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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

Geotechnical Investigation

Proposed Brookstreet
Condominium Development
525 Legget Drive
Ottawa, Ontario

Prepared For

KRP Properties

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Report: PG5673-1 Revision 4

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1.0 Introduction

Paterson Group (Paterson) was commissioned by KRP Properties to conduct a geotechnical investigation for the proposed condominium development (subject site) to be located at 525 Legget Drive in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- Determine the subsoil and groundwater conditions at this site by means of test holes.
- Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

It is understood that the proposed development will consist of a 30-storey condominium building with two levels of underground parking. It is further understood that the building footprint will occupy the south-east portion of the subject site while the footprint of the underground parking level will occupy the majority of the subject site. It is also understood that the proposed building will be provided a connection to the existing hotel building located to the west. Access lanes and landscaped areas are also anticipated.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on February 25 and 26, and March 1, 2021, and consisted of advancing 3 boreholes and 2 test pits to a maximum depth of 18.1 and 3.0 m below the existing ground surface, respectively. Previous investigations were completed by others in November of 2020 consisting of 4 boreholes advanced to a maximum depth of 18.7 m below the ground surface. The test hole locations for the current investigation were determined in the field by Paterson personnel taking into consideration site features and underground services. The locations of the boreholes are shown on Drawing PG5673-1 - Test Hole Location Plan in Appendix 2.

Boreholes were advanced using a low-clearance drill rig operated by a two-person crew and the test pits were extended using a backhoe at the selected locations. The test hole procedure consisted of augering or excavating to the required depths at the selected locations and sampling the overburden soils. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department.

Sampling and In Situ Testing

Soil samples were recovered from the auger flights, using a 50 mm diameter split-spoon sampler, or core recovery barrels. Soils samples collected from the test pits were recovered from the side walls of the open excavation as grab samples. The split-spoon, auger, grab, and rock core samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory for further examination. The depths at which the split-spoon, auger flights, grab, and rock core samples were recovered from the boreholes are shown as SS, AU, G and RC, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of each of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

Diamond drilling was completed at all boreholes as part of the current investigation to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage. The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

The subsurface conditions observed in the test pits were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

A series of groundwater monitoring wells were installed within the boreholes as part of the current investigation to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All monitoring wells should be decommissioned in accordance with Ontario Regulations O.Reg 903 by a qualified licensed well technician and prior to construction.

3.2 Field Survey

The test hole locations were selected by Paterson personnel in a manner to provide general coverage of the proposed development, taking into consideration existing site features. The ground surface elevations were referenced to a geodetic datum. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG5673-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

The analytical testing results of two (2) soil samples from an adjacent site were analyzed to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples were submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing at-grade asphalt parking lot associated with the neighboring buildings. The ground surface east of the parking lot slopes downward gradually towards an existing storm water management pond. The area between the pond and the parking lot is generally landscaped with trees and occupied by a 1-storey pump house structure towards the south-east portion of the subject site. It is expected that the pump house structure will be removed prior to the construction of the proposed building.

The subject site is bordered to the west by a 5-storey above-ground parking structure and further by a multi-storey hotel building, to the north by additional parking lanes and Terry Fox Drive, and to the east and south by the pond.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of pavement structure overlying fill material consisting of silty sand, crushed stone and/or silty clay that extends to depths up to 2.0 m. A very stiff to stiff silty clay deposit was encountered below the fill layer at all boreholes. The silty clay deposit was observed to be underlain by a glacial till deposit, consisting of silty clay with clay, sand, gravel, cobbles and boulders at BH 1-21. Practical refusal to augering was encountered throughout the site.

Reference should be made to the Soil Profile and Test Data sheets in the attachments for specific details of the soil profiles encountered at each test hole location.

Bedrock

Bedrock was cored at all boreholes as part of the current investigation to a maximum depth of 18.1 m. The recovery values and RQD values for the bedrock cores were calculated. The recorded recovery values were between 98 and 100%, while the RQD values varied between 75 and 100%. Based on these results the quality of the bedrock ranges from good to excellent.

Unconfined compressive strength was undertaken by others as part of the previous investigation on six bedrock core samples. The results indicated the UCS ranged between 74 to 151 MPa, with five of the six samples ranging between 109 and 151 MPa.

Based on available geological mapping, the bedrock in the subject area consists of interbedded sandstone and dolomite of the March formation, with an anticipated overburden drift thickness of 0 to 10 m depth.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells on March 3, 2021. Where possible, groundwater levels were also measured within the existing monitoring wells completed by others. The measured groundwater level (GWL) readings are presented in Table 1 below.

Table 1 - Summary of Groundwater Levels

Borehole Number	Measured Groundwater Level		Recording Date
	Depth (m)	Elevation (m)	
Groundwater Levels Based on Boreholes Investigation (Report PG5673)			
BH 1-21	0.43	75.19	March 3, 2021
BH 2-21	0.44	75.19	March 3, 2021
BH 3-21	0.06	75.30	March 3, 2021
Groundwater Levels Based on Previous Investigation by Others (November 2020)			
20-01	-0.29	76.50	December 9, 2020
	0.07	76.14	March 3, 2021
20-03	1.30	74.50	December 2, 2020
	1.64	74.12	March 3, 2021
20-04	0.40	76.10	December 3, 2020

Test pits were also advanced in close proximity to the existing pond to verify the hydraulic connection throughout the overburden. Based on our observations, the overburden is not hydraulically connected to the existing pond due to the relatively dry material and negligible infiltration across the bedrock surface that was encountered at the time of investigation. Groundwater levels are subject to seasonal fluctuations. Therefore, levels could differ at the time of construction.

Hydraulic Conductivity Testing

Following the completion of the slug testing, the test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent with zero-storage assumption, and a screen length significantly greater than the monitoring well diameter. The assumption regarding aquifer storage is considered to be appropriate for groundwater flow through the overburden aquifer. The assumption regarding screen length and well diameter is considered to be met based on a screen length of 1.5 m and a diameter of 0.03 m.

While the idealized assumptions regarding aquifer extent, homogeneity, and isotropy are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site.

Hvorslev analysis is based on the line of best fit through the field data (hydraulic head recovery vs. time), plotted on a semi-logarithmic scale. In cases where the initial hydraulic head displacement is known with relative certainty, such as in this case where a physical slug has been introduced, the line of best fit is considered to pass through the origin.

Based on the above test methods, the monitoring wells screened in the good to excellent quality sandstone bedrock displayed hydraulic conductivity values ranging between **1.5×10^{-4}** and **4.8×10^{-8}** m/sec. The values measured within the monitoring wells are consistent with similar quality bedrock that Paterson has encountered on other sites, and typical published values for sandstone bedrock. These values typically range from 1×10^{-6} to 1×10^{-10} m/sec for bedrock. The range in hydraulic conductivity values is due to the variability quality of the bedrock. The results of the hydraulic conductivity testing are presented in Appendix 1.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed multi-storey building is expected to be founded on conventional footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the two levels of underground parking. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

The above and other considerations are discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Due to the relatively shallow bedrock depth at the subject site and the anticipated founding level for the proposed building, all existing overburden material will be excavated from within the proposed building footprint. Bedrock removal should be required for the construction of the parking garage levels.

Bedrock Removal

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the effects on the existing services, buildings and other structures should be addressed. A pre-blast or construction survey located in proximity of the blasting operations should be conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Excavation side slopes in sound bedrock could be completed with almost vertical side walls. A minimum of 1 m horizontal bench should remain between the bottom of the overburden and the top of the bedrock surface to provide an area for potential sloughing or a stable base for the overburden shoring system.

Vibration Considerations

Construction operations could cause vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain a cooperative environment with the residents.

The following construction equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles or sheet piling will require these pieces of equipment. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Bedrock Excavation Face Reinforcement

Horizontal rock anchors and/or chain link fencing connected to the excavation face may be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations.

Fill Placement

Fill placed for grading beneath the building footprint should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

5.3 Foundation Design

Bearing Resistance Values

Shallow footings placed on a clean, surface sounded bedrock bearing surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5. Footings placed on a clean, surface-sounded bedrock bearing surface will be subjected to negligible postconstruction total and differential settlements.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures, or open joints which can be detected from surface sounding with a rock hammer.

For the footings at depth for the parking garage and building foundation, a factored bearing resistance value at ULS of **6,000 kPa**, incorporating a geotechnical resistance factor of 0.5 could be used if founded on bedrock provided the bedrock is free of seams, fractures, and voids within 1.5 m below the founding level.

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential postconstruction total and differential settlements.

Frictional Resistance

An unfactored coefficient of friction of 0.7 is considered applicable for the design of concrete footings supported on clean, surface sounded bedrock at this site.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to silty clay bearing medium when a plane extending down and out from the bottom edges of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A heavily fractured, weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as presented in Drawing PG5673-1 - Test Hole Location Plan, attached to the present letter report. Paterson field personnel placed 18 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio.

The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 15, 3 and 2 m away from the first and last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the foundation of the building. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on our testing results, the average overburden shear wave velocity is **310 m/s**, while the bedrock shear wave velocity is **2,561 m/s**. It is understood that the overburden will be completely removed as part of the proposed building and footings will be placed directly on the bedrock surface.

Based on this, the V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

$$V_{s30} = \frac{Depth_{of\ interest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{s_{Layer1}}(m/s)} + \frac{Depth_{Lay}\quad(m)}{V_{s_{Layer2}}(m/s)}\right)}$$

$$V_{s30} \quad \frac{30\,m}{\left(\frac{30\,m}{2{,}561\,m/s}\right)}$$

$$V_{s30=}\ 2{,}561\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed building founded on bedrock at an approximate geodetic elevation between 71.0 to 72.0 m is **2,561 m/s**. Therefore, a **Site Class A** is applicable for design of the proposed building as per Table 4.1.8.4.A of the OBC 2012.

The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings. Alternatively, excavated bedrock could be used as select subgrade material around the proposed building footings, provided the excavated bedrock is suitably crushed to 50 mm in its longest dimension and approved by the geotechnical consultant at the time of placement. The upper 200 mm below the basement floor slab should consist of a 19 mm clear crushed stone.

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor. This is discussed further in Section 6.1 of this report.

5.6 Basement Wall

It is understood that the basement walls are to be poured against a dampproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m3 (effective 15.5 kN/m3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face.

Where the soil is to be retained, there are several combinations of backfill materials and retaining soils for the basement walls of the subject structure. However, the conditions should be designed by assuming the retaining soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component ($\triangle P_{AE}$).

Lateral Earth Pressures

The static horizontal earth pressure (P_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $Ko \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45\text{-}a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that centre to centre spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Based on the testing results, the unconfined compressive strength of the sandstone bedrock below the subject site ranges between 74 and 151 MPa, and typically above 100 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing subsoils information, a **Rock Mass Rating (RMR) of 64** was assigned to the bedrock. Therefore, Hoek and Brown parameters (**m and s**) were taken as **1.231** and **0.00293**, respectively.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required.

For our calculations, the following parameters were used.

Table 2 - Parameters Used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Sandstone Hoek and Brown parameters	m=1.231 and s=0.00293
Unconfined compressive strength - Sandstone bedrock	80 MPa
Unit weight - Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 3.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.3	1	3.3	500
	3.3	1	4.3	750
	4.4	1.1	5.5	1000
	5.4	1.1	6.5	1200
125	2	1.5	3.5	750
	2.7	1.6	4.3	1000
	3.9	1.6	5.5	1500
	5.1	1.6	6.7	2000

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Design

Car only parking and heavy traffic areas are anticipated at this site. The subgrade material will consist of glacial till and bedrock throughout the exterior and lowest basement level of the subject site, respectively. The proposed pavement structures are shown in Tables 4 and 5.

Table 4 - Recommended Pavement Structure - Car-Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soils, bedrock or OPSS Granular B Type I or II material placed over in situ soil or bedrock	

Table 5 - Recommended Pavement Structure - Heavy-Truck Traffic and Loading Areas

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soils, bedrock or OPSS Granular B Type I or II material placed over in situ soil or bedrock	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is understood the proposed structure will be founded at an approximate elevation of 71 m and upon sound and relatively watertight bedrock. It is also anticipated that the basement walls are to be poured against a dampproofing system, which will be placed against the exposed bedrock face. It is anticipated that the proposed basement levels will be located within a groundwater storage area, which is anticipated to be confined to seams and fissures located below the bedrock surface.

Therefore, it is recommended that a perimeter foundation drainage system be provided for the proposed structure in conjunction with carrying out grouting of water bearing fissures and seams during the bedrock removal process. A composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) should extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe.

Further, it is recommended that the geotechnical consultant carry out inspections throughout the building excavation process to review the presence of any significant bedrock fissures being water bearing and causing significant water infiltration volumes.

Although the sound sandstone should be relatively watertight, any significant water infiltration from vertical fissures and seams throughout the bedrock surface are recommended to be reviewed by the geotechnical consultant and, if necessary, significant fissures should be grouted with a cementitious grout to reduce the volume of water infiltration to allow for a relatively dry excavation base.

For preliminary design purposes, the following groundwater infiltration control system for the foundation walls is recommended:

- ☐ Line drill the excavation perimeter (usually a 150 to 200 mm spacing).
- ☐ Mechanical bedrock removal along the foundation walls can be undertaken up to 150 mm from the finished vertical excavation face.
- ☐ Grind the bedrock surface up to the outer face of the line drill holes to ensure a satisfactory surface for the below grade architectural drainage system.

- ❒ Where bedrock overbreaks, shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface, as required based on site inspection by Paterson.
- ❒ If required, carry out grouting of water bearing fissures and seams throughout the bedrock face with a cementitious grout and under the direction of the geotechnical consultant.
- ❒ Place a composite drainage layer, such as Delta Drain 6000 or equivalent against the prepared bedrock surface. The composite drainage layer should extend from finished grade to underside of footing level.
- ❒ Pour foundation wall against the composite drainage system.

Interior Perimeter and Underfloor Drainage

The interior perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water from the buildings foundation drainage system to the buildings sump pit(s). The interior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock and surrounded by a minimum of 150 mm of 19 mm clear crushed stone.

For design purposes, 150 mm diameter sleeves should be cast in the foundation wall at the footing interface and placed at 3 m centres to allow the infiltration of water to flow to the interior perimeter drainage pipe. The spacing of the underfloor drainage system should be confirmed by the geotechnical consultant once design details for the proposed development are finalized and reviewed for suitability at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the building should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

6.2 Protection of Footings Against Frost Action

Perimeter footings, pile caps, and grade beams of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover alone, or a combination of soil cover in conjunction with foundation insulation should be provided in this regard.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored, or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated using the parameters provided in Table 6.

Table 6 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight(γ'), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Underpinning of Adjacent Structures

Based on the shallow depth to bedrock encountered throughout the subject site, it is anticipated the existing structures adjacent to the subject site are founded on bedrock. However, in areas where the excavation for the proposed underground parking structure will be within the lateral support zone of footings for the existing buildings that may be founded within the overburden, an underpinning program will be required to safely transfer the existing building loads down to a lower founding elevation (i.e.- bedrock). An underpinning program, such as a 2 to 3 m wide excavation completed with a piano key style excavation technique and extending to the proposed underside of footing of the parking structure, would be adequate. The excavation should be infilled with a lean concrete to the existing structures underside of footing prior to excavating adjacent sections.

Once details of the proposed building are finalized, it is recommended that details of the underpinning program be prepared in conjunction with the projects structural engineer. It is recommended that an assessment be completed by the geotechnical engineer at the time of excavation to confirm founding conditions of the existing structure.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular. A minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on bedrock subgrade. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay materials will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations through the overburden materials and bedrock should be low to moderate and controllable using open sumps.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) will be required for this project as it is anticipated that more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP. All water takings under a PTTW are required to be reported to the MECP Water Taking Reporting Systems (WTRS).

Long-Term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's cistern/sump pit. Provided the bedrock sidewalls are prepared satisfactorily throughout the bedrock removal process and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (i.e.- less than 40,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction once groundwater infiltration levels are observed.

Impact on Neighboring Properties

Based on our observations, the long-term groundwater level is expected to be below the bedrock surface and groundwater lowering is not anticipated under short-term conditions due to construction of the proposed building. The neighboring structures are expected to be founded on bedrock. Issues are not expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

It is further expected that the adjacent SWM pond will have a pond base directly over the bedrock surface. Based on conventional pond construction, it is expected that a clay liner is most likely in place over the bedrock surface. However, the presence of the clay liner will not influence the proposed plan to limit groundwater infiltration by means of grout injections into open fractures observed during rock removal for the proposed building. Details of our recommended grout in-filling program are presented below.

It should be further noted that the bedrock quality is indicative of good to excellent quality based on the noted RQD values and the results of the hydraulic conductivity testing indicate that low groundwater infiltration rates should be encountered. However, to further reduce groundwater infiltration any water bearing seams observed across the bedrock excavation face will be sealed by means of bentonite/cement grout injections during the bedrock removal portion of the excavation program, which will further limit groundwater infiltration into the open excavation and under long-term conditions.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and tarpaulins or other suitable means. The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be considered if such activities are to be completed during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing from an adjacent site show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive to very aggressive corrosive environment.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by the geotechnical consultant:

- Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction.
- Review the bedrock stabilization and excavation requirements, if applicable.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than KRP Properties or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
March 29, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B. Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ☐ KRP Properties (Digital copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

HYDRAULIC CONDUCTIVITY ANALYSIS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Brookstreet Condominium Development
535 Legget Drive - Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 February 25

FILE NO. PG5673

HOLE NO. BH 1-21

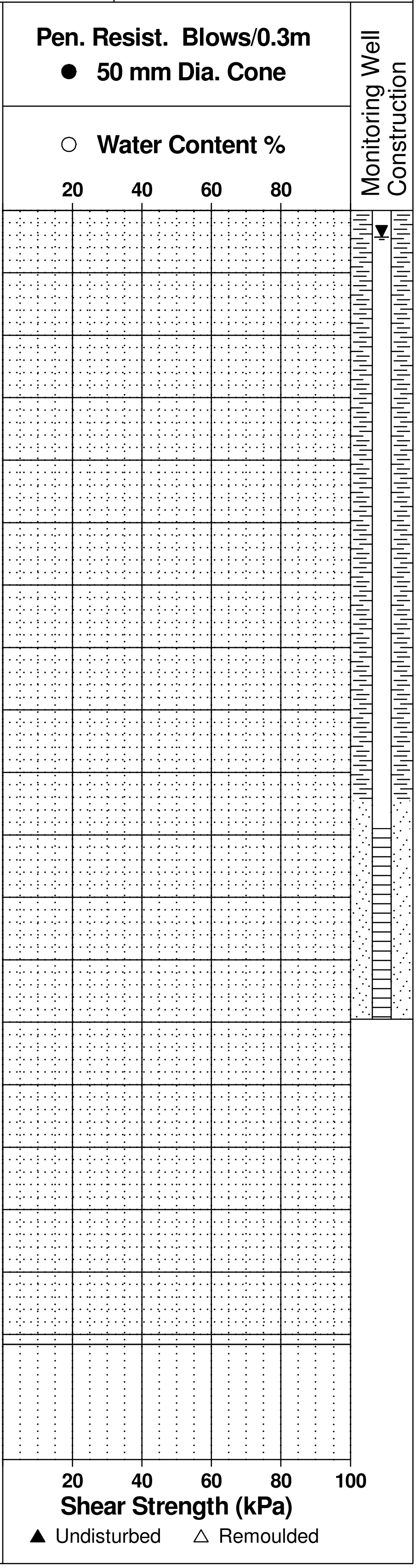
SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic Concrete	0.10
FILL: Crushed stone, some sand	1.24
Brown **SILTY CLAY** trace sand and gravel	1.37
GLACIAL TILL: Brown silty clay with gravel, cobbles and boulders	1.98
BEDROCK: Good to excellent grey sandstone with interbedded dolostone - Approximately 15 to 25 mm deep seam encountered at 10.2 m depth - Vertical seam encountered from 12.5 to 12.8 m depth	18.16

End of Borehole

(GWL @ 0.43 m depth - March 3, 2021)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
AU	2		
SS	3	50	17
SS	4	72	+50
RC	1	100	100
RC	2	100	92
RC	3	100	98
RC	4	100	97
RC	5	100	100
RC	6	100	75
RC	7	100	98
RC	8	100	83
RC	9	100	100
RC	10	100	88
RC	11	100	100

DEPTH (m)	ELEV. (m)
0	75.62
1	74.62
2	73.62
3	72.62
4	71.62
5	70.62
6	69.62
7	68.62
8	67.62
9	66.62
10	65.62
11	64.62
12	63.62
13	62.62
14	61.62
15	60.62
16	59.62
17	58.62
18	57.62



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Brookstreet Condominium Development
535 Legget Drive - Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE 2021 February 26
FILE NO. PG5673
HOLE NO. BH 2-21

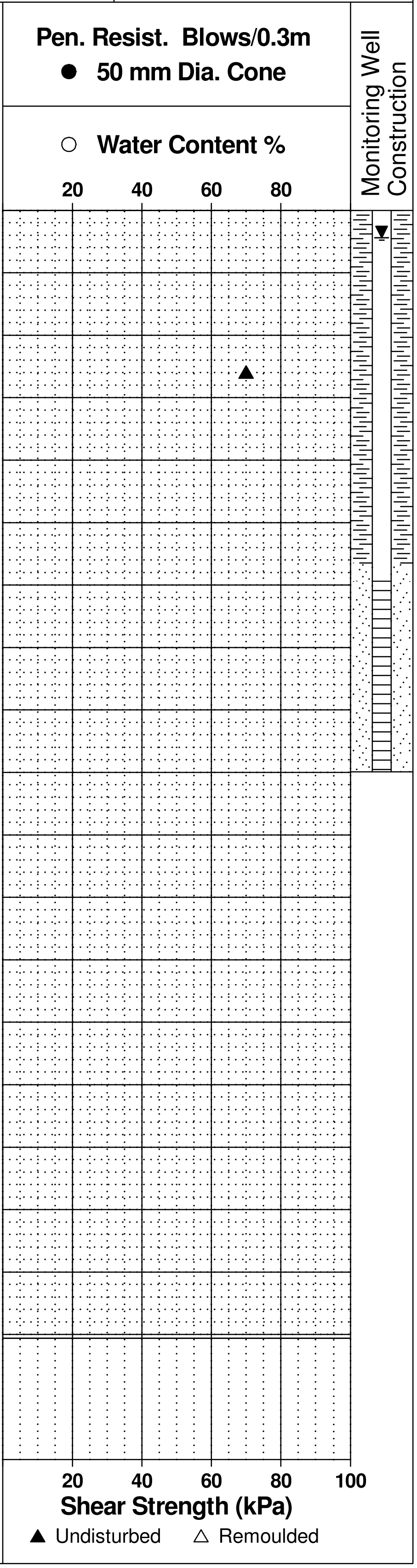
SOIL DESCRIPTION

GROUND SURFACE

SOIL DESCRIPTION	Depth (m)
Asphaltic Concrete	0.10
FILL: Brown silty sand with crushed stone	0.46 0.76
FILL: Brown silty sand with crushed stone and asphalt	
Very stiff to stiff brown **SILTY CLAY**, some sand	2.59
BEDROCK: Good to excellent quality grey sandstone with interbedded dolostone - Approximately 15 to 25 mm deep seam encountered at 11.1 m depth	18.06
End of Borehole (GWL @ 0.44 m depth - March 3, 2021)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
AU	2		
SS	3	100	8
SS	4	100	5
SS	5	67	+50
RC	1	100	82
RC	2	100	87
RC	3	100	97
RC	4	100	97
RC	5	100	98
RC	6	100	98
RC	7	98	95
RC	8	100	91
RC	9	100	100
RC	10	100	95
RC	11	100	93

DEPTH (m)	ELEV. (m)
0	75.63
1	74.63
2	73.63
3	72.63
4	71.63
5	70.63
6	69.63
7	68.63
8	67.63
9	66.63
10	65.63
11	64.63
12	63.63
13	62.63
14	61.63
15	60.63
16	59.63
17	58.63
18	57.63



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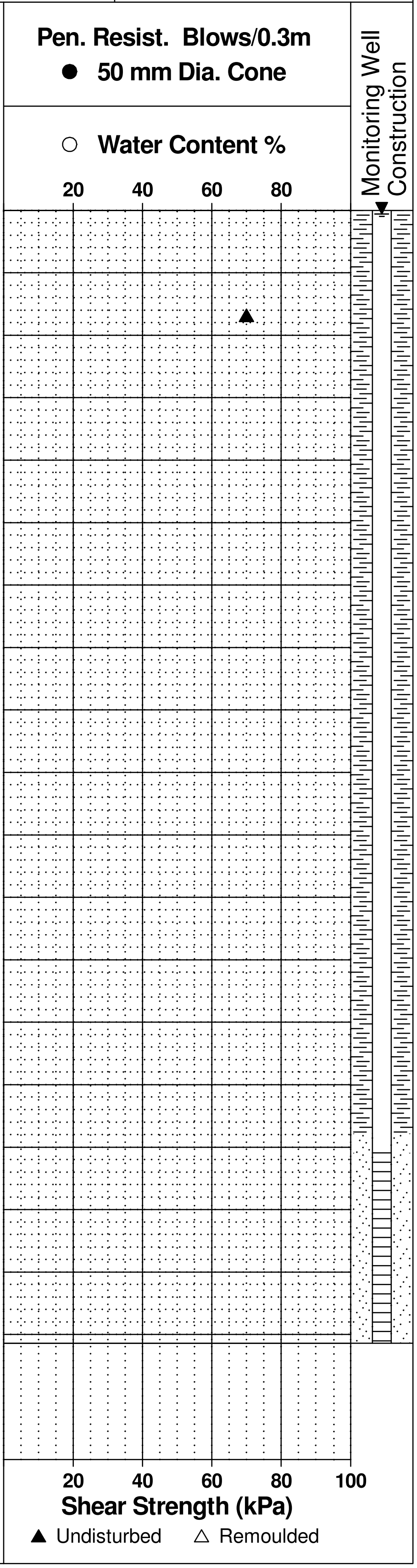
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Brookstreet Condominium Development
535 Legget Drive - Ottawa, Ontario

DATUM Geodetic
FILE NO. PG5673
REMARKS
HOLE NO. BH 3-21
BORINGS BY CME-55 Low Clearance Drill
DATE 2021 March 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	75.36
Asphaltic Concrete 0.08	AU	1				
FILL: Brown silty sand with crushed 0.76 stone and gravel 0.99	AU	2				
	SS	3	71	7	1	74.36
FILL: Grey silty clay, with crushed stone, trace sand, gravel and topsoil 2.06	SS	4	37	+50	2	73.36
Very stiff to stiff brown **SILTY CLAY** some sand	RC	1	100	87	3	72.36
BEDROCK: Good to excellent quality grey sandstone with interbedded dolostone	RC	2	100	100	4	71.36
					5	70.36
	RC	3	100	100	6	69.36
	RC	4	100	100	7	68.36
	RC	5	100	92	8	67.36
					9	66.36
	RC	6	100	100	10	65.36
- Approximately 15 to 25 mm deep seam encountered at 10.5 m depth	RC	7	100	95	11	64.36
					12	63.36
	RC	8	100	100	13	62.36
	RC	9	100	98	14	61.36
					15	60.36
	RC	10	100	100	16	59.36
	RC	11	100	100	17	58.36
18.14					18	57.36
End of Borehole						
(GWL @ 0.06 m depth - March 3, 2021)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Brookstreet Condominium Development
535 Legget Drive - Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5673

REMARKS

HOLE NO. TP 1-21

BORINGS BY Backhoe

DATE 2021 February 26

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
GROUND SURFACE						0	76.01		
FILL: Brown silty clay to clayey silt, some organics, trace sand and crushed stone		G	1						
		G	2						
						1	75.01		
		G	3						
						2	74.01		
		G	4						
2.45		G	5						
GLACIAL TILL: Brown silty clay, some sand, gravel, cobbles and boulders									
3.05		G	6			3	73.01		
End of Test Pit Refusal to excavation on bedrock surface at 3.05 m depth (Open hole GWL at 3.00 m depth)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Brookstreet Condominium Development
535 Legget Drive - Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5673

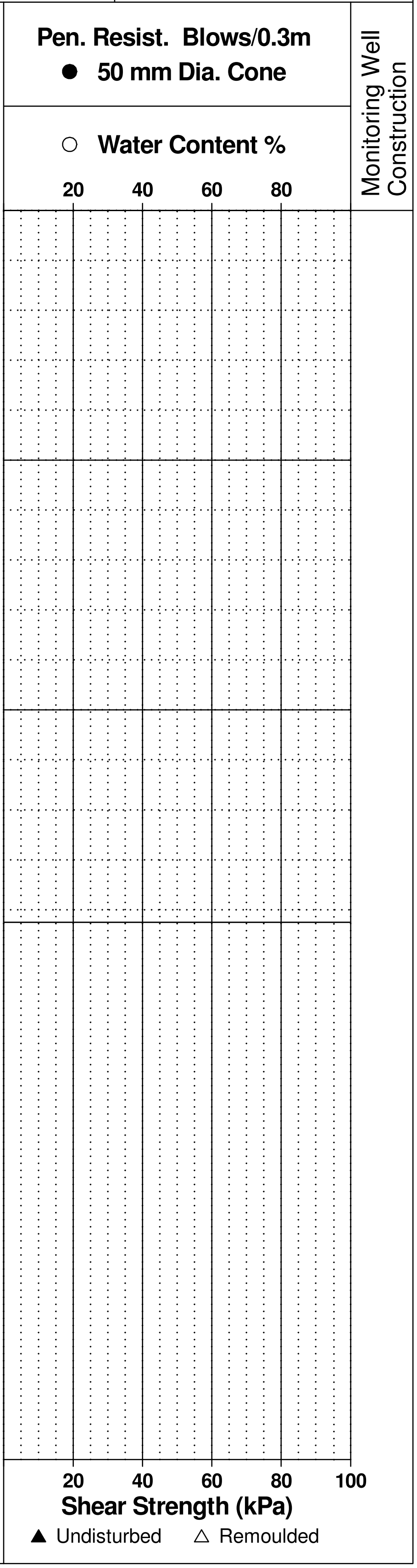
REMARKS

HOLE NO. TP 2-21

BORINGS BY Backhoe

DATE 2021 February 26

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	75.63
FILL: Brown silty clay, some sand and crushed stone		G	1				
						1	74.63
1.45							
Very stiff brown **SILTY CLAY**		G	2				
1.95							
						2	73.63
GLACIAL TILL: Brown silty clay some sand, gravel, cobbles and boulders		G	3				
2.85							
End of Test Pit Refusal to excavation on bedrock surface 2.85 m depth (Open hole GWL at 2.61 m depth)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

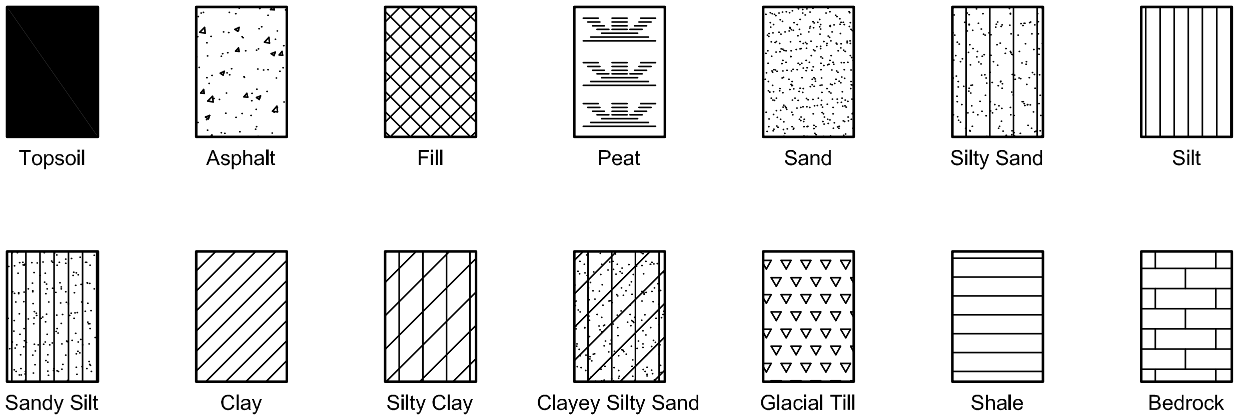
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

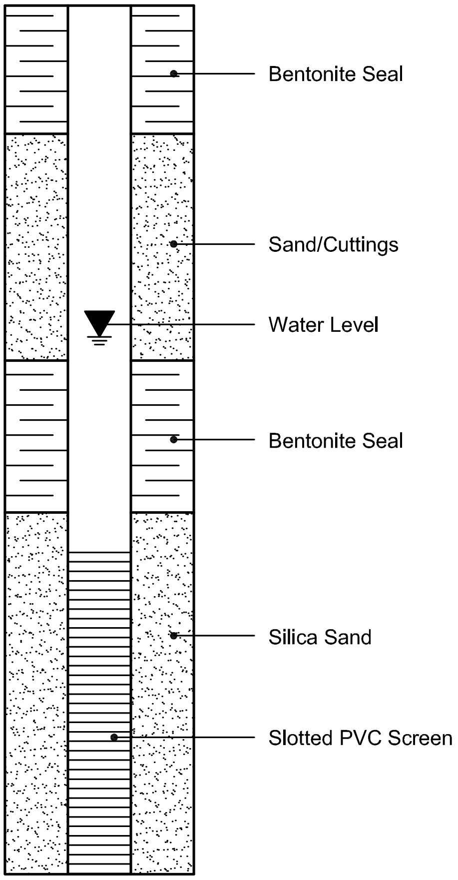
SYMBOLS AND TERMS (continued)

STRATA PLOT

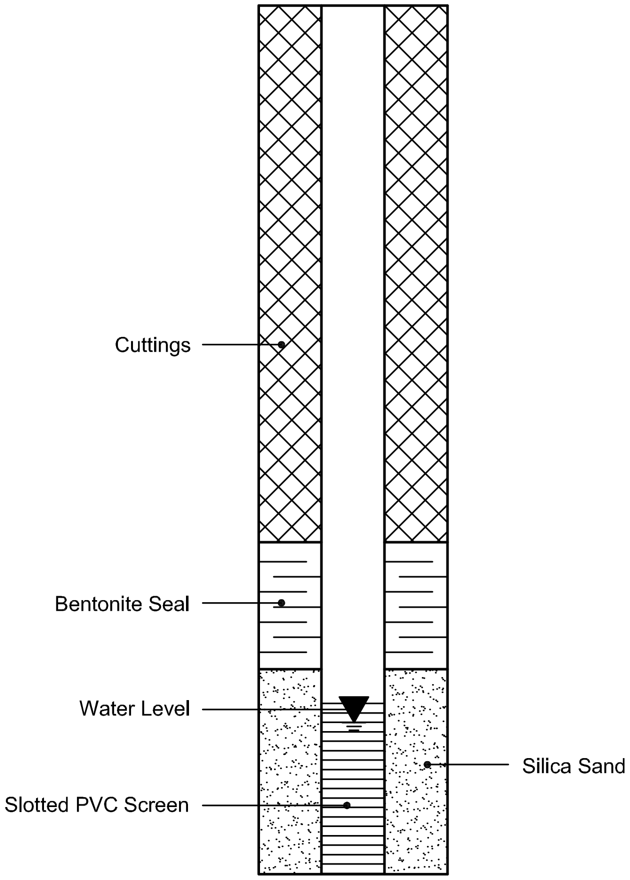


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PROJECT: 20396226

RECORD OF BOREHOLE: 20-01

SHEET 1 OF 3

LOCATION: N 5023374.3 ;E 350498.6

BORING DATE: November 12-13, 2020

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

ADDITIONAL LAB. TESTING

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION ∇
		GROUND SURFACE	76.21				
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE					Flush Mount Casing
		FILL - (SW) gravelly SAND, trace to some silt, angular; grey (PAVEMENT STRUCTURE)	0.05				
		FILL - (SW/GW) gravelly SAND to sandy GRAVEL, trace to some silt, angular; grey (PAVEMENT STRUCTURE)	75.91 0.30				
1				1	SS	26	Cuttings
		(CI/CH) SILTY CLAY to CLAY, trace sand; grey brown, highly fissured, thin lamination of silty sand (WEATHERED CRUST); cohesive w>PL, very stiff	74.69 1.52	2	SS	6	
2		Possible Bedrock	74.08 2.13				Bentonite Seal
		Borehole continued on RECORD OF DRILLHOLE 20-01	2.27				
3							
4							
5							
6							
7							
8							
9							
10							

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: KCP

RECORD OF DRILLHOLE: 20-01

PROJECT: 20396226

SHEET 2 OF 3

LOCATION: N 5023374.31 ;E 350498.56

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 12-13, 2020

DRILL RIG: CME 75

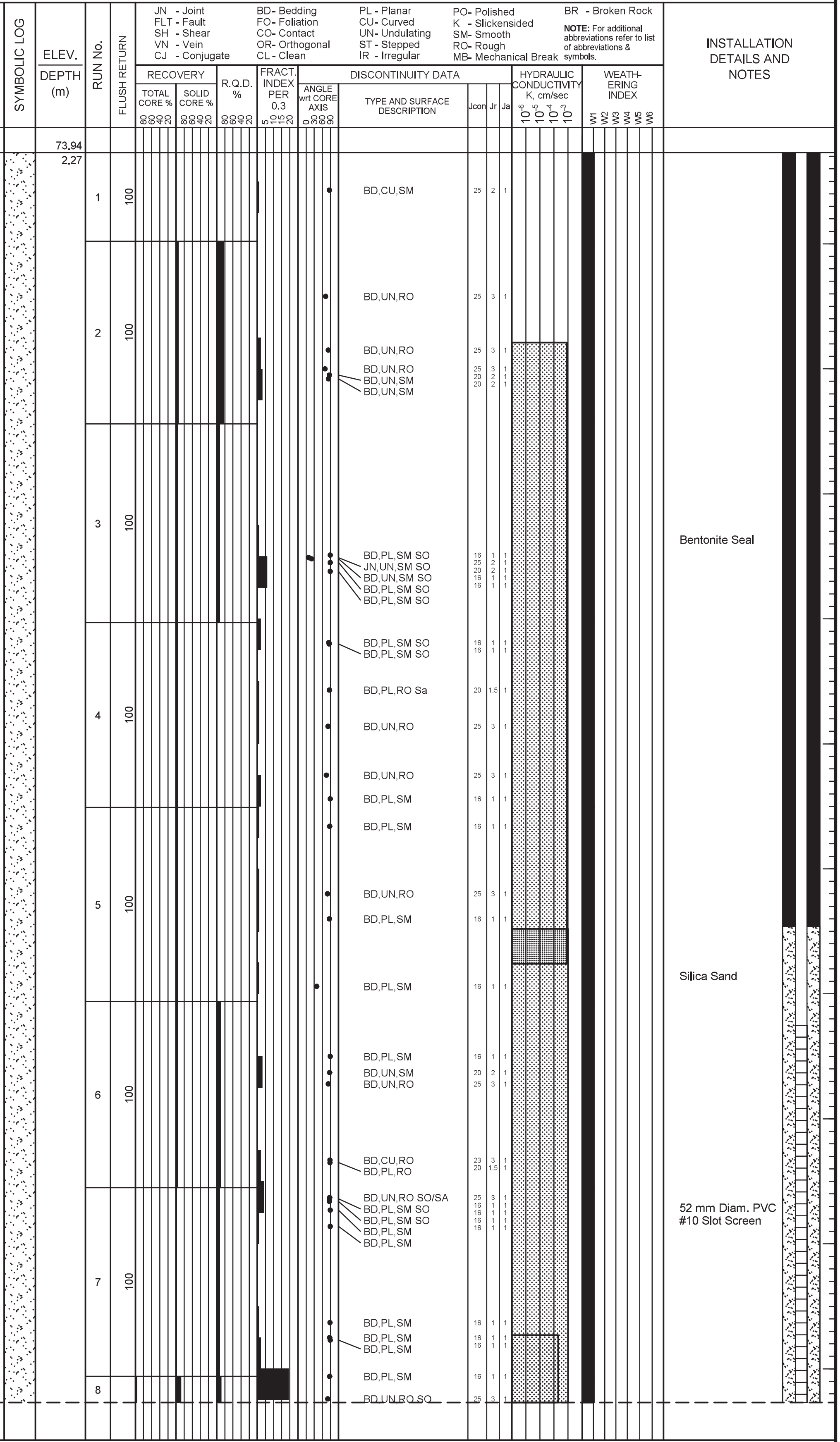
DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SH - Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Clean
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)
		BEDROCK SURFACE	73.94 2.27
3	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, dark to light grey, fine to medium grained, non to faintly porous, strong to very strong SANDSTONE - Broken core at 2.57 m depth, shows signs of rock curling. Possible sandstone boulder or weathered sandstone slab on top of bedrock	
5		- Broken core from 5.52 m to 5.55 m	
10		- Broken core from 10.63 m to 10.64 m	
12		- Broken/lost core from 12.06 m to 12.24 m	
		CONTINUED NEXT PAGE	



TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja
BD,CU,SM	25	2	1
BD,UN,RO	25	3	1
BD,UN,RO	25	3	1
BD,UN,RO	25	3	1
BD,UN,SM	20	2	1
BD,UN,SM	20	2	1
BD,PL,SM SO	16	1	1
JN,UN,SM SO	25	2	1
BD,UN,SM SO	20	2	1
BD,PL,SM SO	16	1	1
BD,PL,SM SO	16	1	1
BD,PL,SM SO	16	1	1
BD,PL,SM SO	16	1	1
BD,PL,RO Sa	20	1.5	1
BD,UN,RO	25	3	1
BD,UN,RO	25	3	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,UN,RO	25	3	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,UN,SM	20	2	1
BD,UN,RO	25	3	1
BD,CU,RO	23	3	1
BD,PL,RO	20	1.5	1
BD,UN,RO SO/SA	25	3	1
BD,PL,SM SO	16	1	1
BD,PL,SM SO	16	1	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,UN,RO SO	25	3	1

DEPTH SCALE
1 : 50

LOGGED: RI
CHECKED: KCP

PROJECT: 20396226

RECORD OF DRILLHOLE: 20-01

SHEET 3 OF 3

LOCATION: N 5023374.31 ;E 350498.56

INCLINATION: -90° AZIMUTH: —

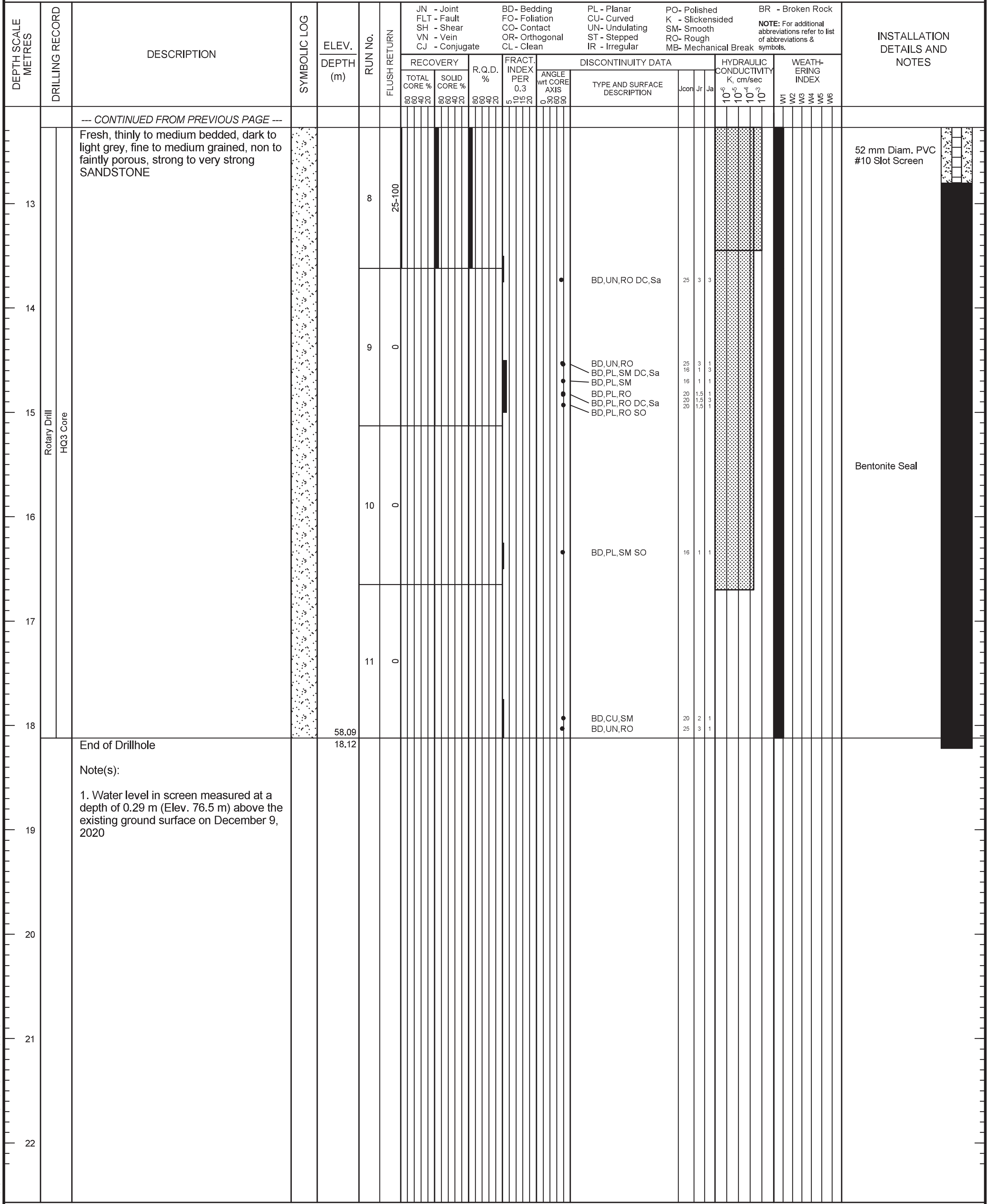
DRILLING DATE: November 12-13, 2020

DRILL RIG: CME 75

DRILLING CONTRACTOR: Downing 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.
SH - Shear	CO- Contact	UN- Undulating	SM- Smooth	
VN - Vein	OR- Orthogonal	ST - Stepped	RO- Rough	
CJ - Conjugate	CL - Clean	IR - Irregular	MB- Mechanical Break	



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DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN
Fresh, thinly to medium bedded, dark to light grey, fine to medium grained, non to faintly porous, strong to very strong SANDSTONE		8	25-100
		9	0
		10	0
		11	0
	58.09 18.12		
End of Drillhole			

Note(s):

1. Water level in screen measured at a depth of 0.29 m (Elev. 76.5 m) above the existing ground surface on December 9, 2020

DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja
BD,UN,RO DC,Sa	25	3	3
BD,UN,RO	25	3	1
BD,PL,SM DC,Sa	16	1	3
BD,PL,SM	16	1	1
BD,PL,RO	20	1.5	1
BD,PL,RO DC,Sa	20	1.5	3
BD,PL,RO SO	20	1.5	1
BD,PL,SM SO	16	1	1
BD,CU,SM	20	2	1
BD,UN,RO	25	3	1

INSTALLATION DETAILS AND NOTES:

52 mm Diam. PVC
#10 Slot Screen

Bentonite Seal

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: KCP

PROJECT: 20396226

RECORD OF BOREHOLE: 20-03

SHEET 1 OF 3

LOCATION: N 55023407.1 ;E 350521.8

BORING DATE: November 13, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

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

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

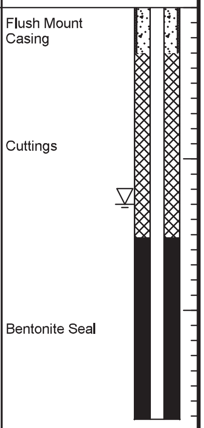
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

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

ADDITIONAL LAB. TESTING

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m
	GROUND SURFACE	75.76			
0	ASPHALTIC CONCRETE	0.05			
	FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE)	75.40 0.36			
	FILL - (SW/GW) sandy GRAVEL to gravelly SAND, angular; grey (PAVEMENT STRUCTURE)	74.95 0.81			
1	TOPSOIL - (CI/CH) SILTY CLAY to CLAY, trace sand; dark grey brown, highly fissured, contains black organic mottling and organic matter (rootlets); cohesive, w>PL, very stiff	74.62 1.14	1	SS	6
2	(CI/CH) SILTY CLAY to CLAY, trace sand; grey brown, highly fissured, thin laminations of silty sand (WEATHERED CRUST); cohesive, w>PL, very stiff		2	SS	5
		73.04 2.72	3	SS	>50
3	Borehole continued on RECORD OF DRILLHOLE 20-03				
4					
5					
6					
7					
8					
9					
10					

PIEZOMETER OR STANDPIPE INSTALLATION



DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: KCP

PROJECT: 20396226

RECORD OF DRILLHOLE: 20-03

SHEET 2 OF 3

LOCATION: N 55023407.10 ;E 350521.80

INCLINATION: -90°   AZIMUTH: ---

DRILLING DATE: November 13, 2020

DRILL RIG: CME 75

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

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

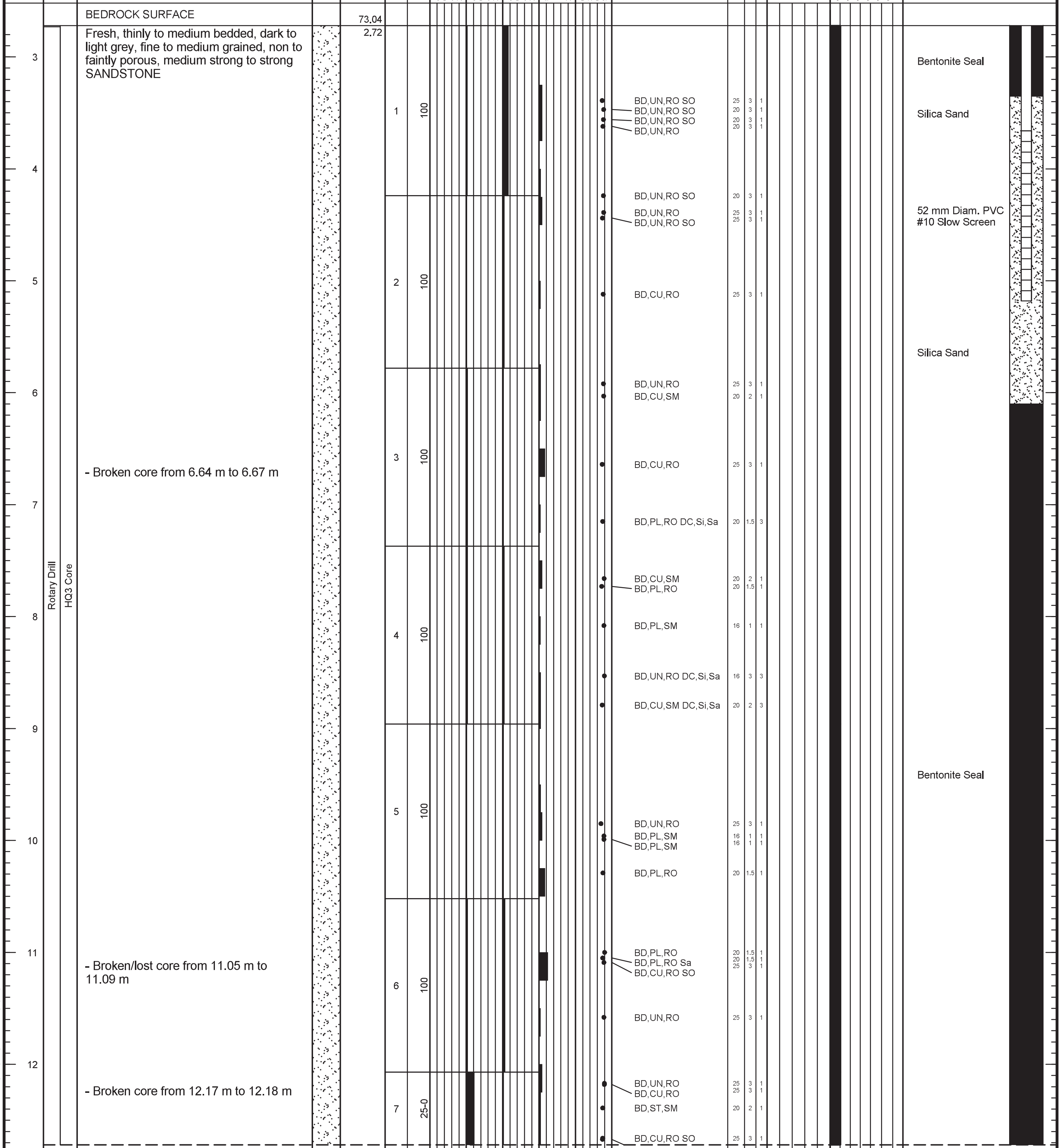
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Clean

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.



DESCRIPTION	ELEV. DEPTH (m)
BEDROCK SURFACE	73.04 2.72
Fresh, thinly to medium bedded, dark to light grey, fine to medium grained, non to faintly porous, medium strong to strong SANDSTONE	
- Broken core from 6.64 m to 6.67 m	
- Broken/lost core from 11.05 m to 11.09 m	
- Broken core from 12.17 m to 12.18 m	

RUN No.	FLUSH RETURN
1	100
2	100
3	100
4	100
5	100
6	100
7	25-0

TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja
BD,UN,RO SO	25	3	1
BD,UN,RO SO	20	3	1
BD,UN,RO SO	20	3	1
BD,UN,RO	20	3	1
BD,UN,RO SO	20	3	1
BD,UN,RO	25	3	1
BD,UN,RO SO	25	3	1
BD,CU,RO	25	3	1
BD,UN,RO	25	3	1
BD,CU,SM	20	2	1
BD,CU,RO	25	3	1
BD,PL,RO DC,Si,Sa	20	1.5	3
BD,CU,SM	20	2	1
BD,PL,RO	20	1.5	1
BD,PL,SM	16	1	1
BD,UN,RO DC,Si,Sa	16	3	3
BD,CU,SM DC,Si,Sa	20	2	3
BD,UN,RO	25	3	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,PL,RO	20	1.5	1
BD,PL,RO	20	1.5	1
BD,PL,RO Sa	20	1.5	1
BD,CU,RO SO	25	3	1
BD,UN,RO	25	3	1
BD,UN,RO	25	3	1
BD,CU,RO	25	3	1
BD,ST,SM	20	2	1
BD,CU,RO SO	25	3	1

INSTALLATION DETAILS AND NOTES

Bentonite Seal

Silica Sand

52 mm Diam. PVC
#10 Slow Screen

Silica Sand

Bentonite Seal

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: KCP

PROJECT: 20396226

RECORD OF DRILLHOLE: 20-03

SHEET 3 OF 3

LOCATION: N 55023407.10 ;E 350521.80

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 13, 2020

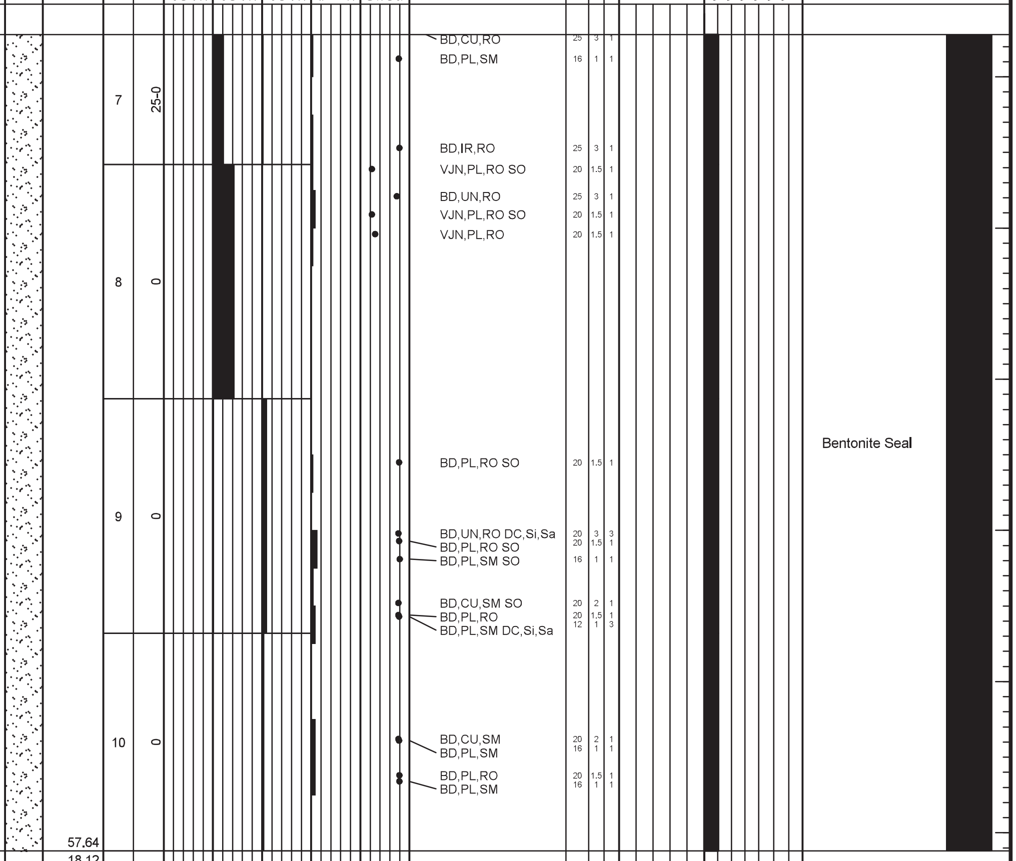
DRILL RIG: CME 75

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SH - Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Clean
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)
		--- CONTINUED FROM PREVIOUS PAGE ---	
13	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, dark to light grey, fine to medium grained, non to faintly porous, medium strong to strong SANDSTONE - Vertical joint from 13.29 m to 13.91 m - Vertical joint from 13.73 m to 14.25 m - Vertical joint from 13.91 m to 14.12 m - Broken core from 16.56 m to 16.57 m	
18		End of Drillhole Note(s): 1. Water level in screen measured at a depth of 1.3 m (Elev. 74.5 m) on December 2, 2020	57.64 18.12

RUN No.	FLUSH RETURN
7	25-0
8	0
9	0
10	0

TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja
BD,CU,RO	25	3	1
BD,PL,SM	16	1	1
BD,IR,RO	25	3	1
VJN,PL,RO SO	20	1.5	1
BD,UN,RO	25	3	1
VJN,PL,RO SO	20	1.5	1
VJN,PL,RO	20	1.5	1
BD,PL,RO SO	20	1.5	1
BD,UN,RO DC,Si,Sa	20	3	3
BD,PL,RO SO	20	1.5	1
BD,PL,SM SO	16	1	1
BD,CU,SM SO	20	2	1
BD,PL,RO	20	1.5	1
BD,PL,SM DC,Si,Sa	12	1	3
BD,CU,SM	20	2	1
BD,PL,SM	16	1	1
BD,PL,RO	20	1.5	1
BD,PL,SM	16	1	1

19

20

21

22

GENERIC - ROCK 20396226.GPJ GAL-MISS.GDT 1/14/21

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: KCP

PROJECT: 20396226

RECORD OF BOREHOLE: 20-04

SHEET 1 OF 3

LOCATION: N 5023422.6 ;E 350487.4

BORING DATE: November 13, 2020

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	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		75.65					
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■						Flush Mount Casing Silica Sand
		FILL - (SW) gravelly SAND; angular; grey (PAVEMENT STRUCTURE)	▨	0.05					Bentonite Seal
		FILL - (ML/SM) gravelly SILTY SAND to sandy SILT, some plastic fines; dark brown, contains cobbles and boulders; non-cohesive, moist, compact	▨	75.24 0.41	1	SS	13		
1		FILL - (ML/SM) SILTY SAND to sandy SILT, some plastic fine; dark brown, contains organic matter (rootlets); non-cohesive, moist to wet, compact	▨	74.48 1.17					Grout
			▨	74.13 1.52					
		FILL - (CI/CH) SILTY CLAY to CLAY, trace sand; dark brown to grey brown, highly fissured; cohesive, w>PL	▧	1.68	2	SS	4		63.5 mm VSP Pipe
2		(CI/CH) SILTY CLAY to CLAY, trace sand; grey brown; highly fissured, thin laminations of silty sand (WEATHERED CRUST); cohesive, w>PL, very stiff		73.34 2.31					
		Borehole continued on RECORD OF DRILLHOLE 20-04							
3									
4									
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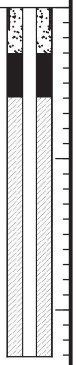
6									
7									
8									
9									
10									



DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: KCP

PROJECT: 20396226

RECORD OF DRILLHOLE: 20-04

SHEET 2 OF 3

LOCATION: N 5023422.60 ;E 350487.43

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 13, 2020

DRILL RIG: CME 75

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SH - Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Clean
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.	FLUSH RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.3 5 10 15 20	ANGLE wrt CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	WEATHERING INDEX W1 W2 W3 W4 W5 W6	INSTALLATION DETAILS AND NOTES
		BEDROCK SURFACE		73.34														
	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, dark to light grey, fine to medium grained, non to faintly porous, medium strong to strong SANDSTONE		2.31								BD,UN,RO Sa	22	3	1			
												BD,UN,RO	25	3	1			
3					1	100												
												BD,PL,SM DC,Ca	16	1	3			UCS = 109 MPa
4												BD,UN,RO So	25	3	1			
												BD,UN,RO So	25	3	1			
												JN,PL,SM So	16	1	1			
					2	100						BD,CU,RO	25	3	1			
												BD,UN,RO	25	3	1			
5																		
												BD,UN,RO So BD,UN,SM So	25 20	3 2	1 1			
6												BD,PL,RO So	20	1.5	1			
					3	100												
												BD,PL,RO BD,PL,SM	20 16	1.5 1	1 1			
												BD,PL,RO BD,PL,RO So BD,PL,RO So	20 20 20	1.5 1.5 1.5	1 1 1			
7												BD,UN,RO So	25	3	1			Grout
												BD,UN,RO So	25	3	1			63.5 mm VSP Pipe
					4	100												
8																		
												BD,UN,RO So	25	3	1			
9																		
					5	100												UCS = 126 MPa
												BD,PL,SM	16	1	1			
10												BD,UN,RO So	20	1.5	1			
		- Broken/lost core and fractured rock from 10.59 m to 10.65 m										BD,UN,RO BD,PL,SM So	25 16	3 1	1 1			
					6	100-0												
11																		
		- Broken core from 11.61 m to 11.62 m										BD,PL,RO BD,PL,RO	20 20	1.5 1.5	1 1			
												BD,UN,RO	25	3	1			
12					7	0												
												BD,PL,SM	16	1	1			

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: KCP

PROJECT: 20396226

RECORD OF DRILLHOLE: 20-04

SHEET 3 OF 3

LOCATION: N 5023422.60 ;E 350487.43

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 13, 2020

DRILL RIG: CME 75

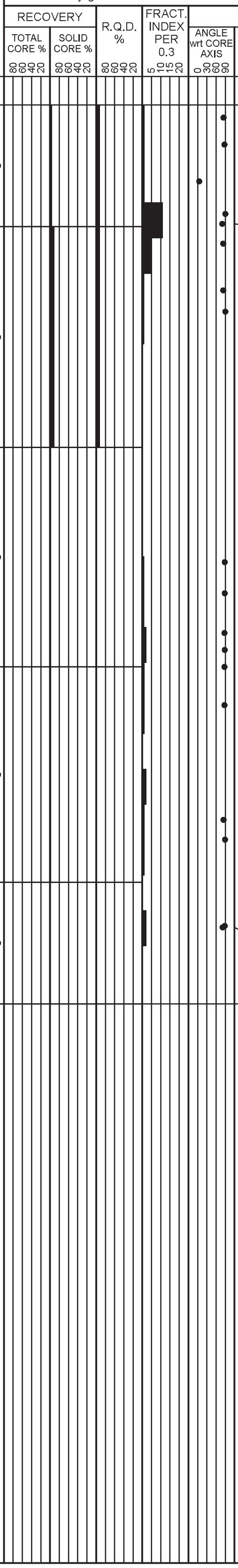
DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SH - Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Clean
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 RETURN	INSTALLATION DETAILS AND NOTES
		--- CONTINUED FROM PREVIOUS PAGE ---				
13 14 15 16 17 18	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, dark to light grey, fine to medium grained, non to faintly porous, medium strong to strong SANDSTONE - HVJN from 12.59 m to 13.62 m - Lost core from 13.15 m to 13.17 m - Broken/lost core form 13.17 m to 13.29 m	56.99 18.66	7 8 9 10 11	0 0 0 0 0	Grout 63.5 mm VSP Pipe UCS = 135 MPa
19 20 21 22		End of Drillhole Note(s): 1. Water level in screen measured at a depth of 0.4 m (Elev. 76.1 m) above the existing ground surface on December 3, 2020				



DISCONTINUITY DATA

TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja
BD,IR,RO	25	3	1
BD,PL,RO So	20	1.5	1
BD,PL,H IN,Cal	25	4	0.75
HVJN,PL,RO So	20	1.5	1
BD,PL,RO	20	1.5	1
BD,CU,SM DC,Si,Sa	20	2	3
BD,PL,SM DC,Si,Sa	16	1	3
BD,PL,SM So	16	1	1
BD,UN,RO	25	3	1
BD,PL,SM So	16	1	1
BD,UN,RO So	25	3	1
BD,UN,RO So	25	3	1
BD,PL,SM	16	1	1
BD,CU,RO	25	3	1
BD,UN,RO So	25	3	1
BD,UN,RO So	25	3	1
BD,UN,RO	25	3	1
BD,UN,RO	25	3	1

HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WEATHERING INDEX W1 W2 W3 W4 W5 W6

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: KCP

PROJECT: 20396226

RECORD OF BOREHOLE: 20-05

SHEET 1 OF 3

LOCATION: N 5023456.7 ;E 350451.9

BORING DATE: November 17, 2020

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

ADDITIONAL LAB. TESTING

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	75.67				
0	ASPHALTIC CONCRETE	0.05				Asphalt
	FILL - (SW) gravelly SAND, trace to some silt, angular; grey (PAVEMENT STRUCTURE)	75.11 0.56				
1	FILL - (SW) gravelly SAND, trace silt; brown, contains cobbles and boulders (PAVEMENT STRUCTURE); non-cohesive, moist, compact	74.15 1.52	1	SS	19	Cuttings
	(CI/CH) SILTY CLAY to CLAY, trace sand; grey brown, highly fissured (WEATHERED CRUST); cohesive, w>PL, very stiff	73.84 1.86	2	SS	3	Bentonite Seal
2	(CL/CI) sandy SILTY CLAY, some gravel; grey brown; cohesive, w>PL, very stiff					
	Borehole continued on RECORD OF DRILLHOLE 20-05					

Depth scale: 0 1 2 3 4 5 6 7 8 9 10

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: KCP

PROJECT: 20396226

RECORD OF DRILLHOLE: 20-05

SHEET 2 OF 3

LOCATION: N 5023456.74 ;E 350451.85

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 17, 2020

DRILL RIG: CME 75

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

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

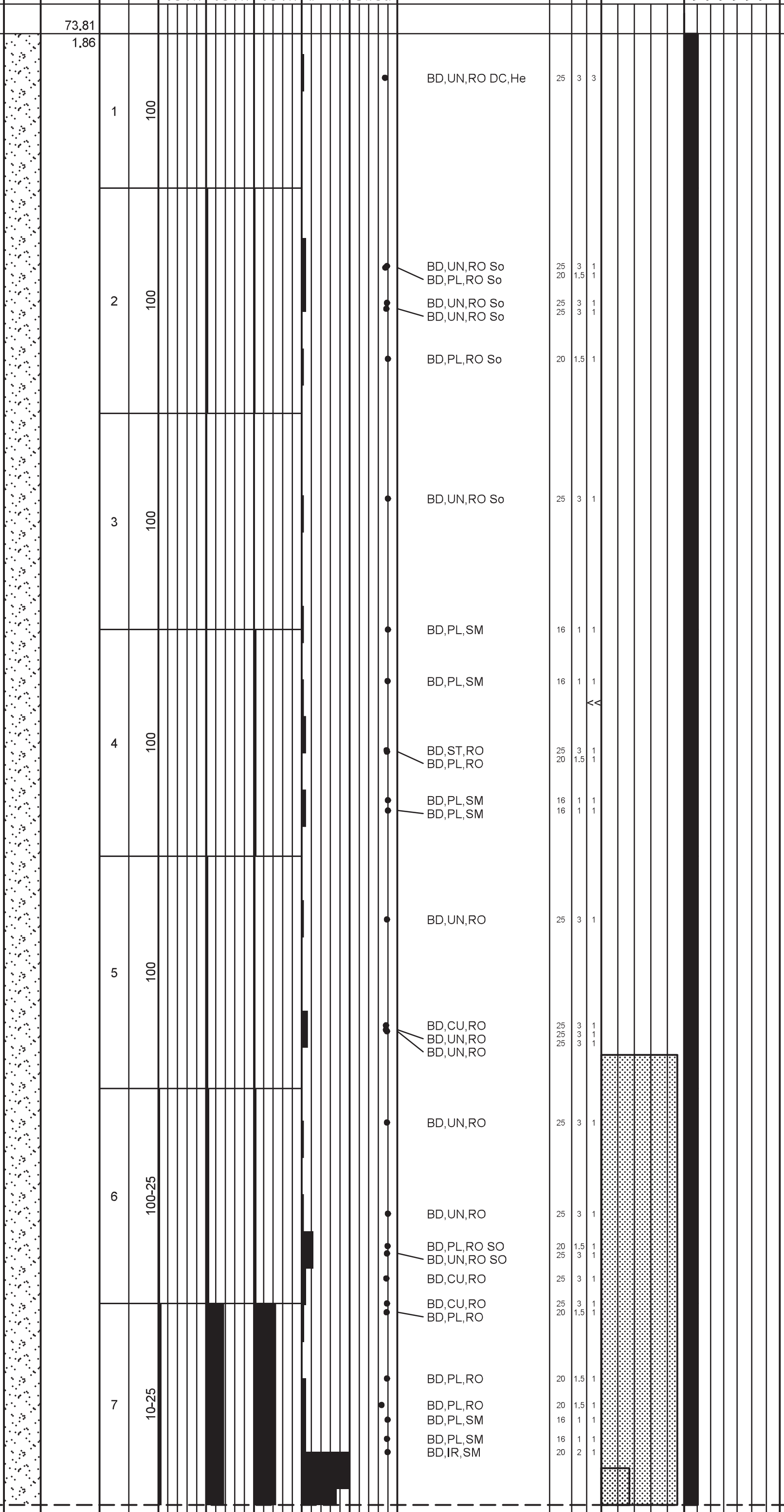
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Clean

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 RETURN	INSTALLATION DETAILS AND NOTES
		BEDROCK SURFACE	73.81 1.86			
2	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, dark to light grey, fine to medium gravel, non to faintly porous, medium strong to strong SANDSTONE		1	100	
3						UCS = 124 MPa
		- Broken core from 3.44 m to 3.45 m		2	100	
4						
5				3	100	
6						
		- Broken core from 6.73 m to 6.74 m		4	100	Bentonite Seal
7						
8				5	100	
		- Lost core from 8.63 m to 8.64 m				
9						UCS = 151 MPa
				6	100-25	
10		- Lost core from 10.10 m to 10.15 m				
11				7	10-25	
		- Broken/lost core from 11.50 m to 11.91 m				

DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja
BD,UN,RO DC,He	25	3	3
BD,UN,RO So	25	3	1
BD,PL,RO So	20	1.5	1
BD,UN,RO So	25	3	1
BD,UN,RO So	25	3	1
BD,PL,RO So	20	1.5	1
BD,UN,RO So	25	3	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,ST,RO	25	3	1
BD,PL,RO	20	1.5	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,UN,RO	25	3	1
BD,CU,RO	25	3	1
BD,UN,RO	25	3	1
BD,UN,RO	25	3	1
BD,UN,RO	25	3	1
BD,UN,RO	25	3	1
BD,PL,RO SO	20	1.5	1
BD,UN,RO SO	25	3	1
BD,CU,RO	25	3	1
BD,CU,RO	25	3	1
BD,PL,RO	20	1.5	1
BD,PL,RO	20	1.5	1
BD,PL,RO	20	1.5	1
BD,PL,SM	16	1	1
BD,PL,SM	16	1	1
BD,IR,SM	20	2	1

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: KCP

PROJECT: 20396226

RECORD OF DRILLHOLE: 20-05

SHEET 3 OF 3

LOCATION: N 5023456.74 ;E 350451.85

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: November 17, 2020

DRILL RIG: CME 75

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SH - Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
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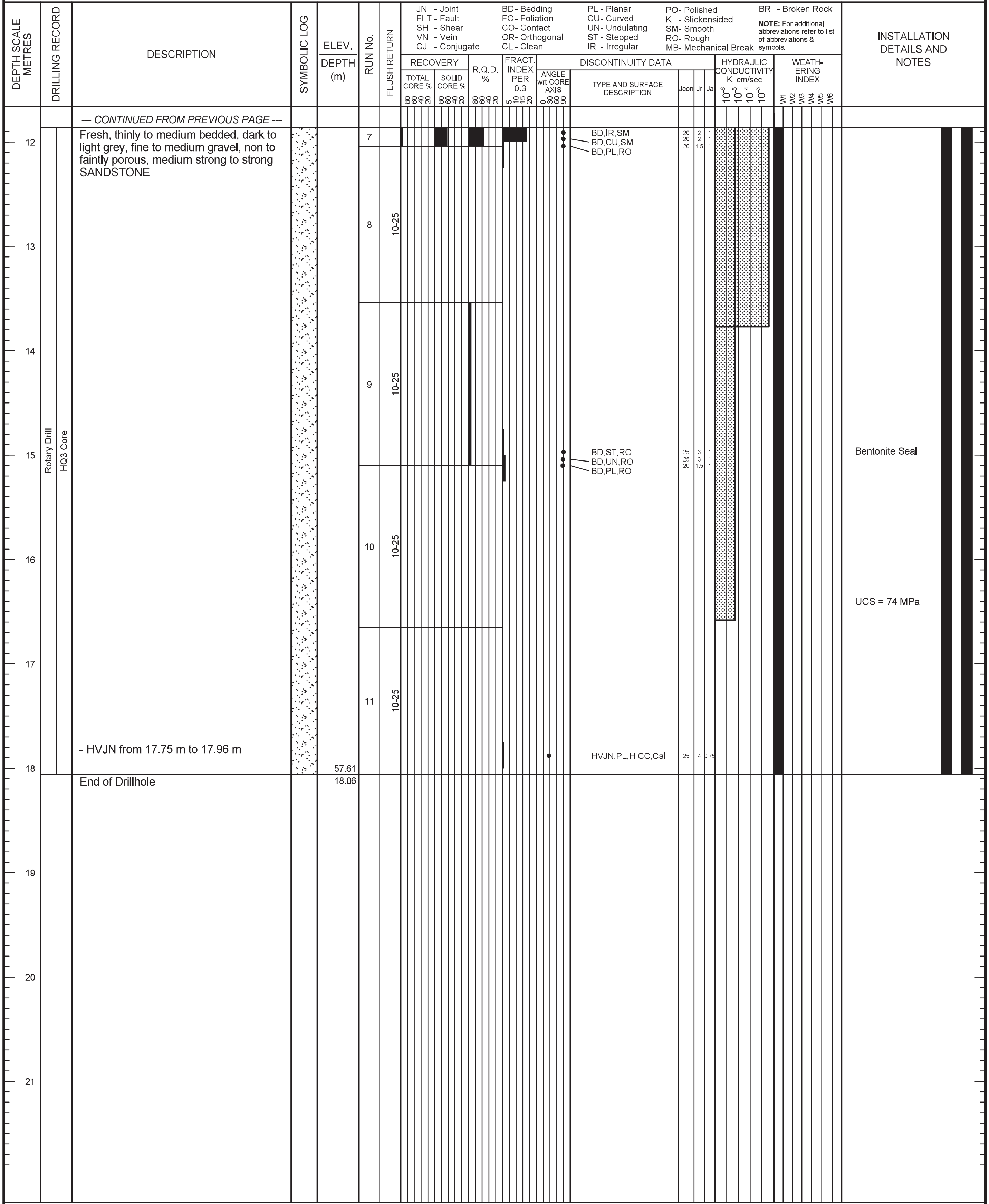
CL - Clean
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
RO- Rough
MB- Mechanical Break
BR - Broken Rock

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



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	INSTALLATION DETAILS AND NOTES
		--- CONTINUED FROM PREVIOUS PAGE ---								
12	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, dark to light grey, fine to medium gravel, non to faintly porous, medium strong to strong SANDSTONE		7		BD,IR,SM BD,CU,SM BD,PL,RO	20 20 20	2 2 1.5	1 1 1	
13				8	10-25					
14				9	10-25					
15						BD,ST,RO BD,UN,RO BD,PL,RO	25 25 20	3 3 1.5	1 1 1	Bentonite Seal
16				10	10-25					UCS = 74 MPa
17				11	10-25					
		- HVJN from 17.75 m to 17.96 m				HVJN,PL,H CC,Cal	25	4	0.75	
18			57.61 18.06							
		End of Drillhole								
19										
20										
21										

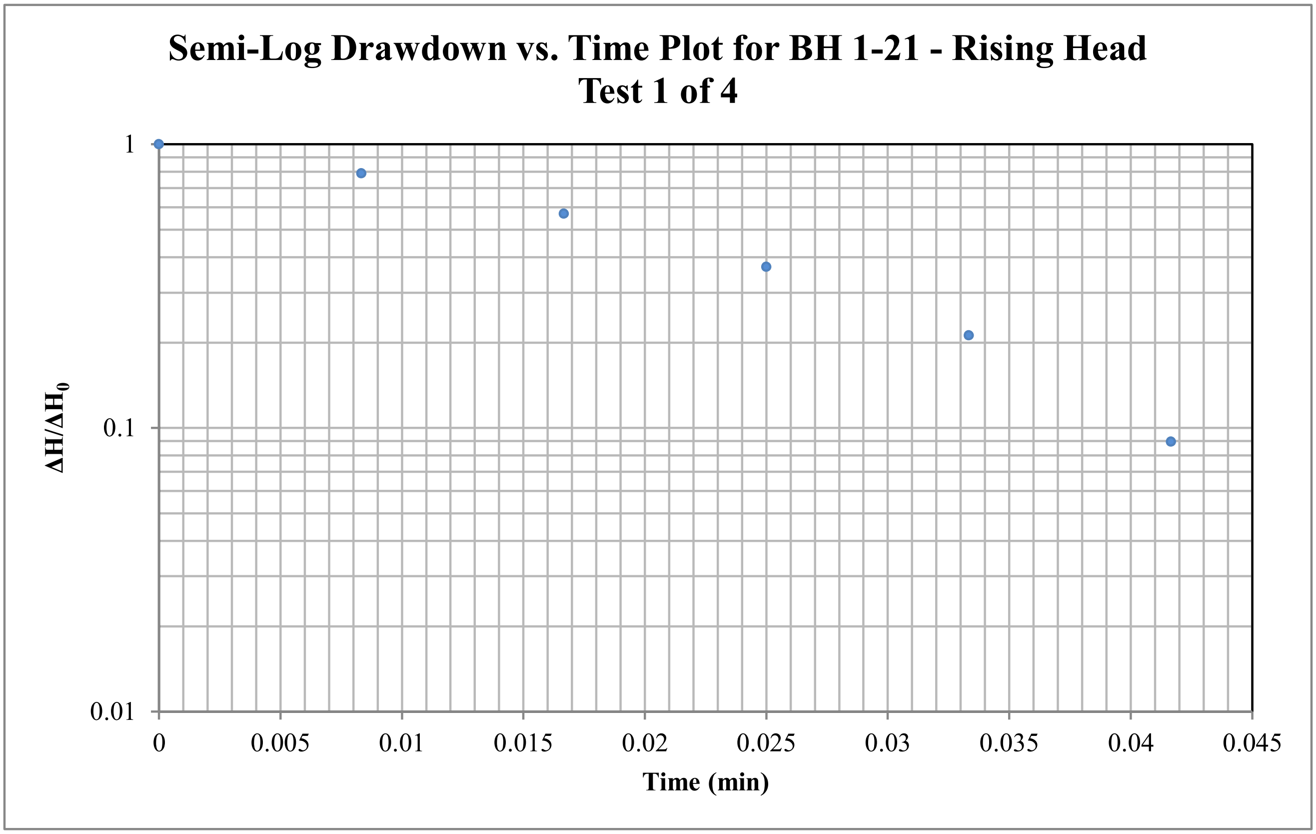
DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: KCP

Hvorslev Hydraulic Conductivity Analysis

Project: KRP Properties - 535 Legget Drive
Test Location: BH 1-21
Test: 1 of 4 Rising Head
Date: March 3, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.64264

Well Parameters:

L	3.048 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

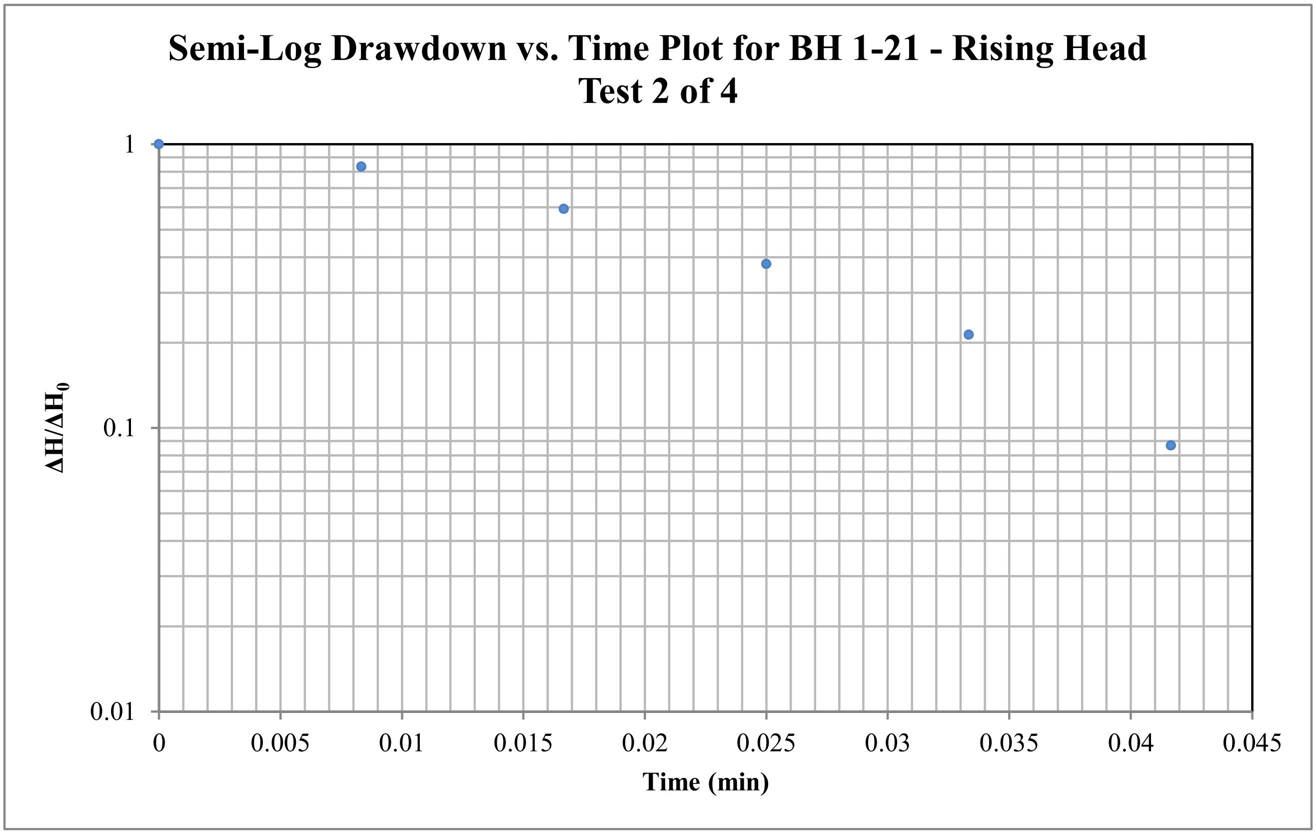
Data Points (from plot):

t*: 0.025 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.44E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: KRP Properties - 535 Legget Drive
Test Location: BH 1-21
Test: 2 of 4 Rising Head
Date: March 3, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.64264

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

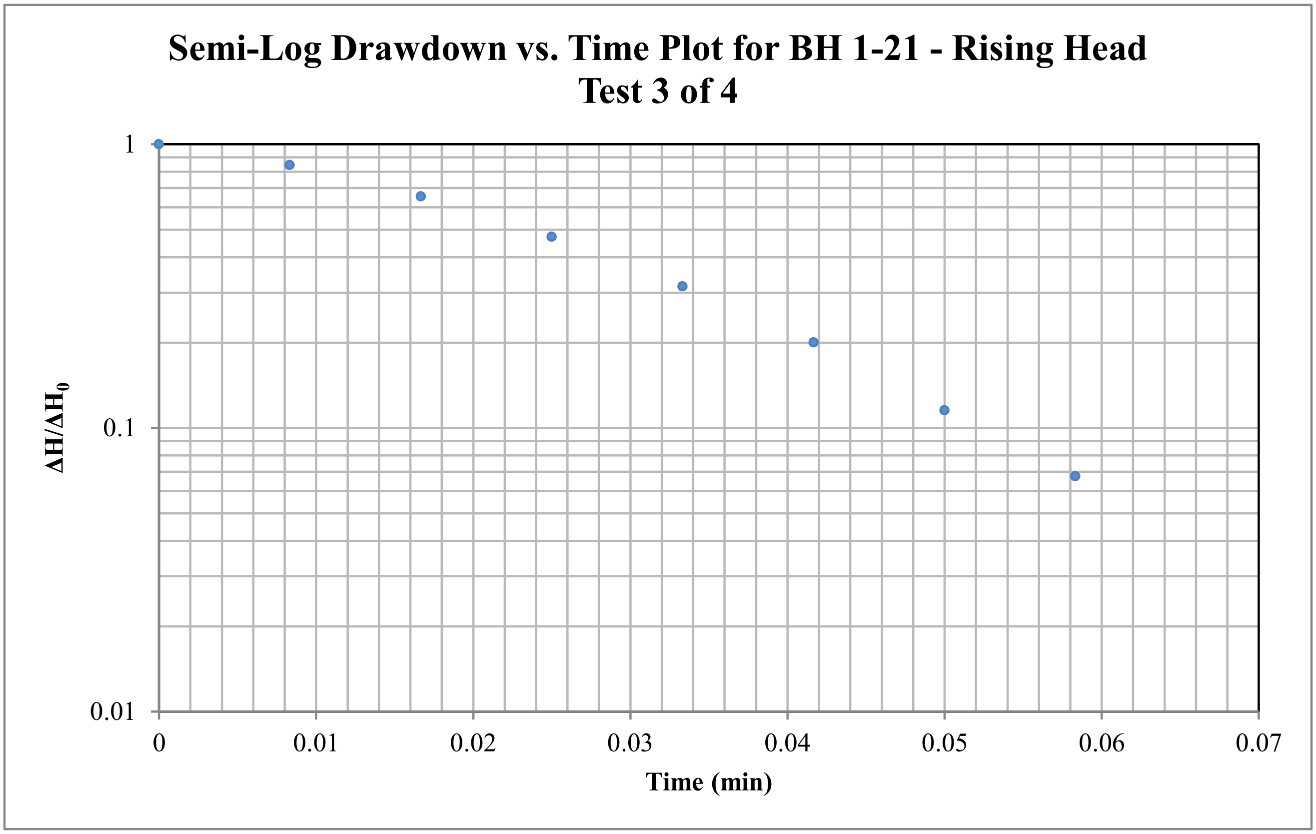
Data Points (from plot):

t*: 0.025 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.42E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: KRP Properties - 535 Legget Drive
Test Location: BH 1-21
Test: 3 of 4 Rising Head
Date: March 3, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t*}\ln\left(\frac{\Delta H *}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.64264

Well Parameters:

L	3.048 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

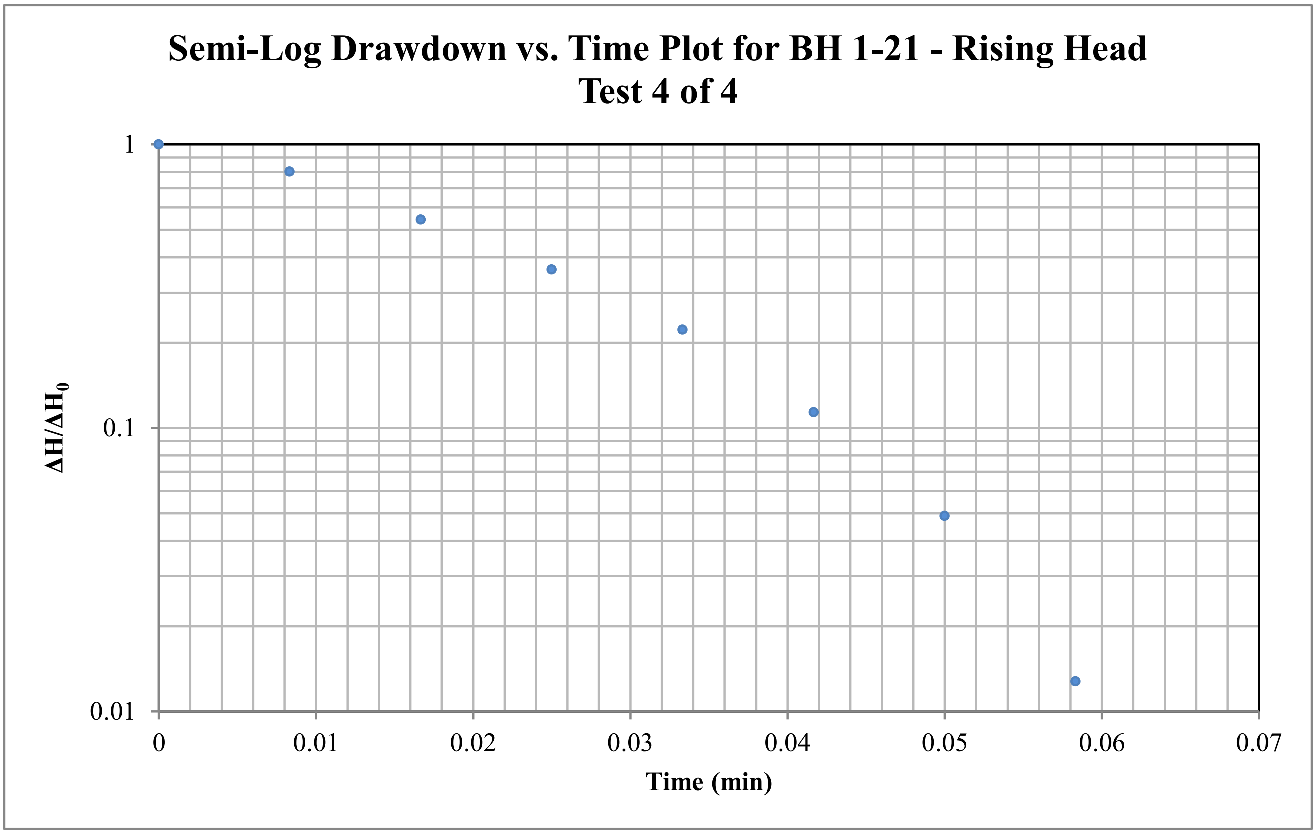
Data Points (from plot):

t*: 0.030 minutes $\Delta H*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.18E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: KRP Properties - 535 Legget Drive
Test Location: BH 1-21
Test: 4 of 4 Rising Head
Date: March 3, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.64264

Well Parameters:

L	3.048 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

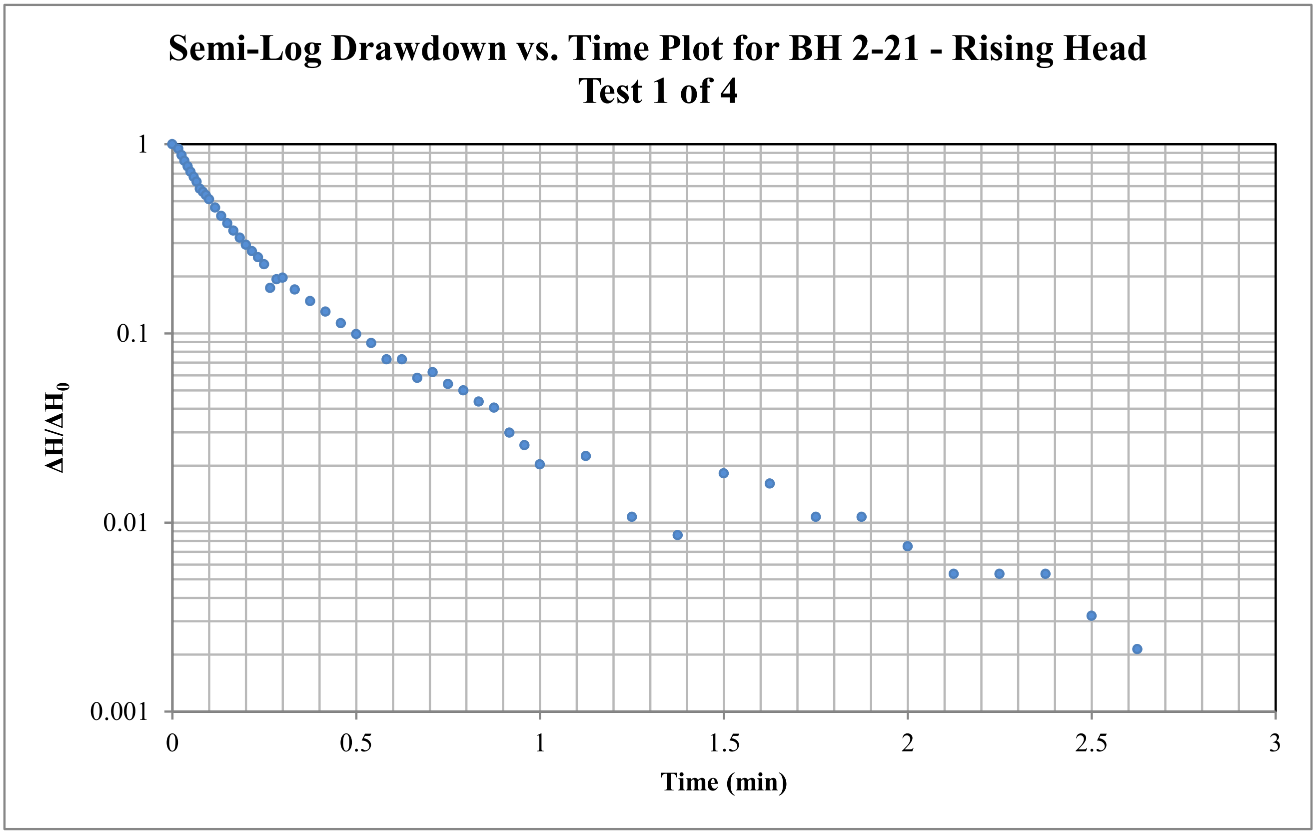
Data Points (from plot):

t*: 0.025 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.46E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: KRP Properties - 535 Legget Drive
Test Location: BH 2-21
Test: 1 of 4 Rising Head
Date: March 3, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.64264

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

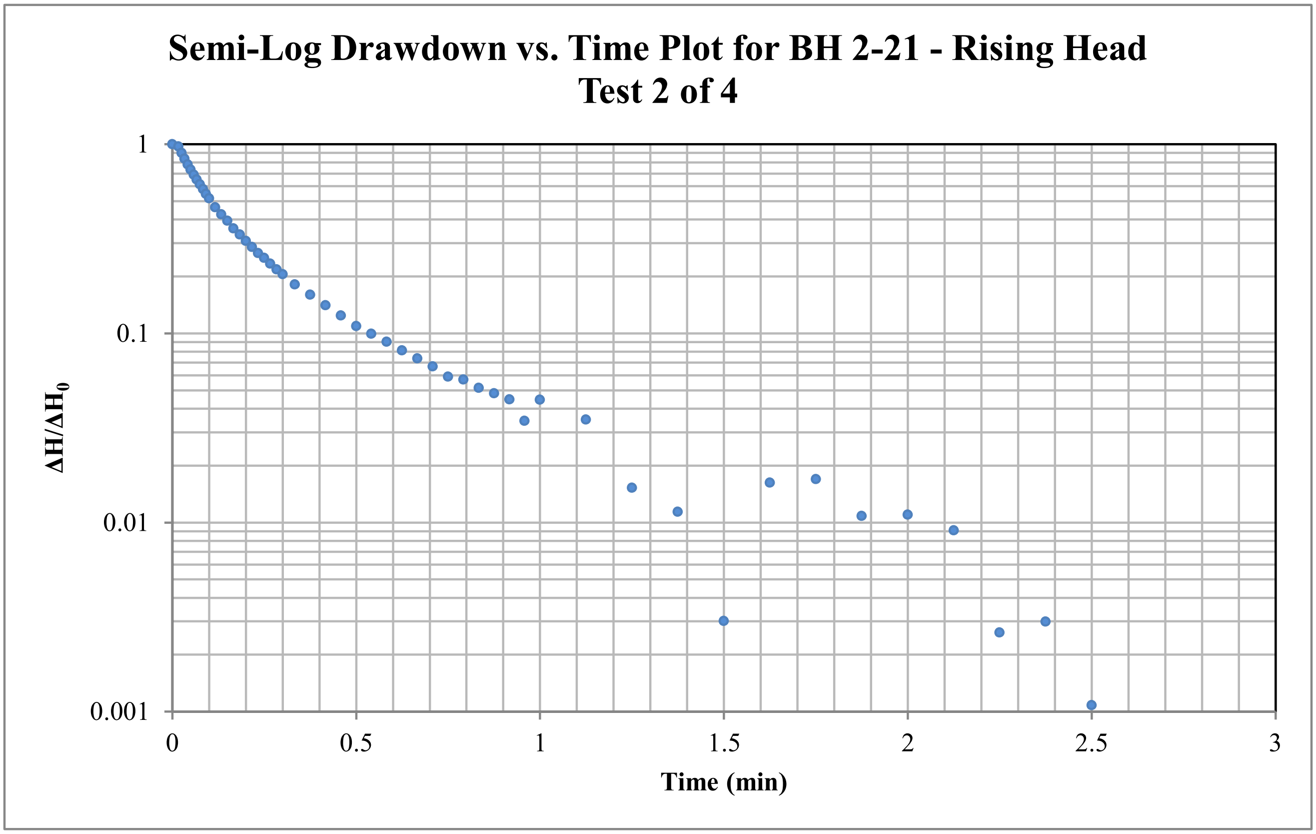
Data Points (from plot):

t*: 0.156 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.31E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: KRP Properties - 535 Legget Drive
Test Location: BH 2-21
Test: 2 of 4 Rising Head
Date: March 3, 2021



Semi-Log Drawdown vs. Time Plot for BH 2-21 - Rising Head Test 2 of 4

Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.64264

Well Parameters:

L	3.048 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

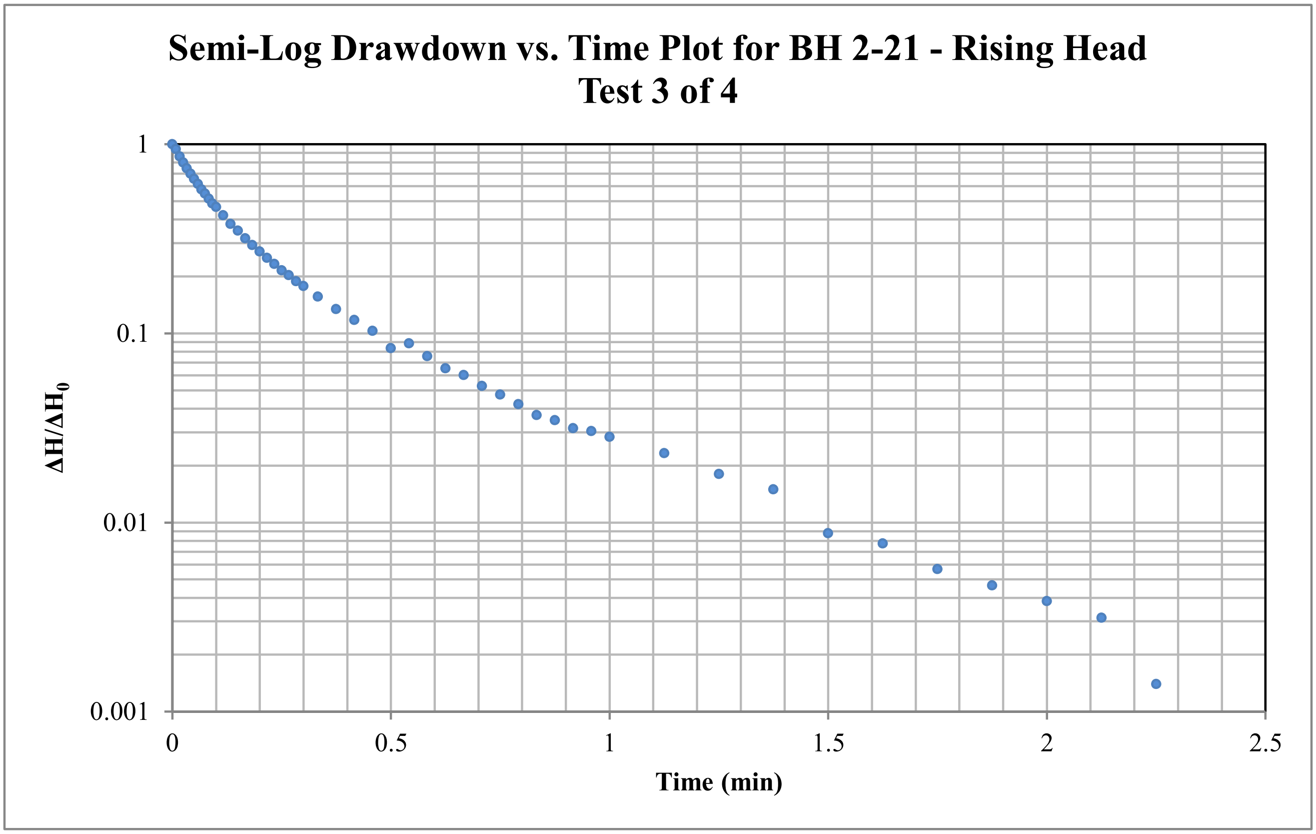
Data Points (from plot):

t^*: 0.153 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.35E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: KRP Properties - 535 Legget Drive
Test Location: BH 2-21
Test: 3 of 4 Rising Head
Date: March 3, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.64264

Well Parameters:

L	3.048 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

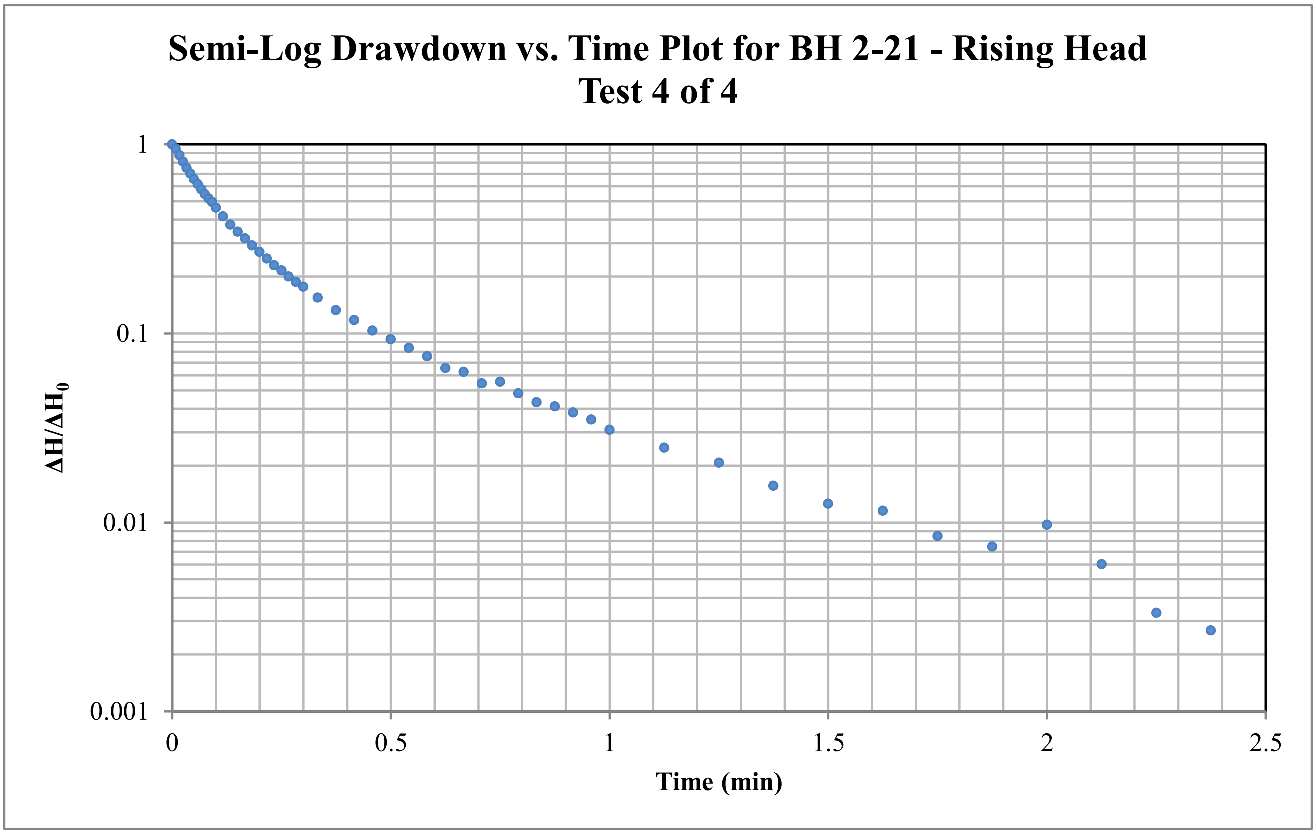
Data Points (from plot):

t*: 0.138 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.60E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: KRP Properties - 535 Legget Drive
Test Location: BH 2-21
Test: 4 of 4 Rising Head
Date: March 3, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.64264

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

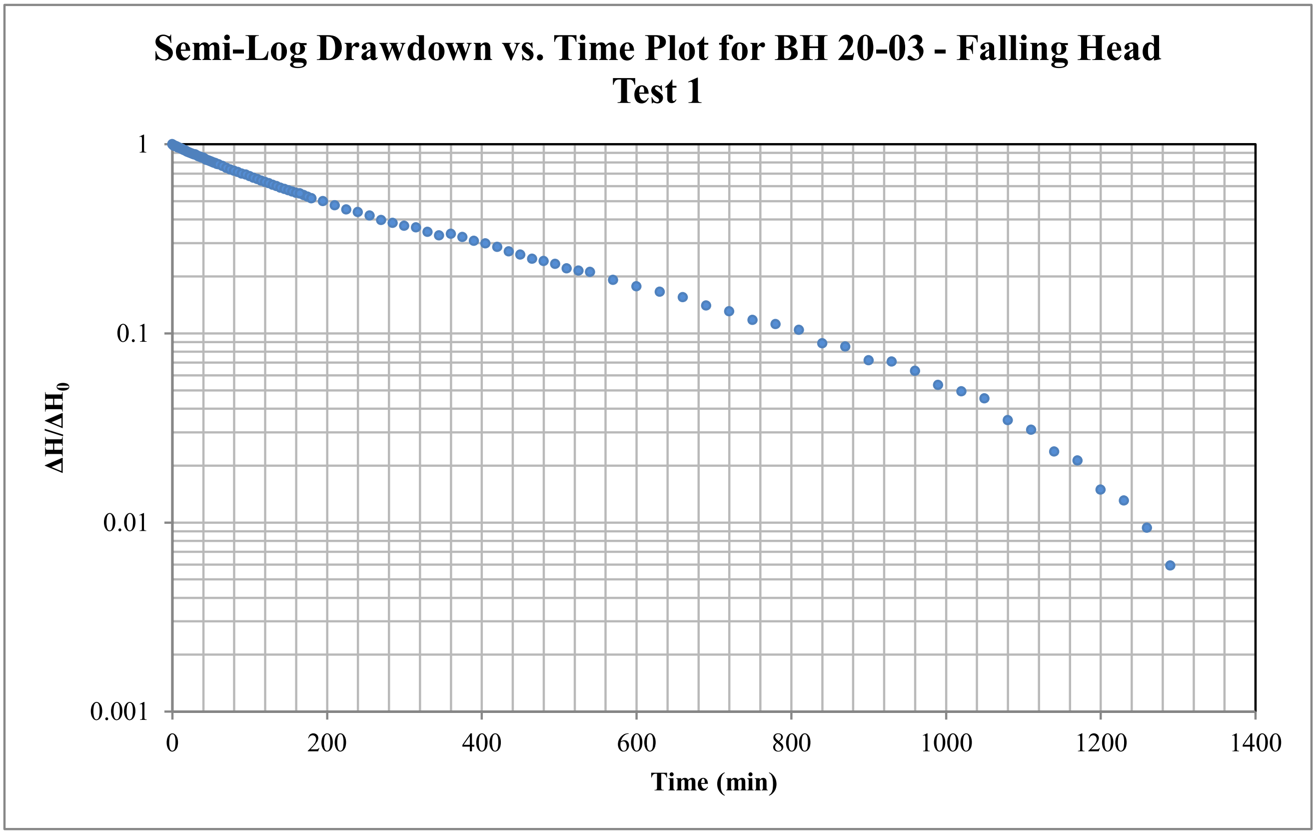
Data Points (from plot):

t*: 0.137 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.64E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: KRP Properties - 535 Legget Drive
Test Location: BH 20-03
Test: 1 of 1 Falling Head
Date: March 4, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.33873

Well Parameters:

L	1.524	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

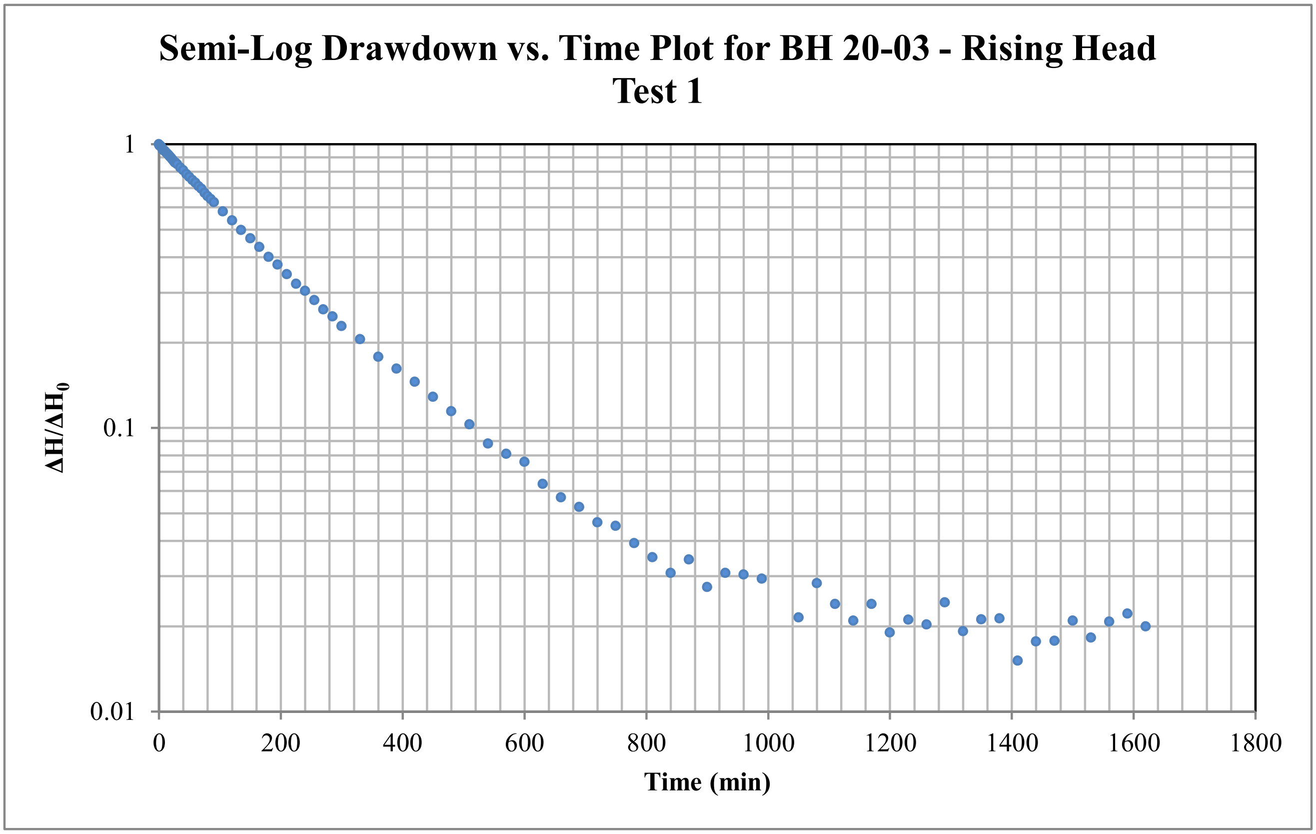
Data Points (from plot):

t*: 301.118 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 4.77E-08 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: KRP Properties - 535 Legget Drive
Test Location: BH 20-03
Test: 1 of 1 Rising Head
Date: March 5, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t*} \ln\left(\frac{\Delta H *}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.33873

Well Parameters:

L	1.524	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

Data Points (from plot):

t*: 199.102 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 7.21E-08 m/sec

Certificate of Analysis

Order # **G3817**

Client: **J.D. Paterson and Associates**

Client Ref: **5392** Project: **G8175**

Note - DL is the lowest detection limit normally attainable by the laboratory. Samples requiring dilution may be reported as being less than an elevated detection limit.

Sample ID: TP7-G7

Matrix: Soil

Paracel ID: G3817.1

Date Sampled: 05/18/01

Parameter	**units**	**DL**	**Result**
Chloride	ug/g	2.0	**38**
pH	pH units	0.050	**6.4**
Sulphate	ug/g	10	**240**
Resistivity	ohm.m	0.10	**26**

Certificate of Analysis

Order # **G3737**

Client: **J.D. Paterson and Associates**

Client Ref: **5402** Project: **G8175**

Note - DL is the lowest detection limit normally attainable by the laboratory. Samples requiring dilution may be reported as being less than an elevated detection limit.

Sample ID: BH31 SS4

Matrix: Soil

Paracel ID: G3737.1

Date Sampled: 05/09/01

Parameter	**units**	**DL**	**Result**
Chloride	ug/g	2.0	**62**
pH	pH units	0.050	**8.4**
Sulfate	ug/g	10	**80**
Resistivity	ohm.m	0.10	**20**

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5673-1 – TEST HOLE LOCATION PLAN

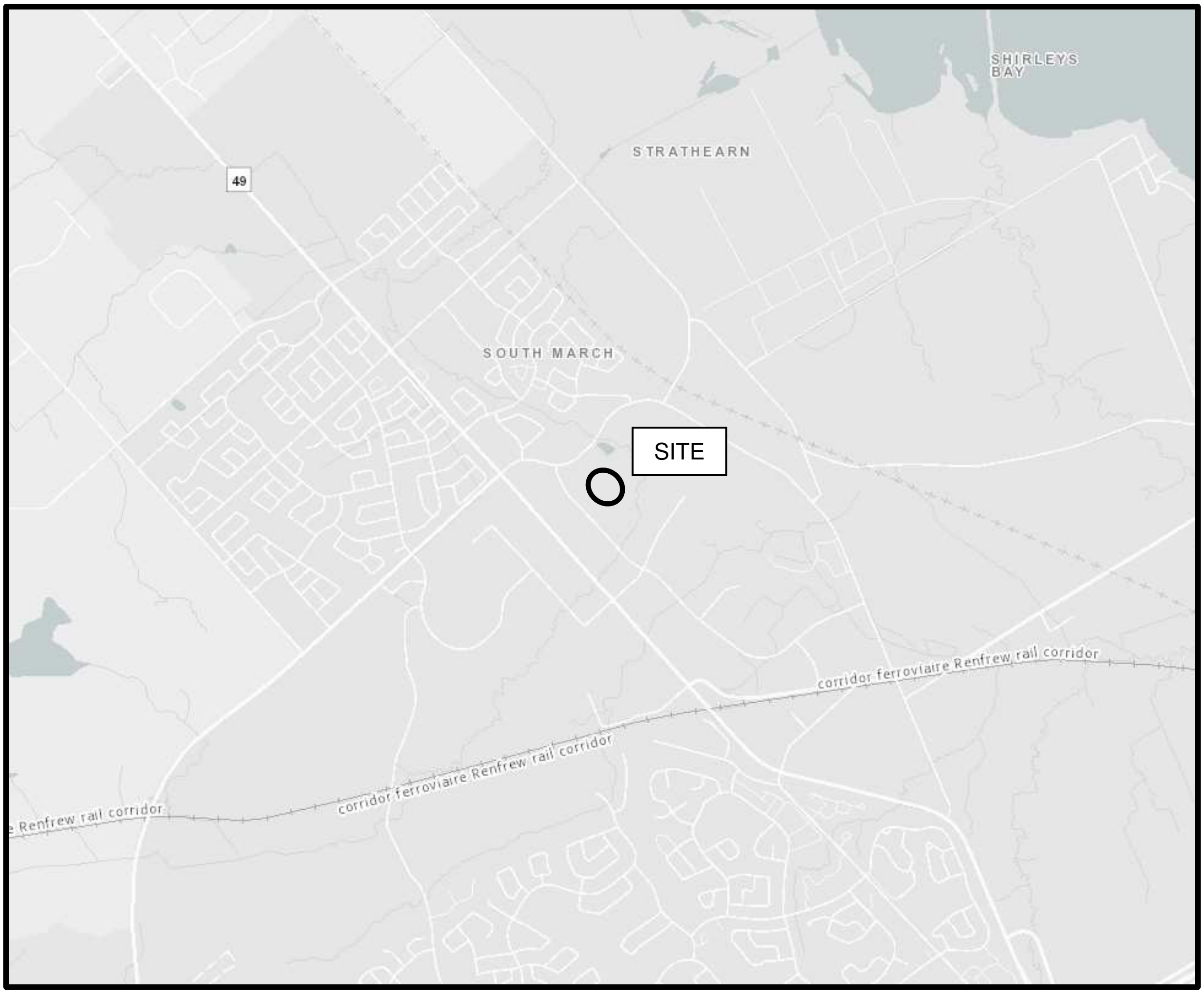


FIGURE 1

KEY PLAN

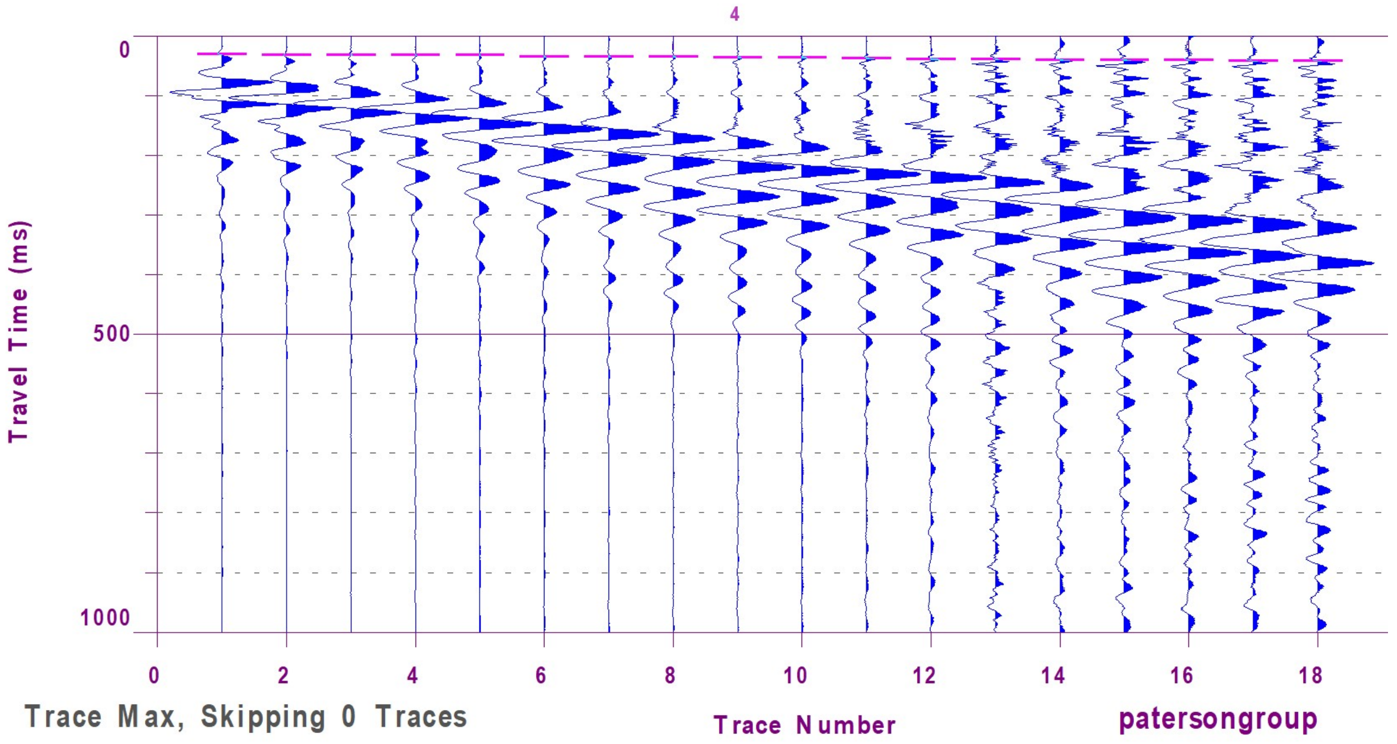


Figure 2 – Shear Wave Velocity Profile at Shot Location -15 m

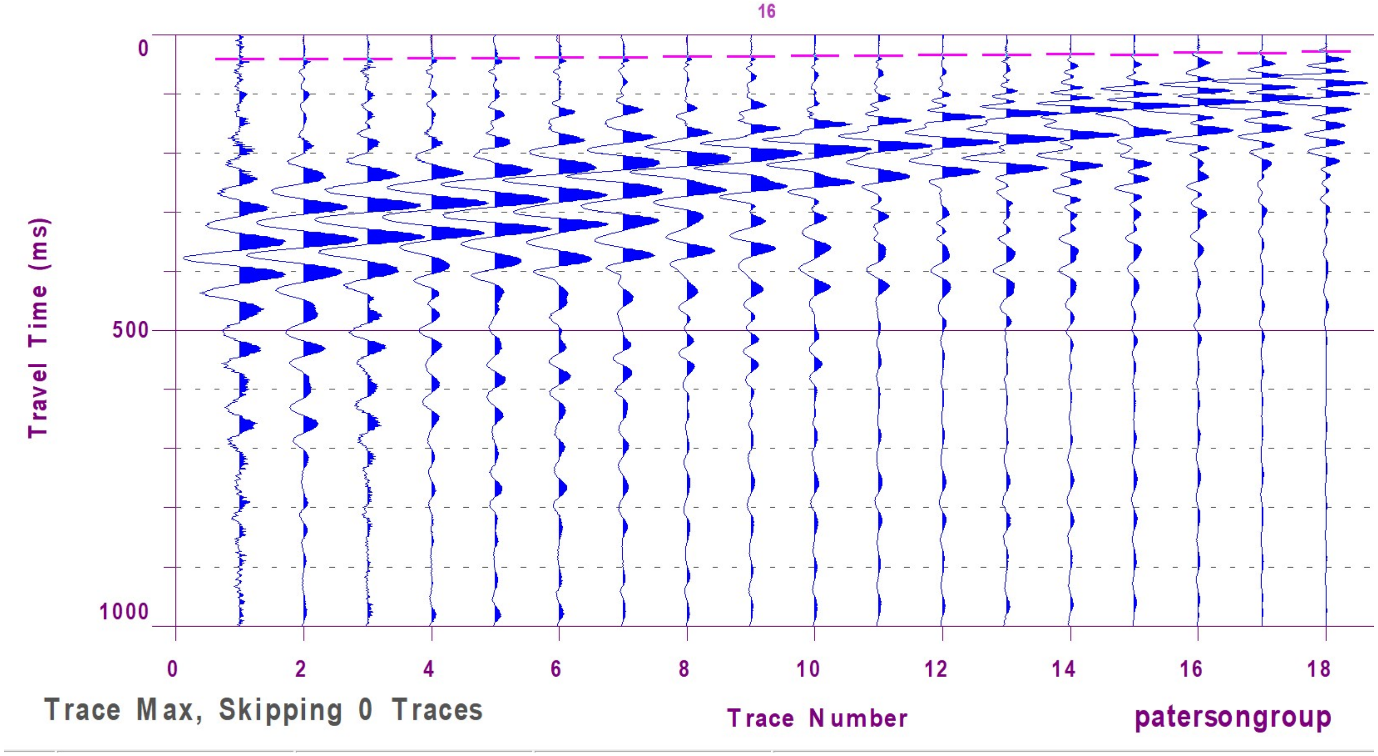
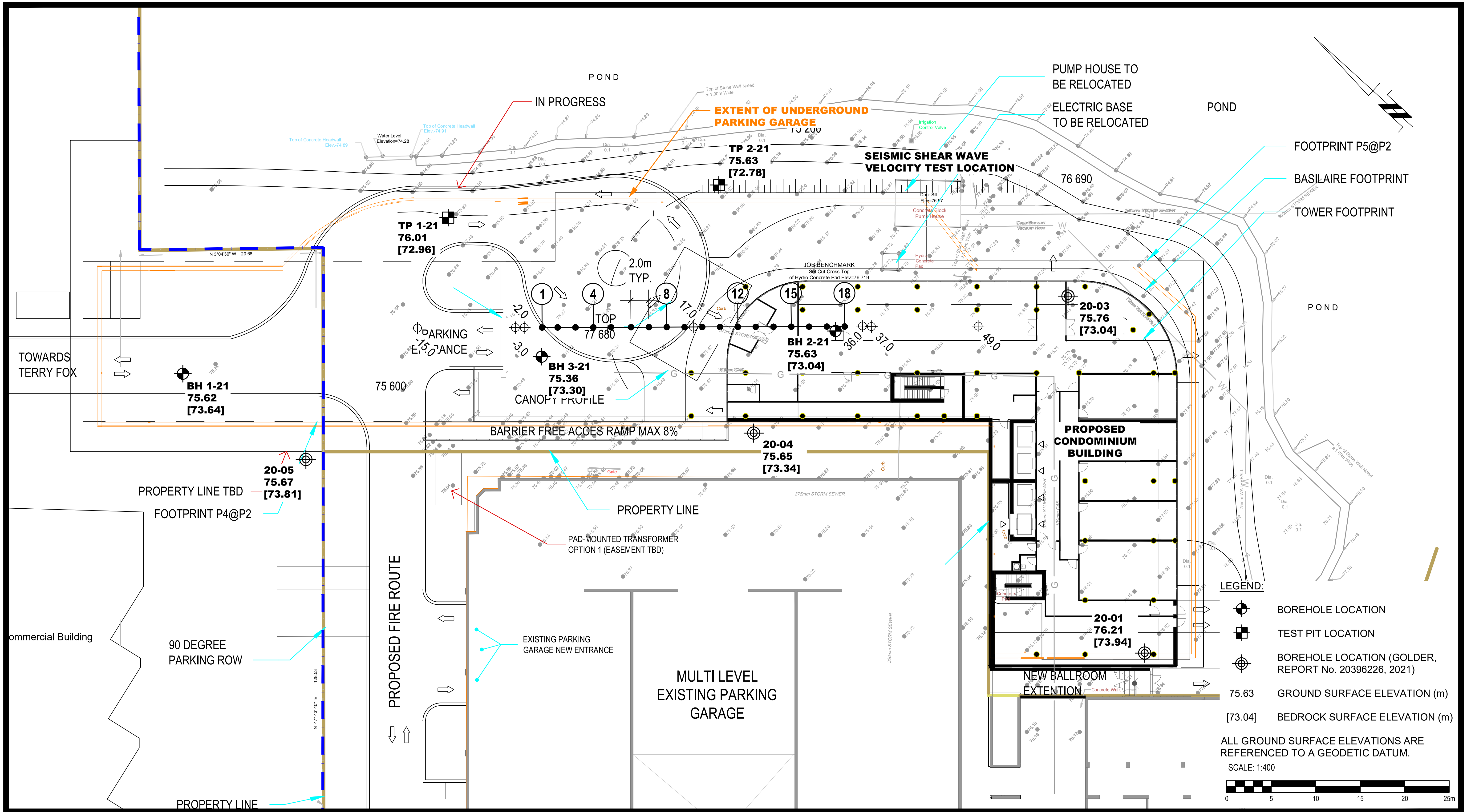


Figure 3 – Shear Wave Velocity Profile at Shot Location 49 m



LEGEND:

⊕	BOREHOLE LOCATION
▣	TEST PIT LOCATION
⊕	BOREHOLE LOCATION (GOLDER, REPORT No. 20396226, 2021)
75.63	GROUND SURFACE ELEVATION (m)
[73.04]	BEDROCK SURFACE ELEVATION (m)

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:400

patersongroup
consulting engineers

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Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
2	UPDATED TO LATEST CONCEPTUAL PLAN	01/31/2022	DJG
1	SHEAR WAVE VELOCITY TEST ADDED EXTENT OF UNDERGROUND PARKING UPDATED	08/06/2021	FC

KRP PROPERTIES

GEOTECHNICAL INVESTIGATION
PROPOSED BROOKSTREET CONDOMINIUM DEVELOPMENT
525 LEGGET DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400
Date: 03/2021
Drawn by: NFRV
Report No.: PG5673-1
Checked by: DP
Dwg. No.: PG5673-1
Approved by: DJG
Revision No.: 2

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