



FINAL

Geotechnical Investigation and Visual Slope Inspection– Proposed Residential Development

6 John Street, Picton, Ontario

Prepared for:

JAMR Developments

42 Redwood Drive
Belleville, ON K8N 0K4

December 11, 2023

Pinchin File: 327548



Geotechnical Investigation and Visual Slope Inspection– Proposed Residential Development

6 John Street, Picton, Ontario
JAMR Developments

December 11, 2023
Pinchin File: 327548
FINAL

Issued to: JAMR Developments
Issued on: December 11, 2023
Pinchin File: 327548
Issuing Office: Kanata, ON

Author:

Megan Keon, EIT.
Project Manager, Geotechnical Services
613.592.3387
mkeon@pinchin.com

Reviewer:

Jeff Dietz, P.Eng.
Senior Technical Manager, Geotechnical Services
519.589.3768
jdietz@pinchin.com



TABLE OF CONTENTS

1.0	INTRODUCTION AND SCOPE	1
2.0	SITE DESCRIPTION AND GEOLOGICAL SETTING	2
3.0	GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY	2
4.0	SUBSURFACE CONDITIONS	3
4.1	Borehole Soil Stratigraphy	3
4.2	Bedrock	4
4.3	Groundwater Conditions	5
5.0	GEOTECHNICAL DESIGN RECOMMENDATIONS	5
5.1	General Information	5
5.2	Site Preparation	6
5.3	Open Cut Excavations and Dewatering	6
5.4	Site Servicing	8
5.4.1	<i>Pipe Bedding and Cover Materials for Flexible and Rigid Pipes</i>	8
5.4.2	<i>Trench Backfill</i>	9
5.4.3	<i>Frost Protection</i>	10
5.5	Foundation Design	10
5.5.1	<i>Shallow Foundations Bearing on Glacial Till</i>	10
5.5.2	<i>Shallow Foundations Bearing on Bedrock</i>	12
5.5.3	<i>Site Classification for Seismic Site Response & Soil Behaviour</i>	12
5.5.4	<i>Foundation Transition Zones</i>	13
5.5.5	<i>Estimated Settlement</i>	14
5.5.6	<i>Building Drainage</i>	14
5.5.7	<i>Shallow Foundations Frost Protection & Foundation Backfill</i>	14
5.6	Floor Slabs	15
5.7	Asphaltic Concrete Pavement Structure Design for Parking Lot and Driveways	16
5.7.1	<i>Discussion</i>	16
5.7.2	<i>Pavement Structure</i>	16
5.7.3	<i>Pavement Structure Subgrade Preparation and Granular up Fill</i>	17
6.0	SLOPE STABILITY ASSESSMENT	17
6.1	Visual Slope Inspection and Findings	18
6.2	Photos of Slope	18
7.0	SITE SUPERVISION & QUALITY CONTROL	19
8.0	TERMS AND LIMITATIONS	20



FIGURES

Figure 1 – Key Map

Figure 2 – Borehole Location Plan

APPENDICES

APPENDIX I	Abbreviations, Terminology and Principle Symbols used in Report and Borehole Logs
APPENDIX II	Pinchin’s Borehole Logs
APPENDIX III	Laboratory Testing Reports for Soil Samples
APPENDIX IV	Report Limitations and Guidelines for Use



1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by JAMR Developments (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed residential development to be located at 6 John Street, Picton, Ontario (Site). The Site location is shown on Figure 1.

Based on information provided by the Client, it is Pinchin's understanding that the development will consist of a two slab-on-grade (i.e. no basement level) townhouse blocks containing four units each. The proposed development will also include new Site services and asphalt surfaced driveways.

Pinchin's geotechnical comments and recommendations are based on the results of the Geotechnical Investigation and our understanding of the project scope.

The purpose of the Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of six (6) sampled boreholes (Boreholes BH1 to BH6), at the Site.

Based on a desk top review and the results of the Geotechnical Investigation, the following geotechnical data and engineering design recommendations are provided herein:

- A detailed description of the soil and groundwater conditions;
- Site preparation recommendations;
- Open cut excavations;
- Anticipated groundwater management;
- Site service trench design;
- Foundation design recommendations including soil bearing resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) design;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;
- Seismic Site classification for seismic Site response;
- Concrete floor slab-on-grade support recommendations;
- Asphaltic concrete pavement structure design for parking areas and access roadways;
- Visual assessment of the slope; and
- Potential construction concerns.

Abbreviations terminology and principle symbols commonly used throughout the report, borehole logs and appendices are enclosed in Appendix I.



2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on the northeast corner of the intersection of John Street and Union Street in Picton, Ontario. The Site is currently developed with one two-storey residential dwelling near the centre of the Site. The Client has asked Pinchin to analyze two slopes on Site by conducting visual inspections. The main slope generally runs from the top of slope on the east property line to the toe of the slope on the west property line or John Street. The elevation is approximately 4.4 m and it covers approximately 35 m horizontally. The other slope that required visual inspection starts at the top of the same slope but extends down to the toe of the slope along Union Street. The elevation between the two is approximately 2.35 m and extends approximately 2.6 m horizontally. The lands adjacent to the Site are developed with single-family residential dwellings and are not as affected by the slopes as they have either been constructed on top of the slope or at the base of the slope.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is located on Paleozoic bedrock (Ontario Geological Survey 2010. Surficial geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 128-REV). The underlying bedrock at this Site is of the Shadow Lake Formation consisting of limestone, dolostone, shale, arkose, and sandstone (Ontario Geological Survey 2011. 1:250 000 scale bedrock geology of Ontario; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site on October 19, 2023 by advancing a total of six (6) sampled boreholes (Boreholes BH1 through BH6) throughout the Site. The boreholes were advanced to termination on probable bedrock at depths of approximately 1.1 to 2.8 metres below existing ground surface (mbgs). The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a CME55 drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.75 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) "N" values (ASTM D1586). The SPT "N" values were used to assess the compactness condition of the non-cohesive soil.

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. The groundwater observations and measurements recorded are included on the appended borehole logs.



The boreholes locations and ground surface elevations were located at the Site relative to a local benchmark by Pinchin personnel. After completing the survey, Pinchin provided the Client with preliminary depths to bedrock and local ground elevations. InSite Project Consulting Inc. provided the following geodetic Temporary Benchmark (TBM) information to Pinchin in an email on November 13, 2023. The TBM is the same location Pinchin had previously used as a local benchmark:

- TBM: Top of nut on fire hydrant, at the approximate location shown on Figure 2; and
- Elevation: 105.29 m (Geodetic Elevation).

The field investigation was monitored by experienced Pinchin personnel. Pinchin logged the drilling operations and identified the soil samples as they were retrieved. The recovered soil samples were sealed into plastic bags and carefully transported to an independent and accredited materials testing laboratory for detailed analysis and testing. All soil samples were classified according to visual and index properties by the project engineer.

The field logging of the soil and groundwater conditions was performed to collect geotechnical engineering design information. The borehole logs include textural descriptions of the subsoil in accordance with a modified Unified Soil Classification System (USCS) and indicate the soil boundaries inferred from non-continuous sampling and observations made during the borehole advancement. These boundaries reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The modified USCS classification is explained in further detail in Appendix I. Details of the soil and groundwater conditions encountered within the boreholes are included on the Borehole Logs within Appendix II.

Select soil samples collected from the boreholes were submitted to a material testing laboratory to determine the grain size distribution of the soil. A copy of the laboratory analytical reports is included in Appendix III. In addition, the collected samples were compared against previous geotechnical information from the area, for consistency and calibration of results.

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises surficial organics and surficial granular fill overlying glacial till and bedrock to the maximum borehole termination depth of approximately 2.8 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT testing, and groundwater measurements.



Surficial organics was encountered in Boreholes BH2, BH3, BH4 and BH6 and ranged in thickness between 50 and 100 mm. Granular fill was encountered in Boreholes BH1 and BH5 in the boreholes advanced through the existing driveway on Site. The granular fill was 25 to 300 mm thick.

Glacial till was encountered underlying the surficial organics and granular fill within all boreholes and extended to the underlying probable bedrock surface encountered at depths ranging between 1.1 and 2.8 mbgs. The glacial till comprised silty sand containing some gravel and some clay or sand and gravel containing some silt. The non-cohesive glacial till had a loose to very dense relative density based on SPT 'N' values of 10 to 84 blows per 300 mm penetration of a split spoon sampler. The loose portions of the glacial till were all within 0.5 m of ground surface. The results of two particle size distribution analyses completed on samples of the glacial till are provided in Appendix III and indicate that the samples contain 17 to 44% gravel, 37 to 39% sand, and 17 to 47% silt and clay. The moisture content in one of the samples tested was 10%, indicating that the soil was moist at the time of sampling. The glacial till was described as damp during the fieldwork.

4.2 Bedrock

Bedrock was inferred due to auger refusal within all of the boreholes at depths ranging between 1.1 and 2.8 mbgs. The following table outlines the boreholes advanced during this field investigation and the associated geodetic ground and probable bedrock elevations:

Borehole #	Coordinates	Geodetic Ground Surface Elevation (m)	Geodetic Probable Bedrock Surface Elevation (m)
BH1	44.00739°N, 77.12838°W	104.41 m	102.12 m
BH2	44.00734°N, 77.12823°W	106.03 m	104.20 m
BH3	44.00735°N, 77.12806°W	108.81 m	107.24 m
BH4	44.00761°N, 77.12805°W	106.87 m	104.03 m
BH5	44.00759°N, 77.12824°W	104.38 m	102.55 m
BH6	44.00773°N, 77.12818°W	103.48 m	102.35 m



4.3 Groundwater Conditions

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs.

Groundwater was not encountered within the boreholes during the field investigation and due to the sloping nature of the Site and the shallow bedrock surface underlying the Site, Pinchin anticipates that the groundwater will not be encountered throughout the construction of the development for excavations in the overburden soils with the possible exception of localized areas of perched groundwater.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

The recommendations presented in the following sections of this report are based on the information available regarding the proposed construction, the results obtained from the geotechnical investigation, and Pinchin's experience with similar projects. Since the investigation only represents a portion of the subsurface conditions, it is possible that conditions may be encountered during construction that are substantially different than those encountered during the investigation. If these situations are encountered, adjustments to the design may be necessary. A qualified geotechnical engineer should be on-Site during the foundation preparation to ensure the subsurface conditions are the same/similar to what was observed during the investigation.

Based on information provided by the Client, it is Pinchin's understanding that the development will consist of a two slab-on-grade (i.e. no basement level) townhouse blocks containing four units each, in the west half of the Site, fronting onto John Street. The proposed development will also include new Site services and asphalt surfaced driveways. Proposed finished grades for the development have not been provided to Pinchin; however, due the proposed proximity of the proposed townhouse blocks to John Street, it is anticipated that finished grades at the fronts of the townhouses will be close to existing.



5.2 Site Preparation

The existing surficial organics and granular material is not considered suitable to remain below the proposed buildings, and driveways and will need to be removed. In calculating the approximate quantity of topsoil to be stripped, we recommend that the topsoil thicknesses provided on the individual borehole logs be increased by 50 mm to account for variations and some stripping of the mineral soil below. In addition, the existing residential dwelling and the associated concrete slabs and debris will need to be removed prior to commencing construction.

Pinchin recommends that any engineered fill required at the Site be compacted in accordance with the criteria stated in the following table:

Type of Engineered Fill	Maximum Loose Lift Thickness (mm)	Compaction Requirements	Moisture Content (Percent of Optimum)
Structural fill to support foundations and floor slabs	200	100% SPMDD	Plus 2 to minus 4
Subgrade fill beneath parking lots and access roadways	300	98% SPMDD	Plus 2 to minus 4

Prior to placing any fill material at the Site, the subgrade should be inspected by a qualified geotechnical engineer, and loosened/soft pockets should be sub excavated and replaced with engineered fill.

Engineered structural fill must extend at least 1 m beyond the edges of proposed footings and then downwards and outwards to approved subgrade at 1 horizontal to 1 vertical.

It is recommended that any fill required to raise grades below the proposed building addition comprise imported Ontario Provincial Standards and Specifications (OPSS) 1010 Granular 'B' material. If the work is carried out during very dry weather, water may have to be added to the material to improve compaction.

A qualified geotechnical engineering technician should be on site to observe fill placement operations and perform field density tests at random locations throughout each lift, to indicate the specified compaction is being achieved.

5.3 Open Cut Excavations and Dewatering

It is anticipated that the foundations will be constructed at conventional frost depths, approximately 1.5 metres below finished floor elevation. Due to the existing slope on the site, excavations at the rear of the townhouse blocks may be more than 5 m below existing grades. Excavations will also be required for construction of retaining walls which are proposed near the east limit of the property. All excavations should be made in conformance to the following recommendations.



Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of surficial organics, granular fill and natural glacial till material. Groundwater was not encountered within the boreholes advanced; however, there is potential for localized perched groundwater.

Where workers must enter trench excavations deeper than 1.2 m, the trench excavations should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act (OHSA), Ontario Regulation 213/91, Construction Projects, July 1, 2011, Part III - Excavations, Section 226. Alternatively, the excavation walls may be supported by either closed shoring, bracing, or trench boxes complying with sections 235 to 239 and 241 under O. Reg. 231/91, s. 234(1). The use of trench boxes can most likely be used for temporary support of vertical side walls. The appropriate trench should be designed/confirmed for use in this soil deposit.

Based on the OHSA, the natural glacial till soils would be classified as Type 3 soil and temporary excavations in these soils must be sloped at an inclination of 1 horizontal to 1 vertical (H to V) from the base of the excavation. Excavations extending below the groundwater table would be classified as a Type 4 soil and temporary excavations will have to be sloped back at 3 horizontal to 1 vertical from the base of the excavation. Excavations through more than one soil type must be sloped as per the requirements of the soil type with the highest number.

Depending on the finished floor elevation of the lowest level of the townhouse buildings, excavation may also be required through bedrock at the rear of the townhouse blocks. The upper limit of probable bedrock was recorded at the borehole locations, but no coring of the bedrock was completed. As such, the quality of the bedrock is not known. Excavations through weathered bedrock may be possible using large equipment with ripping teeth, hoe ram, or hydraulic hammer. Excavations through un-weathered competent bedrock may require blasting.

Excavations in bedrock can be done with near vertical sidewalls. Loose bedrock pieces should be removed from the sidewalls regularly.

In addition to compliance with the OHSA, the excavation procedures must also be in compliance to any potential other regulatory authorities, such as federal and municipal safety standards.

Alternatively, the excavation walls may be supported by either closed shoring, or bracing, complying with sections 235 to 239 and 241 under O. Reg. 231/91, s. 234(1). Pinchin would be pleased to provide further recommendations on shoring design once the building plans have been completed.

Moderate groundwater inflow through the glacial till or bedrock material should be expected. It is believed that this groundwater inflow can be controlled using a gravity dewatering system with perimeter interceptor ditches and high-capacity pumps.



Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions. If construction commences during wet periods (typically spring or fall), there is a greater potential that the groundwater elevation could be higher and/or perched groundwater may be present. Any potential precipitation of perched groundwater should be able to be controlled from pumping from filtered sumps.

Prior to commencing excavations, it is critical that all existing surface water and potential surface water is controlled and diverted away from the Site to prevent infiltration and subgrade softening. At no time should excavations be left open for a period of time that will expose them to precipitation and cause subgrade softening.

All collected water is to discharge a sufficient distance away from the excavation to prevent re-entry. Sediment control measures, such as a silt fence should be installed at the discharge point of the dewatering system. The utmost care should be taken to avoid any potential impacts on the environment.

It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. The method used should not adversely impact any nearby structures. Excavations to conventional design depths for the building foundations are not expected to require a Permit to Take Water or a submission to the Environmental Activity and Sector Registry (EASR). It is the responsibility of the contractor to make this application if required.

5.4 Site Servicing

5.4.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade conditions beneath the Site services will consist of glacial till and bedrock. No support problems are anticipated for flexible or rigid pipes founded on the glacial till or bedrock. Service pipes require an adequate base to ensure proper pipe connection and positive flow is maintained post construction. As such, pipe bedding should be placed to be of uniform thickness and compactness. The pipe bedding and cover material should conform to OPSD 802.010 and 802.013 specifications for flexible pipes and to OPSD 802.031 to 802.033 with Class 'B' bedding for rigid pipes.

For pipes installed within bedrock trenches, the following is recommended:

- Install 300 mm of 19 mm clear stone gravel (OPSS 1004) or Granular 'A' (OPSS 1010) below the pipe extending up the sides to the spring line;
- If clear stone is used as bedding material, then a non-woven geotextile (Terrafix 360R or equivalent) is to be placed over the clear stone and pipe extending up vertically along the side walls of the bedrock and pipe a minimum distance of 500 mm;



- The pipe cover material should consist of either a Granular 'B' Type I (OPSS 1010) with a maximum particle diameter size of 26.5 mm or bedding sand and should extend to a minimum of 300 mm above the top of the pipe; and
- If rock shatter is present a non-woven geotextile (Terrafix 360R or equivalent) may be required to prevent the migration of fines from the bedding material into the rock shatter. Where blasting is required for Site services, over blast of at least 600 mm of rock shatter should be performed. Over blast material may stay in the trench.

All granular fill material is to be placed in maximum 200 mm thick loose lifts compacted to a minimum of 98% SPMDD.

If constant groundwater infiltration becomes an issue, then an approximate 150 mm granular pad consisting of 19 mm clear stone gravel (OPSS 1004) wrapped in a non-woven geotextile (Terrafix 270R or equivalent) should be considered. The clear stone should contain a minimum of 50% crushed particles. Water collected within the stone should be controlled through sumps and filtered pumps.

5.4.2 *Trench Backfill*

Where the adjacent material consists of bedrock, the trench can be backfilled with well graded blast rock fill, with a gradation similar to OPSS 1010 Granular 'B' Type I. The soil should be placed to the underside of the granular subbase of the pavement structure and be compacted in maximum 300 mm thick lifts to 98% SPMDD within 4% of the optimum moisture content. This is recommended to provide soil compatibility and help minimize potential abrupt differential frost heave between surrounding natural materials similar in composition.

All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Quality control will be the utmost importance when selecting the material. The selection of the material should be done as early in the contract as possible to allow sufficient time for gradation and proctor testing on representative samples to ensure it meets the projects specifications.

It is anticipated that imported material may be required to backfill the trenches at the Site. Imported material should consist of a Granular 'A', Granular 'B' Type I, or Select Subgrade Material (OPSS 1010). Heavy construction equipment and truck traffic should not cross any pipe until at least 1 m of compacted soil is placed above the top of the pipe.

Post compaction settlement of finer grained soil can be expected, even when placed to compaction specifications. As such, fill materials should be installed as far in advance as possible before finishing the roadway in order to mitigate post compaction settlements.



5.4.3 Frost Protection

It is understood that the municipal standards at this Site require water services to have 1.8 to 2.0 m of soil cover. If the required soil cover cannot be provided, then the pipe should be insulated with a rigid polystyrene insulation (DOW Styrofoam HI40, or equivalent) or a pre-insulated pipe be utilized.

The insulation design configuration may either consist of placing horizontal insulation to a specified design distance beyond the outside edge of the pipe or an inverted “U” surrounding the top and sides of the pipe. Any method chosen requires suitable design and installation in accordance with the manufacture’s recommendations. To accommodate the placement of horizontal insulation a wider excavation trench may be required.

5.5 Foundation Design

5.5.1 Shallow Foundations Bearing on Glacial Till

The existing glacial till soil is considered suitable to support the proposed building, provided all of the pavement structure, fill, topsoil are removed, and the subgrade prepared as above.

Conventional shallow strip footings established on the glacial till may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States of 125 kPa, and a factored geotechnical bearing resistance of 175 kPa at Ultimate Limit States (ULS).

As the actual service loads were not known at the time of this report, these should be reviewed by the project structural engineer to determine if SLS or ULS governs the footing design.

It is noted that there is a potential for weaker subgrade soil to be encountered between the investigation locations. Pinchin presumes that any areas of weaker subgrade soil will consist of small pockets of soft/loose natural soil which can be compacted to match the density of the remainder of the Site. As such, the material must be compacted to a minimum of 100% Standard Proctor Maximum Dry Density (SPMDD) prior to installing the concrete formwork. Any soft/loose areas which are not able to achieve the recommended 100% SPMDD are to be removed and replaced with a low strength concrete.

Pinchin notes that a qualified geotechnical engineering consultant should be on-Site during the proof roll and foundation preparation activities to verify the recommended level of compaction is achieved and to verify the design assumptions and recommendations. This is especially critical with respect to the recommended soil bearing pressures. If variations occur in the soil conditions between the borehole locations, site verification and site review by Pinchin is recommended to provide appropriate recommendations at that time.



The natural subgrade soil is sensitive to change in moisture content and can become loose/soft if subjected to additional water or precipitation. As well, it could be easily disturbed if travelled on during construction. Once it becomes disturbed it is no longer considered adequate to support the recommended design bearing pressures. It is recommended that a working slab of lean concrete (mud slab) be placed in the footing areas immediately after excavation and inspection to protect the founding soils during placement of formwork and reinforcing steel.

In addition, to ensure and protect the integrity of the subgrade soil during construction operations, the following is recommended:

- Prior to commencing excavations, it is critical that all existing surface water, potential surface water and perched groundwater are controlled and diverted away from the work Site to prevent infiltration and subgrade softening. At no time should excavations be left open for a period of time that will expose them to inclement weather conditions and cause subgrade softening;
- The subgrade should be sloped to a sump outside the excavation to promote surface drainage and the collected water pumped out of the excavation. Any potential precipitation or seepage entering the excavations should be pumped away immediately (not allowed to pond);
- The footing areas should be cleaned of all deleterious materials such as topsoil, organics, fill, disturbed, caved materials or loosened bedrock pieces;
- Any potential large cobbles or boulders (i.e. greater than 200 mm in diameter) within the subgrade material are to be removed and replaced with a similar soil type not containing particles greater than 200 mm in diameter. It is critical that particles greater than 200 mm in diameter are not in contact with the foundation to prevent point loading and overstressing; and
- If the excavated subgrade soil remains open to weather conditions and groundwater seepage, sidewall stability and suitability of the subgrade soil will need to be verified prior to construction.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided and maintained above freezing at all times.



5.5.2 Shallow Foundations Bearing on Bedrock

Probable bedrock, assessed by auger refusal, was encountered within all of the boreholes at approximately 1.1 to 2.8 mbgs. During detailed design stage the bedrock may be cored and proven if a higher bearing pressure than provided below is desired.

For conventional shallow strip and spread footings established directly on the weathered bedrock surface, a factored geotechnical bearing resistance of 400 kPa may be used at ULS. Higher bearing resistances may be available on the unweathered bedrock; however, the bedrock should be cored to confirm this recommendation.

Prior to installing foundation formwork, the bedrock is to be reviewed by a geotechnical engineer. SLS does not apply to foundations bearing directly on bedrock, since the loads required for unacceptable settlements to occur would be much larger than the factored ULS and would be limited to the elastic compression of the bedrock and concrete.

The bearing resistance of 400 kPa (weathered bedrock) assumes the bedrock is cleaned of all overburden material and any loose rock pieces. The bedrock should be cleaned with air or water pressure exposing clean sound bedrock. If construction proceeds during freezing weather conditions water should not be allowed to pool and freeze in bedrock depressions. All concrete should be installed and maintained above freezing temperatures as required by the concrete supplier.

The bedrock is to be relatively level with slopes not exceeding 10 degrees from the horizontal. Where the bedrock slope exceeds 10 degrees from the horizontal and does not exceed 25 degrees from the horizontal, shear dowels can be incorporated into the design to resist sliding. Where rock slopes are steeper, the bedrock is to be levelled and stepped as required. The change in vertical height will be a function of the rock quality at the proposed foundation location and will need to be determined at the time of construction.

As an alternative to levelling the bedrock, where the bedrock surface is irregular and jagged, it may be more practical to provide a level benching over these areas by pouring lean mix concrete (minimum 10 MPa) prior to constructing the foundations. This decision is made on Site since each situation will depend on the Site-specific bedrock conditions.

5.5.3 Site Classification for Seismic Site Response & Soil Behaviour

The following information has been provided to assist the building designer from a geotechnical perspective only. These geotechnical seismic design parameters should be reviewed in detail by the structural engineer and be incorporated into the design as required.



The seismic site classification has been based on the 2012 OBC. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A of the OBC. The site classification is based on the average shear wave velocity in the top 30 m of the site stratigraphy. If the average shear wave velocity is not known, the site class can be estimated from energy corrected Standard Penetration Resistance (N60) and/or the average undrained shear strength of the soil in the top 30 m.

The boreholes advanced at this Site extended to a maximum depth of approximately 2.8 mbgs and were terminated in on the underlying bedrock surface. SPT “N” values within the glacial till ranged between 10 and greater than 50 blows per 300 mm. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class C. A Site Class C has an average shear wave velocity (Vs) of between 360 and 760 m/s.

5.5.4 *Foundation Transition Zones*

Excessive differential settlements can occur where the subgrade support material types differ below the underside of continuous strip footings, (i.e., glacial till to bedrock). As such, where strip footings transition from one material to another the transition between the materials should be suitably sloped or benched to mitigate differential settlements.

Pinchin also recommends the following transition precautions to mitigate/accommodate potential differential settlements:

- For strip footings, the transition zones should be adequately reinforced with additional reinforced steel lap lengths or widened footings;
- Steel reinforced poured concrete foundation walls; and
- Control joints throughout the transition zone(s).

The above recommendations should be reviewed by the structural engineer and incorporated into the design as necessary.

Where strip footings are founded at different elevations, the subgrade soil is to have a maximum slope of 2 H to 1 V, with the concrete footing having a maximum rise of 600 mm and a minimum run of 600 mm between each step, as detailed in the 2012 Ontario Building Code (OBC). The lower footing should be installed first to mitigate the risk of undermining the upper footing.

Individual spread footings are to be spaced a minimum distance of one and a half times the largest footing width apart from each other to avoid stress bulb interaction between footings. This assumes the footings are at the same elevation.



Foundations may be placed at a higher elevation relative to one another provided that the slope between the outside face of the foundations are separated at a minimum slope of 2H: 1V with an imaginary line drawn from the underside of the foundations. The lower footing should be installed first to mitigate the risk of undermining the upper footing.

5.5.5 Estimated Settlement

All individual spread footings should be founded on uniform subgrade soils, reviewed and approved by a licensed geotechnical engineer.

Foundations installed in accordance with the recommendations outlined in the preceding sections are not expected to exceed total settlements of 25 mm and differential settlements of 19 mm.

All foundations are to be designed and constructed to the minimum widths as detailed in the 2012 OBC.

5.5.6 Building Drainage

To assist in maintaining the building dry from surface water seepage, it is recommended that exterior grades around the buildings be sloped away at a 2% gradient or more, for a distance of at least 2.0 m. Roof drains should discharge a minimum of 1.5 m away from the structure to a drainage swale or appropriate storm drainage system.

Exterior perimeter foundations drains are not required, where the finished floor elevation is established a minimum of 150 mm above the exterior final grades or that the exterior gradient is properly sloped to divert surface water away from the building. Exterior foundations of portions of the buildings with floor level lower than exterior grades must be provided with footing weepers draining to a sump.

5.5.7 Shallow Foundations Frost Protection & Foundation Backfill

In the Picton, Ontario area, exterior perimeter foundations bearing on soil, for heated buildings require a minimum of 1.5 m of soil cover above the underside of the footing to provide soil cover for frost protection. Footings bearing on sound bedrock do not require this soil cover for frost protection.

Where the foundations on soil for heated buildings do not have the minimum 1.5 m of soil cover frost protection, they should be protected from frost with a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product. If required, Pinchin can provide appropriate foundation frost protection recommendations as part of the design review.

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular 'B' Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The existing glacial till



material is not considered suitable for reuse as foundation wall backfill where free draining backfill is required, unless a manufactured drainage layer is provided on the exterior of foundation wall. The backfill must be brought up evenly on both sides of foundation walls not designed to resist lateral earth pressure. All backfill material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD below the interior of building and exterior hard landscaping areas; and 95% SPMDD below exterior soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.6 Floor Slabs

Prior to the installation of the engineered fill material, all surficial organics, granular fill and deleterious material should be removed to the underlying organic free in-situ soil. The natural subgrade soil is to be proof roll compacted with a minimum 10 tonne non-vibratory steel drum roller to observe for weak/soft spots. It is noted that some locations will not be accessible by the steel drum roller; as such, these locations can be proof roll compacted with a minimum 450 kg vibratory plate compactor.

The in-situ inorganic silt material encountered within the boreholes is considered adequate for the support of the concrete floor slabs provided it is proof roll compacted as outlined above. Any soft area(s) encountered during proof rolling should be excavated and replaced with a similar soil type.

Once the subgrade soil is exposed it is to be inspected and approved by a qualified geotechnical engineering consultant to ensure that the material conforms to the soil type and consistency observed during the subsurface investigation work.

Based on the in-situ soil conditions, it is recommended to establish the concrete floor slab on a minimum 300 mm thick layer of Granular “A” (OPSS 1010) compacted to 100% SPMDD. Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone placed over the approved subgrade. Any required up fill should consist of a Granular “B” Type I or Type II (OPSS 1010).

The installation of a vapour barrier may be required under the floor slab. If required, the vapour barrier should conform to the flooring manufacturer’s and designer’s requirements. Consideration may be given to carrying out moisture emission and/or relative humidity testing of the slab to determine the concrete condition prior to flooring installation. To minimize the potential for excess moisture in the floor slab, a concrete mixture with a low water-to-cement ratio (i.e. 0.5 to 0.55) should be used.



The following table provides the unfactored modulus of subgrade reaction values:

Material Type	Modulus of Subgrade Reaction (kN/m³)
Granular A (OPSS 1010)	85,000
Granular “B” Type I (OPSS 1010)	75,000
Granular “B” Type II (OPSS 1010)	85,000
Glacial Till	50,000

The values in the table above are for loaded areas of 0.3 m by 0.3 m.

5.7 Asphaltic Concrete Pavement Structure Design for Parking Lot and Driveways

5.7.1 Discussion

Driveway access will be constructed for the proposed buildings. The in-situ glacial till is considered a sufficient bearing material for an asphaltic concrete pavement structure provided all organics and deleterious materials are removed prior to installing the engineered fill material.

At this time Pinchin is unaware of the proposed final grades for the driveways. As such, provided the pavement structure overlies the in-situ glacial till material, the following pavement structure is recommended.

5.7.2 Pavement Structure

The following table presents the minimum specifications for a flexible asphaltic concrete pavement structure:

Pavement Layer	Compaction Requirements	Driveways
Surface Course Asphaltic Concrete HL-3 (OPSS 1150)	92% MRD as per OPSS 310	35 mm
Binder Course Asphaltic Concrete HL-4 (OPSS 1150)	92 % MRD as per OPSS 310	45 mm
Base Course: Granular “A” (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm
Subbase Course: Granular “B” Type I (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	300 mm

Notes:

- I. Prior to placing the pavement structure, the subgrade soil is to be proof rolled with a smooth drum roller without vibration to observe weak spots and the deflection of the soil; and
- II. The recommended pavement structure may have to be adjusted according to the Township of Picton standards. Also, if construction takes place during times of substantial precipitation and the subgrade soil becomes wet and disturbed, the granular thickness may have to be increased to compensate for the weaker subgrade soil. In addition, the granular fill material thickness may have to be temporarily increased to allow heavy construction equipment access the Site, in order to avoid the subgrade from “pumping” up into the granular material.



Performance grade PG 58-28 asphaltic concrete should be specified for Marshall mixes.

5.7.3 Pavement Structure Subgrade Preparation and Granular up Fill

The proper placement of base and subbase fill materials becomes very important in addressing the proper load distribution to provide a durable pavement structure.

The pavement subgrade materials should be thoroughly proof-rolled prior to placement of the Granular 'B' subbase course. If any unstable areas are noted, then the Granular 'B' thickness may need to be increased to support pavement construction traffic. This should be left as a field decision by a qualified geotechnical engineer at the time of construction, but it is recommended that additional Granular 'B' be carried as a provisional item under the construction contract.

Where fill material is required to increase the grade to the underside of the pavement structure it should consist of Granular 'B' Type I (OPSS 1010). The up fill material is to be placed in maximum 300 mm thick lifts compacted to 98% SPMDD within 4% of the optimum moisture content.

Samples of both the Granular 'A' and Granular 'B' Type I aggregates should be tested for conformance to OPSS 1010 prior to utilization on Site and during construction. All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Post compaction settlement of fine grained soil can be expected, even when placed to compaction specifications. As such, fill material should be installed as far in advance as possible before finishing the parking lot and access roadways for best grade integrity.

Where the subgrade material types differ below the underside of the pavement structure, the transition between the materials should be sloped as per frost heave taper OPSD 205.60.

6.0 SLOPE STABILITY ASSESSMENT

At the time that the proposal was completed, The Corporation of the County of Prince Edward (PEC) had requested that a qualified professional evaluate if the existing slope will remain stable post development. As such, Pinchin completed a visual condition assessment at the time of the geotechnical field investigation to determine if a global slope stability analysis is required or not.

The visual slope condition assessment was completed while referencing the procedure outlined in the Ontario Ministry of Natural Resources (MNR) "Technical Guide – River and Stream Systems: Erosion Hazard Limit". The visual inspection consisted of systematically performing a criterion-based slope failure risk assessment based on visual and tactile observations of the Site conditions, surface conditions, topography, erosion, slope stability/instability signs, the performance/non-performance of existing structures, and potential impact of the proposed development.



Based on the MNR Technical Guide mentioned above, the slope stability rating chart indicates that slopes with a rating score of less than 24 have low potential for instability and require a Site inspection only, followed by a summary outlining the observations made during the slope inspection. If the slope scores a rating of 25 or greater, there is a slight to moderate potential for instability and a detailed report is required.

6.1 Visual Slope Inspection and Findings

A visual slope inspection was completed on October 19, 2023 at the same time that the geotechnical field investigation was completed. Pinchin analyzed the main slope that was sloping downwards towards John Street. Based on the Visual Slope Inspection completed, Pinchin assessed a rating score of 19. Additionally, the steeper slope down to Union Street at the southeast corner of the Site was also assessed. Pinchin assessed a rating score of 22 for the steeper slope. Based on the assessed slope ratings, no further detailed analysis is required due to the apparent low potential for instabilities.

6.2 Photos of Slope

The following photos are representative of the multiple slopes on Site:



Photo 1: Slope in front of house along John Street.



Photo 2: Slope from east side of property line down to John Street/west property line.



Photo 3: Slope along Union Street looking down from Borehole BH3, shows vegetation on side of slope along with lack of seepage from slope face.

7.0 SITE SUPERVISION & QUALITY CONTROL

It is recommended that all geotechnical aspects of the project be reviewed and confirmed under the appropriate geotechnical supervision, to routinely check such items. This includes but is not limited to inspection and confirmation of the undisturbed natural subgrade material prior to subgrade preparation, pouring any foundations or footings, backfilling, or engineered fill installation to ensure that the actual conditions are not markedly different than what was observed at the borehole locations and geotechnical components are constructed as per Pinchin's recommendations. Compaction quality control of engineered fill material (full-time monitoring) is recommended as standard practice, as well as regular



sampling and testing of aggregates and concrete, to ensure that physical characteristics of materials for compliance during installation and satisfies all specifications presented within this report.

8.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of JAMR Developments (Client) in order to evaluate the subsurface conditions at 6 John Street, Picton, Ontario. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practises in the field of geotechnical engineering for the Site. Classification and identification of soil, and geologic units have been based upon commonly accepted methods employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Conclusions derived are specific to the immediate area of study and cannot be extrapolated extensively away from sample locations.

Performance of this Geotechnical Investigation to the standards established by Pinchin is intended to reduce, but not eliminate, uncertainty regarding the subgrade soil at the Site, and recognizes reasonable limits on time and cost.

Regardless how exhaustive a Geotechnical Investigation is performed, the investigation cannot identify all the subsurface conditions. Therefore, no warranty is expressed or implied that the entire Site is representative of the subsurface information obtained at the specific locations of our investigation. If during construction, subsurface conditions differ from then what was encountered within our test location and the additional subsurface information provided to us, Pinchin should be contacted to review our recommendations. This report does not alleviate the contractor, owner, or any other parties of their respective responsibilities.

This report has been prepared for the exclusive use of the Client and their authorized agents. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of the third parties. If additional parties require reliance on this report, written authorization from Pinchin will be required. Pinchin disclaims responsibility of consequential financial effects on transactions or property values, or requirements for follow-up actions and costs. No other warranties are implied or expressed. Furthermore, this report should not be construed as legal advice.

The liability of Pinchin or our officers, directors, shareholders or staff will be limited to the lesser of the fees paid or actual damages incurred by the Client. Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be liable for damages resulting from the negligence of Pinchin. Pinchin will not be liable for any losses or damage if the Client has failed, within a period of two years following the date upon which the claim is discovered (Claim Period), to commence legal proceedings against Pinchin to recover such losses or damage unless the laws of the jurisdiction which governs the



Claim Period which is applicable to such claim provides that the applicable Claim Period is greater than two years and cannot be abridged by the contract between the Client and Pinchin, in which case the Claim Period shall be deemed to be extended by the shortest additional period which results in this provision being legally enforceable.

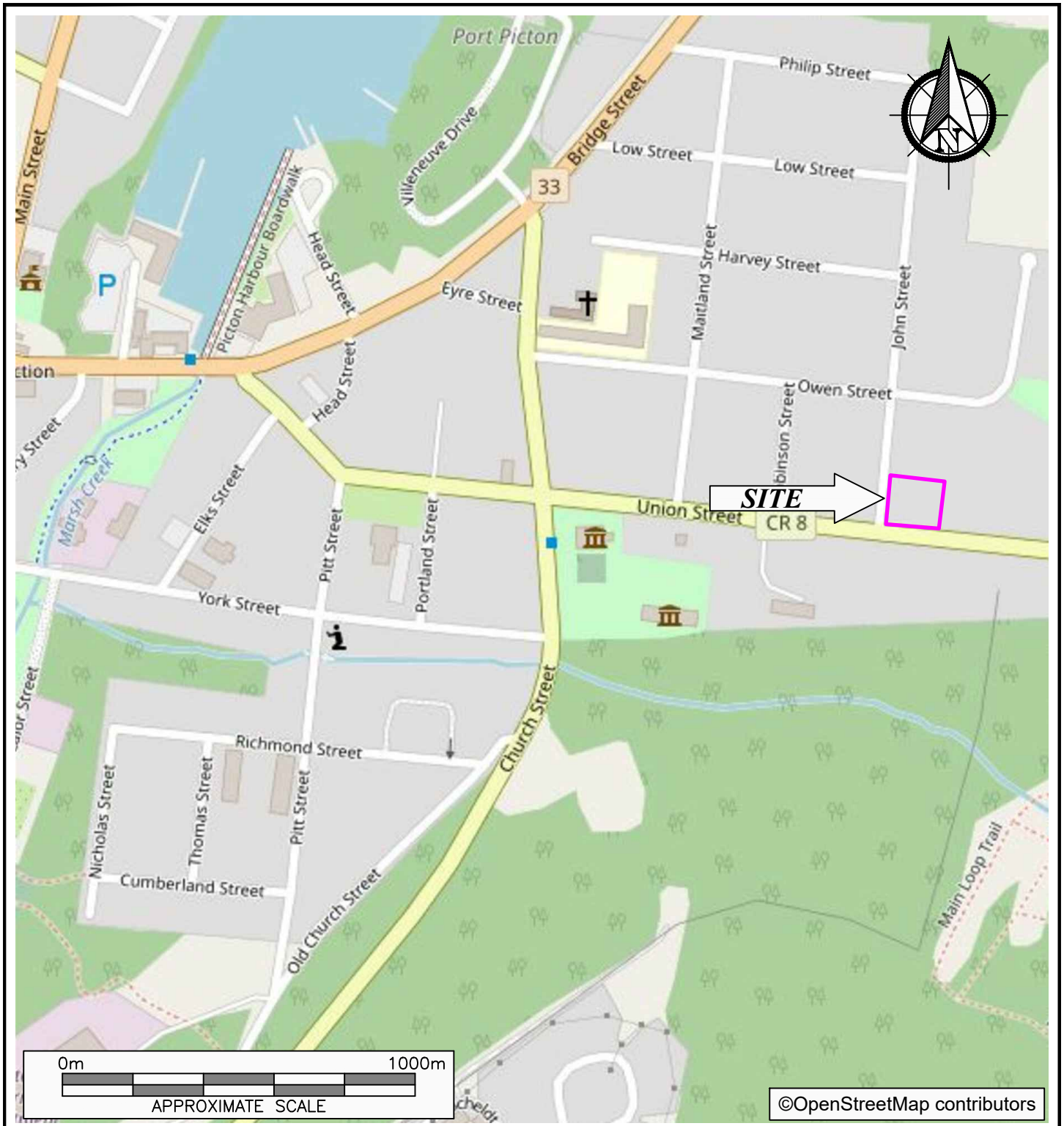
Pinchin makes no other representations whatsoever, including those concerning the legal significance of its findings, or as to other legal matters touched on in this report, including, but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect to regulatory compliance issues, regulatory statutes are subject to interpretation and these interpretations may change over time. Please refer to Appendix IV, Report Limitations and Guidelines for Use, which pertains to this report.

Specific limitations related to the legal and financial and limitations to the scope of the current work are outlined in our proposal, the attached Methodology and the Authorization to Proceed, Limitation of Liability and Terms of Engagement which accompanied the proposal.

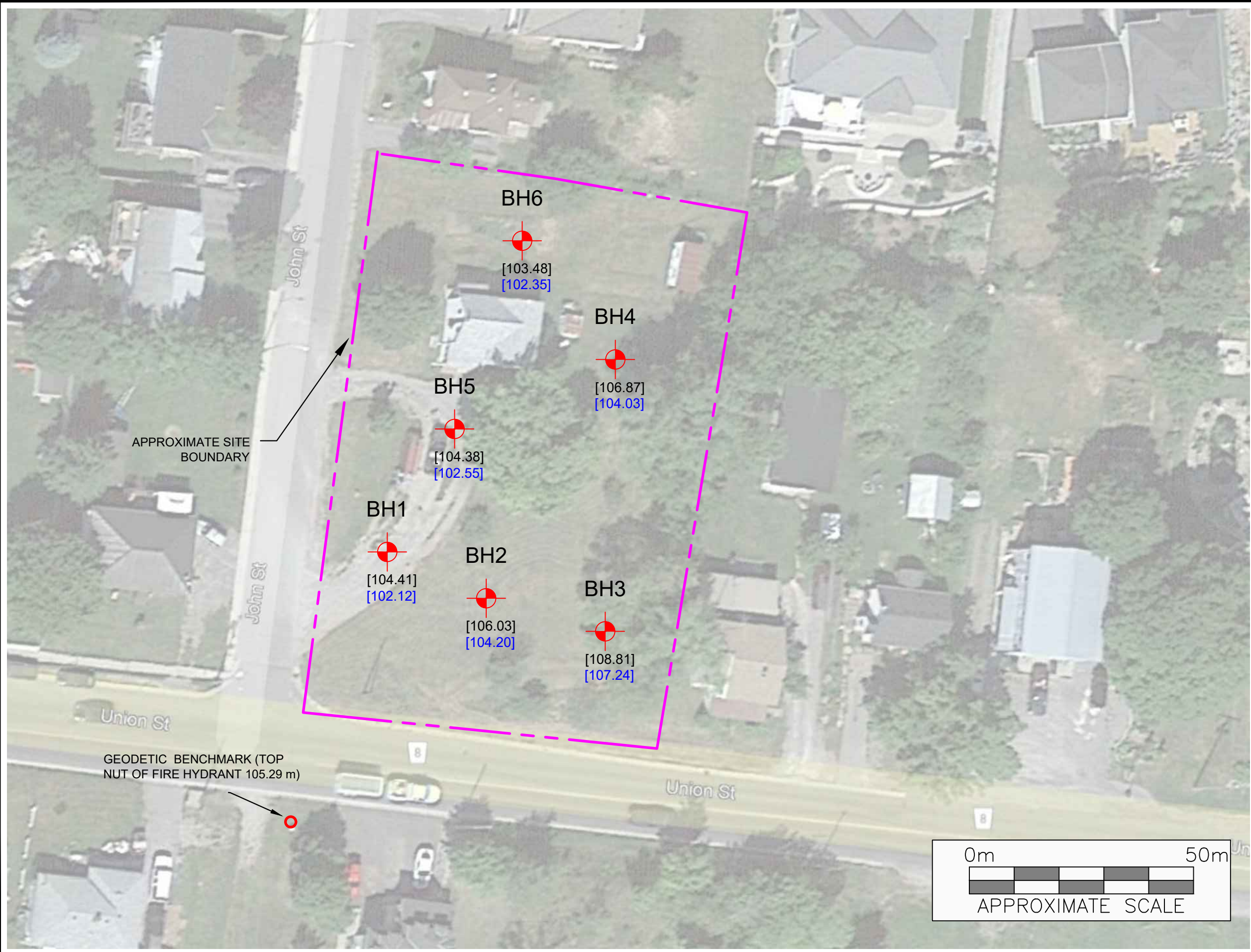
Information provided by Pinchin is intended for Client use only. Pinchin will not provide results or information to any party unless disclosure by Pinchin is required by law. Any use by a third party of reports or documents authored by Pinchin or any reliance by a third party on or decisions made by a third party based on the findings described in said documents, is the sole responsibility of such third parties. Pinchin accepts no responsibility for damages suffered by any third party as a result of decisions made or actions conducted. No other warranties are implied or expressed.

\\pinchin.com\Ott\Job\327000s\0327548.000 JAMR,6JohnStreet,GEO,INV\Deliverables\327548 FINAL GEO Report 6 John Street Picton ON JAMR Dec 11 2023.docx
Template: Master Geotechnical Investigation Report – Ontario, GEO, September 2, 2021


FIGURES



PROJECT NAME			GEOTECHNICAL INVESTIGATION
CLIENT NAME			JAMR DEVELOPMENTS
PROJECT LOCATION			6 JOHN STREET, PICTON, ONTARIO
FIGURE NAME		KEY MAP	FIGURE NO. 1
APPROXIMATE SCALE AS SHOWN	PROJECT NO. 327548	DATE DECEMBER 2023	



LEGEND

-  BOREHOLE LOCATION
- [XX.XX] GEODETIC GROUND ELEVATION (m)
- [XX.XX] GEODETIC REFUSAL ELEVATION (m)



PROJECT NAME
GEOTECHNICAL INVESTIGATION

CLIENT NAME
JAMR DEVELOPMENTS

PROJECT LOCATION
**6 JOHN STREET, PICTON,
ONTARIO**

FIGURE NAME
BOREHOLE LOCATION PLAN

APPROXIMATE SCALE AS SHOWN	PROJECT NO. 327548
--------------------------------------	------------------------------

DATE DECEMBER 2023	FIGURE NO. 2
------------------------------	------------------------

APPENDIX I
Abbreviations, Terminology and Principle Symbols used in Report and
Borehole Logs

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

AS	Auger Sample	w	Washed Sample
SS	Split Spoon Sample	HQ	Rock Core (63.5 mm diam.)
ST	Thin Walled Shelby Tube	NQ	Rock Core (47.5 mm diam.)
BS	Block Sample	BQ	Rock Core (36.5 mm diam.)

In-Situ Soil Testing

Standard Penetration Test (SPT), “N” value is the number of blows required to drive a 51 mm outside diameter split barrel sampler into the soil a distance of 300 mm with a 63.5 kg weight free falling a distance of 760 mm after an initial penetration of 150 mm has been achieved. The SPT, “N” value is a qualitative term used to interpret the compactness condition of cohesionless soils and is used only as a very approximation to estimate the consistency and undrained shear strength of cohesive soils.

Dynamic Cone Penetration Test (DCPT) is the number of blows required to drive a cone with a 60 degree apex attached to “A” size drill rods continuously into the soil for each 300 mm penetration with a 63.5 kg weight free falling a distance of 760 mm.

Cone Penetration Test (CPT) is an electronic cone point with a 10 cm² base area with a 60 degree apex pushed through the soil at a penetration rate of 2 cm/s.

Field Vane Test (FVT) consists of a vane blade, a set of rods and torque measuring apparatus used to determine the undrained shear strength of cohesive soils.

Soil Descriptions

The soil descriptions and classifications are based on an expanded Unified Soil Classification System (USCS). The USCS classifies soils on the basis of engineering properties. The system divides soils into three major categories; coarse grained, fine grained and highly organic soils. The soil is then subdivided based on either gradation or plasticity characteristics. The classification excludes particles larger than 75 mm. To aid in quantifying material amounts by weight within the respective grain size fractions the following terms have been included to expand the USCS:

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm		
Silt	0.002 to 0.06 mm	“trace”, trace sand, etc.	1 to 10%
Sand	0.075 to 4.75 mm	“some”, some sand, etc.	10 to 20%
Gravel	4.75 to 75 mm	Adjective, sandy, gravelly, etc.	20 to 35%
Cobbles	75 to 200 mm	And, and gravel, and silt, etc.	>35%
Boulders	>200 mm	Noun, Sand, Gravel, Silt, etc.	>35% and main fraction

Notes:

- Soil properties, such as strength, gradation, plasticity, structure, etcetera, dictate the soils engineering behaviour over grain size fractions; and
- With the exception of soil samples tested for grain size distribution or plasticity, all soil samples have been classified based on visual and tactile observations. The accuracy of visual and tactile observation is not sufficient to differentiate between changes in soil classification or precise grain size and is therefore an approximate description.

The following table outlines the qualitative terms used to describe the compactness condition of cohesionless soil:

Cohesionless Soil	
Compactness Condition	SPT N-Index (blows per 300 mm)
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

The following table outlines the qualitative terms used to describe the consistency of cohesive soils related to undrained shear strength and SPT, N-Index:

Cohesive Soil		
Consistency	Undrained Shear Strength (kPa)	SPT N-Index (blows per 300 mm)
Very Soft	<12	<2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

Note: Utilizing the SPT, N-Index value to correlate the consistency and undrained shear strength of cohesive soils is only very approximate and needs to be used with caution.

Soil & Rock Physical Properties

General

W	Natural water content or moisture content within soil sample
γ	Unit weight
γ'	Effective unit weight
γ_d	Dry unit weight
γ_{sat}	Saturated unit weight
ρ	Density
ρ_s	Density of solid particles
ρ_w	Density of Water
ρ_d	Dry density
ρ_{sat}	Saturated density e Void ratio
n	Porosity
S_r	Degree of saturation
E_{50}	Strain at 50% maximum stress (cohesive soil)

Consistency

W_L	Liquid limit
W_P	Plastic Limit
I_P	Plasticity Index
W_S	Shrinkage Limit
I_L	Liquidity Index
I_C	Consistency Index
e_{max}	Void ratio in loosest state
e_{min}	Void ratio in densest state
I_D	Density Index (formerly relative density)

Shear Strength

C_u, S_u	Undrained shear strength parameter (total stress)
C'_d	Drained shear strength parameter (effective stress)
r	Remolded shear strength
τ_p	Peak residual shear strength
τ_r	Residual shear strength
ϕ'	Angle of interface friction, coefficient of friction = $\tan \phi'$

Consolidation (One Dimensional)

C_c	Compression index (normally consolidated range)
C_r	Recompression index (over consolidated range)
C_s	Swelling index
m_v	Coefficient of volume change
c_v	Coefficient of consolidation
T_v	Time factor (vertical direction)
U	Degree of consolidation
σ'_o	Overburden pressure
σ'_p	Preconsolidation pressure (most probable)
OCR	Overconsolidation ratio

Permeability

The following table outlines the terms used to describe the degree of permeability of soil and common soil types associated with the permeability rates:

Permeability (k cm/s)	Degree of Permeability	Common Associated Soil Type
$> 10^{-1}$	Very High	Clean gravel
10^{-1} to 10^{-3}	High	Clean sand, Clean sand and gravel
10^{-3} to 10^{-5}	Medium	Fine sand to silty sand
10^{-5} to 10^{-7}	Low	Silt and clayey silt (low plasticity)
$>10^{-7}$	Practically Impermeable	Silty clay (medium to high plasticity)

Rock Coring

Rock Quality Designation (RQD) is an indirect measure of the number of fractures within a rock mass, Deere et al. (1967). It is the sum of sound pieces of rock core equal to or greater than 100 mm recovered from the core run, divided by the total length of the core run, expressed as a percentage. If the core section is broken due to mechanical or handling, the pieces are fitted together and if 100 mm or greater included in the total sum.

RQD is calculated as follows:

$$\text{RQD (\%)} = \frac{\sum \text{Length of core pieces} > 100 \text{ mm} \times 100}{\text{Total length of core run}}$$

The following is the Classification of Rock with Respect to RQD Value:

RQD Classification	RQD Value (%)
Very poor quality	<25
Poor quality	25 to 50
Fair quality	50 to 75
Good quality	75 to 90
Excellent quality	90 to 100

APPENDIX II
Pinchin's Borehole Logs



Log of Borehole: BH2

Project #: 327548

Logged By: MK

Project: Geotechnical Investigation

Client: JAMR Developments

Location: 6 John Street, Picton, Ontario

Drill Date: October 19, 2023

Project Manager: MK

SUBSURFACE PROFILE				SAMPLE													
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value			Shear Strength	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis	
									20	40	60	Δ kPa Δ					
0		Ground Surface	106.03	No Monitoring Well Installed													
		Organics ~75 mm	0.00														
		Glacial Till Silty sand, some gravel, some clay, brown, damp, compact			SS	1	40	18									
1					SS	2	30	10									
					SS	3	20	15									
2		End of Borehole	104.20														
		Borehole terminated at 1.8 mbgs due to refusal on probable bedrock. At drilling completion, groundwater was not encountered in the open boreholes.	1.83														
3																	

Contractor: Canadian Environmental Drilling & Contractors

Grade Elevation: 106.03 m

Drilling Method: Solid Stem Auger/Split Spoon

Top of Casing Elevation: N/A

Well Casing Size: N/A

Sheet: 1 of 1



Log of Borehole: BH3

Project #: 327548

Logged By: MK

Project: Geotechnical Investigation

Client: JAMR Developments

Location: 6 John Street, Picton, Ontario

Drill Date: October 19, 2023

Project Manager: MK

SUBSURFACE PROFILE					SAMPLE										
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value	Shear Strength	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis	
0		Ground Surface	108.81	No Monitoring Well Installed											
		Organics ~50 mm	0.00												
		Glacial Till Silty sand, some gravel, some clay, brown, damp, compact			SS	1	20	19							
1					SS	2	30	25							
		End of Borehole	107.24												
		Borehole terminated at 1.6 mbgs due to refusal on probable bedrock. At drilling completion, groundwater was not encountered in the open boreholes.	1.57												
2															
3															

Contractor: Canadian Environmental Drilling & Contractors

Grade Elevation: 108.81 m

Drilling Method: Solid Stem Auger/Split Spoon

Top of Casing Elevation: N/A

Well Casing Size: N/A

Sheet: 1 of 1



Log of Borehole: BH4

Project #: 327548

Logged By: MK

Project: Geotechnical Investigation

Client: JAMR Developments

Location: 6 John Street, Picton, Ontario

Drill Date: October 19, 2023

Project Manager: MK

SUBSURFACE PROFILE				SAMPLE													
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value			Shear Strength △ kPa △	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis	
									20	40	60						
0		Ground Surface	106.87	No Monitoring Well Installed													
		Organics ~75 mm	0.00														
		Glacial Till Silty sand, some gravel, some clay, brown, damp, compact to very dense			SS	1	30	18									
1					SS	2	70	84									
2					SS	3	70	61									
				SS	4	60	48										
3		End of Borehole	104.03 2.83														
		Borehole terminated at 2.8 mbgs due to refusal on probable bedrock. At drilling completion, groundwater was not encountered in the open boreholes.															

Contractor: Canadian Environmental Drilling & Contractors

Grade Elevation: 106.87 m

Drilling Method: Solid Stem Auger/Split Spoon

Top of Casing Elevation: N/A

Well Casing Size: N/A

Sheet: 1 of 1



Log of Borehole: BH5

Project #: 327548

Logged By: MK

Project: Geotechnical Investigation

Client: JAMR Developments

Location: 6 John Street, Picton, Ontario

Drill Date: October 19, 2023

Project Manager: MK

SUBSURFACE PROFILE					SAMPLE										
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 □ 40 □ 60 □	Shear Strength △ kPa △ 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis	
0		Ground Surface	104.38	↑ No Monitoring Well Installed ↓											
		Granular Fill ~25 mm	0.00												
		Glacial Till Sand and gravel, some silt, brown, damp, loose to dense				SS	1	30	6						
1						SS	2	60	41						GS.
			102.55												
			1.83		SS	3	50	43							
2		End of Borehole													
		Borehole terminated at 1.8 mbgs due to refusal on probable bedrock. At drilling completion, groundwater was not encountered in the open boreholes.													
3															

Contractor: Canadian Environmental Drilling & Contractors

Grade Elevation: 104.38 m

Drilling Method: Solid Stem Auger/Split Spoon

Top of Casing Elevation: N/A

Well Casing Size: N/A

Sheet: 1 of 1



Log of Borehole: BH6

Project #: 327548

Logged By: MK

Project: Geotechnical Investigation

Client: JAMR Developments

Location: 6 John Street, Picton, Ontario

Drill Date: October 19, 2023

Project Manager: MK

SUBSURFACE PROFILE				SAMPLE												
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value			Shear Strength Δ kPa Δ	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
									20	40	60					
0		Ground Surface	103.48	↑ No Monitoring Well Installed ↓												
		Organics ~100 mm	0.00													
		Glacial Till Sand and gravel, some silt, brown, damp, loose to dense			SS	1	60	5								
1			102.35		SS	2	20	39								
		End of Borehole	1.13													
		Borehole terminated at 1.1 mbgs due to refusal on probable bedrock. At drilling completion, groundwater was not encountered in the open boreholes.														
2																
3																

Contractor: Canadian Environmental Drilling & Contractors

Grade Elevation: 103.48 m

Drilling Method: Solid Stem Auger/Split Spoon

Top of Casing Elevation: N/A

Well Casing Size: N/A

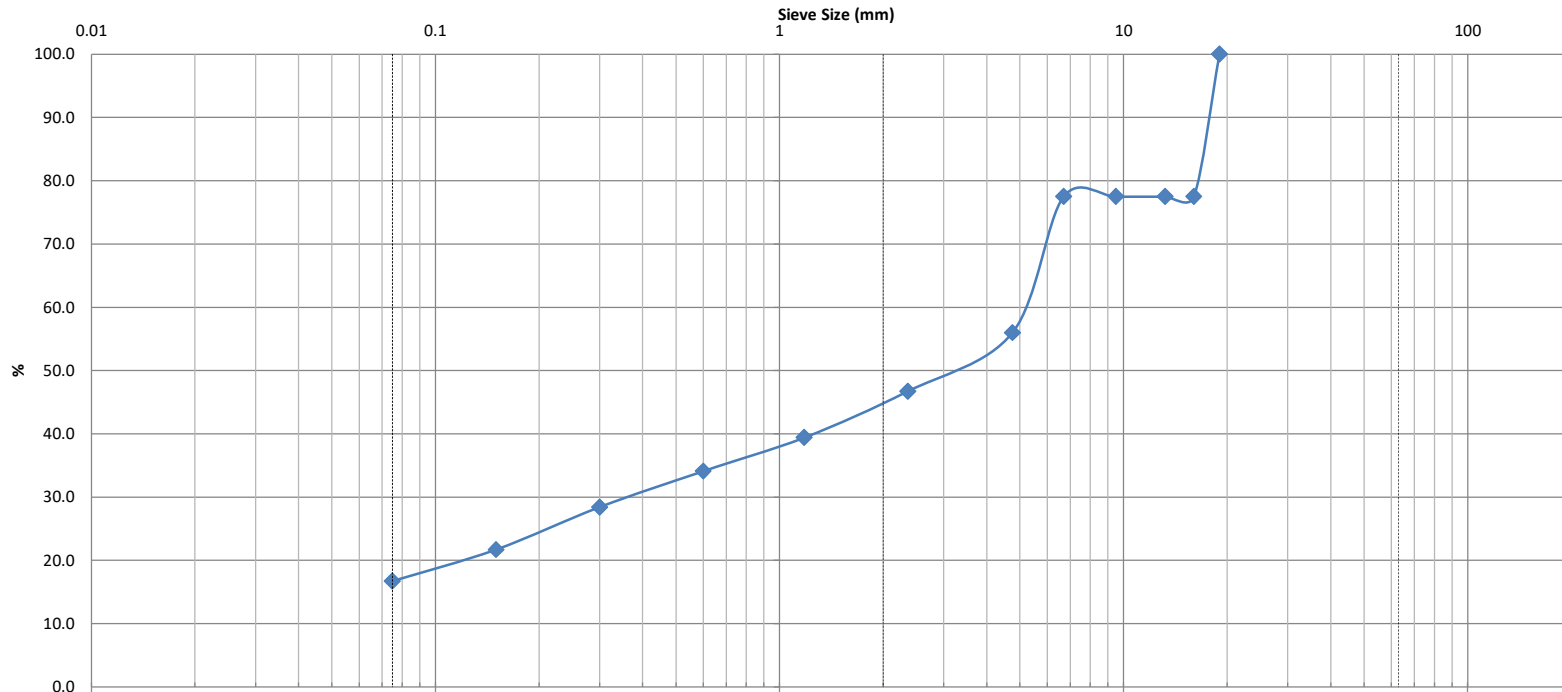
Sheet: 1 of 1

APPENDIX III
Laboratory Testing Reports for Soil Samples



**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Pinchin	DESCRIPTION:	Fine Aggregate	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	Silty Sand w Gravel	LAB NO:	49752
PROJECT:	327551	INTENDED USE:	-	DATE RECEIVED:	26-Oct-23
		PIT OR QUARRY:	-	DATE TESTED:	27-Oct-23
DATE SAMPLED:	-	SOURCE LOCATION:	BH5 SS2	DATE REPORTED:	6-Nov-23
SAMPLED BY:	Client	SAMPLE LOCATION:	2.5'-4.5'	TESTED BY:	CP



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
	19	5.2	0.37	0.034	44.0	39.3		16.7		0.77	152.9

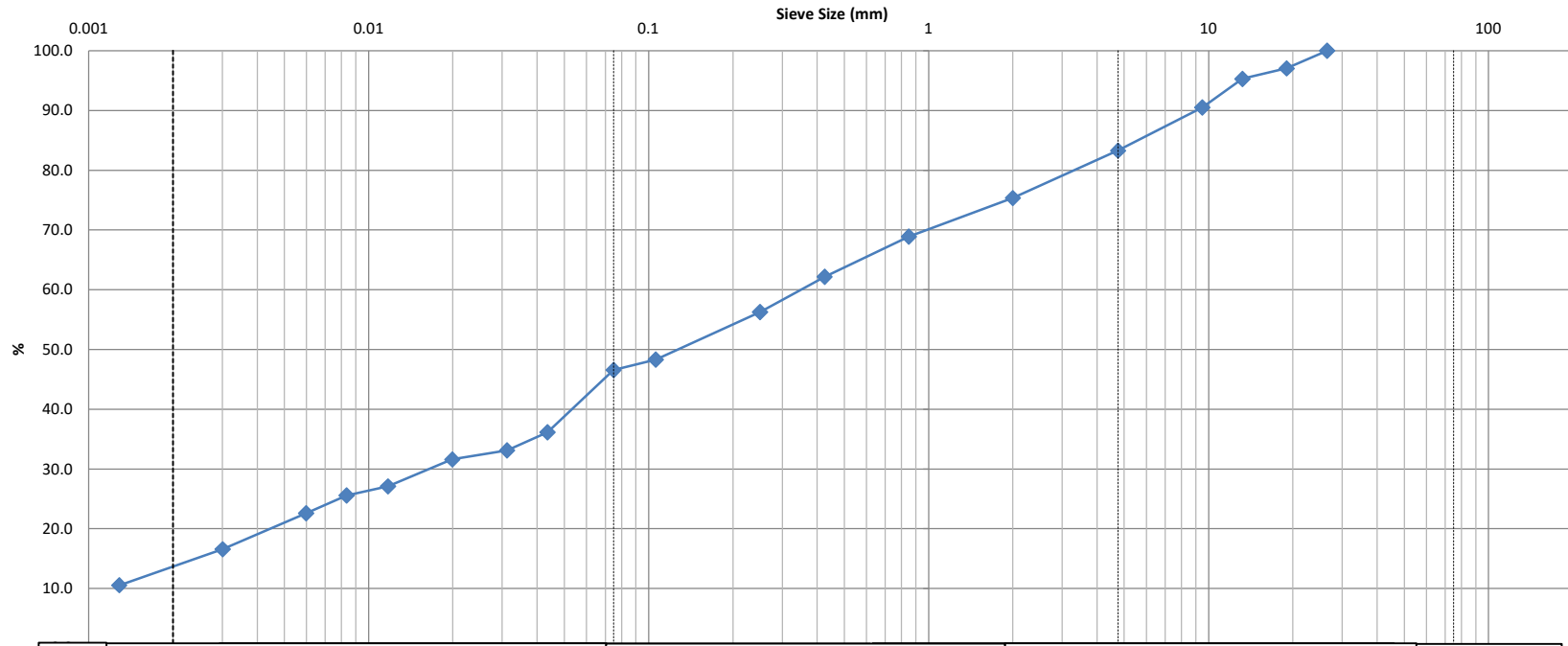
Comments:

REVIEWED BY:	Curtis Beadow		Joe Fosyth, P. Eng.	
	<i>[Signature]</i>		<i>[Signature]</i>	



**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Pinchin	DEPTH:	0.0 - 7.0	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH1 SS1	LAB NO:	49753
PROJECT:	327548			DATE RECEIVED:	26-Oct-23
DATE SAMPLED:	25-Oct-23			DATE TESTED:	30-Oct-23
SAMPLED BY:	Client			DATE REPORTED:	6-Nov-23
				TESTED BY:	DK



Clay	Silt			Sand			Gravel		Cobble
				Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
	D100	D60	D30	D10	Gravel (%)	10.0					
					16.7	Sand (%)	36.7	Silt (%)	32.6	Clay (%)	14.0

Comments:

REVIEWED BY:	Curtis Beadon		Joe Forsyth, P. Eng.	
	<i>[Signature]</i>		<i>[Signature]</i>	

APPENDIX IV
Report Limitations and Guidelines for Use

REPORT LIMITATIONS & GUIDELINES FOR USE

This information has been provided to help manage risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report was prepared for the exclusive use of the Client and their authorized agents, subject to the conditions and limitations contained within the duly authorized work plan. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of the third parties. If additional parties require reliance on this report, written authorization from Pinchin will be required. Pinchin disclaims responsibility of consequential financial effects on transactions or property values, or requirements for follow-up actions and costs. No other warranties are implied or expressed. Furthermore, this report should not be construed as legal advice.

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical report is based on the existing conditions at the time the study was performed, and Pinchin's opinion of soil conditions are strictly based on soil samples collected at specific test hole locations. The findings and conclusions of Pinchin's reports may be affected by the passage of time, by manmade events such as construction on or adjacent to the Site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations.

LIMITATIONS TO PROFESSIONAL OPINIONS

Interpretations of subsurface conditions are based on field observations from test holes that were spaced to capture a 'representative' snap shot of subsurface conditions. Site exploration identifies subsurface conditions only at points of sampling. Pinchin reviews field and laboratory data and then applies professional judgment to formulate an opinion of subsurface conditions throughout the Site. Actual subsurface conditions may differ, between sampling locations, from those indicated in this report.

LIMITATIONS OF RECOMMENDATIONS

Subsurface soil conditions should be verified by a qualified geotechnical engineer during construction. Pinchin should be notified if any discrepancies to this report or unusual conditions are found during construction.

Sufficient monitoring, testing and consultation should be provided by Pinchin during construction and/or excavation activities, to confirm that the conditions encountered are consistent with those indicated by the test hole investigation, and to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated. In addition, monitoring, testing and consultation by Pinchin should be completed to evaluate whether or not earthwork activities are completed in

accordance with our recommendations. Retaining Pinchin for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions. However, please be advised that any construction/excavation observations by Pinchin is over and above the mandate of this geotechnical evaluation and therefore, additional fees would apply.

MISINTERPRETATION OF GEOTECHNICAL ENGINEERING REPORT

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having Pinchin confer with appropriate members of the design team after submitting the report. Also retain Pinchin to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having Pinchin participate in pre-bid and preconstruction conferences, and by providing construction observation. Please be advised that retaining Pinchin to participation in any 'other' activities associated with this project is over and above the mandate of this geotechnical investigation and therefore, additional fees would apply.

CONTRACTORS RESPONSIBILITY FOR SITE SAFETY

This geotechnical report is not intended to direct the contractor's procedures, methods, schedule or management of the work Site. The contractor is solely responsible for job Site safety and for managing construction operations to minimize risks to on-Site personnel and to adjacent properties. It is ultimately the contractor's responsibility that the Ontario Occupational Health and Safety Act is adhered to, and Site conditions satisfy all 'other' acts, regulations and/or legislation that may be mandated by federal, provincial and/or municipal authorities.

SUBSURFACE SOIL AND/OR GROUNDWATER CONTAMINATION

This report is geotechnical in nature and was not performed in accordance with any environmental guidelines. As such, any environmental comments are very preliminary in nature and based solely on field observations. Accordingly, the scope of services do not include any interpretations, recommendations, findings, or conclusions regarding the, assessment, prevention or abatement of contaminants, and no conclusions or inferences should be drawn regarding contamination, as they may relate to this project. The term "contamination" includes, but is not limited to, molds, fungi, spores, bacteria, viruses, PCBs, petroleum hydrocarbons, inorganics, pesticides/insecticides, volatile organic compounds, polycyclic aromatic hydrocarbons and/or any of their by-products.

Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be held liable for damages resulting from the negligence of Pinchin. Pinchin will not be liable for any losses or damage if the Client has failed, within a period of two years following the date upon which the claim is discovered within the meaning of the Limitations Act, 2002 (Ontario), to commence legal proceedings against Pinchin to recover such losses or damage.