REPORT

GEOTECHNICAL AND HYDROGEOLOGICAL INVESTIGATION

1047 RICHMOND ROAD, OTTAWA, ONTARIO

Submitted to:

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1.0 INTRODUCTION

This report presents the results of geotechnical and hydrogeological investigation carried out at the site of a proposed residential development located at 1047 Richmond Road in Ottawa, Ontario.

The purpose of this investigation was to assess the general subsurface conditions at the site by means of a limited number of boreholes. Based on an interpretation of the factual information obtained, a general description of the soil, bedrock, and groundwater conditions is presented. These interpreted subsurface conditions and available project details were used to provide 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

The site of the proposed development is located at 1047 Richmond Road in Ottawa, Ontario. The site is about 2.5 acres and is currently occupied by a single-story commercial building (a car dealership) which consists of a building located in approximately the middle of the site, surrounded by parking areas.

The site is bordered to the east by a residential tower, to the south by Richmond Road, to the west by New Orchard Avenue and the north by a low-rise residential building. The project limits and the location of the proposed development are shown on Figure 1.

Based on the updated design scheme provided to Golder, the site will be developed into three residential buildings including two 38 and 40 storey towers with 6 storey podiums (Towers A and B), and a smaller 6 storey building (Tower C). The proposed development also includes a new park, an outdoor amenity and various access roadways and parking areas. The development includes three levels of underground parking which will encompass the entire development site excluding the future park in the southwest corner.

3.0 PROCEDURE

The field work for the current geotechnical investigation was carried out between September 21 and 30, 2021, in conjunction with the Phase 2 Environmental Site Assessment (ESA). During that time, ten boreholes (numbered 21-01 to 21-10) were advanced at the approximate locations shown on Figure 1.

The boreholes were advanced with a track-mounted hollow stem auger drill rig supplied and operated by CCC Geotechnical and Environmental Drilling of Ottawa, Ontario. The boreholes were advanced to depths ranging from 7.6 m to 15.5 m below the existing ground surface. Refusal to augering was encountered at all of the boreholes at depths ranging from 1.6 to 4.8 m below the existing ground surface.

Upon encountering refusal to augering, boreholes 21-01 to 21-05 were further advanced to a depth of about 7.6 m into the bedrock using pneumatic hammer rock drilling. No rock cores were recovered from these boreholes. Boreholes 21-06 to 21-10 were further advanced for 7.5 and 13.9 m into the bedrock using rotary diamond drilling techniques while retrieving HQ sized core.

Standard Penetration Tests (SPTs) were carried out within the overburden at various intervals of depth in general conformance with ASTM D 1586. Soil samples were recovered using 35 mm inside diameter split-spoon sampling equipment.

Monitoring wells were sealed in all the boreholes (with the exception of 21-08) to allow for subsequent measurements of stabilized groundwater levels as well as to perform in-situ hydraulic conductivity testing. The monitoring wells consist of 51 mm inside diameter rigid PVC pipe with 3.0 m long slotted screen sections, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. Measurement of the groundwater levels was completed on October 05, 2021.

At borehole 21-08, a 63.5 mm inside diameter rigid PVC casing was grouted over the full depth of the borehole to allow for Vertical Seismic Profile (VSP) testing to determine the shear wave velocity profile of the soil and rock.

The fieldwork was supervised by Golder staff who logged the boreholes, directed the in-situ testing, and collected the soil and rock samples retrieved in the boreholes. The samples obtained during the fieldwork were brought to our laboratory for further examination and laboratory testing.

The laboratory testing included determination of natural water content, grain size distribution on selected soil samples, and Uniaxial Compressive Strength (UCS) testing on selected bedrock samples.

Two samples of soil from boreholes 21-06 and 21-10 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 borehole locations were marked in the field and surveyed by Golder. The positions and ground surface elevations at the borehole locations were determined 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 Universal Transverse Mercator (UTM Zone 09) coordinate system. The elevations are referenced to Geodetic datum (CGVD28).

4.0 SUBSURFACE CONDITIONS

4.1 General

The following information on the subsurface conditions is provided in this report:

- Borehole records are provided in Appendix A
- Laboratory test results are provided in Appendix B, and on the relevant borehole records
- Rock core photographs are provided in Appendix C
- Results of the basic chemical analyses are provided in Appendix D
- Results of geophysical testing are provided in Appendix E
- Results of in-situ hydraulic conductivity testing are provided in Appendix F

In general, the subsurface conditions at this site consist of fill, or fill underlain by a deposit of glacial till which is in turn underlain by dolostone bedrock with shale, limestone, and sandstone interbeds.

The following sections present a more detailed overview of the subsurface conditions encountered during the field investigation.

4.2 Pavement Structure

Pavement structure was encountered in all of the boreholes. The pavement structure consists of 50 to 100 mm of asphaltic concrete overlying 110 to 540 mm thick granular base and subbase layers.

4.3 Fill

Fill was encountered below the pavement structure at all of the borehole locations. The fill consists of sand, silty sand to gravelly silty sand. The fill extends to depths ranging between 0.9 and 2.4 m below the existing ground surface at the borehole locations.

The results of SPT tests carried out within the fill gave ‘N’ values ranging from 1 to 35 blows per 0.3 m of penetration, indicating a very loose to dense (but typically compact) state of packing.

The measured natural water content of two samples of fill were about 10%.

The result of grain size distribution testing carried out on two sample of the fill is provided on Figures B-1 and B-2 in Appendix B.

4.4 Glacial Till

A discontinuous deposit of glacial till exists below the fill, and was encountered in the boreholes 21-04, 21-05, 21-08, and 21-10. The glacial till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand. The glacial till deposit (where encountered) was fully penetrated to depths ranging from 3.1 to 4.8 m below the existing surface.

The results of standard penetration tests carried out within the glacial till gave SPT ‘N’ values ranging from 46 to greater than 50 blows per 0.3 m of penetration, indicating a dense to very dense state of packing. High SPT ‘N’ values can also be indicative of cobbles and boulders and not the density of the soil matrix.

The measured natural water content of eight samples of glacial till ranged from 7 to 14%.

The result of grain size distribution testing carried out on one sample of the glacial till is provided on Figure B-3 in Appendix B.

4.5 Bedrock

Refusal to augering was encountered in all of the boreholes at depths ranging from 1.6 to 4.8 m below the existing ground surface. The bedrock was cored in boreholes 21-06 to 21-10 to depths ranging between 9.4 and 15.5 m below the existing ground surface.

In boreholes 21-02, 21-03 and 21-06 to 21-09, a zone of weathered/disturbed bedrock (which could be penetrated by augering and SPT sampling) was encountered at depths ranging from 0.9 to 3.1 m. The thickness of this zone was about 0.5 to 1.7 m at these borehole locations.

The following table summarizes the ground surface elevations, depth to bedrock, bedrock surface elevations and core lengths at the borehole locations:

Borehole Number	Ground Surface Elevation (m)	Bedrock Depth (m)	Core Length (m)	Bedrock Surface Elevation (m)
21-01	65.7	1.8	N/A[1]	63.9
21-02	65.5	3.1	N/A[1]	62.4
21-03	65.2	3.1	N/A[1]	62.1
21-04	65.1	3.7	N/A[1]	61.4
21-05	65.5	3.7	N/A[1]	61.8
21-06	65.0	1.9	7.5	63.1
21-07	66.1	1.6	8.1	64.4
21-08	64.6	3.2	12.3	61.4
21-09	65.9	1.7	13.8	64.2
21-10	65.9	4.8	10.7	61.1

Note: [1] No bedrock core recovery due to pneumatic hammer rock drilling

The bedrock encountered in the cored boreholes typically consists of medium grey dolostone with shale, limestone, and sandstone interbeds to a depth ranging from 9.1 to 13.2 m below the existing ground surface.

In boreholes 21-08 to 21-10, light grey sandstone with thin partings of shale was encountered below the dolostone layer at depths of 9.1 and 13.2 m below the existing ground surface, respectively.

Rock Quality Designation (RQD) values for dolostone and sandstone bedrock measured in the boreholes range from about 0 to 100% but are more typically in the range of 75 to 100% indicating good to excellent quality rock. In general, the RQD values increase with depth.

Nine UCS tests were carried out on core specimens of the bedrock, and measured UCS values range from 86 to 144 MPa, indicating strong rock. The results of the UCS tests are included in Appendix B. The UCS values are also presented in Figures B-4 and B-5 in Appendix B.

Photographs of the recovered bedrock core are presented in Appendix C.

4.6 Groundwater Conditions

In-situ hydraulic conductivity testing was carried out in monitoring wells installed in Boreholes 21-01 through 21-07, 21-09 and 21-10. An insufficient amount of water was present at monitoring wells 21-01, 21-07 and 21-09 to allow for testing to occur. The testing method at monitoring well 21-06 involved the rapid removal of water from the well using a dedicated foot valve and tubing, and measurement of the recovery of the water level over time. At monitoring wells 21-02, 21-03, 21-04, 21-05 and 21-10, a solid cylindrical slug was lowered quickly into the well and the change of the water level over time was recorded.

The data collected during the falling-head tests on monitoring wells 21-02, 21-03, 21-04, 21-05 and 21-10 were analyzed using the Hvorslev method (Hvorslev, 1951) to provide an estimate of the horizontal hydraulic conductivity of the bedrock adjacent to the test intervals. During the rising head test on monitoring well 21-06, the groundwater level was drawn down into the monitoring well screen; therefore, these data were analysed using the Bouwer and Rice (1976)[1]. The relevant calculations are included in Appendix F.

Summary of In-situ Hydraulic Conductivity Testing Results

Borehole	Geologic Unit of Screened Interval	Depth of Screened Interval (m)	Ground Surface Elevation (m)	Groundwater Level		Date of Measurement	Hydraulic Conductivity (cm/s)
				Depth below ground surface* (m)	Elevation (m)		
21-01	Dolostone	4.57 - 7.62	65.73	7.60	58.13	Oct. 5, 2021	Insufficient water for testing
21-02	Dolostone	3.96 - 7.01	65.46	3.32	62.14	Oct. 5, 2021	2x10^{-3}*
21-03	Dolostone	4.57 - 7.62	65.24	3.22	62.02	Oct. 5, 2021	1x10^{-4}*
21-04	Dolostone	4.57 - 7.62	65.09	2.70	62.39	Oct. 5, 2021	4x10^{-4}*
21-05	Dolostone	4.57 - 7.62	65.47	3.94	61.53	Oct. 5, 2021	2x10^{-4}*
21-06	Dolostone	6.33 - 9.38	65.00	6.84	58.16	Oct. 5, 2021	1x10^{-6}**
21-07	Dolostone	6.68 - 9.73	66.07	9.34	56.73	Oct. 5, 2021	Insufficient water for testing
21-09	Dolostone	6.63 - 9.68	65.90	Dry	Dry	Oct. 5, 2021	Not tested
21-10	Sandstone	12.40 - 15.45	65.89	8.85	57.04	Oct. 5, 2021	1x10^{-3}*

Notes: *analysed using Hvorslev (1951) method
**analysed using Bouwer and Rice (1976) method

The groundwater level measurement results indicate that the groundwater level in the bedrock ranged from 2.7 m to 9.3 m below the existing ground surface. The results of the rising head test analyses indicate that the hydraulic conductivity (K) of the bedrock at the borehole locations ranged from about 1×10^{-6} to 2×10^{-3} cm/s.

It is expected that the groundwater levels will be subject to fluctuations both seasonally and as a result of precipitation events. Groundwater levels may also be currently influenced by the excavations currently taking place on the south side of Richmond Road and may change as construction in that area is completed.

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[1] Bouwer, H. and R.C. Rice, 1976. A slug test method for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells, Water Resources Research, vol. 12, no. 3, pp.423-428.

4.7 Corrosion Testing

Two samples of soil from boreholes 21-06 and 21-10 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 Number	Sample Number	Depth Intervals (m)	Chlorides (%)	Sulphates (%)	pH	Resistivity (Ohm-cm)
21-06	2	1.5 – 1.9	0.007	<0.01	8.9	4,350
21-10	3	2.3 – 2.7	<0.002	0.01	8.4	6,670

5.0 DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the available information described herein and project requirements. The information in this portion of the report is provided for planning and design purposes for the guidance of the design engineers and architects. Where comments are made on construction, they are provided only to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities, costs, sequencing and the like.

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

5.2 Seismic Considerations

5.2.1 Seismic Zone

The site falls within the Western Québec Seismic Zone (WQSZ) according to the Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montréal to Témiscaming, and which encompasses the Ottawa area. Within the WQSZ recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montréal-Maniwaki axis. Historical seismicity within the WQSZ includes the 1935 Témiscaming event which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2 and the 1944 Cornwall-Massena event which had a magnitude of 5.6. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

5.2.2 Site Class

Vertical Seismic Profiling (VSP) geophysical testing was carried out within borehole 21-08 to evaluate the average shear wave velocity of the upper 30 m of soil/bedrock at the site (see Figure 1 for the VSP testing location). The results of the shear wave velocity test are included in Appendix E.

The VSP test results indicate that the average shear wave velocity in the upper 30 m from the bedrock surface (V_{s30}) was about 1,700 m/s. Based on this value, it is considered that a Site Class “A” designation is appropriate for the site for all structures founded on rock.

5.3 Frost Protection

The presence of frost-susceptible soils within the frost penetration depth will require that isolated, unheated exterior footings adjacent to surfaces which are cleared of snow cover during winter months be provided with a minimum of 1.8 m of earth cover (or equivalent insulation). Exterior foundations of heated structures should be provided with a minimum of 1.5 m of earth cover (or equivalent insulation).

If sufficient earth cover cannot be provided, foundation insulation details can be provided during detailed design.

The foundations of the proposed residential towers and podiums with three underground parking levels are expected to be placed on or within the bedrock at depth, which is unlikely to be highly frost susceptible and will be below the depth of frost. As such, frost protection is not required for the footings founded on bedrock at depth.

5.4 Foundations

Based on our understanding of the proposed development (in particular the three levels of underground parking which cover the entire footprint of the buildings) it is assumed that the foundations for the high-rise towers as well as the mid-rise podiums would likely consist of spread footings placed on the relatively shallow bedrock.

5.4.1 Bearing Resistance

In general, subsurface conditions encountered during the investigation consist of fill, or fill underlain by glacial till over dolostone/sandstone bedrock. It is considered that the proposed residential towers and podiums can be supported on spread footings placed on or within the competent bedrock.

The factored bearing resistance at Ultimate Limit States (ULS) for spread footing foundations founded on bedrock may be taken as 4,800 kPa for all areas where the foundations are three stories below the existing grade (which includes all of the currently proposed buildings).

These values are applicable provided that the bedrock surface is acceptably cleaned of soil and loose bedrock (i.e., any bedrock that can be easily removed with a hydraulic excavator). The settlement of footings at the corresponding service (unfactored) load levels will be less than 25 mm. Serviceability Limit States (SLS) conditions generally do not govern foundation design in rock.

Should there be localized locations within the excavation where the bedrock surface, following excavation and removal of any weathered rock, is below the planned founding level, then the footing level may be lowered such that the footing will bear directly on the unweathered bedrock. Alternatively, the subgrade could be raised to the underside of the foundation using mass concrete.

The bedrock surfaces should be inspected by qualified geotechnical personnel to confirm that the expected bearing material has been exposed and that the bearing surface has been adequately prepared and cleaned.

5.4.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the clean surface of sound bedrock could be considered. For cast-in-place concrete footings bearing directly onto the bedrock surface, the coefficient of friction, tan φ’, may be taken as follows:

- Cast-in-place concrete footing to clean sound bedrock: tan φ’ = 0.70

The sliding resistance value is unfactored, and a resistance factor of 0.8 would need to be applied to the sliding resistance in accordance with limit states design.

The resistance to lateral loads could be increased by constructing a shear-key at the bottom of the footings if needed. The design of shear keys would require a specific analysis taking into consideration the magnitude of the horizontal loading, the magnitude of the vertical loading, and any variations in the bearing pressure due to overturning moments.

5.5 Rock Anchors

Rock anchors could potentially be used to resist uplift and/or overturning of the foundation. Rock anchors should consist of grouted anchors installed into the bedrock at depth.

Rock anchors are typically installed in a borehole that is drilled with air-percussion equipment or with rotary diamond drilling equipment with water circulation; those drilling methods can fairly penetrate through boulder/cobblery ground such as exists on this site. A cased hole would be drilled through the overburden (if present) with a socket drilled into the bedrock, the steel anchor inserted, and then the annular space around the bar filled with grout.

Because the rock anchors would be permanent elements of the foundations, a “double corrosion protection” system should be provided.

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

i) failure of the steel tendon or top anchorage

ii) failure of the grout/tendon bond

iii) failure of the rock/grout bond

iv) failure within the rock mass, or rock cone pull-out

Potential failure modes i) and ii) are structural and are best addressed by the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion.

For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as *1,000 kPa* for ULS design purposes. This value should be used in calculating the resistance under ULS conditions. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the preliminary resistance is calculated based on the unit weight (undrained) of the potential mass of rock and soil which could be mobilized by the anchor, and resistance to shear of the rock mass. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \varphi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

Where:

Qr = Factored uplift resistance of the anchor (kN)

φ = Resistance factor (use 0.4)

γ' = Effective unit weight of rock (use 16 kN/m^3 below the groundwater level)

D = Anchor length in metres

θ = One-half of the apex angle of the rock failure cone (use 30°)

Where the anchor load is applied at an angle to the vertical, the anchor capacity should be reduced as follows:

$$Q_r' = Q_r \cos(\alpha)$$

Where:

Qr' = Factored uplift resistance of the anchor subject to inclined load (kN)

Qr = Factored uplift resistance of the anchor (kN)

α = Angle between the load direction and the vertical

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. In the case of group effects for a series of rock anchors in a rectangle with width “a” and length “b” installed to a depth “D”, the equation for the volume of the truncated trapezoid failure zone would be as follows:

$$V = \frac{4}{3} D^3 sin^2\Theta + aD^2 \sin\Theta + bD^2 \sin\Theta + abD$$

Where:

V = Volume of the truncated trapezoid failure zone (m3)

D = Depth of anchor group (m)

a = Width of anchor group (m)

b = Length of the anchor group (m)

θ = One-half of the apex angle of the rock failure cone, use 30°

The ULS factored geotechnical resistance for the truncated trapezoid failure formed by the group of anchors can then be calculated based on the following equation:

$$Q_r = \varphi \gamma' V$$

Where:

Qr = Factored uplift resistance of the anchor (kN)

φ = Resistance factor, use 0.4

γ' = Effective unit weight of rock, use 16 kN/m^3

V = Volume of truncated trapezoid (m^3)

The method described above does not explicitly consider the tensile strength of the rock that must be overcome prior to mobilization of the weight of the rock mass. If required, the tensile strength of the rock mass can be assessed based on the unconfined compressive strength, recovery, and quality of bedrock core obtained. This assessment, however, requires a detailed understanding of the anchor lengths, geometry, loads, etc. and would need to be completed during detailed design.

It is recommended that proof load tests be carried out on the anchors to confirm their resistance. The proof load tests should be carried out in accordance with OPSS 942 *(Prestressed Soil and Rock Anchors).*

A geotechnical professional should be present during the installation and testing of the anchors. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grouted area with minimum voids. Confirmation of sufficient embedment into the rock beneath the foundations should be carried out to make sure that the anchors are being installed in rock of adequate quality. The anchor holes must be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to promote an adequate bond between the grout and the rock. Prestressing of the anchors prior to loading will minimize anchor movement due to service loads.

5.6 Lateral Earth Pressure

Lateral earth pressures acting on the foundation walls of the underground parking are provided in the following sections for the portion of the underground parking within the overburden (or approximate elevations of between 65.5 and 62.5 m) and portion of the underground parking below and within the bedrock (or approximate elevations of between 62.5 and 56.5 m).

The following sections can also be used to estimate the lateral earth pressures on a temporary shoring system that might be required during the excavations.

5.6.1 Underground Parking – Within Overburden

Lateral earth pressures acting on the foundation walls (or temporary retaining system) above bedrock (i.e., within the overburden) will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading may also need to be considered in the design.

5.6.1.1 Static Lateral Earth Pressures

It is assumed that the foundation walls will be non-yielding, and therefore at-rest conditions will apply for those walls. It is assumed that the foundation walls will be drained but if the structures will not be drained, the earth pressure equation below the groundwater level should be used for the depth of the soil below groundwater level. The groundwater level was measured to be between 2.7 and 9.3 m depth at this site.

As a first, but likely conservative approximation, the static lateral earth pressure can be calculated as:

$$\sigma_h(z) = K\,(\gamma \cdot z + q) \text{ (Above the groundwater level)}$$

$$\sigma_{h(z)} = K\,[\gamma d_w + (\gamma - \gamma_w)(z - d_w) + q] + (z - d_w)\,\gamma_w \text{ (Below the groundwater level)}$$

Where:

$\sigma_{h(z)}$ = Lateral earth pressure on the wall at depth z (kPa)
K = Earth pressure coefficient, K_o for restrained structures or K_a for unrestrained structures
γ = Unit weight of retained soil (see table below)
γ_w = Unit weight of water (use 9.81 kN/m^3)
z = Depth below the top of wall (m)
d_w = Depth to groundwater level (see discussion above)
q = Uniform surcharge at ground surface behind the wall to account for traffic, equipment, or stockpiled soil (use 12 kPa)

The pressures are based on the existing fill and native materials behind the wall and the following parameters (unfactored) should be used to estimate the lateral earth pressures:

Material	Existing Fill	Glacial Till	Granular A / Granular B Type II	Clear Stone
Soil Unit Weight:	20 kN/m^3	21.5 kN/m^3	22 kN/m^3	18 kN/m^3
Internal Angle of Friction	Ø = 28°	Ø = 31°	Ø = 35°	Ø = 32 °
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o Passive, K_P	 0.36 0.53 2.77	 0.32 0.48 3.12	 0.27 0.43 3.70	 0.31 0.47 3.25

The above lateral earth pressures have not been factored; factoring of these loads will be required if the foundation wall is being designed in accordance with Limit States Design.

Where the permanent structure is significantly smaller than the excavation and a wide backfilled gallery exists between the structure wall and an adjacent rigid shoring system, then the permanent structure walls should be designed to retain the granular backfill soils using the above formulas, and an at rest earth pressure coefficient given above.

A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the structure. Care must be taken during the compaction operation not to overstress the structure. Heavy construction equipment should be maintained at a distance of at least 1 m away from the structure while the backfill soils are being placed and the backfill should be uniformly raised around the structure. Hand operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.

5.6.1.2 Seismic Lateral Earth Pressures

Seismic loading will result in increased lateral earth pressures acting on the retaining and foundation 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 earthquake-induced dynamic 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).

If the foundation walls are backfilled with granular free draining fill either in a zone with width equal to at least half 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 foundation wall, the following parameters (unfactored) provided in the table below may be used.

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

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

Where:

$\sigma_{h(z)}$ = Static plus seismic lateral earth pressure at depth z, (kPa)

K_a = Static active earth pressure coefficient, (see table above)

K_o = Static at-rest earth pressure coefficient, (see table above)

K = Earth pressure coefficient, K_o for restrained structures or K_a for unrestrained structures

H = Total depth to the bottom of the foundation wall (m)

K_{AE} = Seismic active earth pressure coefficient (see table below)

γ = Unit weight of the backfill soil (kN/m^3) (see table above)

z = Depth below the top of the wall (m)

Seismic (earthquake) loading must be taken into account in the assessment of lateral earth pressures:

- The horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the PGA. For structures which allow lateral yielding, (k_h) is taken as 0.5 times the PGA.
- The following seismic active pressure coefficients (K_{AE}) are for the fill, glacial till and granular backfills:

Seismic Active Pressure Coefficients, K_{AE}

Wall Behavior	Site PGA (2475-year Earthquake)	Existing Fill	Glacial Till	Granular A / Granular B Type II	Clear Stone
Yielding wall	0.244 g	0.44	0.40	0.34	0.38
Non-yielding wall		0.55	0.50	0.43	0.48

The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A mm, where A is the design zonal acceleration ratio of (0.244 g). This corresponds to displacements of up to approximately 40 mm at this site.

It should be noted that the above seismic earth pressure coefficients assume that the back of the wall is vertical and that 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.

5.6.2 Underground Parking – Within Bedrock

It is considered that three design conditions could exist with regards to the lateral earth pressures that will be exerted on the foundation walls founded within the bedrock:

- Case 1: Walls cast directly against the bedrock face
- Case 2: Walls cast against formwork with a narrow-backfilled gallery provided between the foundation wall and the adjacent excavation bedrock face
- Case 3: Walls cast against formwork with a wide backfilled gallery provided between the foundation wall and the adjacent excavation face

<u>Case 1</u>

For the first case (wall cast against the bedrock), there will be no effective lateral earth pressures on the foundation wall. This assumes that any loose blocks or wedges of rock are removed from the face of the excavation or are stabilized prior to constructing the wall, and that any rock stabilization is designed for permanent use (i.e., with appropriate corrosion protection).

<u>Case 2</u>

For the second case, the magnitude of the lateral earth pressure depends on the magnitude of the arching which can develop in the backfill and therefore depends on the width of the backfill, its angle of internal friction, as well as the interface friction angles between the backfill and both the rock face and the foundation wall. The magnitude of the lateral earth pressure can be calculated as:

$$\sigma_h(z) = \frac{\gamma B}{2\tan\delta}\left(1 - e^{-2K\frac{z}{B}\tan\delta}\right) + K\,q$$

Where:

$\sigma_{h(z)}$ = Lateral earth pressure on the foundation wall at depth z (kPa)

K = Earth pressure coefficient (use 0.6)

γ = Unit weight of retained soil (use 20 kN/m^3 for clear stone)

B = Width of backfill between foundation wall and bedrock face (m)

δ = Average interface friction angle at backfill-foundation wall and backfill-rock face interfaces (use 15 degrees)

z = Depth below top of formwork (m)

q = Surcharge at ground surface to account for traffic, equipment, or stock piled materials (use 12 kPa)

Case 3

For the third case, when the width of the backfill is equal to half the wall height or more (i.e., wide backfill), the foundation walls should be designed to resist lateral earth pressures calculated as outlined in Section 5.6.1.

The following should be considered in estimating the lateral earth pressure:

- Hydrostatic groundwater pressures would also need to be considered if the structure is designed to be water-tight.
- It has been assumed that the underground parking level will be maintained at minimum temperatures but will not be permitted to freeze. If these areas are to be unheated, additional guidelines for the design of the foundation wall will be required.

5.7 Excavation

Excavations for the underground parking and foundations will be made through the overburden and underlying dolostone and sandstone bedrock. It is expected that the excavation will extend up to approximately 9 or 10 m below the existing ground surface (to accommodate the three-storeys of underground parking).

5.7.1 Excavation in Overburden

No unusual problems are anticipated with excavating the overburden materials using large hydraulic excavating equipment.

In accordance with the Occupational Health and Safety Act (OHSA) of Ontario, the overburden materials above the groundwater table (i.e., fill and glacial till) would generally be classified as a Type 3 soil and therefore, the side slopes should be stable in the short term at 1 horizontal to 1 vertical. Below the water table, side slopes of 3 horizontal to 1 vertical (Type 4 soil in accordance with the OHSA) will be required.

Where site conditions (such as 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 5.9.

5.7.2 Excavation in Bedrock

The bedrock surface was encountered at depths ranging from about 1.6 to 4.8 m below the existing ground surface. Excavations into the bedrock will be required for the three levels of underground parking.

The bedrock encountered at this site, in general, consists of slightly weathered to fresh dolostone/sandstone. The thin upper portion of the bedrock, however, is highly weathered (as encountered in boreholes 21-02, 21-03 and 21-06 to 21-09). It will likely be possible to carry out the bedrock removal using mechanical methods (such as hydraulic excavators and hoe ramming) for the removal of the highly weathered portion of the bedrock or for shallow excavations into bedrock (such as for service installation).

Where deep excavation of the sound bedrock is required (for the underground parking), it is anticipated that the bedrock removal could be carried out using controlled blasting, potentially in conjunction with hoe ramming and closely spaced line drilling.

The borehole log information (such as bedding and jointing orientations and spacing) suggests that near-vertical excavation walls in the bedrock should stand unsupported for the construction period. The borehole data, however, provides only limited information of the bedding and jointing in the bedrock and therefore the exposed bedrock should be inspected regularly (as the bedrock excavation proceeds) by qualified geotechnical personnel to assess the exposed bedrock surface for potential localized instabilities. Additional temporary rock support system such as rock bolts or shotcrete and mesh might be required to secure localized unstable rock wedges or poor-quality rock. If rock bolts are used to secure the unstable rock wedges (on the rock faces against the foundation walls where they are relied on for long-term support of the rock walls), they should be designed as a long-term / permanent stabilization measure and should have adequate corrosion protection cover.

All loose rock should be removed from the sidewalls during excavation to ensure the safety of workers.

The blasting should be controlled to limit the peak particle velocities at all adjacent structures or services such that blast induced damage will be avoided. This will require blast designs by a specialist in this field (see Section 5.13).

5.8 Groundwater Management

5.8.1 Estimates of Groundwater Taking and Radius of Influence

5.8.1.1 Construction Condition

It is understood that three levels of underground garage parking are being considered. Accordingly, excavation will be through surficial fill and native glacial till, into the underlying bedrock. Based on the groundwater conditions observed in the monitoring wells, excavations will likely extend below the groundwater level. The rate of groundwater inflow to the excavation will depend on many factors, including: the contractor’s schedule and rate of excavation, the size of the excavation, and the time of year at which the excavation is made. Also, there may be instances where precipitation collects in an open excavation and must be rapidly pumped out.

According to O.Reg 63/16 and O.Reg 387/04, if the volume of water to be pumped from excavations for the purpose of construction dewatering is greater than 50,000 L/day and less than 400,000 L/day, the water taking will need to be registered as a prescribed activity in the Environmental Activity and Sector Registry (EASR) and requires the completion of a “Water Taking Plan”. Alternatively, a Permit to Take Water (PTTW) is required from the Ministry of the Environment Conservation and Parks (MECP) if a volume of water greater than 400,000 L/day is to be pumped from an excavation.

It is possible that groundwater elevations encountered during construction may be higher than those observed in October 2021, if, for example, construction occurs during the spring. Therefore, groundwater inflow estimates were completed using a groundwater elevation that is 0.5 m higher than the measured groundwater elevations. Incident precipitation could add approximately 700,000 L/day to the underground parking excavation, based on the proposed footprint, and assuming a 79.2 mm precipitation event (a 10-year event as observed at the Ottawa Airport weather station).

The Dupuit-Forcheimer analytical solution was used to estimate the potential groundwater inflow into the underground parking excavation using the geometric mean hydraulic conductivity measured in the wells screened above and to the depth of the underground parking (all monitoring wells except 21-10). The initial head elevation of the analytical model was assigned a value of 62.9 m (i.e., 0.5 m above the value recorded at monitoring well 21-04). It is assumed that construction dewatering activities would lower the groundwater level to an elevation of 56.0 m for the preliminary analysis. The hydraulic conductivity of the bedrock at this depth was estimated to be approximately 1x10^{-4} cm/s. The amount of dewatering needed for the excavation (including a safety factor of 2) is estimated to be between 105,000 (steady-state inflow) and 450,000 (initial inflow) litres per day (L/day). The radius of influence for the excavation is estimated to be approximately 25 m from the edge of the excavation. Groundwater inflow and dewatering radius of influence calculations are included in Appendix F.

Based on the groundwater conditions observed at the site and depending on how the excavation proceeds, water taking exceeding 400,000 L/day may be required to dewater groundwater from the excavation. As a result, a PTTW may be necessary for the water taking associated with the proposed work.

5.8.1.2 Permanent Condition

The Dupuit-Forcheimer analytical solution was used to estimate the potential groundwater inflow to the drainage system for the three levels of underground parking. The initial groundwater elevation was assumed to be 62.9 m (i.e., 0.5 m above the value recorded at monitoring well 21-04), and it was assumed that the drains would lower the groundwater elevation to elevation 56.5 m for the preliminary analysis. The analytical solution was run using the geometric mean hydraulic conductivity measured in the wells screened at the depth of the underground parking. The steady-state dewatering rate (including a safety factor of 2.0) for the drainage system is estimated to be approximately 92,000 L/day. The radius of influence for the drainage system for steady-state flow was estimated to be approximately 25.0 m from the underground parking (see Appendix F).

5.9 Temporary Shoring

The excavation through the overburden for underground parking will extend to depths of about 1.6 to 4.8 m below the existing ground surface. 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.

For design purposes, a soldier pile and timber lagging system are considered a feasible shoring method that may be considered for the proposed excavations at the site. Due to the presence of shallow bedrock beneath the overlaying deposits, the soldier piles might need to be socketed into the competent bedrock to provide sufficient embedment for toe fixity, or the piles may need to be pinned to bedrock. 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.

Where foundations or settlement-sensitive infrastructure are present within the zone of influence of the shoring, the deflections may need to be greatly limited and therefore soldier pile and timber lagging system might not be feasible. Golder can provide further recommendations and guideline in the detailed design stage when the distance and extent of the excavations with respect to the sensitive structures are determined.

For a soldier pile and lagging system, some form of lateral support to the wall is typically required for excavation depths greater than about 3 to 4 m. 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 since the anchors would likely encroach on their properties. The presence of utilities beneath the adjacent properties, 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.

5.10 Floor Slab

The floor slab of the underground parking will be cast on bedrock.

Provision should be made for at least 150 mm of OPSS.MUNI 1010 Granular A compacted to 100% of the material’s standard Proctor Maximum Dry Density (SPMDD) to form the base for the floor slab. Any engineered fill required to raise the grade to the underside of the Granular A, should consist of OPSS.MUNI 1010 Granular B Type II, or the Granular A bedding can be thickened, as needed. The underslab fill should be placed in maximum 300 mm thick lifts and compacted to at least 100% of the material’s SPMDD using suitable vibratory compaction equipment.

Provision should be made for drainage underneath the floor slab. The details on the permanent dewatering system are provided in Section 5.8.1.2, and subfloor drainage system should be designed to accommodate permanent groundwater inflow.

As a preliminary guideline, the subfloor drainage system may consist of a network of perimeter drains and 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 (fully wrapped in non-woven geotextile and backfilled with free draining granular soils) installed around the perimeter and within the underground parking footprint. The capacity of the subfloor drainage system should be modified during construction as required if higher than anticipated inflows are observed. For preliminary design, the subdrains should be spaced no greater than 6.0 m apart.

Vertical drainage system should be provided to the exterior foundation walls. The drainage system must withstand the design horizontal earth pressures used for foundation wall design and should be connected to the underslab perimeter drainage system (see further discussions below).

5.11 Foundation Wall Backfill and Drainage

The existing fill and glacial till encountered at this site are potentially frost susceptible and should not be used as backfill against the foundation walls. To avoid problems with frost adhesion and heaving as well as to provide drainage, the foundation walls should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for Ontario Provincial Standard Specification (OPSS) Granular B Type I, Granular B Type II, or Granular A. The granular backfill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material’s SPMDD using suitable vibratory compaction equipment. To reduce compaction induced stresses, only light compaction rollers or plate tampers should be used within 1.0 m of the wall.

If the basement walls will be backfilled, vertical drainage membrane such as Miradrain (or similar drainage board) should be installed prior to backfilling. If the wall will be cast against shoring/rock the drainage board should be installed prior to casting the wall.

Any narrow galleries between the foundation walls and shoring wall/exposed bedrock may be backfilled using clear stone (where too narrow for normal compacted granular fill). Where the clear stone is in direct contact with soil, it should be fully wrapped in non-woven geotextile.

The perimeter drainage of the basement wall backfill should be provided by means of a perforated pipe in a surround of 19 mm clear stone, fully wrapped in geotextile, which leads by gravity drainage to an adjacent storm sewer or sump pit.

In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill immediately adjacent to the building and the more frost susceptible backfill placed beyond the wall backfill. To reduce the severity of this differential heaving, the backfill adjacent to the foundation walls should be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 m below finished exterior grade at a slope of 3 horizontal to 1 vertical, or flatter, away from the wall. The granular fill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material’s SPMDD using suitable vibratory compaction equipment.

5.12 Ground Movements

During the excavation of the underground parking area, lateral deformation and vertical settlement of the adjacent ground may occur as a result of installation and deflection of the excavation activities. The ground movements induced could affect the stability or performance of structures and 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.

Protective measures such as temporary shoring for the excavations in soil may need to be adopted where the excavations interfere with the zone of influence of adjacent building foundations or other structures/utilities.

5.13 Vibration Monitoring

A pre-construction or pre-blast survey should be carried out for all of the surrounding structures. Selected existing interior and exterior cracks in the structures should be identified during the pre-construction survey and should be monitored for lateral or shear movements by means of pins, glass plate telltales and/or movement telltales.

The excavation contractor should be required to submit a complete and detailed blasting design and monitoring proposal prepared by a blasting/vibrations specialist prior to commencing blasting. This would have to be reviewed and accepted in relation to the requirements of the blasting specifications.

The contractor should be limited to only small, controlled shots. The following frequency dependent peak vibration limits at the nearest structures and services are suggested as typical vibration criteria commonly adopted for construction projects. If unusually sensitive receptors are identified during construction planning, then specific criteria may need to be adopted for those receptors.

Frequency Range (Hz)	Vibration Limits (mm/s)
< 10	5
10 to 40	5 to 50 (sliding scale)
> 40	50

It is recommended that the monitoring of ground vibration intensities (peak ground vibrations and accelerations) from the blasting operations be carried out both in the ground adjacent to the closest structures and services and within the structures themselves.

If practical, bedrock removal should commence at the furthest points from the closest structure or service to assess the ground vibration attenuation characteristics and to confirm the anticipated ground vibration levels.

Vibration monitoring should be carried out throughout all bedrock removal operations.

5.14 Site Servicing

5.14.1 Pipe Bedding and Cover

At least 150 mm of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface occurs, or if fill material is located below the invert of the pipe, it will be necessary to remove the disturbed material or fill, and place a sub-bedding layer consisting 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% of the material’s SPMDD. 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 or surrounding soil could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from spring line of the pipe to at least 300 mm above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 mm. The cover material should be compacted to at least 95% of the material’s SPMDD.

5.14.2 Trench Backfill

Trench backfill may consist of approved excavated material such as the existing pavement granulars, inorganic fill and glacial till, where the services will be overlain by pavements or other hard surfacing.

It is important for frost heave compatibility that the trench backfill within the frost zone of 1.8 m depth below pavement grade matches the soil exposed on the trench walls. This will require some separation of materials upon excavation. In particular, where the watermain is to be installed beneath existing pavements, the trench backfill should match those existing granular layers.

Trench backfill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of its SPMDD using suitable compaction equipment.

5.15 Pavement Design

In preparation for pavement construction, all disturbed, or otherwise deleterious materials (i.e., those materials containing organic material) should be removed from the roadway areas. To minimize potential for disturbance, the general grade should not be cut to final subgrade level until all services have been installed.

Sections requiring grade raising to the proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow, OPSS Select Subgrade Material (SSM) or granular fill. These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material's SPMDD using suitable compaction equipment.

Below the pavement structure, frost compatibility must be maintained across any new service trenches.
Due to the variability of the soils within the project limits, the subsoil should be inspected by qualified geotechnical personnel to make sure that there is no potential for differential frost heaving.

5.15.1 Pavement Drainage

The surface of the pavement subgrade should be crowned to promote drainage of the roadway granular structure. The subgrade surface should be crowned or sloped to promote drainage of the roadway granular structure. Perforated pipe subdrains should be provided along the low sides of the roadway along the entire length.
The subdrains should be installed in accordance with the City of Ottawa Specification F-4050 "Pipe Subdrain" and as per City of Ottawa Drawing No. R1. The geotextile should consist of a Class I non-woven geotextile to OPSS 1860. The geotextile should have a maximum Apparent Opening Size A.O.S. of 212 μm. The subdrains should be connected to the catch basins such that the pavement structure will be positively drained and will intercept flows within the subbase.

Backfilling of catch basin laterals located below subgrade level should be completed using acceptable native soils or fill which match the material types exposed on the lateral trench walls. This will reduce potential problems associated with differential frost heaving.

5.15.2 Granular Pavement Materials

Good drainage significantly improves the freeze-thaw resistance of the asphaltic concrete and decreases the frequency of transverse cracking, thereby extending the life of the pavement. The granular base and subbase for new construction should consist of Granular A and Granular B Type II (City of Ottawa F-3147), respectively.

5.15.3 Pavement Design

The pavement structure for local roads or parking lots, which will not experience bus or truck traffic (other than school bus and garbage collection), should consist of:

Pavement Component	Thickness (mm)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	400

The pavement structure for roadways which will experience bus and/or truck traffic as well as fire routes should consist of:

Pavement Component	Thickness (mm)
Asphaltic Concrete	120
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	450

The granular base and subbase materials should be uniformly compacted to at least 100% of material's SPMDD using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in accordance with Table 10 of OPSS 310.

The composition of the asphaltic concrete pavement with 90 mm thickness should be as follows:

- Superpave 12.5 mm Surface Course – 40 mm
- Superpave 19.0 mm Base Course – 50 mm

The composition of the asphaltic concrete pavement with 120 mm thickness should be as follows:

- Superpave 12.5 mm Surface Course – 50 mm
- Superpave 19.0 mm Base Course – 70 mm

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 design is based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill and grade raise fill have 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.

5.15.4 Pavement Structure Compaction

Adequate compaction of the granular materials will be essential to the continued acceptable performance of the roadway and parking areas. Compaction should be carried out in conformance with procedures outlined in OPSS 501 "Construction Specification for Compacting" with compacted densities of the various materials being in accordance with Subsection 501.08.02 Method A. The granular base and subbase material should be uniformly compacted to at least 100% of the material's SPMDD using suitable vibratory compaction equipment. Compaction of the asphaltic concrete should be carried out in accordance with OPSS 310, Table 10.

The placement and compaction of any engineered fill, as well as sewer and watermain bedding and backfill, should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction viewpoint. In addition, compaction testing and sampling of the asphaltic concrete used on site should be carried out to make sure that the materials used, and level of compaction achieved during construction meet the project requirements.

5.15.5 Joints, Tie-ins with Existing Pavements, Pavement Resurfacing

At intersections with roadways at the project extents, the new pavement structure should be continued to the limits of construction. At connections to existing pavements, the existing pavement should be milled back beyond the curb return an additional 300 mm to the depth of the new surface course to accept the new surface course asphaltic concrete.

The granular courses and subbase level should be tapered between the new and existing pavements by using 10 horizontal to 1 vertical tapers up or down as required. At driveways and commercial entrances, butt joints may be used.

A tack coat should be provided on all and vertical and milled horizontal surfaces. The tack coat should consist of SS-1 emulsified asphalt diluted with an equal amount of water. The undiluted and emulsified asphalt shall be in conformance with OPSS 1103.

5.16 Site Grading

The subsurface conditions at this site generally consist of fill, or fill underlain by a deposit of glacial till, which are in turn underlain by bedrock.

No practical restrictions apply to the thickness of grade raise fill which may be placed on the site from a foundation design perspective.

5.17 Material Reuse

The existing fill and glacial till materials encountered at this site are not considered to be generally suitable for reuse as structural/engineered fill. Within foundation areas, imported engineered fill such as OPSS Granular B Type II should be used (if required). The existing fill and native overburden soils could however be reused in non-structural areas (i.e., landscaping).

5.18 Trees

The silty clay soils in Ottawa are sensitive to water depletion by trees of high-water demand during periods of dry weather. When trees draw water from the clayey soil, the clay undergoes shrinkage which can result in settlement of adjacent structures.

Based on the results of the geotechnical investigation, the site is not underlain by sensitive silty clay. Therefore, no restrictions on the types or sizes of trees that may be planted or tree to foundation setback distances need to be considered for this development.

5.19 Corrosion and Cement Type

The concentration of sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The sulphate results (see Section 4.7) were compared with Table 3 of Canadian Standards Association Standards A23.1-14 (CSA A23.1) and generally indicate a low degree of sulphate attack potential on concrete structures at the locations of all tested samples. Therefore, concrete made with Type GU Portland cement is considered acceptable for all substructures.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. Generally, the results indicate a moderate potential for corrosion of exposed ferrous metal within the study area, which should be taken into consideration in the design of substructures.

6.0 ADDITIONAL CONSIDERATIONS

If construction is carried out during periods of sustained below freezing temperatures, all subgrade areas should be protected from freezing (e.g., by using insulated tarps and/or heating).

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to document that the correct/expected strata exist and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill, pipe bedding, and pavement base and subbase materials should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction point of view.

At the time of the writing of this report, only conceptual details for the proposed development were available. 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 and to review some of our preliminary recommendations.

7.0 CLOSURE

We trust that this report contains sufficient information for your present purposes. If you have any questions regarding this report or if we can be of further service to you on this project, please reach out us.

Signature page

Golder Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
C. G. HENDRY
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June 27, 2023
PROVINCE OF ONTARIO

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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.

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KEY MAP

SCALE: 1:50,000 m

LEGEND

⊕	APPROXIMATE BOREHOLE LOCATION

REFERENCE(S)

1. PROJECTION: TRANSVERSE MERCATOR, DATUM NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM CGVD28

DRAFT

0 25 50

1:1,000 METRES

CLIENT
FENGATE DEVELOPMENT HOLDINGS LP

PROJECT
GEOTECHNICAL INVESTIGATION
1047 RICHMOND ROAD, OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT
wsp

YYYY-MM-DD	2023-06-27
DESIGNED	----
PREPARED	ZS
REVIEWED	----
APPROVED	----

PROJECT NO.	CONTROL	REV.	FIGURE
21494078	0003	A	1

APPENDIX A

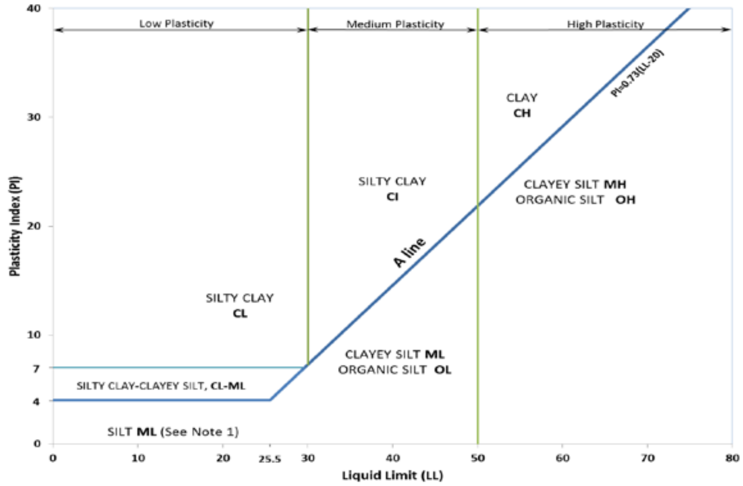
Borehole Records

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 BOREHOLES 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.). Values reported are as recorded in the field and are uncorrected.

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
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
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 to 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 the effects of overburden pressure.
2. Definition of compactness terms are based on SPT ‘N’ ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT ‘N’ value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT ‘N’ value(s) should be considered only an approximate guide to the soil compactness. 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

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
NP	non-plastic
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 $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

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

2 shear strength = (compressive strength)/2

PROJECT: 21494078

RECORD OF BOREHOLE: 21-01

SHEET 1 OF 1

LOCATION: N 5026314.5 ;E 361326.2

BORING DATE: September 24, 2021

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	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ / HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ (ND = Not Detected)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT PERCENT (Wp W Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	65.73							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.00							Flush Mount Casing
		FILL - (SW) gravelly SAND, angular; brown (PAVEMENT STRUCTURE); non-cohesive, moist	0.10 65.43 0.30	1	SS	19	ND		Metals	
1		FILL - (SM) gravelly SILTY SAND; grey to dark brown, trace sand (SP); non-coohesive, moist, compact to very loose		2	SS	4	ND			
			63.90	3	SS	2	ND		PHCs, VOCs	
2	Air Hammer H Bit	BEDROCK (Auger Refusal) (Air hammer from 1.83 m to 7.62 m)	1.83							Bentonite Seal
3										
4										SIlica Sand
5										
6										50 mm Diam. PVC #10 Slot Screen
7			58.11							∇
		End of Borehole	7.62							
8										
9										
10										

Note(s):

1. Water level measured at a depth of 7.63 m (Elev. 58.13 m) on October 5, 2021

2. Borehole log not for geotechnical purposes.

DEPTH SCALE

1 : 50

GOLDER

LOGGED: DG

CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-02

SHEET 1 OF 1

LOCATION: N 5026359.3 ;E 361297.8

BORING DATE: September 21, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

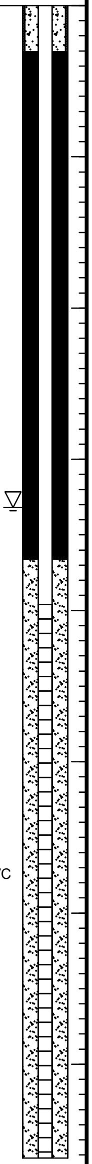
HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ *ND = Not Detected* 100 200 300 400

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ☐ *ND = Not Detected* 100 200 300 400

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

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

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	65.46						
Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.08						Flush Mount Casing
	FILL - (SW) gravellyl SAND, angular; grey (PAVEMENT STRUCTURE); non-cohesive, moist	65.16 0.30	1	SS	22	⊕ ND		
	FILL - (SP) SAND, fine to medium, trace silt; brown; non-cohesive, moist, compact to dense		2	SS	31	⊕ ND	Metals	
	FILL - (SM/GP) SILTY SAND and GRAVEL; dark brown, contains brick fragments and rootlets; non-cohesive, moist, compact	64.24 1.22	3	SS	10	⊕ ND		
	Highly weathered BEDROCK	63.63 1.83	4	SS	>84	⊕ ND		Bentonite Seal
			5	SS	>50	⊕ ND	PHCs, VOCs	
Air Hammer H Bit	BEDROCK (Auger Refusal) (Air hammer from 3.05 M TO 7.62 M)	62.41 3.05						SIlica Sand 50 mm Diam. PVC #10 Slot Screen
	End of Borehole	57.84 7.62						



Note(s):

1. Water level measured at a depth of 3.32 m (Elev. 62.14 m) on October 5, 2021

2. Borehole log not for geotechnical purposes.

DEPTH SCALE

1 : 50

GOLDER

LOGGED: DG

CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-03

SHEET 1 OF 1

LOCATION: N 5026355.1 ;E 361289.2

BORING DATE: September 21 & 22, 2021

DATUM: Geodetic

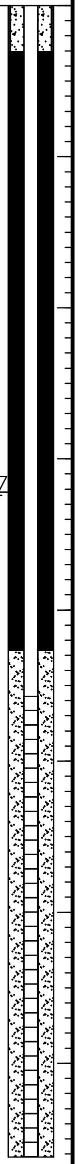
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ / HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ (ND = Not Detected)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	65.24						
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT							Flush Mount Casing
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE); non-cohesive, moist	0.08 64.63	1	SS	43	ND	VOCs	
1		FILL - (SP) SAND, fine to medium, trace silt; brown; non-cohesive, moist, dense	0.61 64.02	2	SS	31	ND	PHCs	
		FILL - (SM) SILTY SAND, some topsoil, trace gravel; dark brown, contains shale fragments; non-cohesive, moist, compact	1.22 63.41	3	SS	12	ND	Metals	
2		Highly weathered BEDROCK	1.83	4	SS	>94	ND		Bentonite Seal
			62.19	5	SS	52/ 6"			
3	Air Hammer H Bit	BEDROCK (Auger Refusal) (Air hammer from 3.05 m to 7.62 m)	3.05						
4									SIlica Sand
5									
6									50 mm Diam. PVC #10 Slot Screen
7			57.62						
		End of Borehole	7.62						

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

WATER CONTENT PERCENT: Wp ├──○W──┤ Wl — 20 40 60 80



Note(s):

1. Water level measured at a depth of 4.22 m (Elev. 62.02 m) on October 5, 2021

2. Borehole log not for geotechnical purposes.

DEPTH SCALE

1 : 50

GOLDER

LOGGED: DG

CHECKED: AG

MIS-BHS 001 21494078.GPJ GAL-MIS.GDT 12/16/21 ZS

PROJECT: 21494078

RECORD OF BOREHOLE: 21-04

SHEET 1 OF 1

LOCATION: N 5026369.7 ;E 361313.7

BORING DATE: September 21, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected (100, 200, 300, 400)

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ☐ ND = Not Detected (100, 200, 300, 400)

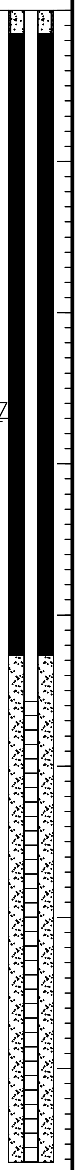
HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3})

WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20, 40, 60, 80)

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
		GROUND SURFACE	65.09
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.05
		FILL - (SM) SILTY SAND, trace gravel; brown to grey brown, contains wood fragments; non-cohesive, moist, loose to compact	62.65 2.44
		(SM) gravelly SILTY SAND; grey brown, contains cobbles and boulders (GLACIAL TILL); non-cohesive, wet, dense	61.43 3.66
	Air Hammer H Bit	BEDROCK (Auger Refusal) (Air hammer from 3.66 m to 7.62 m)	57.47 7.62
		End of Borehole	

SAMPLE NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR	ADDITIONAL LAB. TESTING
1	SS	9	ND	VOCs
2	SS	10	ND	
3	SS	7	ND	
4	SS	14	ND	
5	SS	49	ND	
6	SS	55/ 4"	ND	PHCs

PIEZOMETER OR STANDPIPE INSTALLATION



Flush Mount Casing

Bentonite Seal

SIlica Sand

50 mm Diam. PVC #10 Slot Screen

Note(s):

1. Water level measured at a depth of 2.70 m (Elev. 62.39 m) on October 5, 2021

2. Borehole log not for geotechnical purposes.

DEPTH SCALE

1 : 50

GOLDER

LOGGED: DG

CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-05

SHEET 1 OF 1

LOCATION: N 5026358.2 ;E 361327.9

BORING DATE: September 22/24, 2021

DATUM: Geodetic

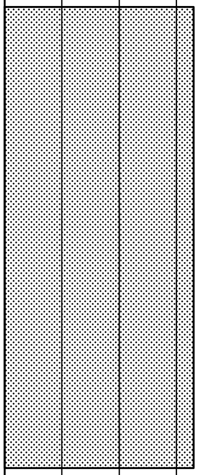
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
	GROUND SURFACE	65.47
Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.08
	FILL - (SP) SAND, fine to coarse, some gravel, trace silt; brown; non-cohesive, moist, compact	64.86 0.61
	FILL - (SM/GW) SILTY SAND and GRAVEL; dark brown, contains wood fragments; non-cohesive, moist, compact	64.02 1.45
	Possible FILL - (SP) SILTY SAND, fine to coarse, trace silt, trace gravel; grey brown; non-cohesive, moist, compact to dense	62.73 2.74
	(SM) gravelly SILTY SAND, non-plastic fines; grey brown, contains cobbles (GLACIAL TILL); non-cohesive, moist, dense	61.82 3.65
Air Hammer H Bit	BEDROCK (Auger Refusal) (Air hammer from 3.65 m to 7.62 m)	57.85 7.62
	End of Borehole	

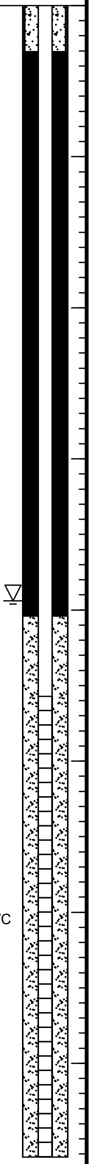
SAMPLE NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ / HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ (ND = Not Detected)	ADDITIONAL LAB. TESTING
1	SS	15	ND	
2	SS	20	ND	PHCs, VOCs
3	SS	52/ 0"	ND	PHCs, VOCs
4	SS	20	ND	
5	SS	39	ND	
6	SS	46	ND	
7	SS	34/ 10"	ND	

HYDRAULIC CONDUCTIVITY, k, cm/s



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

PIEZOMETER OR STANDPIPE INSTALLATION



Note(s):

1. Water level measured at a depth of 3.94 m (Elev 61.53 m) on October 5, 2021

2. Borehole log not for geotechnical purposes.

DEPTH SCALE
1 : 50

GOLDER

LOGGED: DG
CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-06

SHEET 1 OF 2

LOCATION: N 5026317.1 ;E 361275.1

BORING DATE: September 30, 2021

DATUM: Geodetic

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
		GROUND SURFACE		65.00							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT		0.05							Flush Mount Casing
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE); non-cohesive, moist		0.20							
		FILL - (SP) SAND, fine to medium, trace silt; brown; non-cohesive, moist, loose									
1		FILL - (SM) gravelly SILTY SAND; grey brown, contains organic matter, possible cobbles; non-cohesive, moist, loose		64.09 0.91	1	SS	37				Bentonite Seal
		Highly weathered BEDROCK		63.63 1.37	2	SS	>76				
2		Borehole continued on RECORD OF DRILLHOLE 21-06		63.12 1.88							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-06

SHEET 2 OF 2

LOCATION: N 5026317.1 ;E 361275.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 30, 2021

DRILL RIG: CME 55

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

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

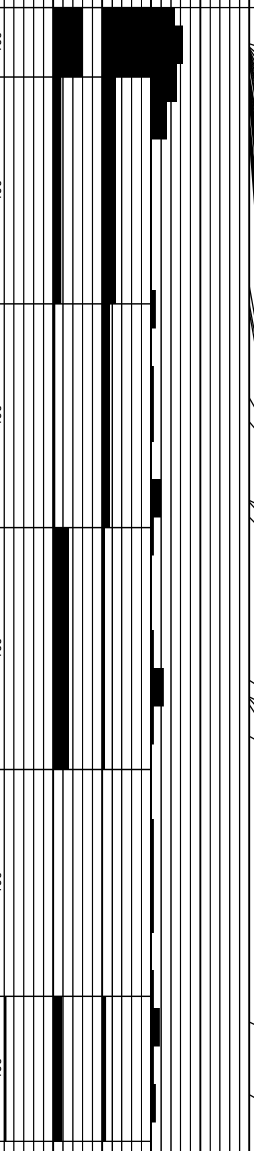
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN
		BEDROCK SURFACE	63.12		
2	Rotary Drill HQ3 Core	Slightly weathered to fresh, medium to thickly bedded, medium grey, fine grained, faintly porous, medium strong DOLOSTONE, interbedded with shale, limestone and sandstone - Broken core from 1.88 m to 2.07 m - Broken core from 2.34 m to 2.38 m - Broken core from 2.41 m to 2.43 m	1.88	1	100
3				2	100
4				3	100
5		- Broken core from 5.11 m to 5.14 m			
6				4	100
		- Broken core from 6.47 m to 6.49 m			
7					
8				5	100
		- Lost core from 8.56 m to 8.59 m			
9			55.62 9.38	6	100
		End of Drillhole Note(s): 1. Water level measured at a depth of 6.84 m (Elev. 58.16 m) on October 5, 2021			
10					
11					

RECOVERY: TOTAL CORE % (80 60 40 20); 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) — TYPE AND SURFACE DESCRIPTION — Jcon — Jr — Ja

BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM SO
BD,PL,SM
BD,PL,SM SO
BD,PL,SM
BD,PL,RO SO
BD,PL,SM SO
BD,PL,SM SO
BD,UN,SM
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM DC,SI <1 mm
BD,UN,SM DC,CL 5 mm
BD,UN,RO DC,CL <1 mm
BD,PL,SM SO
BD,PL,SM

BD,PL,SM
BD,PL,SM
BD,UN,SM SO
BD,PL,SM

AJN,PL,H IN,CA <1 mm

BD,PL,SM
BD,PL,SM DC,CL <1 mm
JN,PL,SM DC,CL <1 mm
BD,PL,SM
BD,PL,SM DC,SI <1 mm
JN,PL,SM

BD,PL,SM
BD,PL,SM

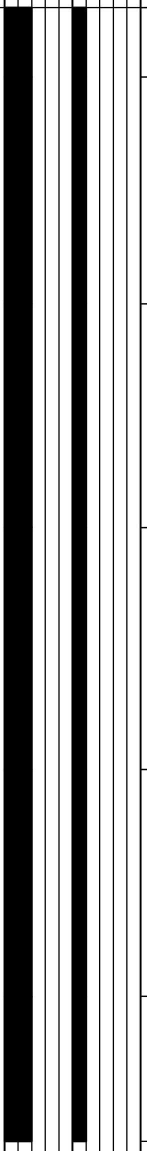
BD,PL,SM

BD,PL,SM
BD,PL,SM DC,SI,SA
2 mm

BD,PL,SM

BD,PL,SM
BD,PL,SM

ROCK STRENGTH INDEX (R4 R3 R2 R1) — WEATHERING INDEX (W1 W2 W3 W4) — Q AVG.



Bentonite Seal

Silica Sand

52 mm Diam. PVC
#10 Slot Screen

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-07

SHEET 1 OF 2

LOCATION: N 5026297.0 ;E 361328.4

BORING DATE: September 30, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	66.07					
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.05					Flush Mount Casing
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE); non-cohesive, moist	65.82 0.25					
		FILL - (SP) SAND, fine to medium, trace sand; brown; non-cohesive, moist	65.64 0.43					
		FILL - (SM) gravelly SILTY SAND; dark brown; non-cohesive, moist, loose	65.16 0.91					Bentonite Seal
1		Highly weathered BEDROCK		1	SS	92		
			64.45	2	SS	50/4"		
		Borehole continued on RECORD OF DRILLHOLE 21-07	1.62					
2								
3								
4								
5								
6								
7								
8								
9								
10								

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-07

SHEET 2 OF 2

LOCATION: N 5026297.0 ;E 361328.4

INCLINATION: -90° AZIMUTH: ---

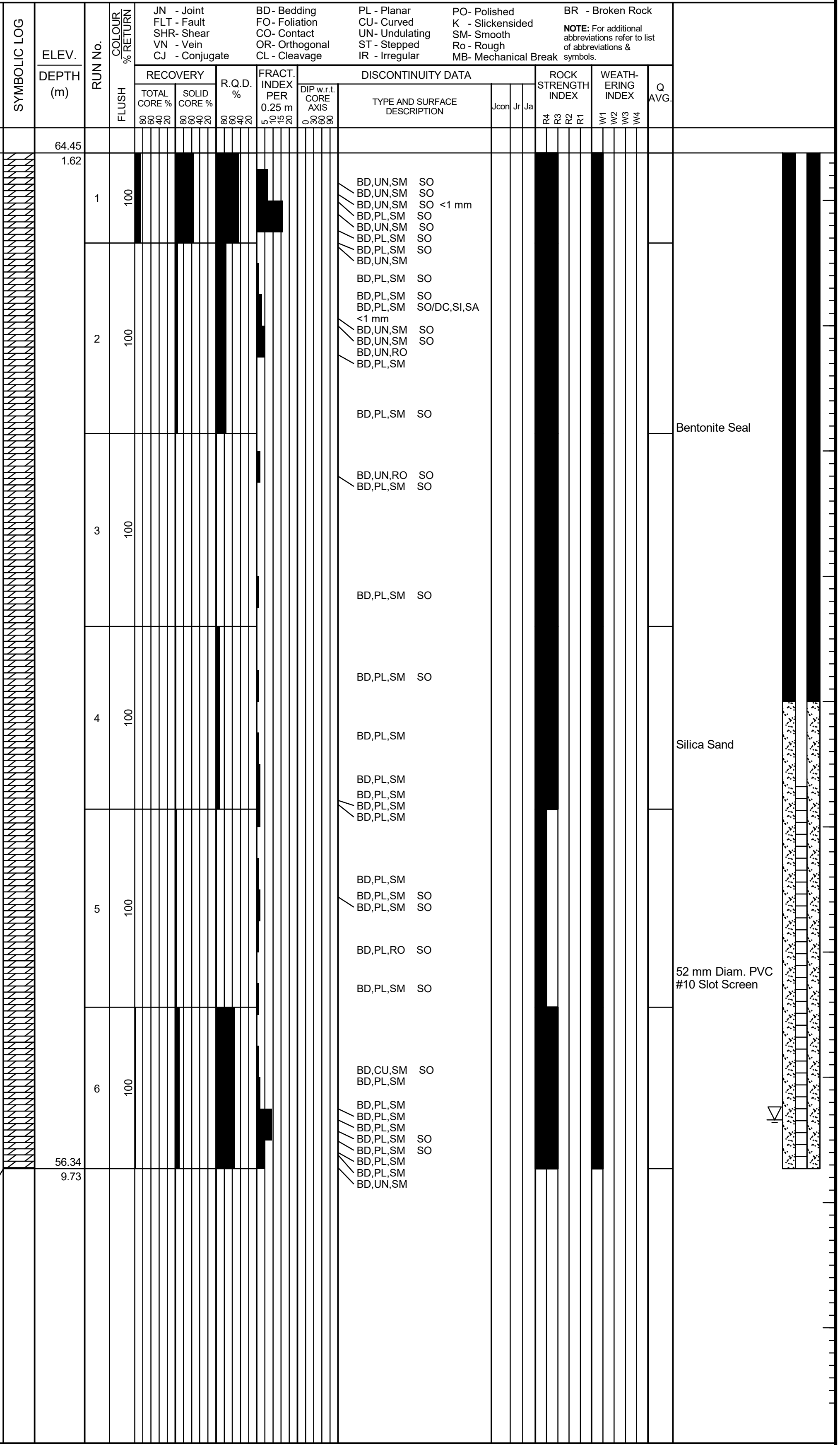
DRILLING DATE: September 30, 2021
DRILL RIG: CME 55
DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

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	ELEV. DEPTH (m)	RUN No.	FLUSH % RETURN
		BEDROCK SURFACE	64.45		
	Rotary Drill HQ3 Core	Slightly weathered to fresh, medium to thickly bedded, medium grey, fine grained, non to faintly porous, medium strong DOLOSTONE, interbedded with shale, limestone and sandstone - Broken core from 1.85 m to 1.86 m - Broken/lost core from 1.95 m to 2.01 m - Broken/lost core from 2.11 m to 2.29 m - Broken core from 2.34 m to 2.37 m - Broken core from 3.21 m to 2.25 m - Broken core from 4.19 m to 4.2 m - Broken core from 7.55 m to 5.56 m - Broken/lost core from 9.43 m to 9.51 m - Broken core from 9.72 m to 9.73 m	1.62	1	100
2				2	100
3				3	100
4				4	100
5				5	100
6				6	100
9.73		End of Drillhole	56.34 9.73		

Depth scale: 2, 3, 4, 5, 6, 7, 8, 9, 10, 11



DISCONTINUITY DATA – TYPE AND SURFACE DESCRIPTION

BD,UN,SM SO
BD,UN,SM SO
BD,UN,SM SO <1 mm
BD,PL,SM SO
BD,UN,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,UN,SM
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO/DC,SI,SA <1 mm
BD,UN,SM SO
BD,UN,SM SO
BD,UN,RO
BD,PL,SM
BD,PL,SM SO
BD,UN,RO SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM SO
BD,PL,SM SO
BD,PL,RO SO
BD,PL,SM SO
BD,CU,SM SO
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM
BD,PL,SM
BD,UN,SM

Bentonite Seal

Silica Sand

52 mm Diam. PVC
#10 Slot Screen

Note(s):

1. Water level measured at a depth of 9.34 m (Elev. 56.73 m) on October 5, 2021

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-08

SHEET 1 OF 3

LOCATION: N 5026385.1 ;E 361306.5

BORING DATE: September 28, 2021

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	TYPE	BLOWS/0.30m
		GROUND SURFACE	64.64			
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.05			
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE); non-cohesive, moist	0.16			
		FILL - (SP) SAND, fine to medium, trace silt; brown; non-cohesive, moist	64.11 0.53			
1		FILL - (SM) gravelly SILTY SAND; dark brown, contains organic matter (rootlets); non-cohesive, moist, loose to compact		1	SS	6
2		(SM) gravelly SILTY SAND; grey to grey brown, trace organic matter, weathered shale and thick laminations to thin beds of sand, fine to medium (GLACIAL TILL); non-cohesive, moist, compact to very dense	62.81 1.83	2	SS	23
				3	SS	56
3		Highly weathered BEDROCK	61.59 3.05	4	SS	50/ 6"
		Borehole continued on RECORD OF DRILLHOLE 21-08	3.2			
4						
5						
6						
7						
8						
9						
10						

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

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-08

SHEET 2 OF 3

LOCATION: N 5026385.1 ;E 361306.5

INCLINATION: -90°   AZIMUTH: ---

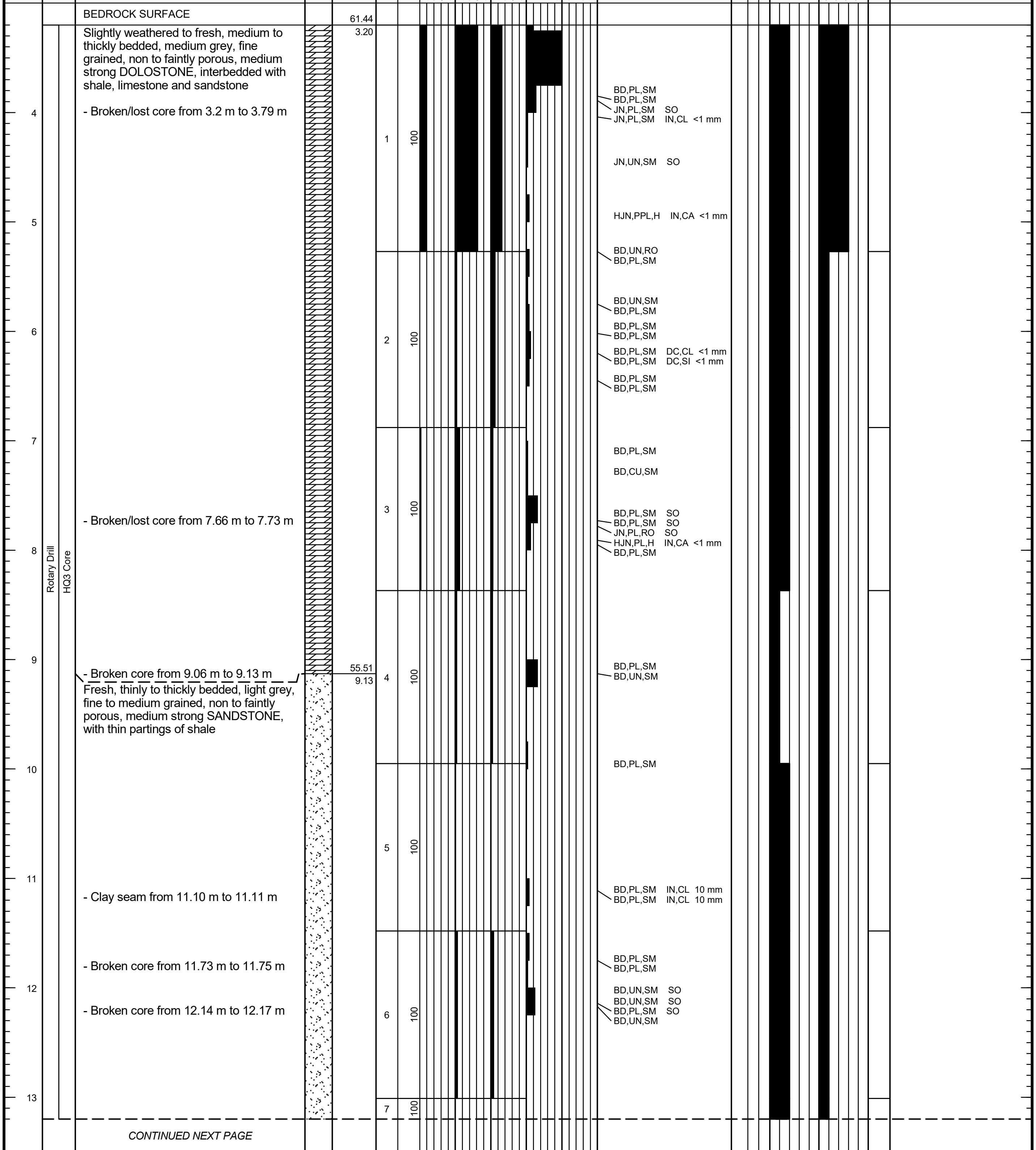
DRILLING DATE: September 28, 2021

DRILL RIG: CME 55

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

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	



ELEV. DEPTH (m)	DESCRIPTION
	BEDROCK SURFACE
61.44 3.20	Slightly weathered to fresh, medium to thickly bedded, medium grey, fine grained, non to faintly porous, medium strong DOLOSTONE, interbedded with shale, limestone and sandstone - Broken/lost core from 3.2 m to 3.79 m - Broken/lost core from 7.66 m to 7.73 m - Broken core from 9.06 m to 9.13 m
55.51 9.13	Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE, with thin partings of shale - Clay seam from 11.10 m to 11.11 m - Broken core from 11.73 m to 11.75 m - Broken core from 12.14 m to 12.17 m

RUN No.	FLUSH % RETURN	TYPE AND SURFACE DESCRIPTION
1	100	BD,PL,SM BD,PL,SM JN,PL,SM SO JN,PL,SM IN,CL <1 mm JN,UN,SM SO HJN,PPL,H IN,CA <1 mm
2	100	BD,UN,RO BD,PL,SM BD,UN,SM BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM DC,CL <1 mm BD,PL,SM DC,SI <1 mm BD,PL,SM BD,PL,SM
3	100	BD,PL,SM BD,CU,SM BD,PL,SM SO BD,PL,SM SO JN,PL,RO SO HJN,PL,H IN,CA <1 mm BD,PL,SM
4	100	BD,PL,SM BD,UN,SM BD,PL,SM
5	100	BD,PL,SM IN,CL 10 mm BD,PL,SM IN,CL 10 mm
6	100	BD,PL,SM BD,PL,SM BD,UN,SM SO BD,UN,SM SO BD,PL,SM SO BD,UN,SM
7	100	

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-08

SHEET 3 OF 3

LOCATION: N 5026385.1 ;E 361306.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 28, 2021

DRILL RIG: CME 55

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

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	ROCK STRENGTH INDEX R4 R3 R2 R1	WEATH-ERING INDEX W1 W2 W3 W4	Q AVG.
		--- CONTINUED FROM PREVIOUS PAGE ---																
14	Rotary Drill HQ3 Core	Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE, with thin partings of shale - Lost core from 13.59 m to 13.60 m			7	100						BD,PL,SM SO BD,UN,SM SO BD,UN,SM SO						
15				49.15 15.49	8	100						BD,UN,RO						
16		End of Drillhole																
17																		
18																		
19																		
20																		
21																		
22																		
23																		

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-09

SHEET 1 OF 3

LOCATION: N 5026279.3 ;E 361293.7

BORING DATE: September 29, 2021

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	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
		GROUND SURFACE		65.90							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT		0.05							Flush Mount Casing
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE)		65.65 0.25							
		FILL - (SP) SAND, fine to medium, trace to some silt; brown; non-cohesive, moist		65.34 0.56							
1		FILL - (SM/ML) gravelly SILTY SAND to sandy SILT; brown to dark brown, contains weathered shale and organic matter; non-cohesive, moist, loose		64.38 1.52	1	SS	5				Bentonite Seal
		Highly weathered BEDROCK		1.65	2	SS	50/ 5"				
		Borehole continued on RECORD OF DRILLHOLE 21-09									
2											
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-09

SHEET 2 OF 3

LOCATION: N 5026279.3 ;E 361293.7

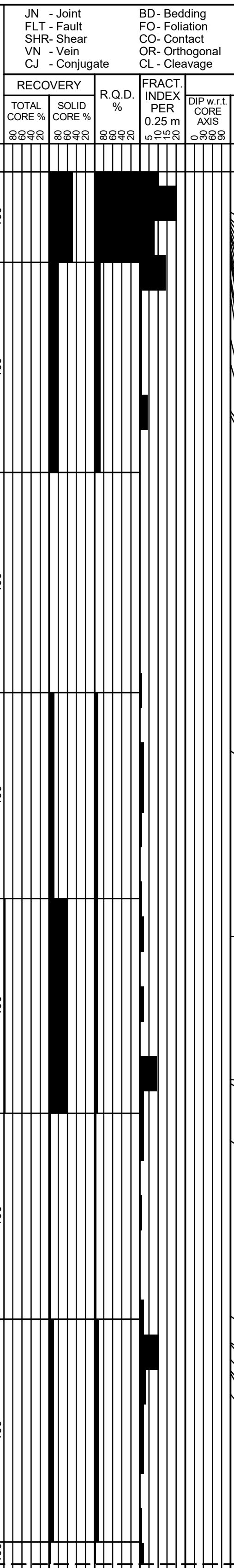
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 29, 2021
DRILL RIG: CME 55
DRILLING CONTRACTOR: CCC Drilling

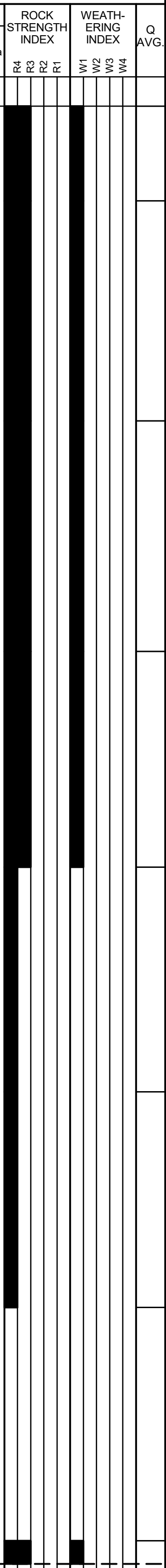
DATUM: Geodetic

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	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION
		BEDROCK SURFACE	64.25 1.65			
2	Rotary Drill HQ3 Core	Fresh, medium to thickly bedded, medium grey, fine grained, non to faintly porous, medium strong DOLOSTONE, interbedded with shale, limestone and sandstone		1	100	BD,UN,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO
3		- Broken core from 1.65 m to 1.92 m - Broken core from 2.3 m to 2.41 m - Broken core from 3.37 m to 3.4 m		2	100	BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,UN,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO JN,UN,RO SO BD,PL,SM BD,PL,SM BD,UN,SM SO BD,PL,SM SO
4 5				3	100	BD,PL,SM SO
6				4	100	BD,PL,SM BD,CU,SM BD,PL,SM BD,PL,SM SO JN,PL,SM SO
7 8		- Broken/lost core from 8.09 m to 8.17 m		5	100	BD,PL,SM SO JN,UN,RO SO JN,PL,RO SO BD,UN,SM SO BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM SO
9				6	100	BD,PL,SM BD,PL,SM SO BD,PL,SM SO BD,PL,SM
10 11		- Broken/lost core from 9.86 m to 9.87 m - Broken core from 10.18 m to 10.26 m - Broken core from 10.73 m to 10.76 m		7	100	BD,PL,SM SO BD,PL,SM BD,PL,SM BD,PL,SM IN,CL 10 mm BD,PL,SM IN,CL 10 mm BD,UN,SM IN,CL 20 mm BD,PL,SM SO BD,PL,SM SO BD,UN,SM SO BD,PL,SM SO
		Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE, with thin partings of shale	54.73 11.17			BD,UN,RO
				8	100	
		CONTINUED NEXT PAGE				



Jcon Jr Ja



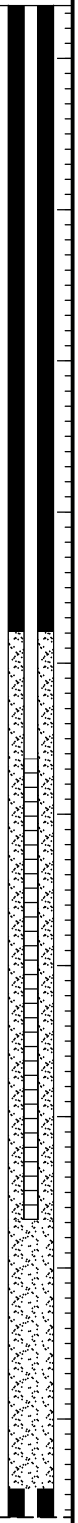
Bentonite Seal

Silica Sand

52 mm Diam. PVC #10 Slot Screen

Silica Sand

Bentonite Seal



DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-09

SHEET 3 OF 3

LOCATION: N 5026279.3 ;E 361293.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 29, 2021

DRILL RIG: CME 55

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

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 DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	ROCK STRENGTH INDEX (R4 R3 R2 R1)	WEATH-ERING INDEX (W1 W2 W3 W4)	Q AVG.	
		--- CONTINUED FROM PREVIOUS PAGE ---																	
12	Rotary Drill HQ3 Core	Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE, with thin partings of shale - Broken core from 11.67 m to 11.68 m - Lost core from 12.42 m to 12.43 m			8	100						BD,UN,SM BD,UN,SM BD,UN,RO BD,UN,SM							
13 14		- Broken core from 13.84 m to 13.85 m			9	100						BD,UN,SM BD,PL,SM SO							Bentonite Seal
15				50.40 15.50	10	100													
16 17 18 19 20 21		End of Drillhole Note(s): 1. Borehole was dry on October 5, 2021																	

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-10

SHEET 1 OF 3

LOCATION: N 5026360.8 ;E 361363.7

BORING DATE: September 29, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

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

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	65.89					
0	ASPHALT						Flush Mount Casing
	FILL - (SM) gravelly SILTY SAND; brown; non-cohesive, moist	0.05					
	FILL - (SM) gravelly SILTY SAND; grey brown, trace organic matter; non-cohesive, moist, compact	65.15 0.74					
1			1	SS	10		
	(SM) gravelly SILTY SAND; grey brown, contains cobbles and boulders (GLACIAL TILL); non cohesive, moist, dense to very dense	64.37 1.52	2	SS	46		
			3	SS	73		Bentonite Seal
3			4	RC	DD		
4			5	RC	DD		
			6	SS	>50		
	Borehole continued on RECORD OF DRILLHOLE 21-10	61.09 4.8					
5							
6							
7							
8							
9							
10							

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-10

SHEET 2 OF 3

LOCATION: N 5026360.8 ;E 361363.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 29, 2021

DRILL RIG: CME 55

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

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

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

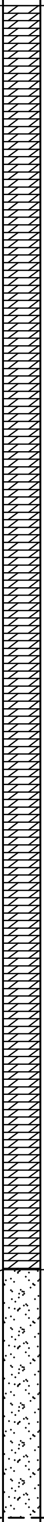
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

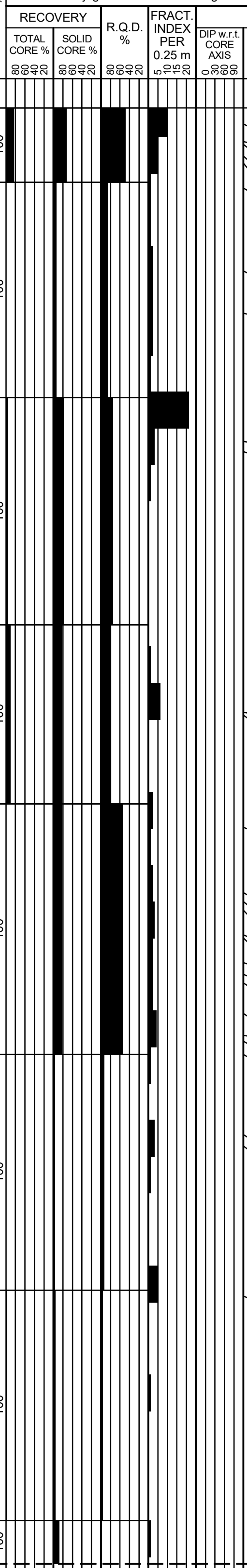
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

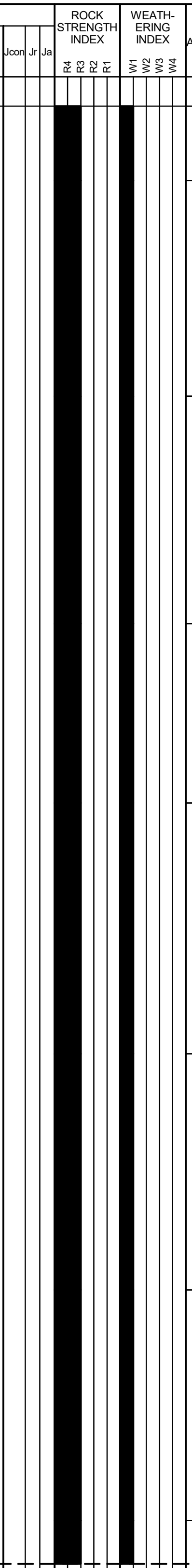
BR - Broken Rock

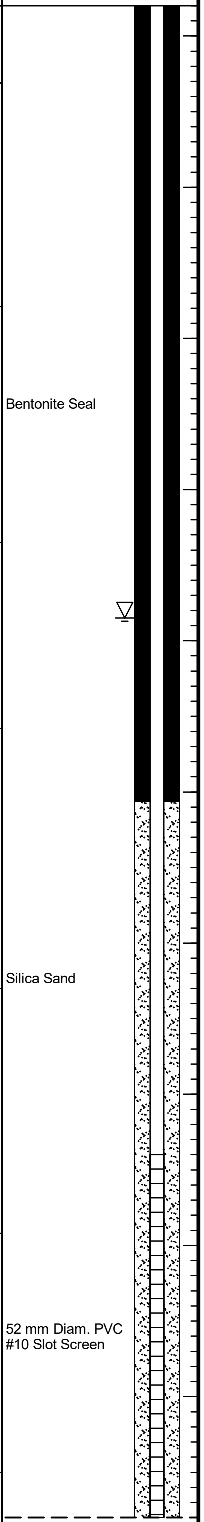
NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN / FLUSH	DISCONTINUITY DATA — TYPE AND SURFACE DESCRIPTION
		BEDROCK SURFACE	61.09			
5	Rotary Drill HQ3 Core	Fresh, medium to thickly bedded. medium grey, fine grained, non to faintly porous, medium strong DOLOSTONE, interbedded with shale, limestone and sandstone - Broken/lost core from 4.8 m to 4.88 m - Broken core from 5.03 m to 5.05 m	4.80	1	100	BD,PL,SM BD,CU,SM SO BD,CU,SM SO BD,UN,SM BD,PL,SM SO BD,PL,SM SO BD,UN,SM CC,CA <1 mm
6				2	100	HJN,PL,H IN,CA <1 mm BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM HJN,PL,H IN BD,PL,SM Ca 3-5 mm BD,PL,SM
7		- Broken/lost core from 6.79 m to 7.02 m - Broken core from 7.09 m to 7.16 m		3	100	BD,CU,SM BD,PL,SM DC,CL <1 mm BD,PL,RO BD,PL,SM BD,PL,SM BD,PL,SM
8						
9		- Broken/lost core from 8.72 m to 8.88 m - Broken core from 8.93 m to 8.97 m		4	100	BD,PL,RO BD,PL,SM BD,UN,SM BD,PL,SM
10				5	100	BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,RO BD,PL,SM BD,CU,SM BD,CU,SM BD,UN,SM BD,PL,SM BD,CU,SM BD,PL,SM BD,UN,SM BD,PL,SM BD,UN,SM BD,PL,SM
11						
12				6	100	BD,PL,SM BD,CU,SM DC,SI <1 mm BD,CU,SM BD,UN,SM DC,SI <1 mm
13		- Broken/lost core from 12.92 m to 12.96 m	52.73 13.16			BD,UN,SM BD,UN,SM
		Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE with thin partings of shale		7	100	BD,PL,SM
14				8	100	JN,PL,RO









CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-10

SHEET 3 OF 3

LOCATION: N 5026360.8 ;E 361363.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 29, 2021
DRILL RIG: CME 55
DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
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OR- Orthogonal
CL - Cleavage
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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 DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	ROCK STRENGTH INDEX R4 R3 R2 R1	WEATH-ERING INDEX W1 W2 W3 W4	Q AVG.	
		--- CONTINUED FROM PREVIOUS PAGE ---																	
15	Rotary Drill HQ3 Core	Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE with thin partings of shale		50.44 15.45	8	100						JN,PL,RO							52 mm Diam. PVC #10 Slot Screen
		End of Drillhole Note(s): 1. Water level measured at a depth 8.85 m (Elev. 57.04 m) on October 5, 2021																	
16																			
17																			
18																			
19																			
20																			
21																			
22																			
23																			
24																			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: AG

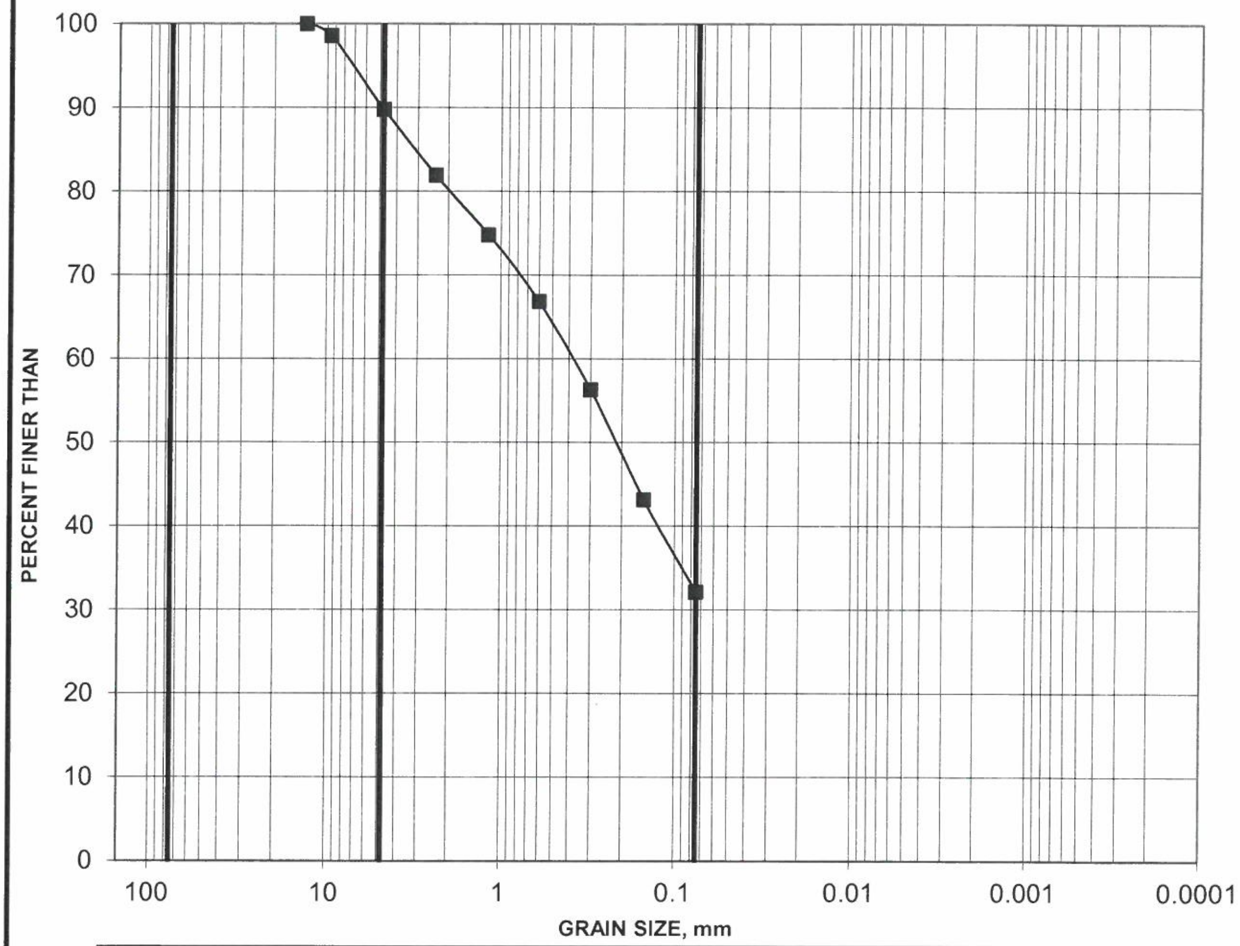
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FIGURE B-1

SILTY SAND (FILL)



<table>
<tr><td rowspan="2">COBBLE SIZE</td><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIU</td><td>FINE</td><td rowspan="2">SILT AND CLAY</td></tr>
<tr><td colspan="2">GRAVEL SIZE</td><td colspan="3">SAND SIZE</td></tr>
</table>

<table>
<tr><td colspan="4"></td><td colspan="4">Constituents (%)</td></tr>
<tr><td></td><td>Borehole</td><td>Sample</td><td>Depth (m)</td><td>Gravel</td><td>Sand</td><td>Silt</td><td>Clay</td></tr>
<tr><td>—■—</td><td>21-01</td><td>2</td><td>0.61-1.22</td><td>10</td><td>58</td><td colspan="2">32</td></tr>
</table>

Project: 21494078/3000

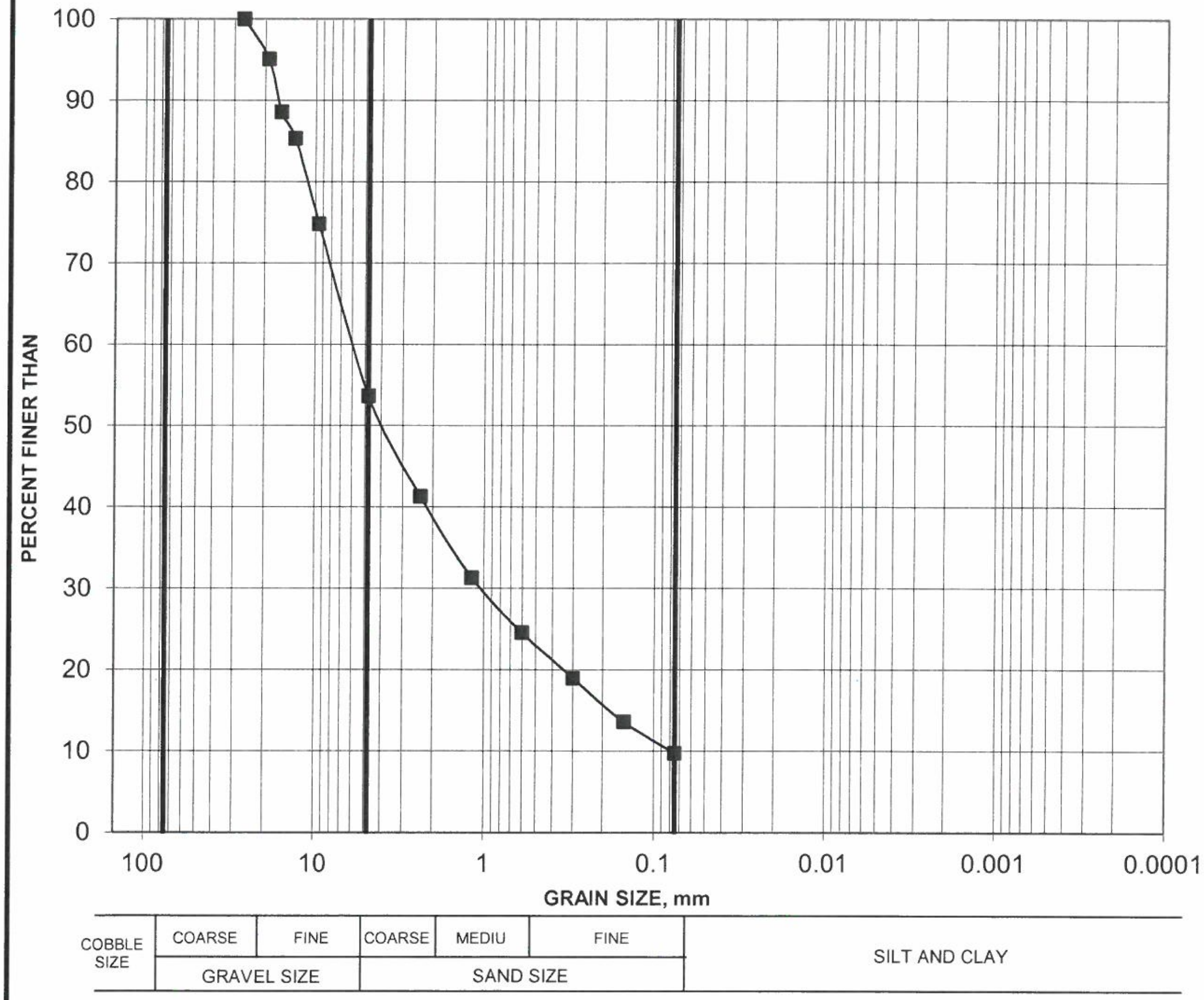
GOLDER
MEMBER OF WSP

Created by:
Checked by: SB

GRAIN SIZE DISTRIBUTION

FIGURE
B-2

GRAVELLY SAND (FILL)



				Constituents (%)			
	Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
—■—	21-02	3	1.22-1.83	46	44	10	

GOLDER
MEMBER OF WSP

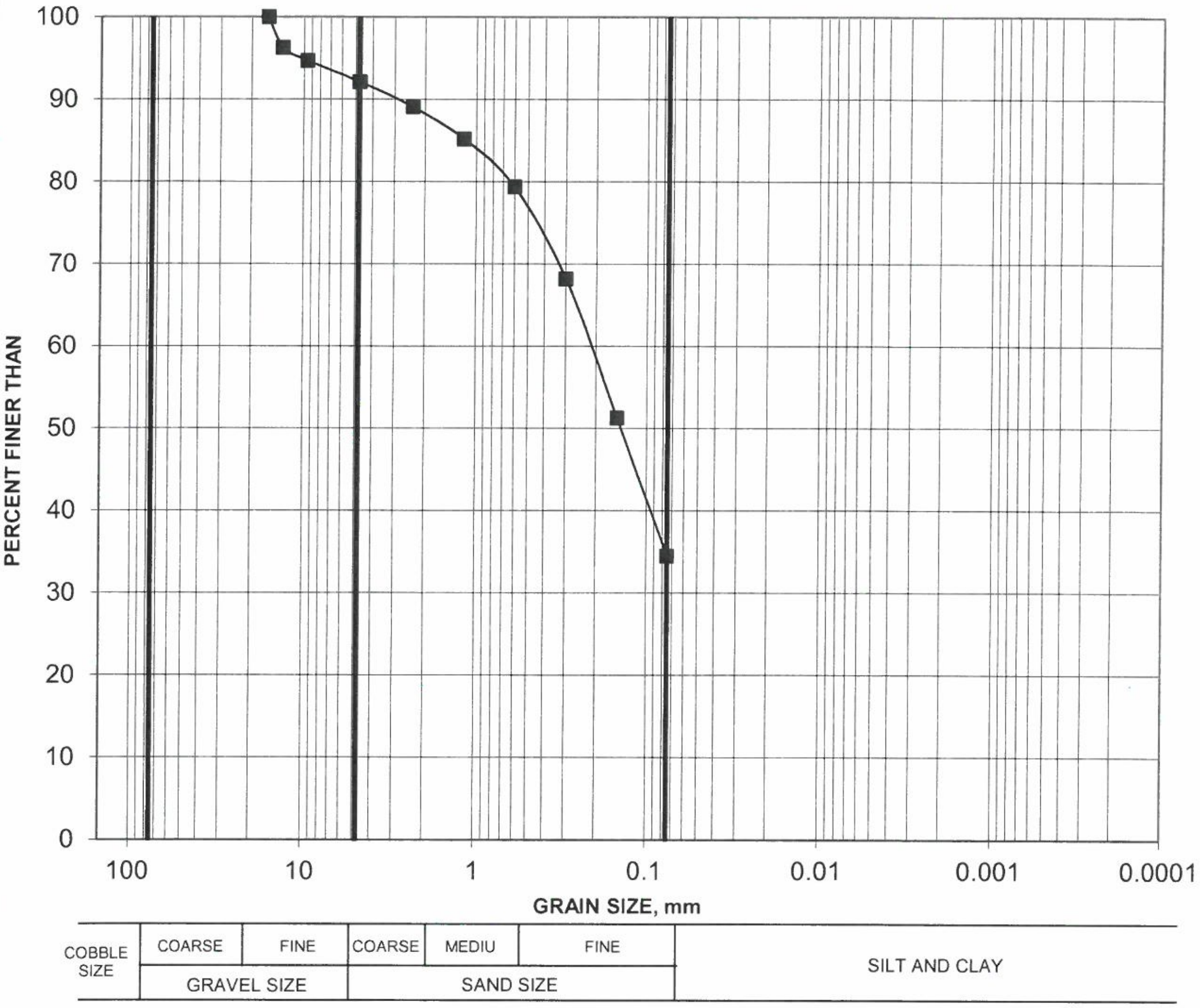
Project: 21494078

Created by: CW

Checked by: JB

GRAIN SIZE DISTRIBUTION	FIGURE B-3

SILTY SAND (GLACIAL TILL)



				Constituents (%)			
	Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■	21-08	3A	2.29-2.44	8	57	35	

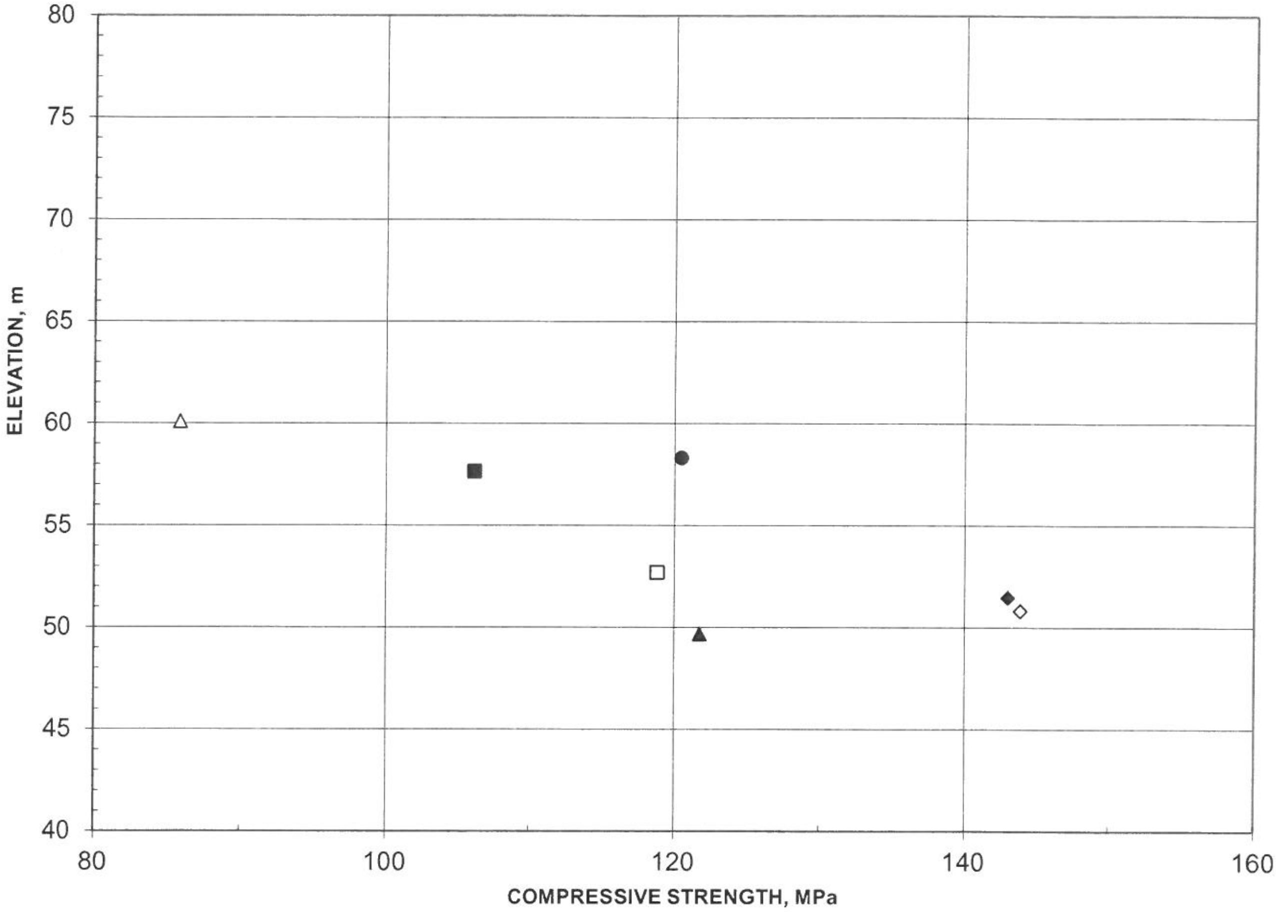
Project: 21494078

GOLDER
MEMBER OF WSP

Created by:
Checked by: JB

ASTM D7012 - Method C
UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE
SUMMARY OF LABORATORY TEST RESULTS

FIGURE
B-4



	Borehole	Depth (m)	L/D	Bulk Density (kg/m^3)	Lithology	UCS (MPa)	Failure Type
—■—	BH21-06 RC1	7.4	2.1	2669	shale/limestone	106	1
—◆—	BH21-08 RC1	13.2	2.1	2610	limestone	143	1
—▲—	BH21-08 RC2	15.0	2.1	2580	limestone	122	1
—●—	BH21-09 RC1	7.6	2	2640	limestone	120	1
—□—	BH21-09 RC2	13.2	2.0	2500	limestone	119	1
—◇—	BH21-09 RC3	15.1	2	2542	limestone	144	1
—△—	BH21-10 RC1	5.8	2.1	2671	shale/limestone	86	1

Notes:

Failure Types

1. Well formed cones on both ends
2. Well formed cones on one end, vertical cracks through cap
3. Columnar vertical craking through both ends
4. Diagonal fracture with no cracking through ends
5. Side fractures at top or bottom
6. Side fractures at both sides of top or bottom

Remarks

- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- Time to failure > 2 and < 15 minutes.

Project: 21494078/3000

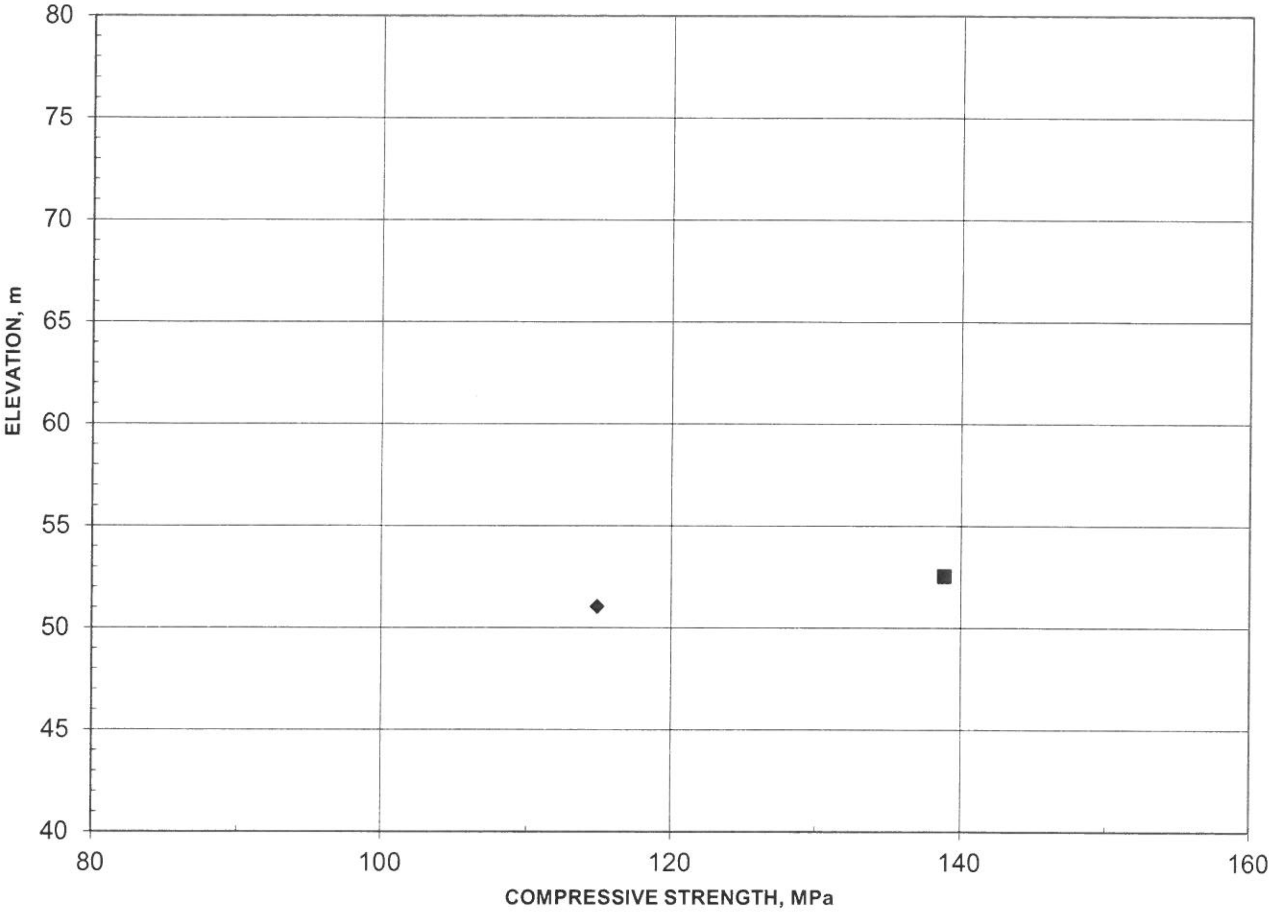
GOLDER
MEMBER OF WSP

Created by:	CW
Checked by:	JB

https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2021/21494078/

ASTM D7012 - Method C
UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE
SUMMARY OF LABORATORY TEST RESULTS

FIGURE
B-5



	Borehole	Depth (m)	L/D	Bulk Density (kg/m^3)	Lithology	UCS (MPa)	Failure Type
—■—	BH21-10 RC2	13.3	2.2	2550	limestone	139	1
—◆—	BH21-10 RC3	14.8	2.2	2543	limestone	115	1

Notes:

Failure Types

1. Well formed cones on both ends
2. Well formed cones on one end, vertical cracks through cap
3. Columnar vertical craking through both ends
4. Diagonal fracture with no cracking through ends
5. Side fractures at top or bottom
6. Side fractures at both sides of top or bottom

Remarks

- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- Time to failure > 2 and < 15 minutes.

Project: 21494078/3000

GOLDER
MEMBER OF WSP

Created by: CW
Checked by: JB

https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2021/21494078/

APPENDIX C

Bedrock Core Photographs

BH 21-06 (Dry)
Rock core from a depth of 1.9 m to 9.4 m
Core Box 1 to 3 of 3

1.9 m

9.4 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

GOLDER

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-06
1 to 3 of 3

BH 21-06 (Wet)
Rock core from a depth of 1.9 m to 9.4 m
Core Box 1 to 3 of 3

1.9 m

9.4 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

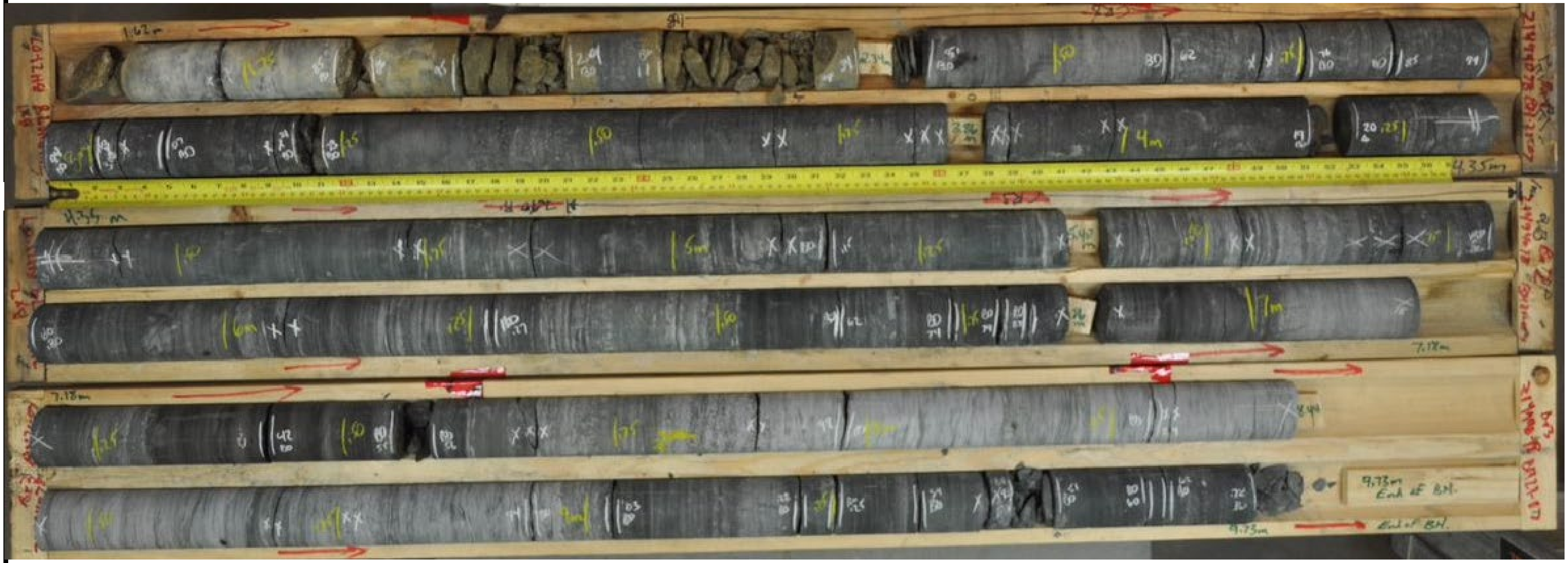
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-06
1 to 3 of 3

BH 21-07 (Dry)
Rock core from a depth of 1.6 m to 9.7 m
Core Box 1 to 3 of 3

1.6 m



9.7 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-07
1 to 3 of 3

BH 21-07 (Wet)
Rock core from a depth of 1.6 m to 9.7 m
Core Box 1 to 3 of 3

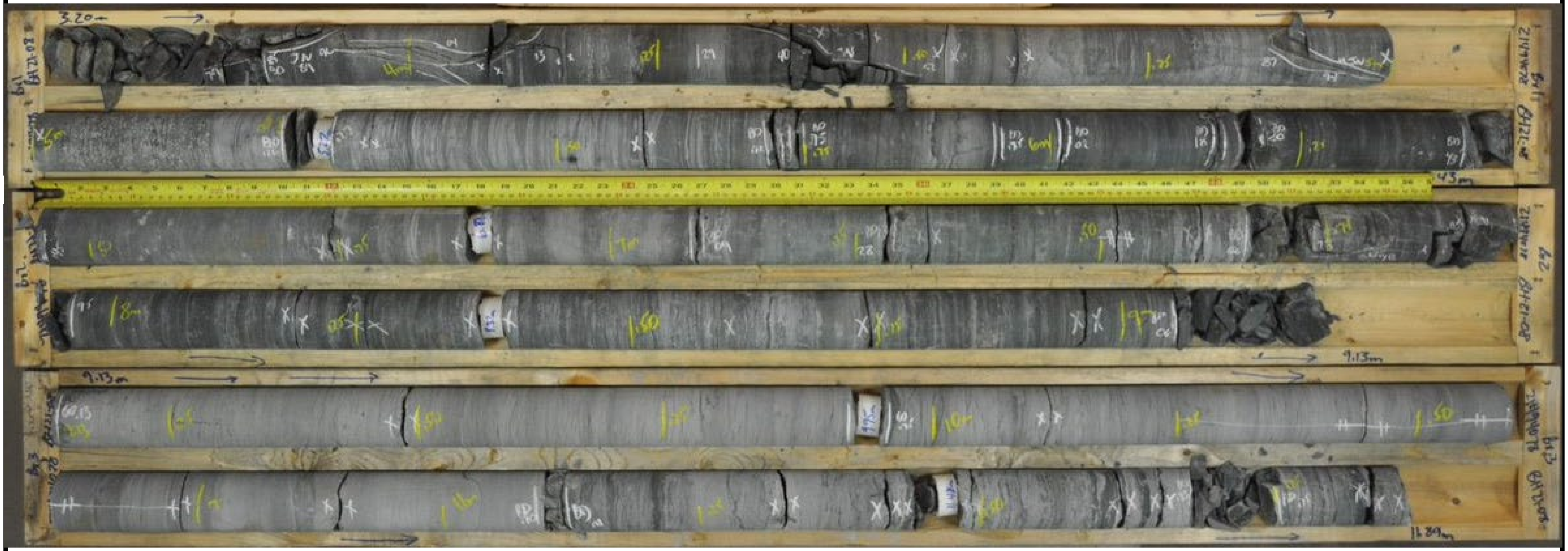
1.6 m

9.7 m

GOLDER	**Environmental Assessment, Geotechnical and Hydrogeological Investigation** **21494078- LPF Ph One Two RSC Richmond** **Ottawa, ON**	Project No. 21494078 Drawn: AG Date: 2021-10-08 Checked: AG Review: WC	**BH 21-07** **1 to 3 of 3**

BH 21-08 (Dry)
Rock core from a depth of 3.2 m to 11.2 m
Core Box 1 to 3 of 5

3.2 m



11.2 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

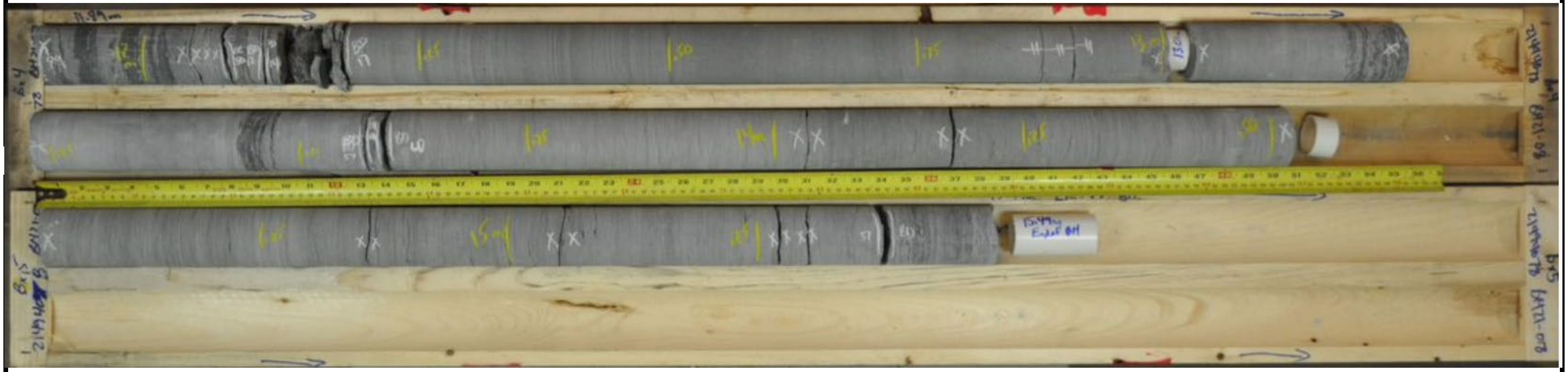
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-08
1 to 3 of 5

BH 21-08 (Dry)
Rock core from a depth of 11.2 m to 15.5 m
Core Box 4 to 5 of 5

11.2 m



15.5 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

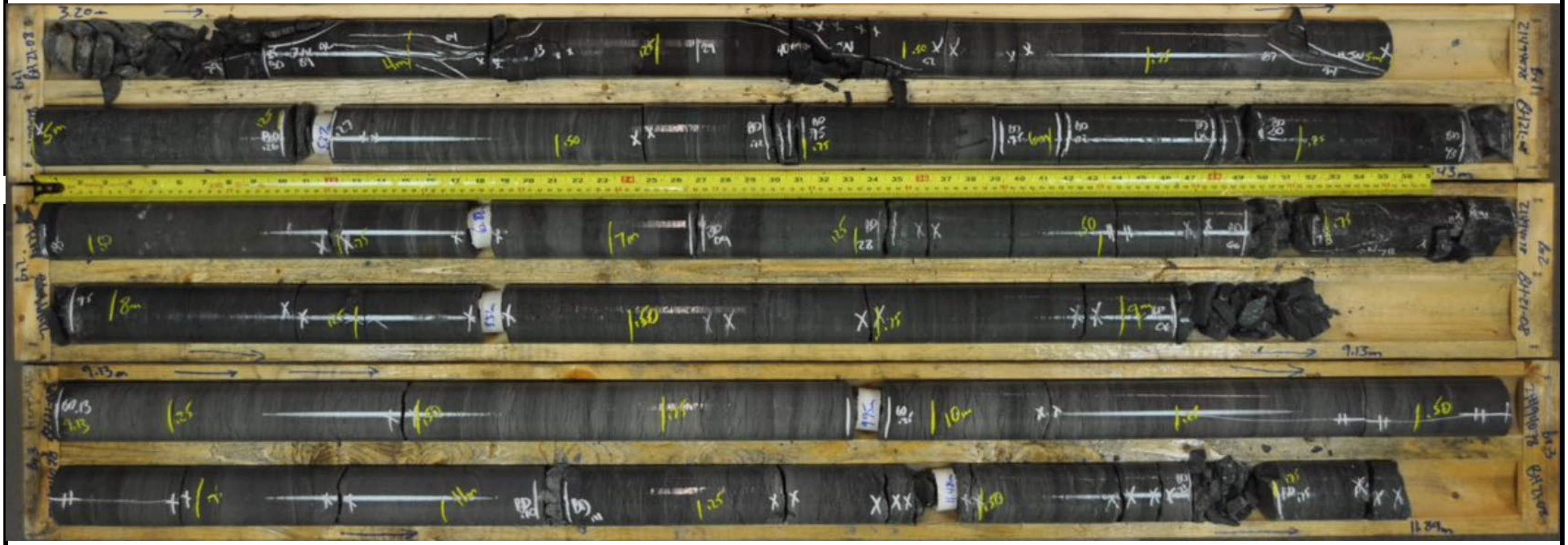
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-08
4 to 5 of 5

BH 21-08 (Wet)
Rock core from a depth of 3.2 m to 11.2 m
Core Box 1 to 3 of 5

3.2 m



11.2 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

GOLDER

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-08
1 to 3 of 5

BH 21-08 (Wet)
Rock core from a depth of 11.2 m to 15.5 m
Core Box 4 to 5 of 5

11.2 m

15.5 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

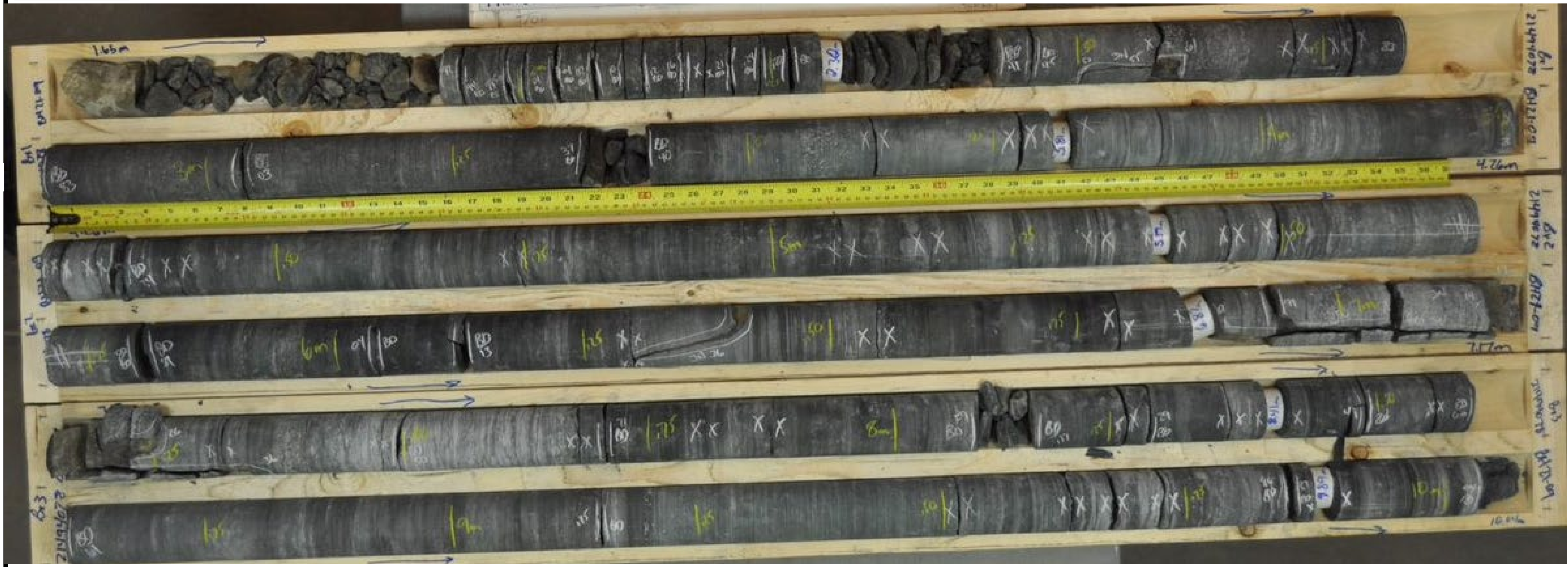
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-08
4 to 5 of 5

BH 21-09 (Dry)
Rock core from a depth of 1.6 m to 10.0 m
Core Box 1 to 3 of 5

1.6 m

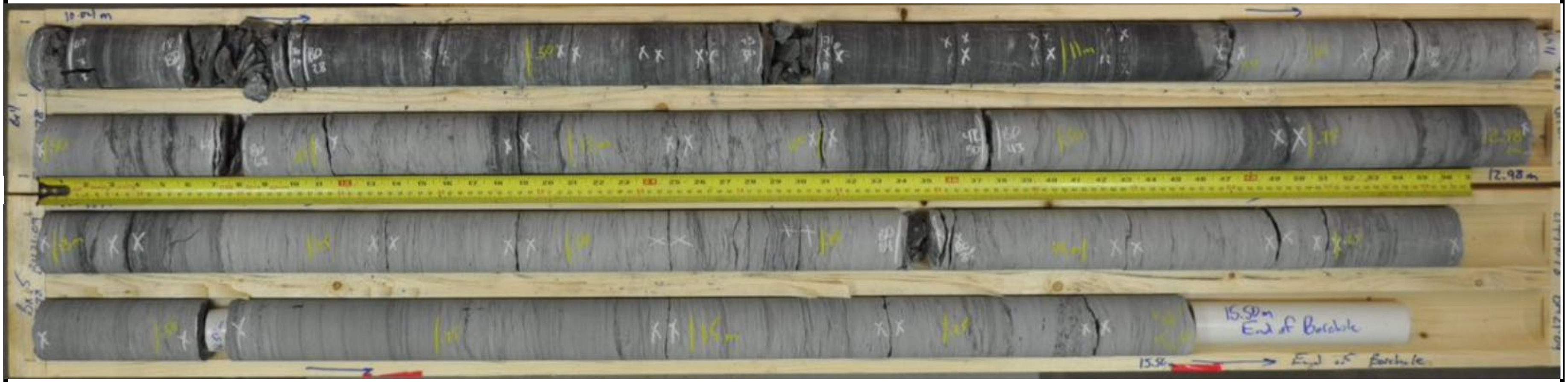


10.0 m

GOLDER	**Environmental Assessment, Geotechnical and Hydrogeological Investigation** **21494078- LPF Ph One Two RSC Richmond** **Ottawa, ON**	Project No. 21494078 Drawn: AG Date: 2021-10-08 Checked: AG Review: WC	**BH 21-09** **1 to 3 of 5**

BH 21-09 (Dry)
Rock core from a depth of 10.0 m to 15.5 m
Core Box 4 to 5 of 5

10.0 m



15.5 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-09
4 to 5 of 5

BH 21-09 (Wet)
Rock core from a depth of 1.6 m to 10.0 m
Core Box 1 to 3 of 5

1.6 m

10.0 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

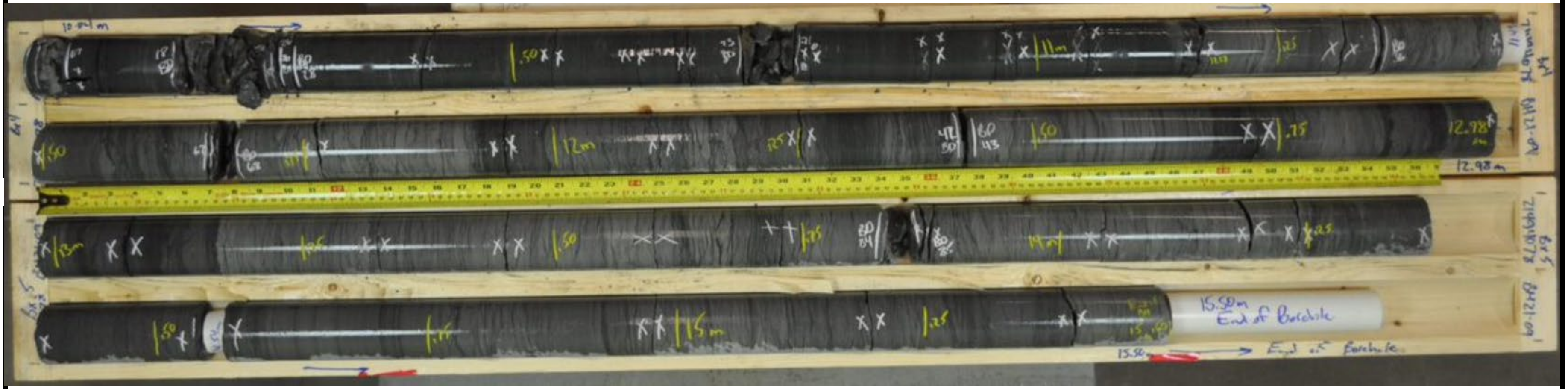
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-09
1 to 3 of 5

BH 21-09 (Wet)
Rock core from a depth of 10.0 m to 15.5 m
Core Box 4 to 5 of 5

10.0 m



15.5 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-09
4 to 5 of 5

BH 21-10 (Dry)
Rock core from a depth of 2.7 m to 12.1 m
Core Box 1 to 3 of 5

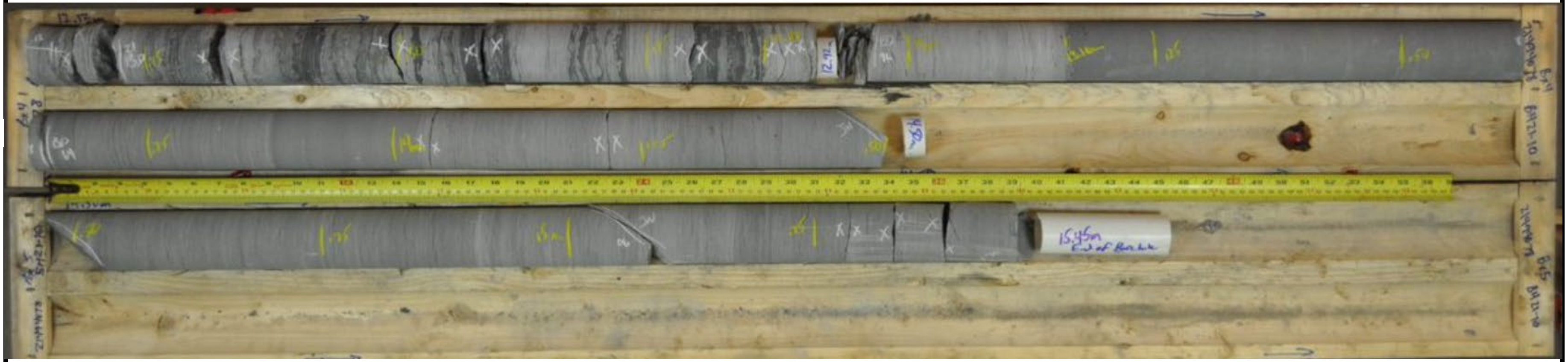
2.7 m

12.1 m

GOLDER	**Environmental Assessment, Geotechnical and Hydrogeological Investigation** **21494078- LPF Ph One Two RSC Richmond** **Ottawa, ON**	Project No. 21494078 Drawn: AG Date: 2021-10-08 Checked: AG Review: WC	**BH 21-10** **1 to 3 of 5**

BH 21-10 (Dry)
Rock core from a depth of 12.1 m to 15.4 m
Core Box 4 to 5 of 5

12.1 m



15.4 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-10
4 to 5 of 5

BH 21-10 (Wet)
Rock core from a depth of 2.7 m to 12.1 m
Core Box 1 to 3 of 5

2.7 m

12.1 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

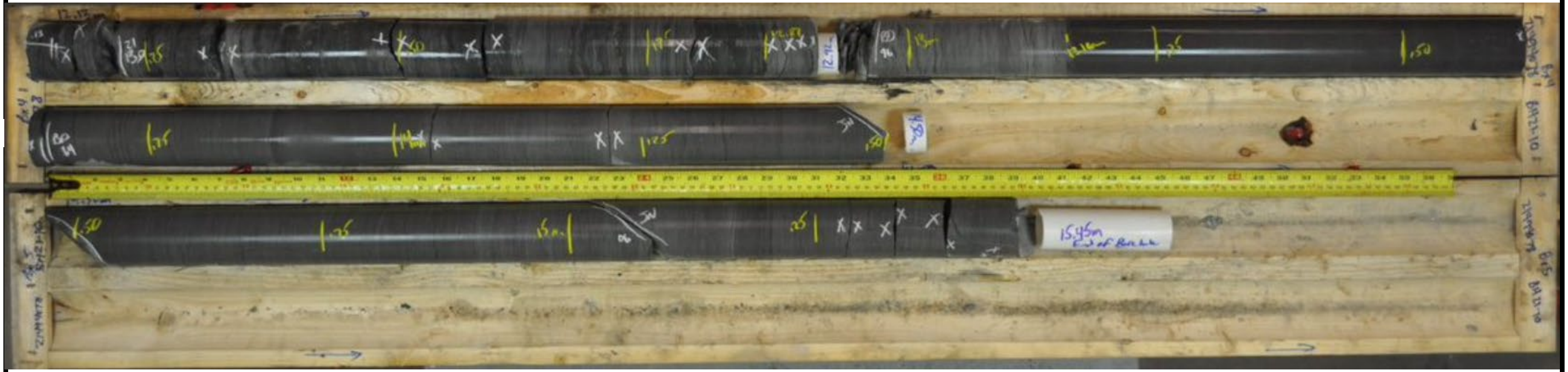
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-10
1 to 3 of 5

BH 21-10 (Wet)
Rock core from a depth of 12.1 m to 15.4 m
Core Box 4 to 5 of 5

12.1 m



15.4 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-10
4 to 5 of 5

APPENDIX D

Results of Basic Chemical Analyses

Certificate of Analysis

Client:	Golder Associates Ltd. (Ottawa) 1931 Robertson Road Ottawa, ON K2H 5B7
Attention:	Ms. Ali Ghirian
PO#:	
Invoice to:	Golder Associates Ltd. (Ottawa)

Report Number:	1964465
Date Submitted:	2021-10-12
Date Reported:	2021-10-15
Project:	21494078
COC #:	881198

				Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1588443 Soil 2021-09-30 21-06 sa2	1588444 Soil 2021-09-27 21-10 sa3
Group	**Analyte**	**MRL**	**Units**	**Guideline**		
Anions	Cl	0.002	%		0.007	<0.002
	SO4	0.01	%		<0.01	0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.24	0.15
	pH	2.00			8.88	8.39
	Resistivity	1	ohm-cm		4350	6670

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

GOLDER
MEMBER OF WSP

TECHNICAL MEMORANDUM

DATE October 27, 2021

21494078

TO Ali Ghirian
Golder Associates Ltd.

FROM Peter Giamou, Christopher Phillips

**EMAIL pgiamou@golder.com;
cphillips@golder.com**

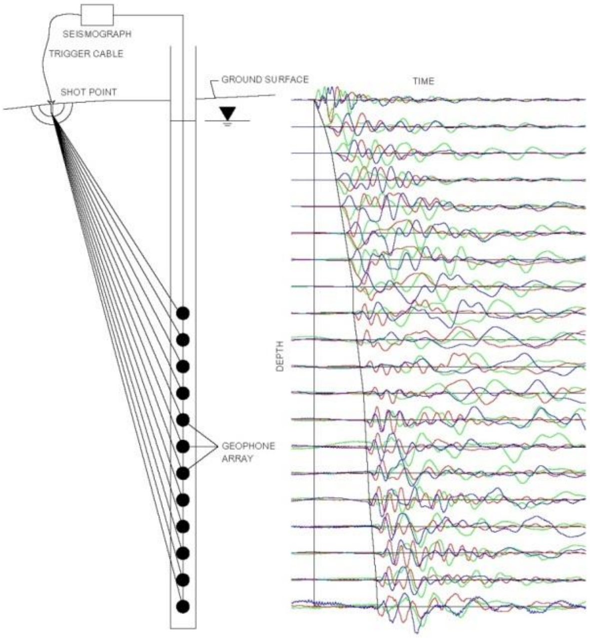
VERTICAL SEISMIC PROFILING RESULTS
1047 RICHMOND ROAD, OTTAWA, ONTARIO

This memorandum presents the results of two Vertical Seismic Profiling (VSP) testing carried out in Borehole 21-08 at 1047 Richmond Road, Ottawa, Ontario. VSP testing was carried out on October 6, 2021. Borehole 21-08 was drilled to an approximate depth of 15 m below the existing ground surface and then cased with a 2.5 inch PVC pipe grouted in place. The borehole consisted of approximately 3.2 m of sandy silt over dolostone and sandstone bedrock to the bottom of the borehole.

Methodology

For the VSP method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in a nearby borehole at a known depth. The active seismic source can be either compression or shear wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear-wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole.

The high-resolution results of a VSP survey are often used for earthquake engineering site classification, as per the National Building Code of Canada (NBCC).



Example 1: Layout and resulting time traces from a VSP survey.

Field Work

The field work was carried out on October 6, 2021, by personnel from the Golder Mississauga office.

At Borehole 21-08, compression and shear-wave seismic energy were generated from a sledge-hammer located 2.00 m from the borehole. The seismic source for the shear-wave test consisted of a 2.4-metre-long, 150 millimetre by 150 millimetre wooden beam, weighted by a vehicle and horizontally struck with a 9.9 Kg sledge-hammer on alternate ends of the beam to induce polarized shear waves. Test measurements started at ground surface and were recorded in the borehole with a 3-component receiver spaced at 1-metre intervals below the ground surface to the maximum depth of the casing (15 m).

The seismic records collected for each source location were stacked a minimum of three times to minimize the effects of ambient background seismic noise on the collected data. The data was sampled at 0.020833 millisecond intervals and a total time window of 0.341 seconds was collected for each seismic shot.

Data Processing

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

1) Compilation of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;
2) Low Pass Filtering of data to remove spurious high-frequency noise;
3) First-break picking of the compression and shear-wave arrivals; and,
4) Calculation of the average compression and shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.).
The seismic records from Borehole 21-08 are presented on the following two plots and show the first-break picks of the compression wave (Figure 1) and shear wave arrivals (Figure 2) overlaid on the seismic waveform traces recorded at the different geophone depths. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

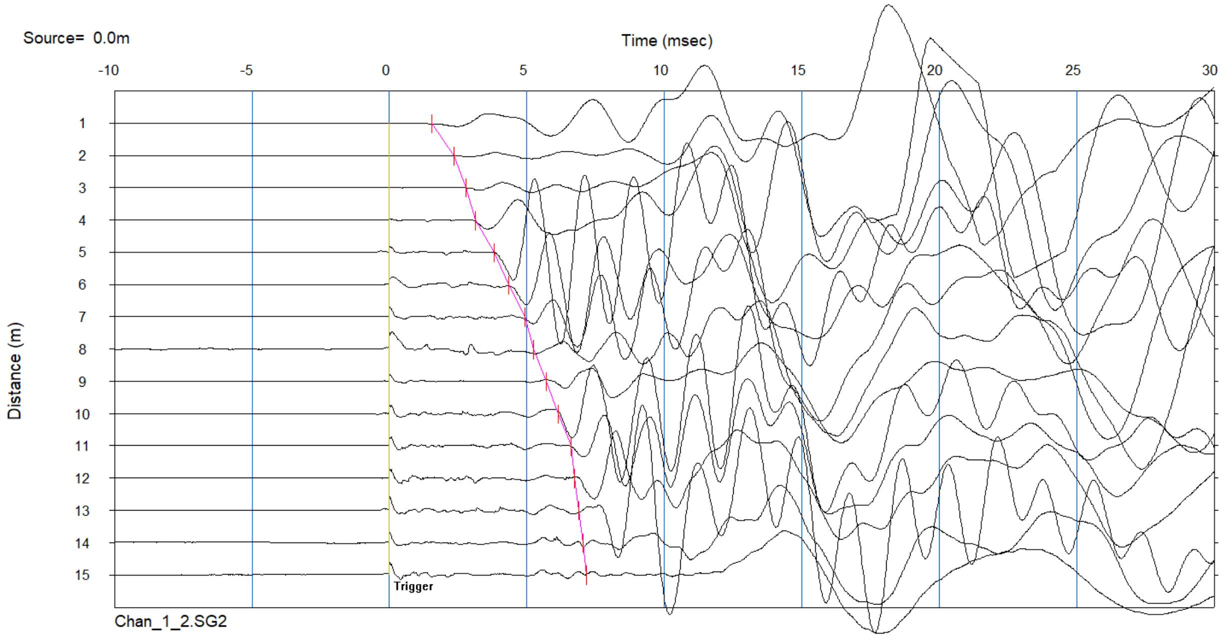


Figure 1: First-break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 21-08.

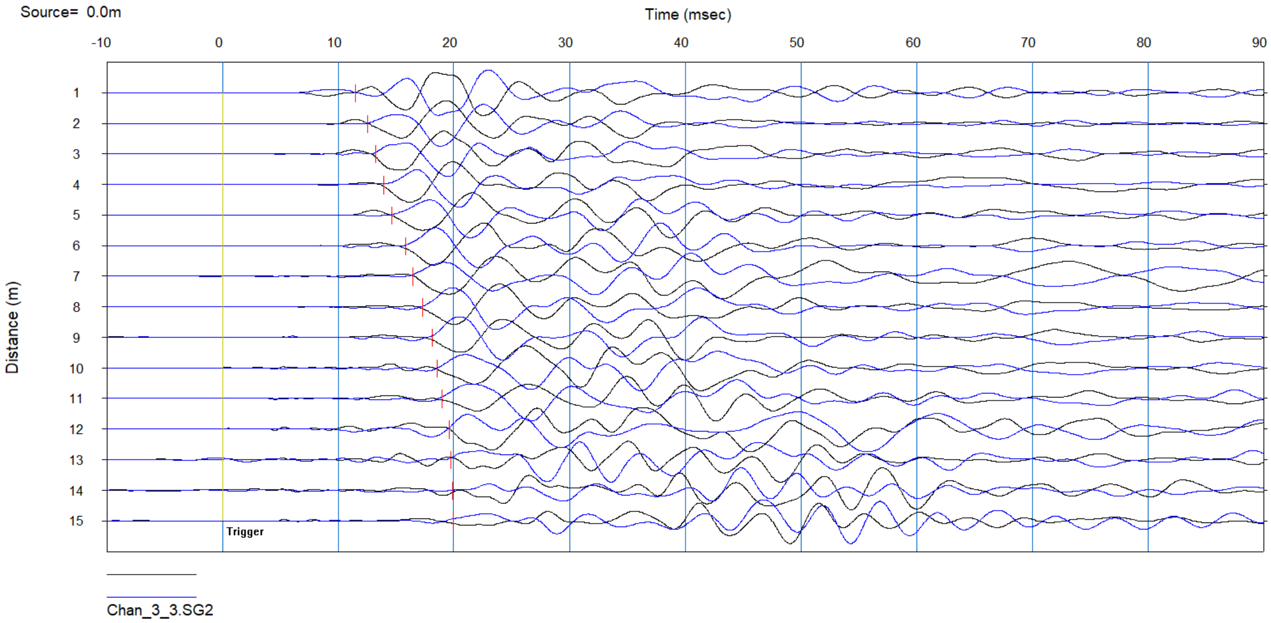


Figure 2: First-break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 21-08.

Results

The VSP results at Borehole 21-08 are summarized in Table 1. The shear wave and compression wave layer velocities were calculated by best-fitting a theoretical travel time model to the field data. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Table 1. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. An estimated bulk density of 2000 kg/m^3 was used for the overburden and an estimated bulk density of 2,600 kg/m^3 was used for the limestone bedrock.

At Borehole 21-08 the average shear wave velocity from ground surface to a depth of 30 metres was measured to be 1,171 metres per second. The average velocity at Borehole BH 21-08 was calculated assuming that the velocity from 15 metres to a depth of 30 metres was constant with an average shear-wave velocity value of 2,800 m/s which is equal to the velocity at the bottom of the borehole.

Limitations

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

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

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

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

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

Closure

We trust that these results meet your current needs. If you have any questions or require clarification, please contact the undersigned at your convenience.

Golder Associates Ltd.

Peter Giamou, B.Sc.,P. Geo
Senior Geophysicst
PG/CRP/jl

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

Attachments: Table 1 – VSP Modeller BH 21-08

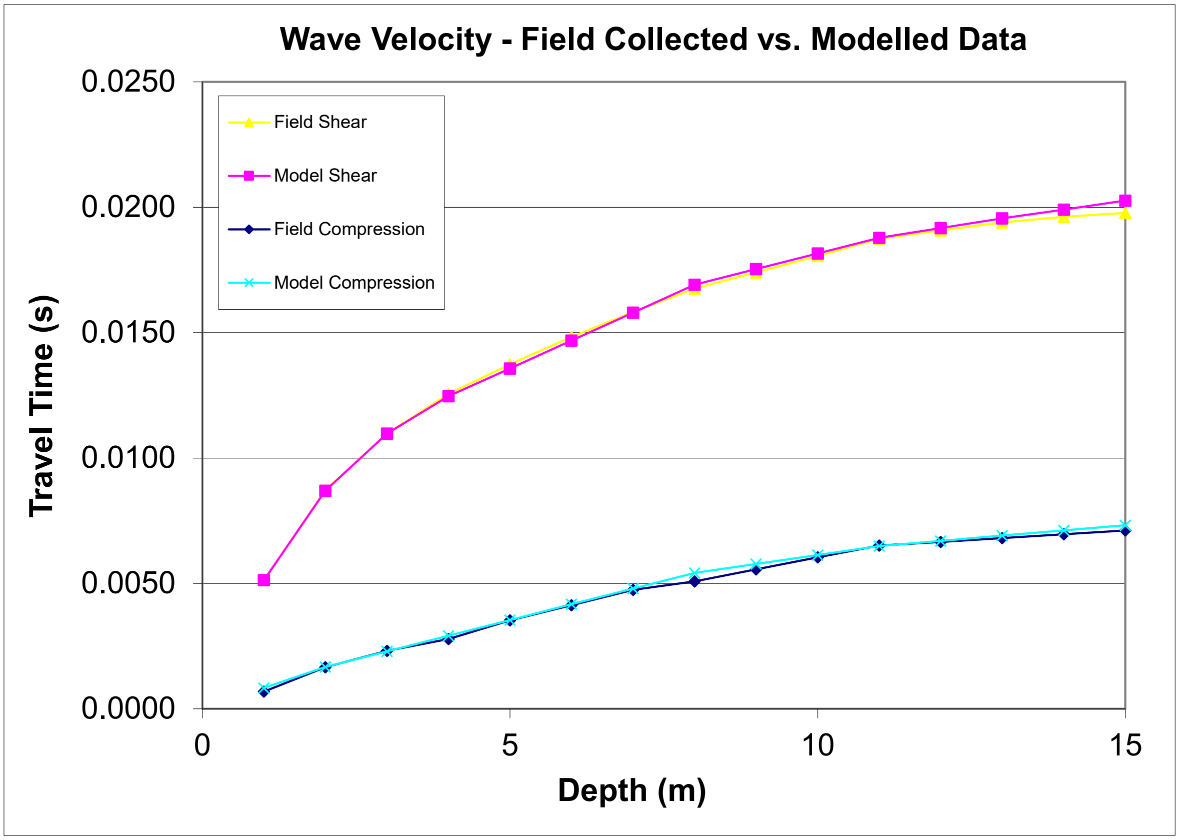
https://golderassociates.sharepoint.com/sites/152441/project files/5 technical work/geotechnical_1047 richmond rd/vsp survey/report/21494078 tech memo vsp model bh21-08 27oct2021.docx

TABLES

TABLE 1- VSP MODELLER BH21-08

TABLE 1
VSP VELOCITY PROFILE
BOREHOLE 21-08

Layer Depth (m)		Velocities (m/s)		Estimated Bulk Density (kg/m^3)	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave	Shear Wave		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1.0	400	195	2000	0.34	76	204	219
1.0	2.0	1200	280	2000	0.47	157	461	2671
2.0	3.0	1600	440	2000	0.46	387	1130	4604
3.0	4.0	1600	670	2600	0.39	1167	3253	5100
4.0	5.0	1600	900	2600	0.27	2106	5343	3848
5.0	6.0	1600	900	2600	0.27	2106	5343	3848
6.0	7.0	1600	900	2600	0.27	2106	5343	3848
7.0	8.0	1600	900	2600	0.27	2106	5343	3848
8.0	9.0	2800	1600	2600	0.26	6656	16741	11509
9.0	10.0	2800	1600	2600	0.26	6656	16741	11509
10.0	11.0	2800	1600	2600	0.26	6656	16741	11509
11.0	12.0	4800	2600	2600	0.29	17576	45430	36469
12.0	13.0	4800	2600	2600	0.29	17576	45430	36469
13.0	14.0	4800	2800	2600	0.24	20384	50638	32725
14.0	15.0	4800	2800	2600	0.24	20384	50638	32725



Notes

1. Depth presented is relative to the ground surface.
2. This table shall be analyzed in conjunction with the accompanying report.

APPENDIX F

Results of In-situ Hydraulic Conductivity Testing

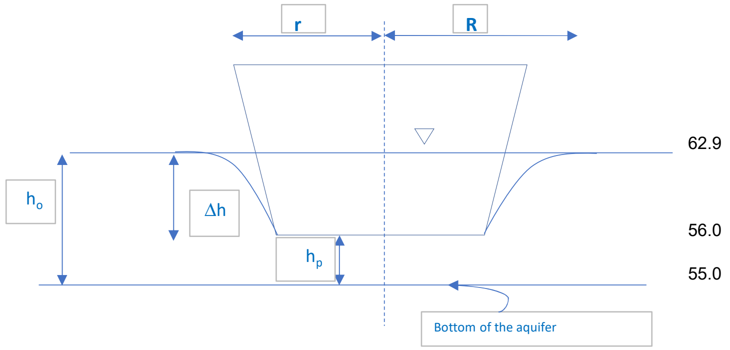
Inflow to Excavation

Dupuit-Forchheimer Equation: $Q=\pi K((h_o^2-hp^2)/\ln(R/r))$

1047 Richmond Road

K (m/sec)	1E-06	
h_0 (m)	7.9	**r - equivalent radius of pit**
h_p (m)	1.0	**R - radius of influence**
r (m)	**52.8**	**SF - safety factor**
SF	**2**	

	SF * Q (m^3/s)	R	Rad of Inf. from edge	m^3/day	L/day
Initial*	5.2E-03	57.8	5.0	450.11	450,108
	2.7E-03	62.8	10.0	234.68	234,677
	1.9E-03	67.8	15.0	162.75	162,751
	1.5E-03	72.8	20.0	126.69	126,687
Steady-State**	1.2E-03	77.8	25.0	104.97	104,972
	1.0E-03	82.8	30.0	90.44	90,437
	9.3E-04	87.8	35.0	80.01	80,008
	8.4E-04	92.8	40.0	72.15	72,148
	7.6E-04	97.8	45.0	66.01	66,005
	7.1E-04	102.8	50.0	61.06	61,065
	6.2E-04	112.8	60.0	53.60	53,596
	5.6E-04	122.8	70.0	48.20	48,202
	5.1E-04	132.8	80.0	44.11	44,110
	4.7E-04	142.8	90.0	40.89	40,891
	4.4E-04	152.8	100.0	38.29	38,286
	3.5E-04	202.8	150.0	30.23	30,232

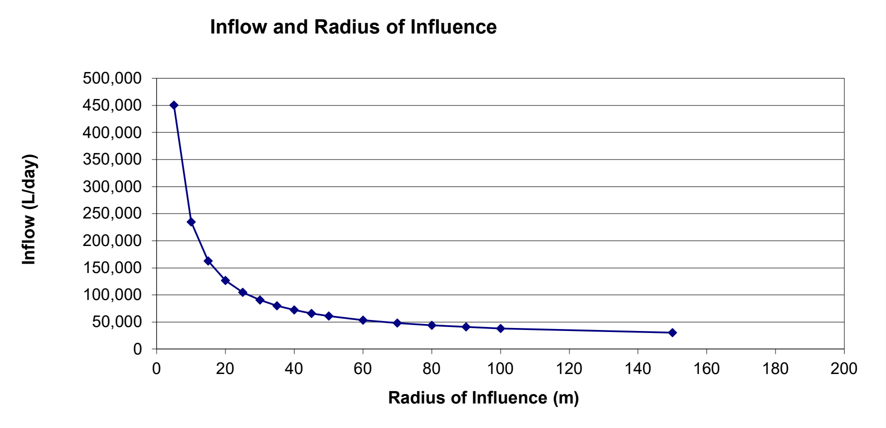


Excavation Assumptions

Width (m)	73	
Length (m)	120	
round surface elevation (masl)	65.50	
Groundwater elevation (masl)	62.89	*0.5 m higher than measured at 21-4*
Bottom of basement (masl)	56.5	*assumed 3 basement levels totalling 9 m depth*
Dewatered level (masl)	56.0	*0.5 m below depth of excavation*
Base of aquifer (masl)	55.0	*assumed 1 m below dewatered level*

Sichart and Kyrieleis Equation: $R=3000\Delta h(K^{1/2})$

Radius of Influence of Excavation (m)	23



Rainfall Amount - Based on a 79.2 mm precipitation event in 24 hours with a return of 10 years

Excavation Area (m^2)	8760
10 year Rainfall event (m)	0.0792
Max Vol Precipiation (L)	693,792

Notes

L - litres
m - metres
mbgs - metres below ground surface
Initial*: Potential worst-case inflow rate when trench is initially rapidly dewatered
Steady-State**: Steady state inflow rate

Prepared by: CAMC
Checked by: BH

Inflow to Excavation

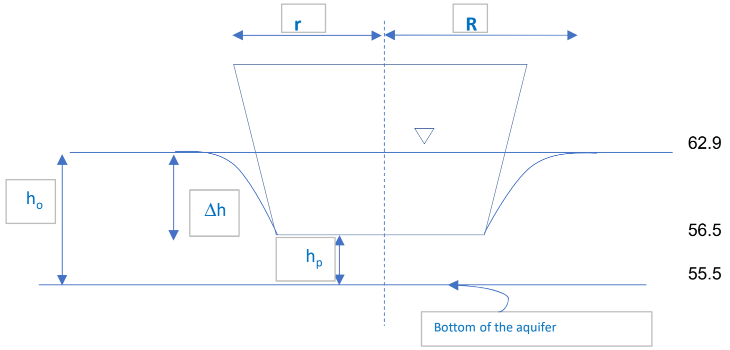
Dupuit-Forchheimer Equation: $Q=\pi K((h_o^2-hp^2)/ln(R/r))$

1047 Richmond Road

K (m/sec)	1E-06
h_0 (m)	7.4
h_p (m)	1.0
r (m)	**52.8**
SF	**2**

r - equivalent radius of pit
R - radius of influence
SF - safety factor

	SF * Q (m^3/s)	R	Rad of Inf. from edge	m^3/day	L/day
	3.8E-03	58.8	6.0	331.12	331,121
	2.2E-03	63.8	11.0	188.25	188,249
	1.6E-03	68.8	16.0	134.57	134,568
	1.2E-03	73.8	21.0	106.37	106,366
Steady-State**	1.1E-03	77.8	25.0	91.88	91,879
	9.2E-04	82.8	30.0	79.16	79,156
	8.1E-04	87.8	35.0	70.03	70,028
	7.3E-04	92.8	40.0	63.15	63,149
	6.7E-04	97.8	45.0	57.77	57,772
	6.2E-04	102.8	50.0	53.45	53,448
	5.4E-04	112.8	60.0	46.91	46,911
	4.9E-04	122.8	70.0	42.19	42,190
	4.5E-04	132.8	80.0	38.61	38,608
	4.1E-04	142.8	90.0	35.79	35,791
	3.9E-04	152.8	100.0	33.51	33,511
	3.1E-04	202.8	150.0	26.46	26,461

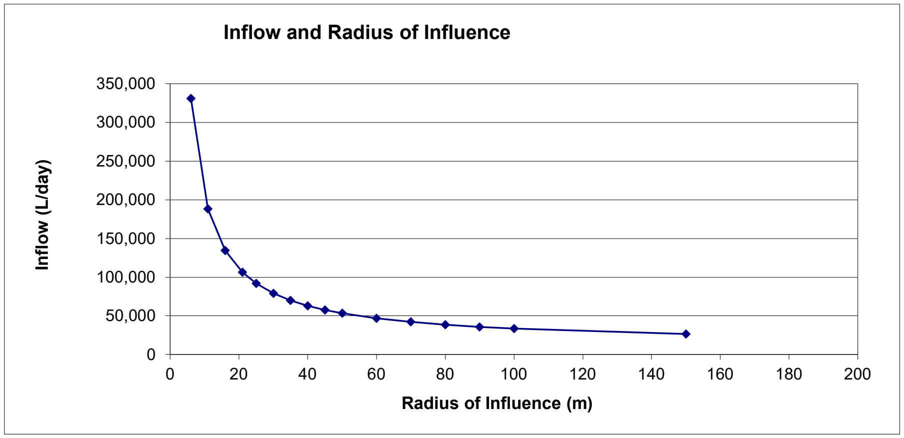


Excavation Assumptions

Width (m)	73	
Length (m)	120	
Ground surface elevation (masl)	65.50	
Groundwater elevation (masl)	62.89	*0.5 m higher than measured at 21-4*
Bottom of basement (masl)	56.5	*assumed 3 basement levels totalling 9 m depth*
Dewatered level (masl)	56.5	*same as the depth of excavation*
Base of aquifer (masl)	55.5	*assumed 1 m below dewatered level*

Sichart and Kyrieleis Equation: $R=3000\Delta h(K^{1/2})$

Radius of Influence of Excavation (m) 21



Notes
L - litres
m - metres
mbgs - metres below ground surface
Initial*: Potential worst-case inflow rate when trench is initially rapidly dewatered
Steady-State**: Steady state inflow rate

Prepared by: CAMC
Checked by: BH

HVORSLEV SLUG TEST ANALYSIS
RISING HEAD TEST 21-2

INTERVAL (metres below ground surface)

Top of Interval = 3.96
Bottom of Interval = 7.01

$$K = \frac{r_c^2}{2L_e} ln\left[\frac{L_e}{2R_e} + \sqrt{1+\left(\frac{L_e}{2R_e}\right)^2}\right]\left[\frac{ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)}\right]$$

where K = (m/sec)

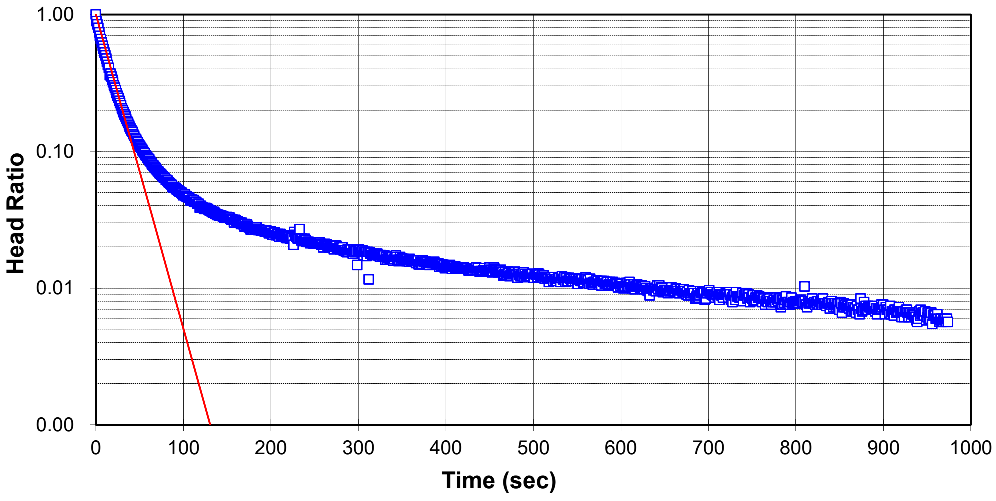
where:
r_c = casing radius (metres)
R_e = filter pack radius (metres)
L_e = length of screened interval (metres)
t = time (seconds)
h_t = head at time t (metres)

INPUT PARAMETERS

r_c = **0.025**
R_e = **0.05**
L_e = **3.1**
t_1 = **0**
t_2 = **29**
h_1/h_0 = **1.00**
h_2/h_0 = **0.22**

RESULTS

K= 2E-05 m/sec
K= 2E-03 cm/sec



Project Name: Fengate/Phase 1, 2 and RSC/Ottawa
Project No.: 21494078
Test Date: 2021-10-05

Analysis By: SPS
Checked By: BH
Analysis Date: 2021-10-06

Golder Associates Ltd.

HVORSLEV SLUG TEST ANALYSIS
RISING HEAD TEST 21-3

INTERVAL (metres below ground surface)

Top of Interval = 4.57
Bottom of Interval = 7.62

$$K = \frac{r_c^2}{2L_e} ln\left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e}\right)^2}\right]\left[\frac{ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)}\right]$$

where K = (m/sec)

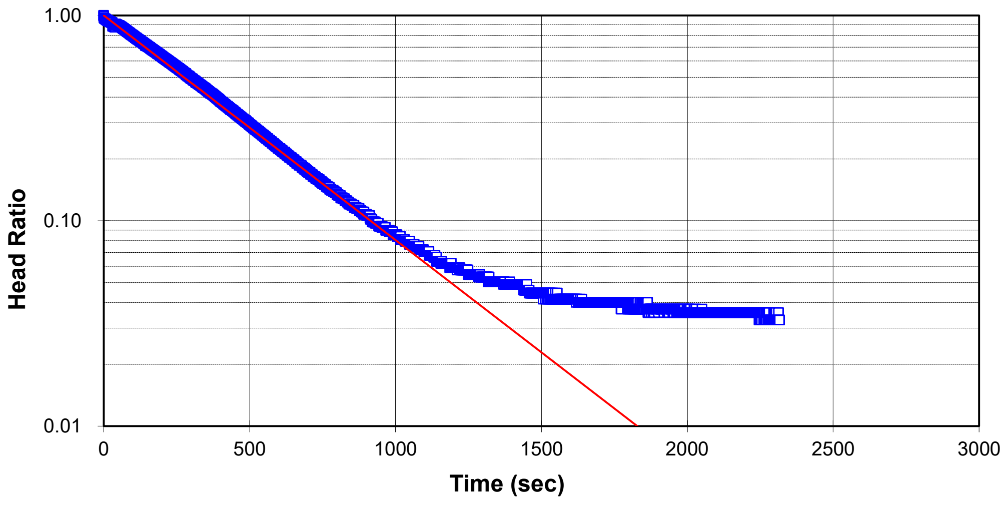
where:
r_c = casing radius (metres)
R_e = filter pack radius (metres)
L_e = length of screened interval (metres)
t = time (seconds)
h_t = head at time t (metres)

INPUT PARAMETERS

r_c =	0.025
R_e =	0.05
L_e =	3.1
t_1 =	0
t_2 =	775
h_1/h_0 =	1.00
h_2/h_0 =	0.14

RESULTS

K=	1E-06	m/sec
K=	1E-04	cm/sec



Project Name: Fengate/Phase 1, 2 and RSC/Ottawa
Project No.: 21494078
Test Date: 2021-10-05

Analysis By: SPS
Checked By: BH
Analysis Date: 2021-10-06

Golder Associates Ltd.

HVORSLEV SLUG TEST ANALYSIS
RISING HEAD TEST 21-4

INTERVAL (metres below ground surface)

Top of Interval = 4.57
Bottom of Interval = 7.62

$$K = \frac{r_c^2}{2L_e} ln\left[\frac{L_e}{2R_e} + \sqrt{1+\left(\frac{L_e}{2R_e}\right)^2}\right]\left[\frac{ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)}\right]$$

where K = (m/sec)

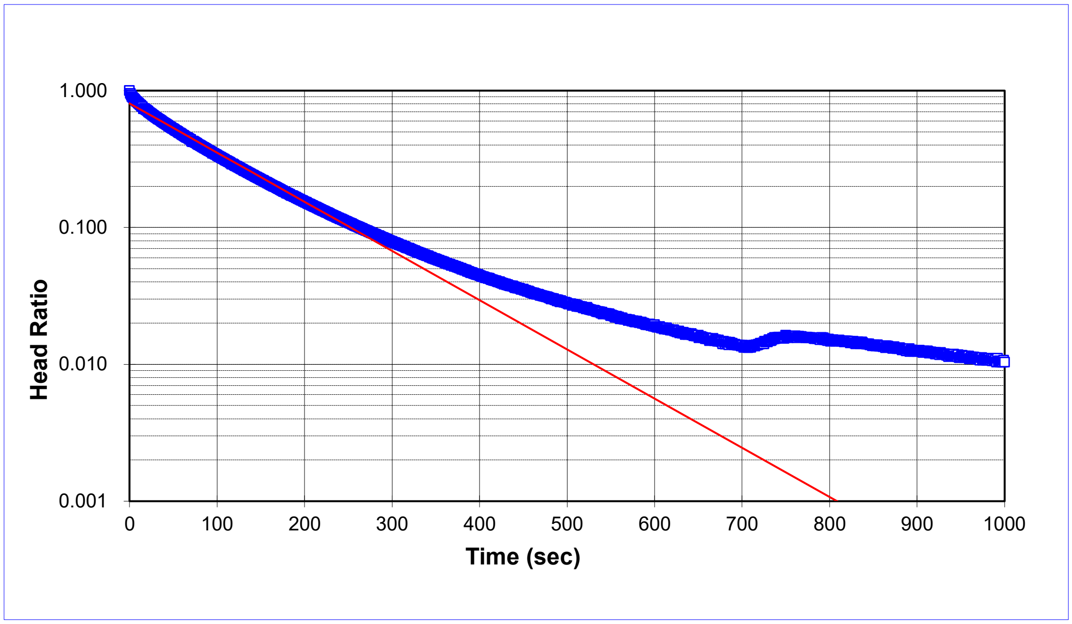
where:
r_c = casing radius (metres)
R_e = filter pack radius (metres)
L_e = length of screened interval (metres)
t = time (seconds)
h_t = head at time t (metres)

INPUT PARAMETERS

r_c = **0.025**
R_e = **0.05**
L_e = **3.1**
t_1 = **41**
t_2 = **199**
h_1/h_0 = **0.57**
h_2/h_0 = **0.16**

RESULTS

K=	**4E-06**	**m/sec**
K=	**4E-04**	**cm/sec**



Project Name: Fengate/Phase 1, 2 and RSC/Ottawa
Project No.: 21494078
Test Date: 2021-10-05

Analysis By: SPS
Checked By: BH
Analysis Date: 2021-10-06

Golder Associates Ltd.

HVORSLEV SLUG TEST ANALYSIS
FALLING HEAD TEST 21-5

INTERVAL (metres below ground surface)

Top of Interval = 4.57
Bottom of Interval = 7.62

$$K = \frac{r_c^2}{2L_e} ln\left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e}\right)^2}\right]\left[\frac{ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)}\right]$$

where K = (m/sec)

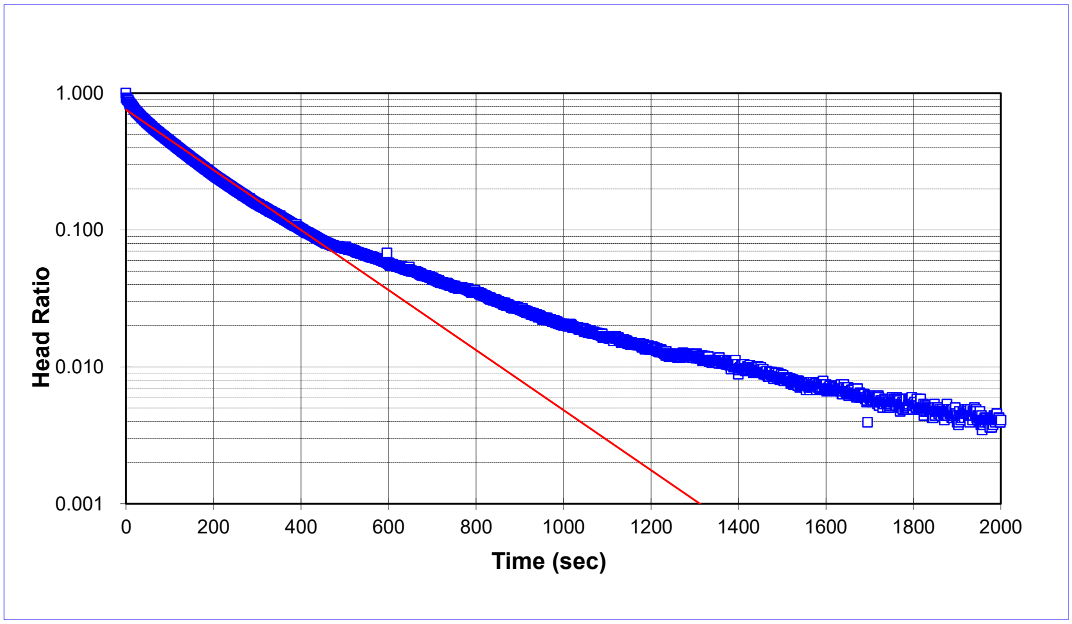
where:
r_c = casing radius (metres)
R_e = filter pack radius (metres)
L_e = length of screened interval (metres)
t = time (seconds)
h_t = head at time t (metres)

INPUT PARAMETERS

r_c = **0.025**
R_e = **0.05**
L_e = **3.1**
t_1 = **69**
t_2 = **376**
h_1/h_0 = **0.53**
h_2/h_0 = **0.11**

RESULTS

K= 2E-06 m/sec
K= 2E-04 cm/sec



Project Name: Fengate/Phase 1, 2 and RSC/Ottawa
Project No.: 21494078
Test Date: 2021-10-05

Analysis By: SPS
Checked By: BH
Analysis Date: 2021-10-06

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BOUWER AND RICE SLUG TEST ANALYSIS
RISING HEAD TEST 21-6

INTERVAL (metres below ground surface)

Top of Interval = 6.33
Bottom of Interval = 9.38

$$K = \frac{r_c^2 \ln\left(\frac{R_e}{r_w}\right)}{2L_e}\frac{1}{t}\ln\frac{y_0}{y_t}$$ where K=m/sec

where:

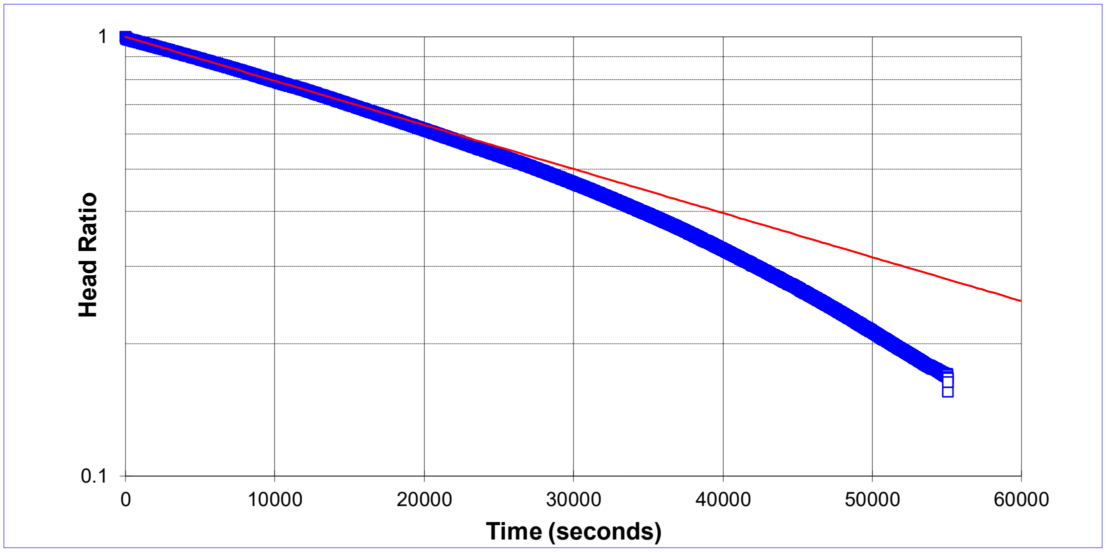
r_c = casing radius (metres);
R_e = effective radius (metres);
L_e = length of screened interval (metres);
r_w = radial distance to undisturbed aquifer (metres)
y_0 = initial drawdown (metres)
y_t = drawdown (metres) at time t (seconds)

INPUT PARAMETERS

r_c =	0.03
r_w =	0.05
L_e =	2.54
$ln(R_e/r_w)$	2.69
y_0 =	1.00
y_t =	0.63
t =	20000

RESULTS

K=	1E-08	m/sec
K=	1E-06	cm/sec



Project Name: Fengate/Phase 1, 2 and RSC/Ottawa
Project No.: 21494078
Test Date: 05-Oct-21

Analysis By: SPS
Checked By: BH
Analysis Date: 06-Oct-21

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HVORSLEV SLUG TEST ANALYSIS
RISING HEAD TEST 21-10

INTERVAL (metres below ground surface)

Top of Interval = 12.40
Bottom of Interval = 15.45

$$K = \frac{r_c^2}{2L_e} ln\left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e}\right)^2}\right]\left[\frac{ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)}\right]$$

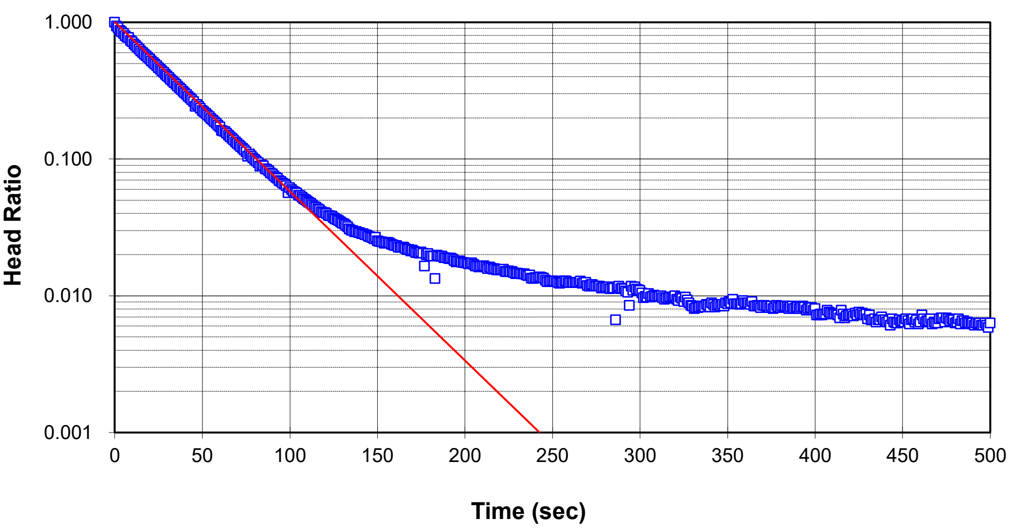
where K = (m/sec)

where:
r_c = casing radius (metres)
R_e = filter pack radius (metres)
L_e = length of screened interval (metres)
t = time (seconds)
h_t = head at time t (metres)

INPUT PARAMETERS	
r_c =	0.025
R_e =	0.05
L_e =	3.1
t_1 =	0
t_2 =	87
h_1/h_0 =	1.00
h_2/h_0 =	0.08

RESULTS

K=	1E-05	m/sec
K=	1E-03	cm/sec



Project Name: Fengate
Project No.: 21494078
Test Date: 2021-10-05

Analysis By: SPS
Checked By: BH
Analysis Date: 2021-10-06

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