

Geotechnical
Engineering

Environmental
Engineering

Hydrogeology

Geological
Engineering

Materials Testing

Building Science

Noise and Vibration
Studies

Geotechnical Investigation

Proposed Building Addition
314 Bell Street South
Ottawa, Ontario

Prepared For

HD & P

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Report: PG5905-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by HD & P to conduct a geotechnical investigation for the proposed building addition to be located at 314 Bell Street South in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report). The objectives of the geotechnical investigation were to:

- ❑ Determine the existing subsoil and groundwater conditions at this site by means of boreholes.
- ❑ 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 proposed development as they are understood at the time of this report.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed building addition will be located along the southern side of the existing structure and consist of a 3-storey structure with 1 basement level. Asphalt-paved parking areas, walkways and landscaped areas are anticipated at finished grades surrounding the proposed building addition.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field investigation was carried out on August 4, 2021, and consisted of advancing a total of 3 boreholes to a maximum depth of 8.3 m. The borehole locations 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 PG5905-1 - Test Hole Location Plan in Appendix 2.

The boreholes were advanced using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The drilling procedure consisted of augering and bedrock coring to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using a 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery 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.

Bedrock samples were recovered at all boreholes using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

The subsurface conditions observed in the test holes 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

Groundwater monitoring wells were installed in all boreholes 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. 902 by a qualified licensed well technician and prior to construction.

Sample Storage

All samples from the current investigation will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The borehole locations and ground surface elevations at each borehole location were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The location of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG5905-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

4.0 Observations

4.1 Surface Conditions

The majority of the subject site is occupied by an existing residential building. A gravel-surfaced access lane and parking area are located at the southern and western limits of the site, respectively.

The subject site is bordered to the north and south by existing residential buildings, to the east by Bell Street South, and to the west by an unnamed access lane. The ground surface across the subject site slopes gently downward from Bell Street South to the west end of the site from approximate geodetic elevation 73.5 to 71.5 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of an approximate 0.9 to 2.4 m thick fill layer underlain by bedrock. The fill material was generally observed to consist of silty sand with crushed stone and trace amounts of topsoil, gravel, cobbles and boulders.

Bedrock

Practical refusal to augering on the bedrock surface was encountered at approximate depths ranging from 0.9 to 2.4 m. The bedrock was observed to consist of grey limestone and was generally of fair to excellent quality based on the RQDs of the bedrock core. At borehole BH 3-21, the bedrock was observed to be weathered and of poor quality to an approximate depth of 1.6 m, becoming fair to good in quality with depth. At boreholes BH 1-21 to BH 3-21, the bedrock was cored to approximate depths ranging from 6.7 to 8.3 m below the existing ground surface.

Based on available geological mapping, bedrock in the area of the subject site consists of interbedded limestone and shale of the Verulam Formation.

4.3 Groundwater

Groundwater level readings were measured in the monitoring wells on August 12, 2021. The measured groundwater level (GWL) readings are presented in Table 1 on the following page.

Table 1 - Summary of Groundwater Level Readings				
Borehole Number	Ground Surface Elevation (m)	Groundwater Levels (m)	Groundwater Elevation (m)	Recording Date
BH 1-21	72.13	2.82	69.31	August 16, 2021
BH 2-21	73.37	2.57	70.80	
BH 3-21	71.72	3.16	68.56	
Note: Ground surface elevations at test hole locations are referenced to a geodetic datum.				

It should be noted that groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour, moisture content and consistency of the recovered soil samples.

Based on these observations, the long-term groundwater level is expected between an approximate 2.5 to 3.5 m depth. However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed building addition. It is recommended that the proposed building addition be founded on conventional spread footings placed on clean, surface sounded bedrock.

Where the footings of the proposed building addition abut the existing building, they should match the existing footing elevations.

Bedrock removal will be required to complete the basement level. Hoe ramming is an option where the bedrock is weathered and/or where only small quantities of bedrock need to be removed. Line drilling and controlled blasting is recommended where large quantities of bedrock need to be removed. 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

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Due to the relatively shallow depth of the bedrock surface and the anticipated founding level for the proposed building, all existing overburden material should be excavated from within the proposed building addition footprint.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be completed prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be 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.

Vibration Considerations

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

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, 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). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This 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 and paved areas 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 can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in lifts and at least 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 use as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane.

5.3 Foundation Design

Footings placed directly on clean, surface sounded limestone bedrock can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance at ULS.

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.

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.

As a general procedure, it is recommended that the footings for the proposed addition that are located adjacent to the existing structure be founded at the same level as the existing footings. This accomplishes three objectives. First, the behaviour of the two structures at their connection will be similar due to the similar bearing medium. Second, there will be minimal stress added to the existing structure from the new structure. Third, the bearing of the new structure will not be influenced by any backfill from the existing structure.

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 a sound bedrock bearing medium, when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. If a higher seismic site class is required (Class A or B), a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

For the proposed building addition, all overburden soil will be removed from the building footprint, leaving the bedrock as the founding medium for the basement floor slab.

It is recommended that the upper 200 mm of sub-floor fill consist of 19 mm clear crushed stone. Further, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided underlying the basement slab. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m³.

Where undrained conditions are anticipated (i.e below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken 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.

Static 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 $K_o \cdot q$ 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:

- $a_c = (1.45 - a_{max}/g)a_{max}$
- 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 Pavement Design

The pavement structures for car only parking areas, heavy truck parking areas and access lanes are presented in Tables 2 and 3, should they be required at the subject site.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas	
Thickness (mm)	Material Description
50	Wear Course - Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
300	SUBBASE - OPSS Granular B Type II
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil.	

Table 3 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas	
Thickness (mm)	Material Description
40	Wear Course - Superpave 12.5 Asphaltic Concrete
50	Binder Course - Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
300	SUBBASE - OPSS Granular B Type II
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil.	

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 99% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 150 mm diameter, perforated and corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipes should have a positive outlet, such as a gravity connection to the storm sewer.

Where insufficient room is available for exterior backfill, it is suggested that a composite drainage system (such as Delta Drain 6000 or equivalent) be secured against the temporary shoring system or vertical bedrock face, and extending to a series of drainage sleeve inlets through the building foundation wall at the footing/foundation wall interface. The drainage sleeves should be at least 150 mm diameter and be spaced 3 m along the perimeter foundation walls. An interior perimeter drainage pipe should be placed along the building perimeter along with the sub-slab drainage system. The perimeter drainage pipe and sub-slab drainage system should direct water to sump pit(s) within the underground level.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration below the basement slab. For preliminary design purposes, we recommend that 150 mm perforated pipes be placed at approximate 6 m centres underlying the basement floor slab. The spacing of the sub-slab drainage system should be confirmed by the geotechnical consultant at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where sufficient space is available for conventional backfilling, the backfill material against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as Delta Drain 6000) connected to a drainage system is provided.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation should be provided in this regard.

Exterior unheated footings, such as isolated piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m, or an equivalent combination of soil cover and foundation insulation.

However, foundations which are founded directly on clean, surface sounded bedrock and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by temporary shoring systems from the start of the excavation until the structure is backfilled. Based on the subsurface conditions encountered and the proposed building setback from the property lines, it is anticipated that sufficient space will be available to slope the excavation.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be 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 be kept away 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.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

In sound bedrock, almost vertical side slopes can be constructed, provided all weathered and loose rock is removed or stabilized with rock anchors. A minimum 1 m horizontal ledge should remain between the unsupported excavation and sound bedrock surface. The ledge will provide an area to allow for potential sloughing. Where sufficient space for the horizontal ledge is not available, it is recommended that concrete blocks be used to retain the overburden soils.

Where the vertical sides are constructed in sound bedrock, bedrock stabilization may be required. Specifically, horizontal anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The requirement for horizontal rock anchors should be evaluated by Paterson during the excavation operations and should be discussed with the structural engineer during the design stage.

Underpinning of Adjacent Structures

Due to the shallow depth to bedrock, it is expected that the existing building to the south of the subject site is founded on bedrock. Therefore, underpinning is not expected to be required at this site. However, an assessment should be completed by the geotechnical engineer at the time of excavation to confirm founding conditions of the existing building adjacent to the proposed building addition, in order to evaluate rock bolt locations and specific rock bolt details, should they be required.

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.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be 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.

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Properties

Based on the subsurface conditions encountered at the subject site, it is anticipated that the adjacent structures are founded on bedrock. Therefore, no adverse effects from short term and long term dewatering are expected for surrounding structures. The short term dewatering during the excavation program will be managed by the excavation contractor.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, 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 also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

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, if required, prior to construction.
- 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 HD & P 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.



Kevin A. Pickard, EIT



Scott S. Dennis, P.Eng.

Report Distribution:

- HD & P (Digital copy)
- Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS SYMBOLS AND TERMS

DATUM Geodetic

FILE NO. **PG5905**

REMARKS

HOLE NO. **BH 1-21**

BORINGS BY CME-55 Low Clearance Drill

DATE August 4, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	72.13						
FILL: Brown silty sand with crushed stone	0.40	AU	1										
FILL: Brown silty sand with gravel, trace clay, occasional cobbles and boulders	1.52	SS	2	50		1	71.13						
BDRCoK: Fair to good quality, grey limestone		RC	1	100	50	2	70.13						
		RC	2	100	70	4	68.13						
		RC	3	100	78	5	67.13						
		RC	4	100	83	6	66.13						
End of Borehole (GWL @ 2.82m - August 16, 2021)	6.70												

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 4, 2021

FILE NO. **PG5905**

HOLE NO. **BH 2-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80	
GROUND SURFACE												
FILL: Crushed stone with sand	0.40	AU	1			0	73.37					
FILL: Brown silty sand, trace clay, gravel and topsoil, occasional cobbles and boulders		SS	2	75	13	1	72.37					
		SS	3	56	50+	2	71.37					
		SS	4	50	50+	3	70.37					
	2.39											
BEDROCK: Excellent to good quality, grey limestone		RC	1	100	89	4	69.37					
		RC	2	100	97	5	68.37					
		RC	3	100	88	6	67.37					
	6.73											
- fair quality by 6.1m depth												
End of Borehole (GWL @ 2.57m - August 16, 2021)												

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic

FILE NO. **PG5905**

REMARKS

HOLE NO. **BH 3-21**

BORINGS BY CME-55 Low Clearance Drill

DATE August 4, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %					
GROUND SURFACE								20	40	60	80		
FILL: Crushed stone with sand	0.40					0	71.72						
FILL: Brown silty sand with gravel, trace clay, occasional cobbles and boulders	0.94	AU	1										
		SS	2	67	50+	1	70.72						
		RC	1	81	23								
		RC	2	100	50	2	69.72						
		RC	3	100	90	3	68.72						
		RC	4	100	81	4	67.72						
BEDROCK: Poor to good quality, grey limestone interbedded with black shale						5	66.72						
		RC	5	100	64	6	65.72						
		RC	6	100	88	7	64.72						
						8	63.72						
End of Borehole	8.33												
(GWL @ 3.16m - August 16, 2021)													
								20	40	60	80	100	
								Shear Strength (kPa)					
								▲ Undisturbed △ Remoulded					

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)
D _{xx}	-	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
D ₁₀	-	Grain size at which 10% of the soil is finer (effective grain size)
D ₆₀	-	Grain size at which 60% of the soil is finer
C _c	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
C _u	-	Uniformity coefficient = D_{60} / D_{10}

C_c and C_u are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < C_c < 3$ and $C_u > 4$

Well-graded sands have: $1 < C_c < 3$ and $C_u > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

C_c and C_u 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
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
W _o	-	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.
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SYMBOLS AND TERMS (continued)

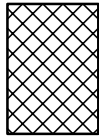
STRATA PLOT



Topsoil



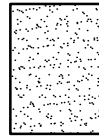
Asphalt



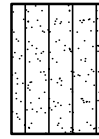
Fill



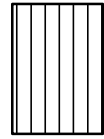
Peat



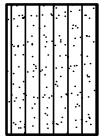
Sand



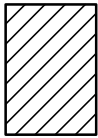
Silty Sand



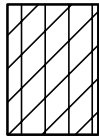
Silt



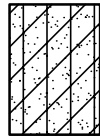
Sandy Silt



Clay



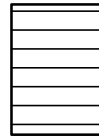
Silty Clay



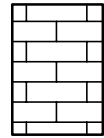
Clayey Silty Sand



Glacial Till



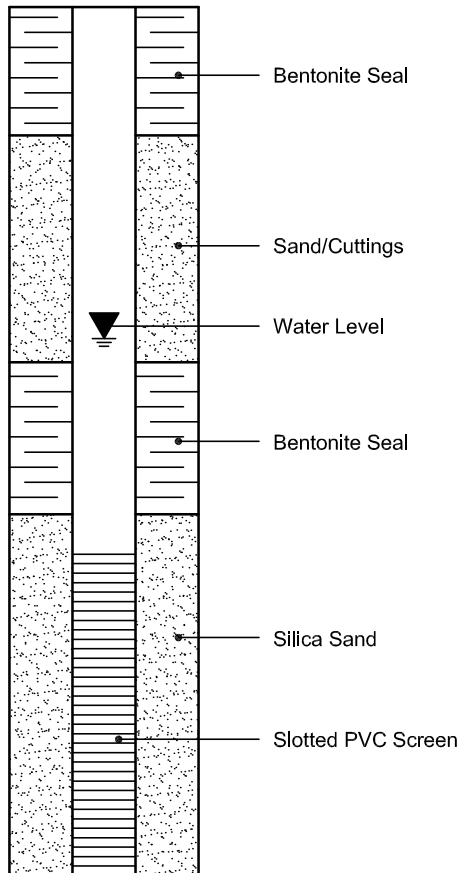
Shale



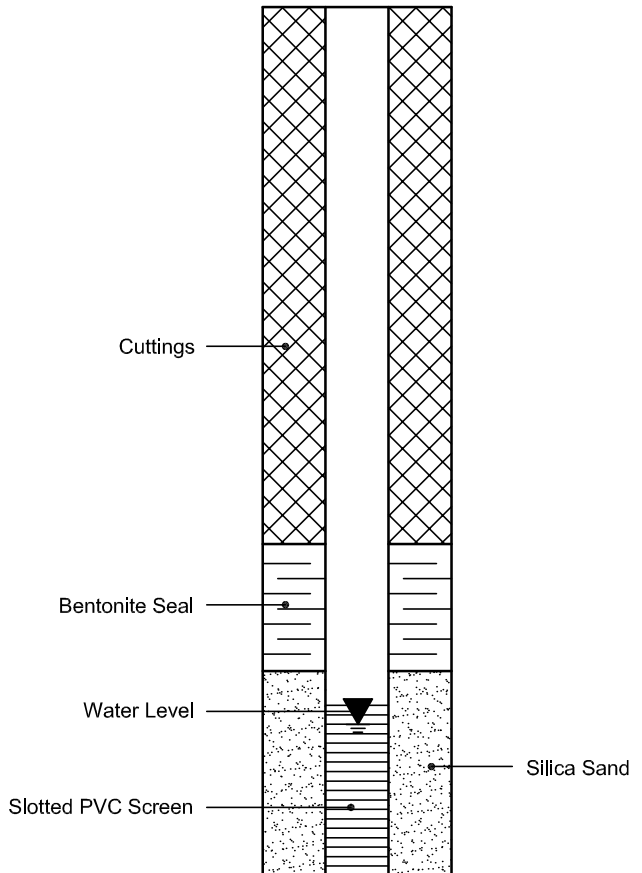
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5905-1 - TEST HOLE LOCATION PLAN

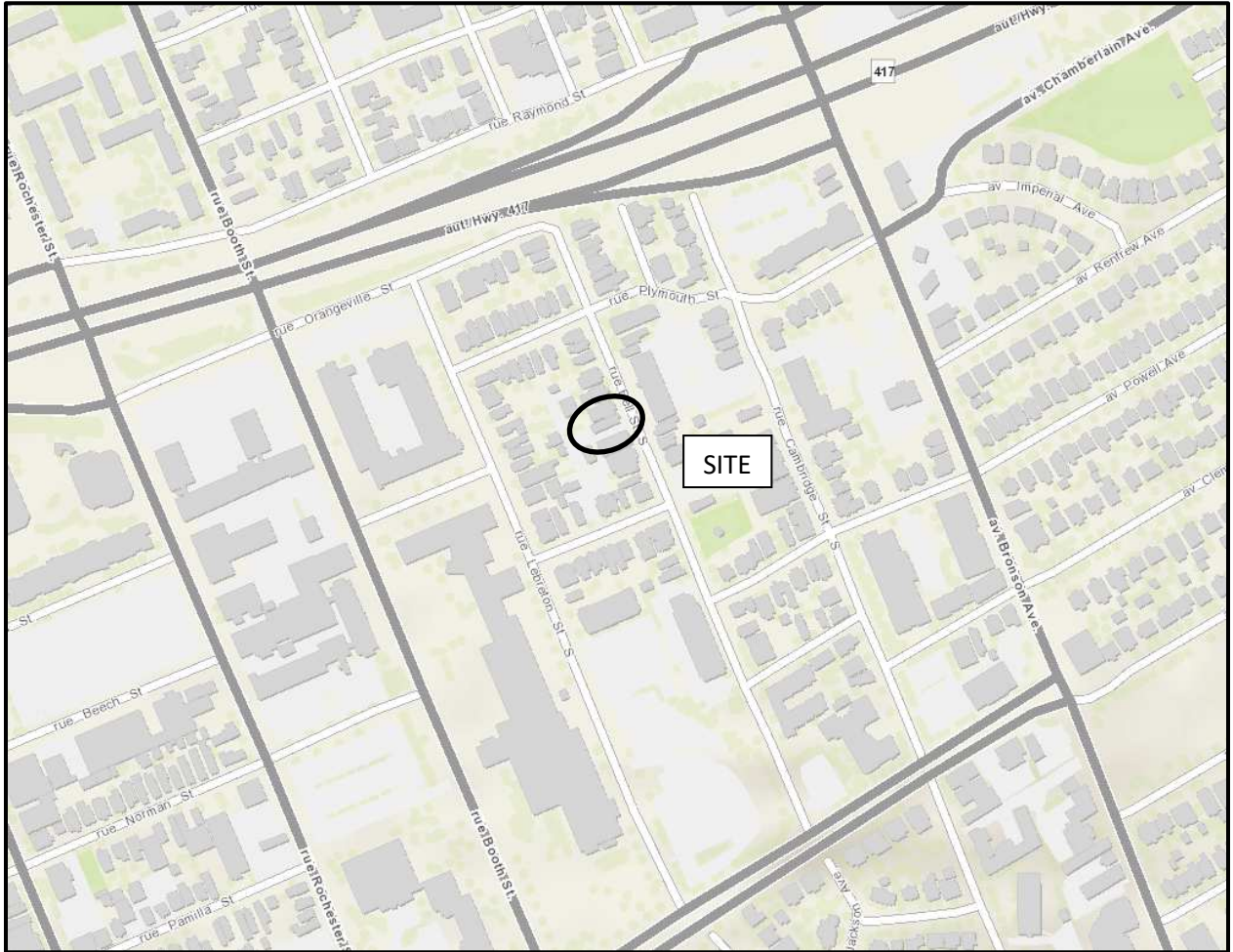
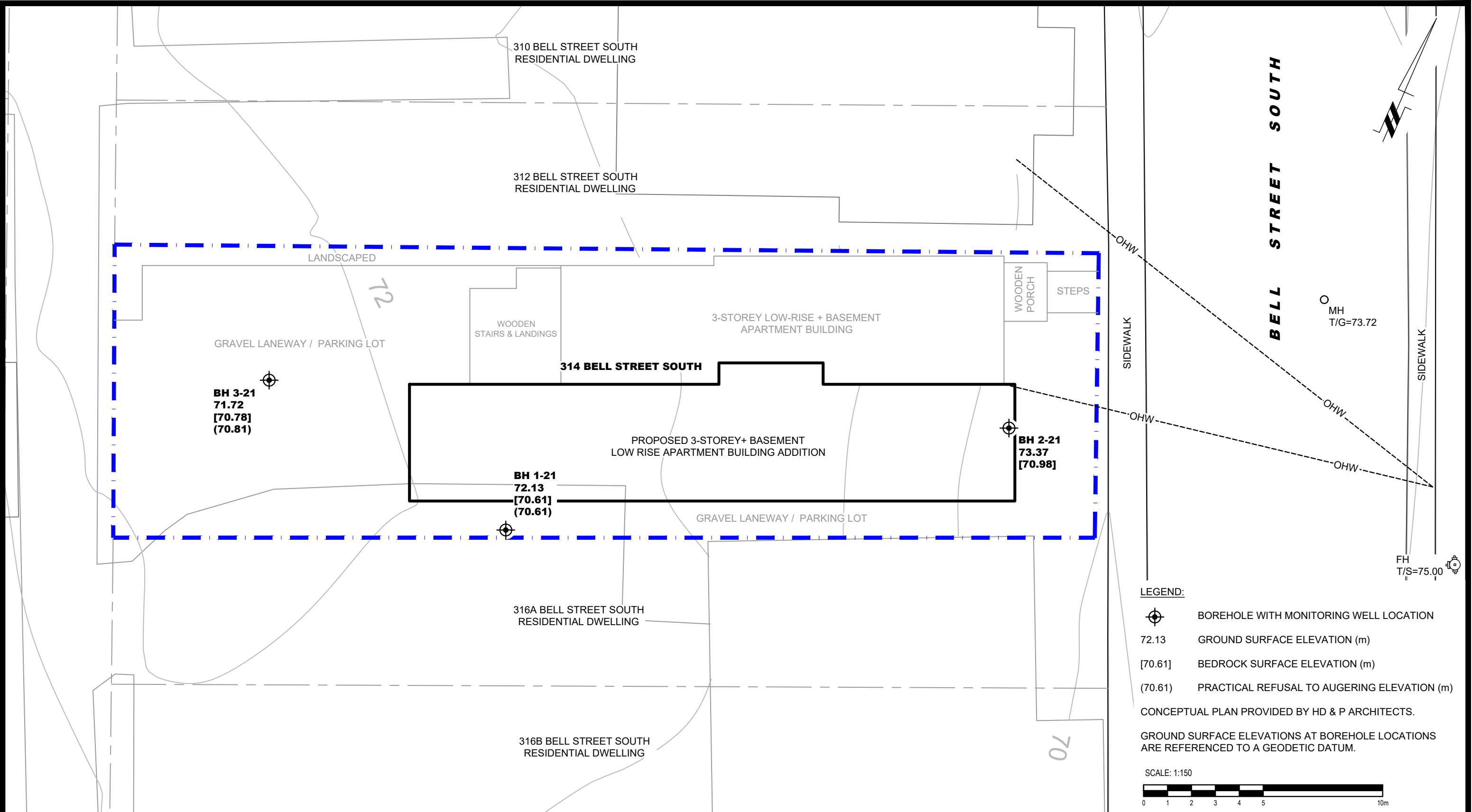


FIGURE 1

KEY PLAN



- LEGEND:**
- BOREHOLE WITH MONITORING WELL LOCATION
 - 72.13 GROUND SURFACE ELEVATION (m)
 - [70.61] BEDROCK SURFACE ELEVATION (m)
 - (70.61) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY HD & P ARCHITECTS.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:150

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NO.	REVISIONS	DATE	INITIAL

HD & P
**GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING ADDITION
314 BELL STREET SOUTH**

TEST HOLE LOCATION PLAN

OTTAWA, ONTARIO

Scale:	1:150	Date:	08/2021
Drawn by:	JM	Report No.:	PG5905-1
Checked by:	KP	Dwg. No.:	PG5905-1
Approved by:	SD	Revision No.:	