING		ASPHALT				
	0.5	COMPACT BROWN SAND AND GRAVEL TRACE SILT, SOME COBBLES (FILL)		1	2" DO	23
				2	"	16
				3	"	11
	232.9 9.5	FIRM GREY SILTY CLAY		4	"	2
	229.4 14.0	COMPACT GREY SANDY SILT TO SILTY SAND WITH GRAVEL TRACE CLAY SOME COBBLES (TILL)		5	"	11
				6	"	20
				7	BX RC	-
				8	2" DO	22
				9	"	14
				10	"	25
ROTARY DRILLING BX CASING	206.4 37.0	VERY DENSE GREY SANDY SILT TO SILTY SAND WITH GRAVEL, TRACE CLAY SOME COBBLES AND BOULDERS NUMEROUS LENSES, POCKETS AND LAYERS OF SAND AND SILTY SAND (TILL)		11	"	96
				12	"	95
				13	"	>100
				14	"	>100
	180.0					





FIRM GREY **SILTY CLAY**

COMPACT GREY SANDY SILT TO SILTY SAND WITH GRAVEL, TRACE CLAY, SOME COBBLES (TILL)

VERY DENSE GREY SANDY SILT TO SILTY SAND WITH GRAVEL, TRACE CLAY SOME COBBLES AND BOULDERS NUMEROUS LENSES, POCKETS AND LAYERS OF SAND AND SILTY SAND (TILL)

FAIRLY SOUND BLACK SHALE **BEDROCK** SOME LIME-STONE BANDS AND [illegible]

SOUND GREY FINE GRAINED LIMESTONE **BEDROCK** SOME SHALL BANDS

80.8 END OF HOLE

NOTES ON CORE

SHALE BEDROCK, HORIZONTALY BEDDED RELATIVELY SOFT FOSSILIFEROUS

CORE LENGTHS
MIN - 1/2"
MAX - 3"
AVG - 2"

FRACTURES MAINLY HORIZONTAL ALONG BEDDING PLANES SOME NEAR VERTICAL CALCITE STRINGERS

CORE LENGTHS
MIN - 1/2"
MAX - 8"
AVG - 3"

W. L. IN PIEZOMETER AT ELEV. 234.4 MAY 22, 1975

VERTICAL SCALE
1 IN. TO 5 FT.

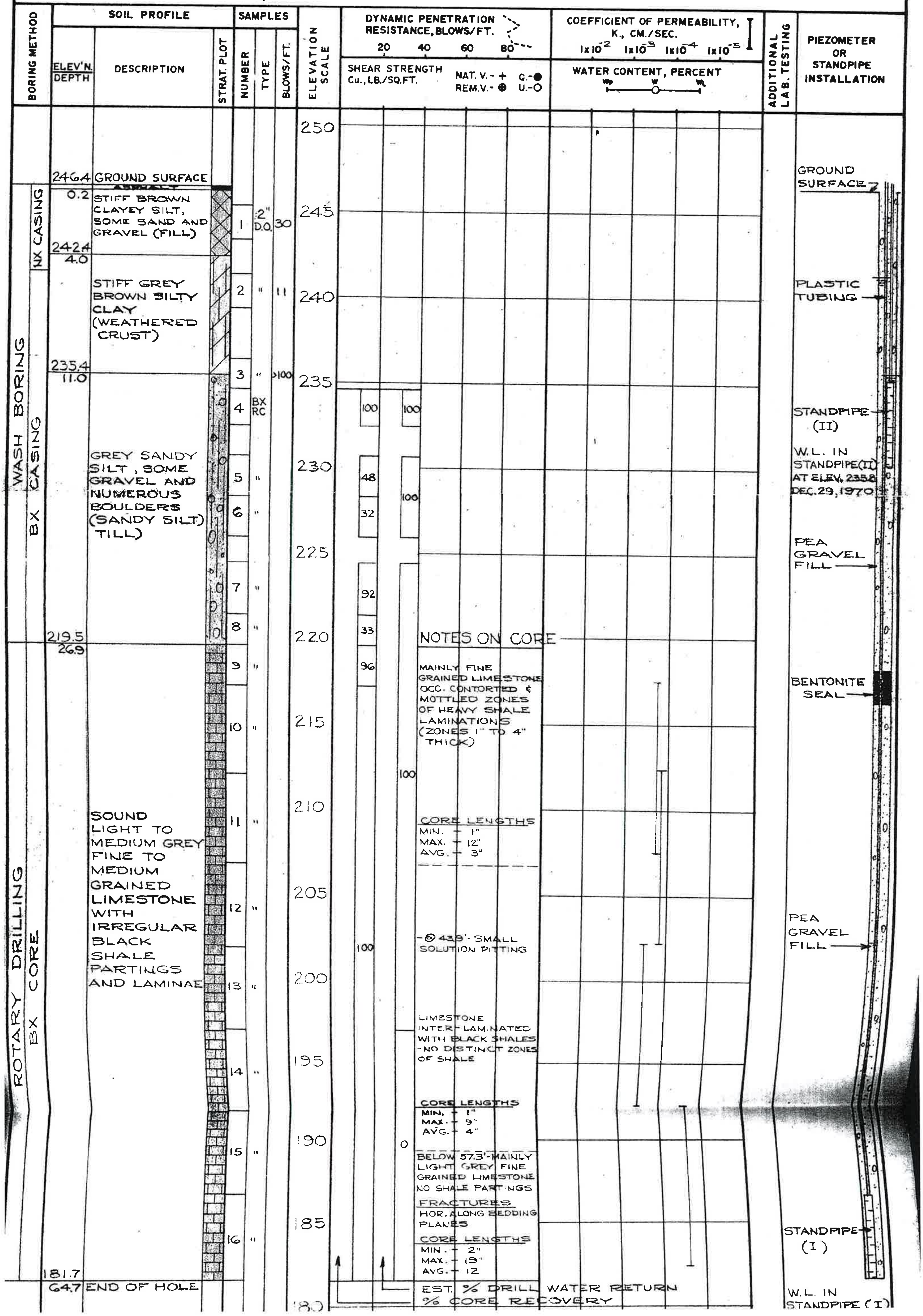
Golder Associates

DRAWN GF
CHECKED EAW

RECORD OF BOREHOLE W-3

LOCATION See Figure 1
BORING DATE DECEMBER 22-29, 1971
DATUM GEODETIC
SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.
PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

70794



ELEV'N. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FT.
246.4	GROUND SURFACE			
	ASPHALT			
0.2	STIFF BROWN CLAYEY SILT, SOME SAND AND GRAVEL (FILL)	1	2" D.O.	30
242.4 4.0	STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)	2	"	11
235.4 11.0	GREY SANDY SILT, SOME GRAVEL AND NUMEROUS BOULDERS (SANDY SILT) TILL)	3	"	>100
		4	BX RC	
		5	"	
		6	"	
		7	"	
		8	"	
219.5 26.9	SOUND LIGHT TO MEDIUM GREY FINE TO MEDIUM GRAINED LIMESTONE WITH IRREGULAR BLACK SHALE PARTINGS AND LAMINAE	9	"	
		10	"	
		11	"	
		12	"	
		13	"	
		14	"	
		15	"	
		16	"	
181.7 64.7	END OF HOLE			

% CORE RECOVERY: 100, 48, 32, 92, 33, 96, 100

EST. % DRILL WATER RETURN: 100, 100, 100, 0

NOTES ON CORE

MAINLY FINE GRAINED LIMESTONE OCC. CONTORTED & MOTTLED ZONES OF HEAVY SHALE LAMINATIONS (ZONES 1" TO 4" THICK)

CORE LENGTHS
MIN. ± 1"
MAX. ± 12"
AVG. ± 3"

-@ 43.9' - SMALL SOLUTION PITTING

LIMESTONE INTER-LAMINATED WITH BLACK SHALES - NO DISTINCT ZONES OF SHALE

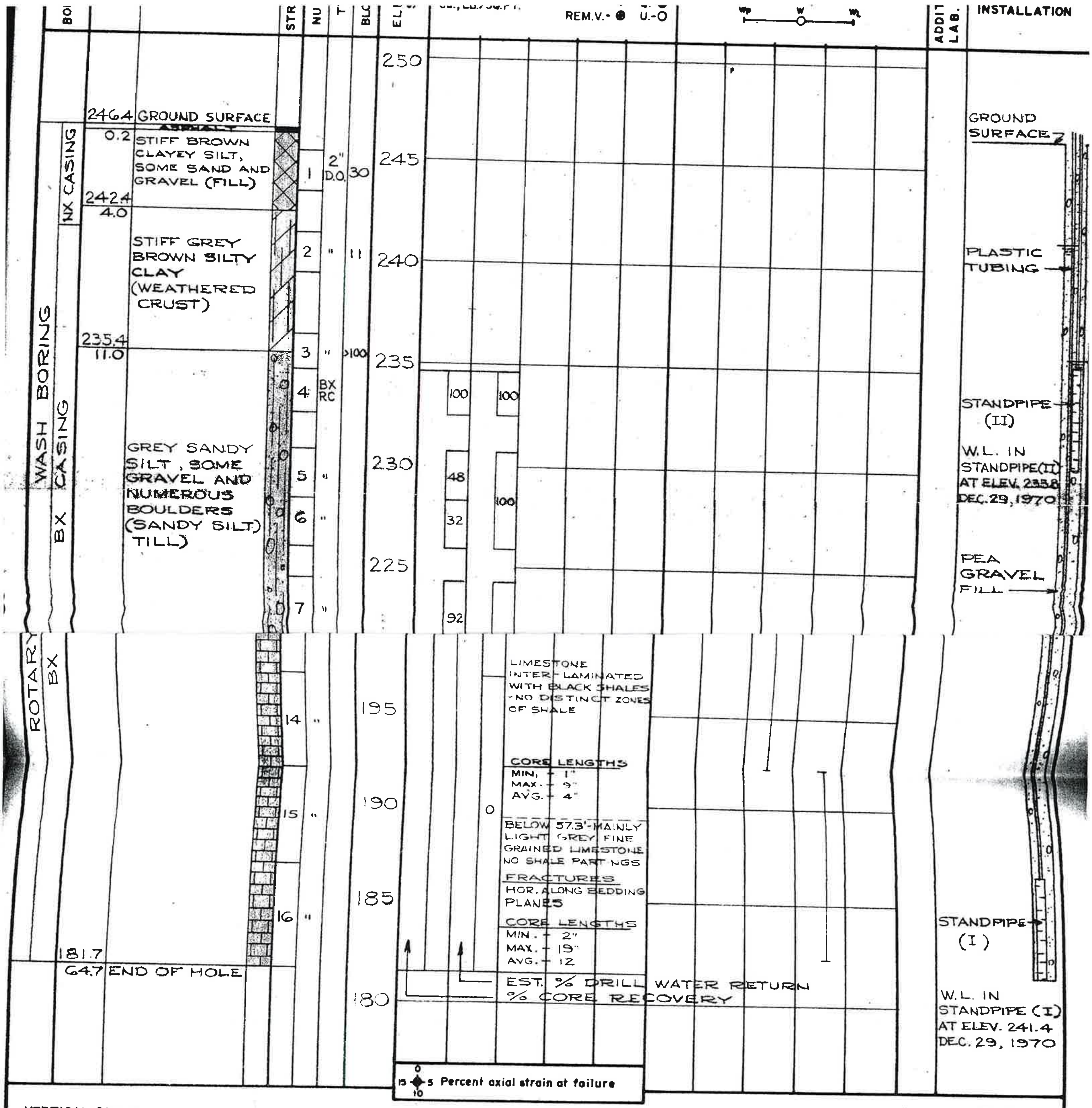
CORE LENGTHS
MIN. ± 1"
MAX. ± 9"
AVG. ± 4"

BELOW 57.3' - MAINLY LIGHT GREY FINE GRAINED LIMESTONE NO SHALE PARTINGS

FRACTURES
HOR. ALONG BEDDING PLANES

CORE LENGTHS
MIN. ± 2"
MAX. ± 19"
AVG. ± 12

GROUND SURFACE
PLASTIC TUBING
STANDPIPE (II)
W.L. IN STANDPIPE (II) AT ELEV. 235.8 DEC. 29, 1970
PEA GRAVEL FILL
BENTONITE SEAL
PEA GRAVEL FILL
STANDPIPE (I)
W.L. IN STANDPIPE (I)



VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN D.N.
CHECKED

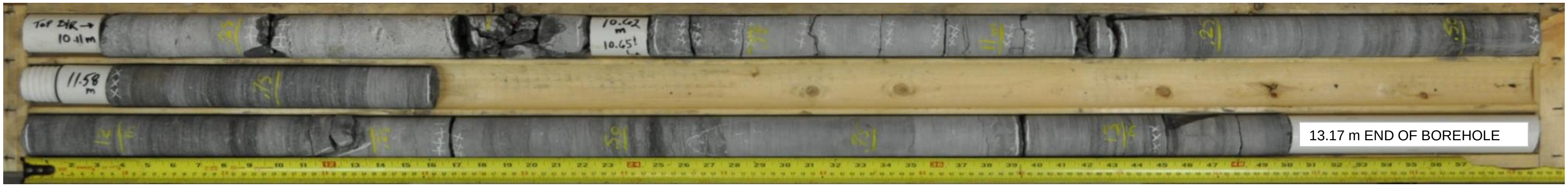
APPENDIX C

Core Photos and Results of UCS Testing

BH 18-01 DRY

10.11 m to 13.17 m

10.11 m START OF RUN



13.17 m END OF BOREHOLE

GOLDER

Geotechnical Investigation

Westgate Mall Phase 1 Development

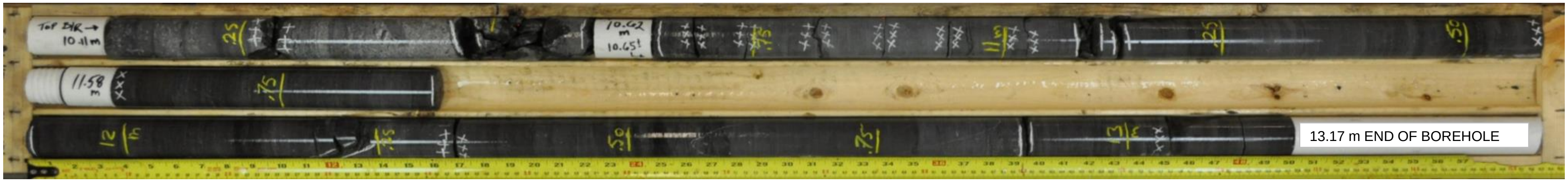
Ottawa, Ontario

Project No.	18106595
Drawn:	SG
Date:	29/10/2018
Checked:	NRL
Review:	NRL

FIGURE C1

BH 18-01 WET

10.11 m to 13.17 m


10.11 m START OF RUN

13.17 m END OF BOREHOLE

Geotechnical Investigation

Westgate Mall Phase 1 Development

Ottawa, Ontario

Project No.	18106595
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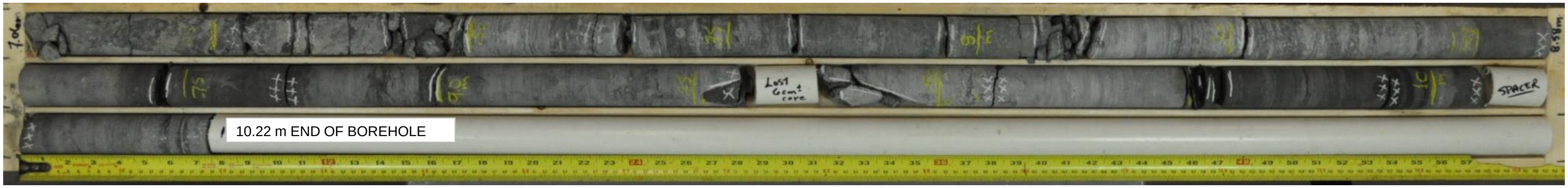
Review:	NRL

FIGURE C2

BH 18-02 DRY

7.06 m to 10.22 m

7.06 m START OF RUN



GOLDER

Geotechnical Investigation

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Ottawa, Ontario

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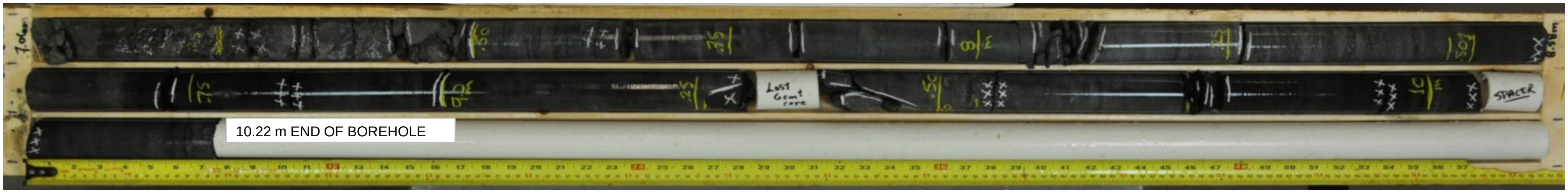
FIGURE C3

BH 18-02 WET

7.06 m to 10.22 m

7.06 m START OF RUN



10.22 m END OF BOREHOLE

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Westgate Mall Phase 1 Development

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Review:	NRL

FIGURE C4

Golder Associates Ltd.
1931 Robertson Road
Ottawa, Ontario
K2H 5B7

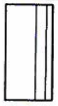
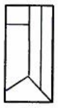
UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE

Project: RioCan Westgate Geotech Ottawa

Project No.: 18106595

Date: October 2, 2018

Location(s): See table below

Bore Hole No.	Depth (m)	Date Tested	Core Size	Diameter (mm)	Density (kg/m³)	Compressive Strength (MPa)	Failure Mode
18-01	11.39-11.50	Sep 27/18	NQ	44.8	2696	122.2	
18-02	8.39-8.50	Sep 27/18	NQ	44.7	2686	122.6	

REMARKS :
- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- Specimen ends prepared with high-strength plaster, but un-restrained.
- L/D ratio's between 2.0:1 and 2.5:1
- Time to failure > 2 and < 15 minutes.
- This report constitutes a testing service only. Interpretation of results will be provided on request only.

TESTING WAS CARRIED OUT IN GENERAL ACCORDANCE WITH ASTM D7012 - Method C

SIGNED: Melanie Ireland

APPENDIX D

Results of Chemical Analysis

Certificate of Analysis

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

Report Number: 1817557
Date Submitted: 2018-09-27
Date Reported: 2018-10-04
Project: 18106595
COC #: 188741

				Lab I.D.	1389955	1389956
				Sample Matrix	Soil	Soil
				Sample Type		
				Sampling Date	2018-09-11	2018-09-12
				Sample I.D.	18-01 SA 7/17.5-19.5	18-03 SA 5/10-12
Group	**Analyte**	**MRL**	**Units**	**Guideline**		
Anions	Cl	0.002	%		0.023	0.068
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.57	0.82
	pH	2.00			8.27	8.71
	Resistivity	1	ohm-cm		1750	1220
Subcontract	SO4	20	ug/g		40	40

Guideline = *** = Guideline Exceedence**

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

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

APPENDIX E

Results of Geophysical Testing

TECHNICAL MEMORANDUM

DATE October 05, 2018 **Project No.** 18106595

TO Sarah Ghadbane,
Golder Associates Ltd.

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

SEISMIC REFRACTION AND NBCC SEISMIC SITE CLASS TESTING RESULTS
WESTGATE MALL, OTTAWA

This technical memorandum presents the results of a Multichannel Analysis of Surface Waves (MASW) test performed for the National Building Code (NBCC 2015) as well as a refraction survey. The seismic testing was carried in the parking lot of the Westgate Shopping Center in Ottawa, Ontario (Figure 1). The geophysical testing was performed by Golder Associates Ltd. (Golder) personnel on September 18, 2018.

Figure 1: Site Map (MASW Line in blue, Refraction Lines in red)

1.0 METHODOLOGY

1.1 MASW

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.

1.2 Seismic Refraction

Seismic refraction employs a seismic source and an array of geophones to measure the rate of propagation of the imparted energy through the ground. In seismic refraction, the first arrival (or first break) of energy is picked in time from the seismic records. For geophones located close to the seismic source location (shot point), the first arrival is the direct wave – the energy that travels directly through the ground from the shot point to the geophone at the velocity of the upper layer. At the ‘critical’ distance and beyond, the first event detected becomes the critically refracted arrival, which is the energy from the wave that travels along the interface between the surface layer and the layer beneath. This requires that the lower layer have a higher velocity than the upper layer such that the refracted energy will at some point “overtake” the energy arriving directly (for example soil over bedrock).

Geophones are typically laid out at 2 m to 25 m intervals depending on the required depth of investigation and occasionally the interval is tightened close to the shot points to improve sampling of the first layer velocity. Shot points for each spread are as follows: 2 off-end shots, two end shots and 1 to 3 mid-spread shots distributed along the geophone array. Continuous refraction profiles are generated by shooting multiple, overlapping (2 or 3 geophones overlap) spreads along a profile line.

Once the seismic refraction records are picked for first arrivals, time-distance graphs are prepared and tables with geophone locations and elevations, arrival times and layer assignments are generated. These data are then interpreted to produce a depth model showing the interpreted layers and the material velocities within the layers.

2.0 FIELD WORK

The MASW field work was conducted on September 18, 2018, by two geophysicists from the Golder Mississauga office. The seismic lines were collected during the night time on the asphalt parking surface.

Despite working at night, the site was relatively noisy and we were limited by the number of shots because of the proximity of hotels and condominiums.

2.1 MASW

For the MASW line, a series of 24 low frequency (4.5 Hz) geophones were laid out at 3 metre intervals. Both active and passive readings were recorded along the MASW line. For the active investigation, a seismic drop of 45 kg and a 9.9 kg sledge hammer were used as seismic sources. Active seismic records were collected with seismic sources located 5, 10, and 15 metres from and collinear to the geophone array. An example of active seismic record collected along the MASW line is shown in Figure 2 below.

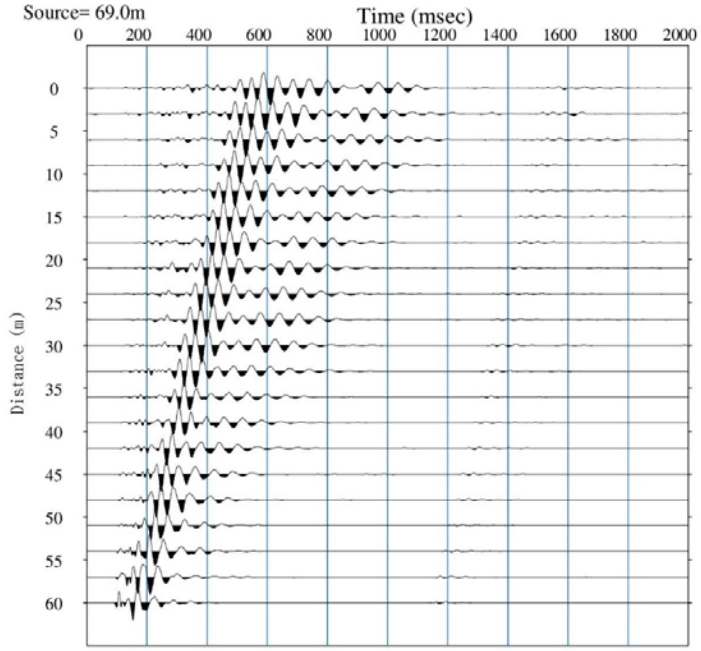


Figure 2: Typical seismic record collected at the site of the MASW line.

2.2 Seismic Refraction

The seismic refraction survey was carried out using a Geometrics Geode seismograph with a 24 geophone array and 4.5 Hz geophones. A sledge hammer and weight drop were used as the seismic source. At each source location, several shots were stacked out to maximize signal-to-noise ratio during the seismic refraction survey, seismic records were stacked. The data were continuously checked in field to ensure that the data quality was satisfactory. The seismic refraction data along Line 1 were noisy but filtering the data allowed us to pick first arrivals (Figure 3). Unfortunately, the data were too noisy along seismic refraction Line 2 to be able to generate a successful velocity model (Figure 4).

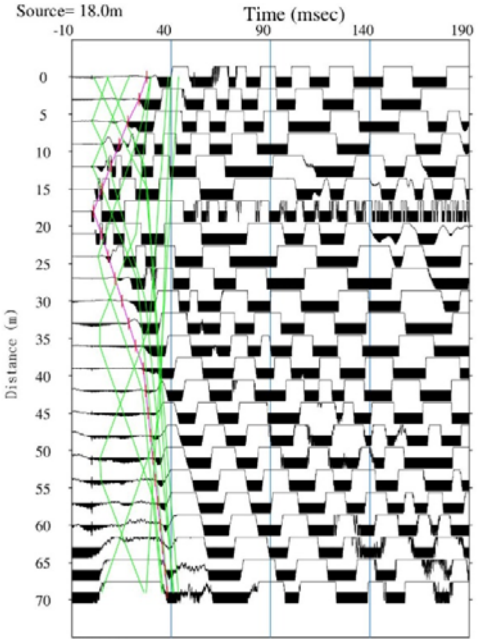


Figure 3: Typical seismic record collected at seismic refraction Line 1.

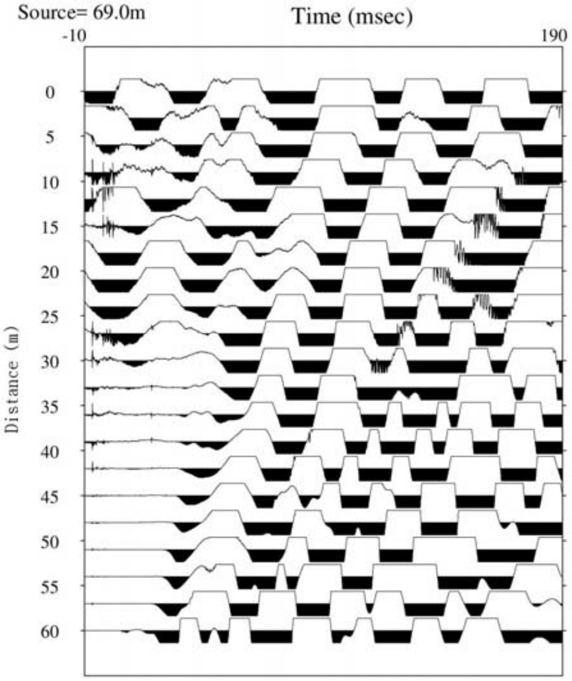


Figure 4: Typical seismic record collected at seismic refraction Line 2.

3.0 DATA PROCESSING

3.1 MASW

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

1) Transformation of the time domain data into the frequency domain using a Fast-Fourier Transform (FFT) for each source location;
2) Calculation of the phase for each frequency component;
3) Linear regression to calculate phase velocity for each frequency component;
4) Filtering of the calculated phase velocities based on the Pearson correlation coefficient (r2) between the data and the linear regression best fit line used to calculate phase velocity;
5) Generation of the dispersion curve by combining calculated phase velocities for each shot location of a single MASW test; and,
6) Generation of the stiffness profile, through forward iterative modelling and matching of model data to the field collected dispersion curve.

Processing of the MASW data was completed using the SeisImager/SW software package (Geometrics Inc.). The calculated phase velocities for a seismic shot point were combined and the dispersion curve generated by choosing the minimum phase velocity calculated for each frequency component as shown on Figure 5. 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 (7 to 27 Hz). The minimum measured surface wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 7 Hz. Passive data were useful to pick more accurately the dispersion curve at lower frequency.

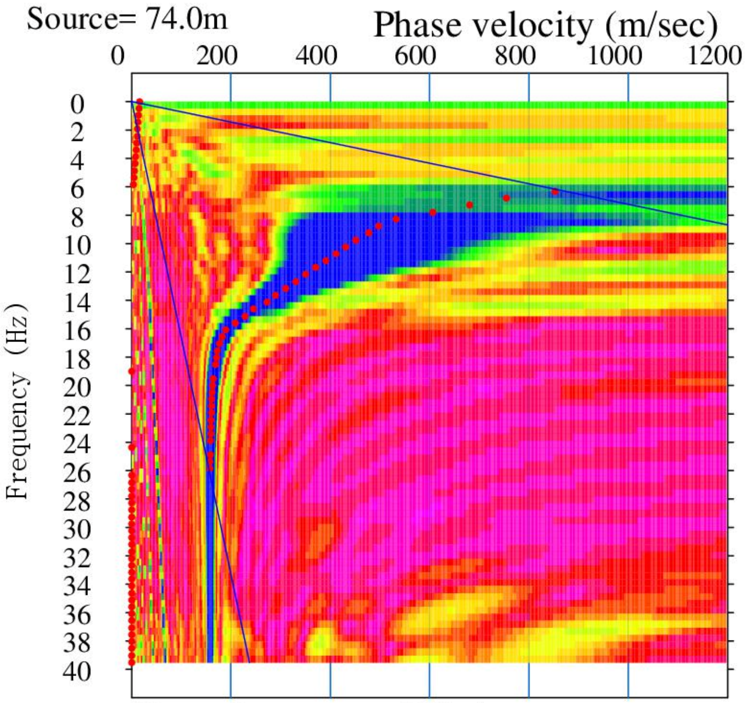


Figure 5: Active MASW Dispersion Curve Picks (red dots) along the MASW line

3.2 Seismic refraction

The Seismic data were transferred from the data collection systems and processed in the office.

The seismic data along Line 1 were processed using the SeisImager software package (Geometrics Inc.). The first step was to manually pick the first arrivals of compressional waves using the Pickwin program of SeisImager package. Overall the data quality of the seismic data was satisfactory. The picked files were input into the Plotrefa module to generate a tomographic model of the subsurface velocity along the survey line.

The seismic refraction subsurface models were exported from its respective software package and contoured using the Surfer Surface Mapping System (Golden Software) using a Kriging algorithm and a cell size of half of the geophone spacing. The contoured models were then imported to AutoCAD (Autodesk Inc.) for interpretation and presentation.

4.0 RESULTS

4.1 MASW

The MASW test results are presented in Figure 6, which present the calculated shear wave velocity profile derived from the field testing along the MASW line. The results along the MASW line have been calculated using weight-drop located at 5 metres from the last geophone. The field collected dispersion curves are compared with the model generated dispersion curves on Figure 7 for the MASW line. There is a satisfactory correlation between the field collected and model calculated dispersion curves, with a root mean squared error of less than 3% along the MASW line.

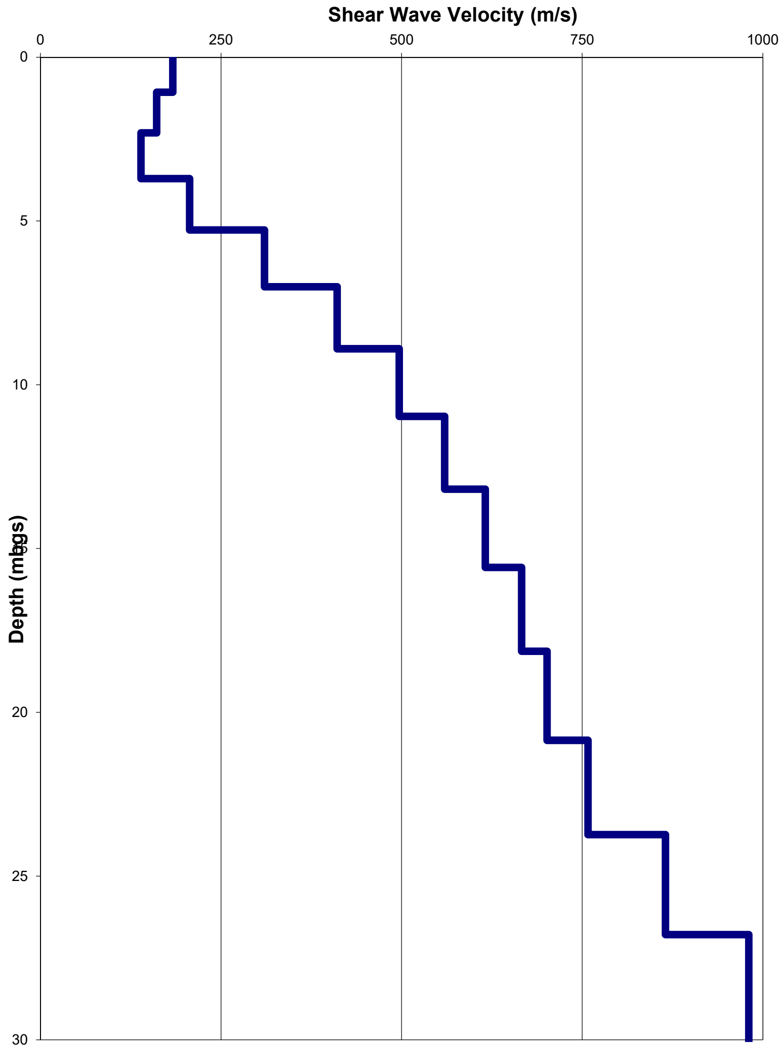


Figure 6: MASW Modelled Shear-Wave Velocity Depth profile along the MASW Line

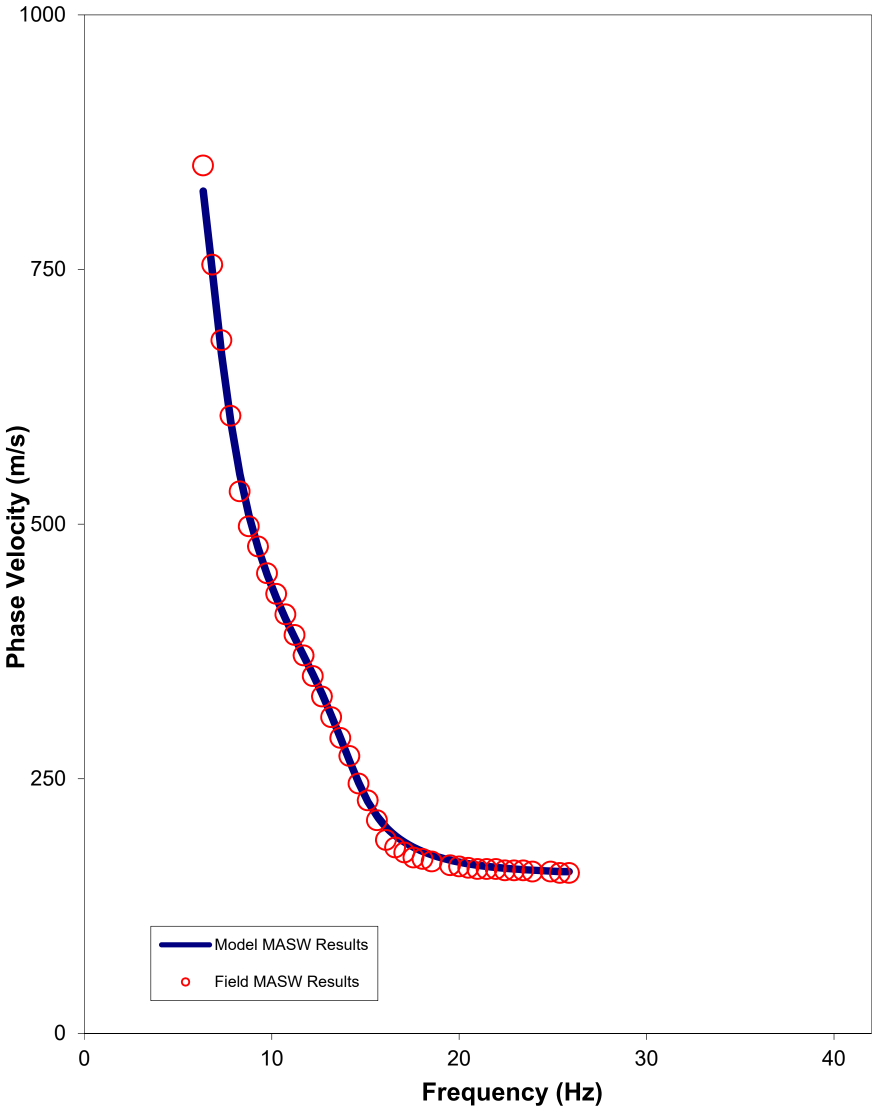


Figure 7: Comparison of Field (red dots) vs. Modelled Data (blue line) along the MASW Line

To calculate the average shear-wave velocity as required by the NBCC 2015, the results were modelled to 30 metres below ground surface. The average shear-wave velocity along the MASW line was found to be 418 m/s (Table 1).

Table 1: Shear-Wave Velocity Profile along the MASW line

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	183	0.005849
1.07	2.31	1.24	161	0.007684
2.31	3.71	1.40	139	0.010072
3.71	5.27	1.57	207	0.007582
5.27	7.01	1.73	310	0.005581
7.01	8.90	1.90	411	0.004615
8.90	10.96	2.06	497	0.004150
10.96	13.19	2.23	560	0.003976
13.19	15.58	2.39	616	0.003881
15.58	18.13	2.55	666	0.003836
18.13	20.85	2.72	701	0.003879
20.85	23.74	2.88	758	0.003806
23.74	26.79	3.05	865	0.003525
26.79	30.00	3.21	981	0.003277
			Vs Average to 30 mbgs (m/s)	**418**

The NBCC 2015 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.

4.2 Seismic Refraction

The plan location of the seismic refraction lines is presented on Figure 1. Interpreted seismic refractions results of Line 1 is presented on Figure 8. One borehole BH-W3 was located near the east end of the line. This borehole indicated a bedrock surface of 8.2 mbgs.

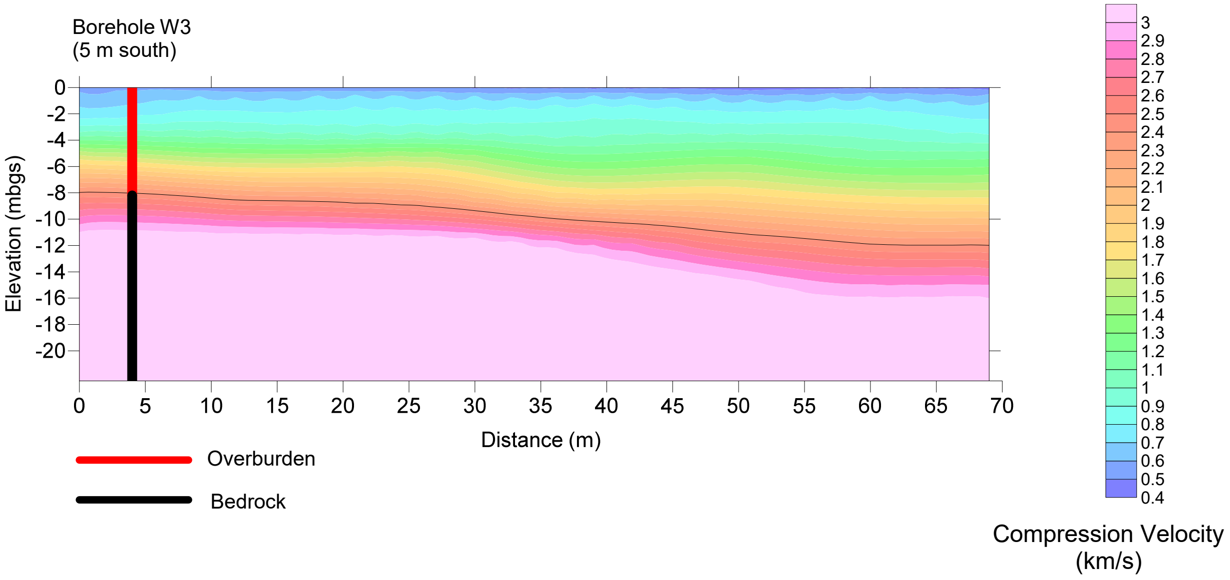


Figure 8: Seismic Refraction Interpreted Section along Line 1

In correlation with the bedrock surface from the existing borehole the compressional velocity contours of 2,400 m/s was chosen to represent the interpreted bedrock surface.

The interpreted bedrock depth ranges from about 8 m to 12 m (Figure 8).

The interpretation presented in this technical memorandum is based on available borehole information at the time of the data processing and on our understanding of the physical properties of typical materials in the area. The geophysical survey results should be further calibrated if more boreholes become available in the future.

5.0 LIMITATIONS

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

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

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

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

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

6.0 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

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