Geotechnical Engineering

Environmental Engineering

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Geotechnical Investigation

Proposed Residential Development
585 Bobolink Ridge - Ottawa

Prepared For

Tamarack (Fernbank) Corp. c/o HP Urban Inc.

July 5, 2021

Report PG5858-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Tamarack (Fernbank) Corp. to conduct a geotechnical investigation for the proposed residential development to be located at 585 Bobolink Ridge in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of test holes.

- ❑ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

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

2.0 Proposed Development

Based on available site plans, it is understood that the proposed development will consist of townhouse-style residential dwellings of slab-on-grade construction or with a basement level. Associated roadways and landscaped areas are also anticipated for the development. It is further anticipated that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on June 11, 2021. At that time, a total of four (4) boreholes were advanced to a maximum depth of 3.8 m below the existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site taking into consideration underground utilities and site features. The locations of the test holes are shown on Drawing PG5858-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed with a track mounted power auger drill rig operated by a two-person crew. The test hole procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden.

All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer.

Sampling and In Situ Testing

Soil samples were collected 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. The auger and split-spoon samples were placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU, and SS 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. This testing was done in general accordance with ASTM D1586-11 - Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

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

Groundwater

Flexible standpipe piezometers were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

Sample Storage

All samples 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 location and ground surface elevation at each test hole location were surveyed by Paterson personnel and referenced to a geodetic datum using a GPS unit. The test hole locations and ground surface elevation at the test hole locations are presented on Drawing PG5858-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples recovered from the subject site were visually examined in our laboratory to review the field logs.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the sulphate potential against subsurface concrete structures. The results are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by gravel access lanes and grass covered areas. The site is relatively flat and approximately at grade with surrounding properties and roadways. The subject site is bordered by residential dwellings to the west, undeveloped land to the north, Robert Grant Avenue to the east and Bobolink Ridge to the south. The geodetic elevation of the existing ground surface across the subject site is approximately 108 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consisted of a 1.1 to 2.6 m thick fill layer which was generally observed to consist of a brown silty sand with gravel, cobbles, boulders, trace clay and trace organics. A 1.3 to 2.7 m thick deposit of glacial till was encountered underlying the fill in BH 3 and BH 4. The glacial till deposit was generally observed to consist of a brown silty sand with gravel, cobbles, boulders, and trace clay. Practical refusal to augering on inferred bedrock was encountered at all test hole locations at depths ranging from 1.7 to 3.8 m below the ground surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the subject site consists of interbedded dolostone and limestone of the Gull River formation with an anticipated drift thickness of 0 to 1 m.

4.3 Groundwater

Groundwater levels were measured in the piezometers in all boreholes on June 16, 2021.The measured groundwater level (GWL) readings are presented in Table 1 on the next page.

Based on field observations of the recovered soil samples, such as moisture levels, colouring and consistency, the long-term groundwater level is anticipated to be located within the bedrock.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Levels

Borehole Number	Ground Surface Elev. (m)	Measured Groundwater Level		Recording Date
		Depth (m)	Elevation (m)	
BH 1	108.32	Dry	n/a	June 16, 2021
BH 2	108.32	Dry	n/a	June 16, 2021
BH 3	108.53	Dry	n/a	June 16, 2021
BH 4	108.01	Dry	n/a	June 16, 2021

Note: Ground surface elevations at test hole locations were surveyed by Paterson and are referenced to a geodetic datum.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is expected that the proposed residential dwellings will be founded on conventional spread footing foundations placed on a approved engineered fill, glacial till, or clean, surface-sounded bedrock bearing surface.

Due to the shallow depth to bedrock encountered throughout the subject site, bedrock removal is anticipated for future service alignments and possibly for the building construction.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, and any deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. The existing fill, where free of organics and deleterious materials, should be proof-rolled by a suitably sized vibratory roller making several passes and approved by Paterson personnel. Areas with poor performing fill should be removed and reinstated with a compacted engineered fill as detailed below.

It is anticipated that the existing fill, free of significant amounts of deleterious material and organics, can be left in place below the proposed building footprint beyond the lateral support zones for footings.

However, it is recommended that the floor slab subgrade surface be sub-excavated a minimum of 300 mm below the underside of the proposed slab and proof-rolled **under dry conditions and above freezing temperatures** by an adequately sized vibratory roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by Paterson personnel at the time of construction. In poor performing areas the fill material should be removed and reinstated with an approved engineered fill, such as OPSS Granular B Type II.

The approved fill subgrade for floor slabs should be covered with a non-woven geotextile, such as Terrafix 270R or equivalent other approved by Paterson, to prevent the subgrade particles from migrating through the voids within the upper compacted engineered fill.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and 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 velocities (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 also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. Footings that are anticipated to be placed on a near-vertical ledge or side wall of bedrock at the time of construction should be reviewed by Paterson personnel to review the suitability for the near-vertical bedrock ledge to support the proposed structure. Improvements such as providing additional lateral support by placement of approved engineered fill for moderately weathered or fractured bedrock may be required and will be verified for applicability at the time of construction.

Vibration Considerations

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

The following construction equipments 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 buildings.

Fill Placement

Fill placed for grading beneath the proposed structure(s) or other settlement sensitive areas should consist of clean imported granular fill unless otherwise specified, 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 engineered fill should be placed in maximum 300 mm thick lifts and compacted using suitable compaction equipment for the specified lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where surface settlement is a minor concern. The backfill materials should be spread in thin lifts and at a minimum compacted by the tracks of the spreading equipment to minimize voids. If the non-specified backfill is to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 98% of the material's SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. This material should be used structurally only to build up the subgrade for roads and paved areas. Where the fill is open-graded, a binding layer of finer granular fill or a woven geotextile, such as Terratrack 200 or equivalent, may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be determined at the time of construction.

Proof-Rolling

For the future driveways and roadways, proof-rolling of the subgrade is required in areas where existing fill, free of significant amounts of organics and deleterious materials, is encountered. It is recommended that the subgrade surface be proof-rolled **under dry and above freezing temperatures** by an adequately sized roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by Paterson personnel at the time of construction.

5.3 Foundation Design

Using continuously applied loads, footings can be designed using the bearing resistance values presented in Table 2.

Table 2 - Bearing Resistance Values

Bearing Surface	Bearing Resistance Values at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Compact Glacial Till	150	225
Engineered Fill Pad	150	225
Sound Bedrock	-	500

Note: A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance values at ULS.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil or clean, sound bedrock bearing surfaces. An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings. A clean surface-sounded bedrock bearing surface should be free of loose materials, and have no near or surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Footings designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively. Footings bearing on a clean, surface-sounded bedrock and designed using the above-noted bearing resistance values will be subjected to negligible post-construction total an differential settlements. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing medium to reduce the potential long-term total and differential settlements. Also, at the soil/bedrock transitions, it is recommended that a minimum depth of 500 mm of bedrock be removed from below the founding elevation for a minimum length of 2 m on the bedrock side. This area should be subsequently reinstated with an engineered fill, such as OPSS Granular A or Granular B Type II and compacted to a minimum of 98% of the material SPMDD.

The width of the sub-excavation should be a minimum of 500 mm greater than the width of the footing. Steel reinforcement, extending a minimum of 3 m on both sides of th 2 m long transition, should be placed in the top portions of the footing and foundation walls.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support. Adequate lateral support is provided to a compact silty sand or glacial till above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in-situ soil of the same or higher capacity as the bearing medium soil.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock. A heavily fractured, weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Permissible Grade Raise Restrictions

Due to the absence of a cohesive soil deposit, a permissible grade raise restriction is not considered applicable for the subject site from a geotechnical perspective.

5.4 Design for Earthquakes

The subject site can be taken as seismic site response **Class C** as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012 for foundations considered at this site. A higher seismic class may be applicable, such as Class A or B, provided the footings are within 3 m of the bedrock surface. However, this would need to be confirmed by performing a seismic shear wave velocity test at the subject site.

The soils underlying the site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Pavement Structure

For design purposes, the following pavement structures presented below could be used for the design of driveways, local residential streets and roadways with bus traffic. It is anticipated that both pavement structures provided would be adequate for use as a fire route.

Table 3 - Recommended Pavement Structure - Driveways	
Thickness (mm)	**Material Description**
50	**Wear Course** - HL 3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill. **NOTE:** Minimum Performance Grade (PG) 58-34 asphalt cement should be used for driveways.	

Table 4 - Recommended Pavement Structure - Local Residential Roadways	
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
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill. **NOTE :** Minimum Performance Grade (PG) 58-34 asphalt cement should be used for local roadways.	

Table 5 - Recommended Pavement Structure Local Collector Roadways with Bus Traffic	
Thickness (mm)	**Material Description**
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Upper Binder Course** - Superpave 19.0 Asphaltic Concrete
50	**Lower Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
550	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill. **NOTE:** Minimum Performance Grade (PG) 64-34 asphalt cement should be used for roadways with bus traffic.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for driveways and local roadways and PG 64-34 asphalt cement should be used for roadways with bus traffic. Paving is to be completed in accordance with applicable City of Ottawa standards. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be subexcavated and replaced with OPSS Granular B Type I or Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase, or other measures that can be recommended at the time of construction as part of the field observation program.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 100 to 150 mm diameter, geotextile-wrapped, perforated and corrugated plastic pipe surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill 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 Miradrain G100N) connected to a drainage system is provided.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

Where footings are founded directly on clean, surface-sounded bedrock with no near-surface cracks or fissures, and is approved by Paterson personnel at the time of the excavation, the minimum soil cover, listed above, is not required.

If the bedrock is cracked or fissured, and is considered to be frost susceptible, the following measures should be implemented at the time of construction on an individual lot basis:

- ❏ Option A - Sub-excavate the weathered bedrock to sound bedrock or to the required frost cover depth. Pour footings at the lower level.

- ❏ Option B - Use insulation to protect footings. It is preferable to pour footings on the insulation overlying weathered bedrock. However, due to the potential undulating bedrock surface, consideration may have to be given to adopting an insulation detail that allows the footings to be poured directly on the weathered bedrock.

Exterior unheated footings, such as those for isolated exterior 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 a combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

Temporary Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavations to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should 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.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material for areas over a soil subgrade. However, the bedding thickness should be increased to 300 mm for areas over a bedrock subgrade, if encountered. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at a minimum to the spring line of the pipe.

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

Generally, it should be possible to re-use the moist (not wet) glacial till above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet sub-excavated soil should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used. All stones greater than 300 mm in their greatest dimension should be removed prior to reuse of site-generated glacial till.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should consist of the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the SPMDD.

6.5 Groundwater Control

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. It is anticipated that groundwater infiltration into the excavations should be moderate, if encountered, and controllable using open sumps.

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

For typical ground or surface water volumes being pumped during the construction phase, 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.

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. Provisions in the contract documents should be provided to protect the excavation walls from freezing, if applicable.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and tarpaulins or other suitable means. The excavation base 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.

The trench excavations should be constructed to avoid the introduction of frozen materials, snow or ice into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving during construction. Also, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure.

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ 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 and proof rolling.
- ❑ 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 in the report are in accordance with Paterson's present understanding of the project. Paterson request permission to review the recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions encountered during construction differ from the test pit locations, Paterson requests immediate notification to permit reassessment of the recommendations provided herein.

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

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 Tamarack (Fernbank) Corp. c/o HP Urban Inc. or their agent(s) is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Nicole Patey B.Eng.

David J. Gilbert P.Eng.

Report Distribution:

❏ Tamarack (Fernbank) Corp. c/o HP Urban Inc (1 digital copy)
❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 585 Bobolink Ridge
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 11, 2021

FILE NO. PG5858

HOLE NO. BH 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	108.32		
		AU	1						
FILL: Brown silty sand with gravel, cobbles, boulders and rock fragments		SS	2	36	50+				
						1	107.32		
1.65		SS	3	0	50+				
End of Borehole									
Practical refusal to augering at 1.65m depth.									
(Piezometer dry - June 16, 2021)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 585 Bobolink Ridge
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 11, 2021

FILE NO. PG5858

HOLE NO. BH 2-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	108.32
FILL: Dark brown silty sand with crushed stone (to 0.30)		AU	1				
FILL: Brown silty sand with gravel, trace clay, occasional cobbles and boulders		SS	2	50	71	1	107.32
		SS	3	27	50+		
						2	106.32
(to 2.57)		SS	4	33	50+		
End of Borehole							

Practical refusal to augering at 2.57m depth.

(Piezometer dry - June 16, 2021)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 585 Bobolink Ridge
Ottawa, Ontario

DATUM Geodetic

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BORINGS BY Track-Mount Power Auger

DATE June 11, 2021

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand, some organics	0.10	AU	1		
FILL: Brown silty sand with gravel and cobbles, occasional boulders and rock fragments	1.83	SS	2	17	21
		SS	3	27	50+
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders, trace clay	3.10	SS	4	50	71
		SS	5	50	50+

DEPTH (m)	ELEV. (m)
0	108.53
1	107.53
2	106.53
3	105.53

End of Borehole

Practical refusal to augering at 3.10m depth.

(Piezometer dry - June 16, 2021)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 585 Bobolink Ridge
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 11, 2021

FILE NO. PG5858

HOLE NO. BH 4-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	108.01
FILL: Dark brown silty sand, trace organics		AU	1				
1.07		SS	2	67	26	1	107.01
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders, trace clay		SS	3	75	68	2	106.01
		SS	4	40	50+		
		SS	5	33	35	3	105.01
3.81							

End of Borehole

Practical refusal to augering at 3.81m depth.

(Piezometer dry - June 16, 2021)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

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)

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

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

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

CONSOLIDATION TEST

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

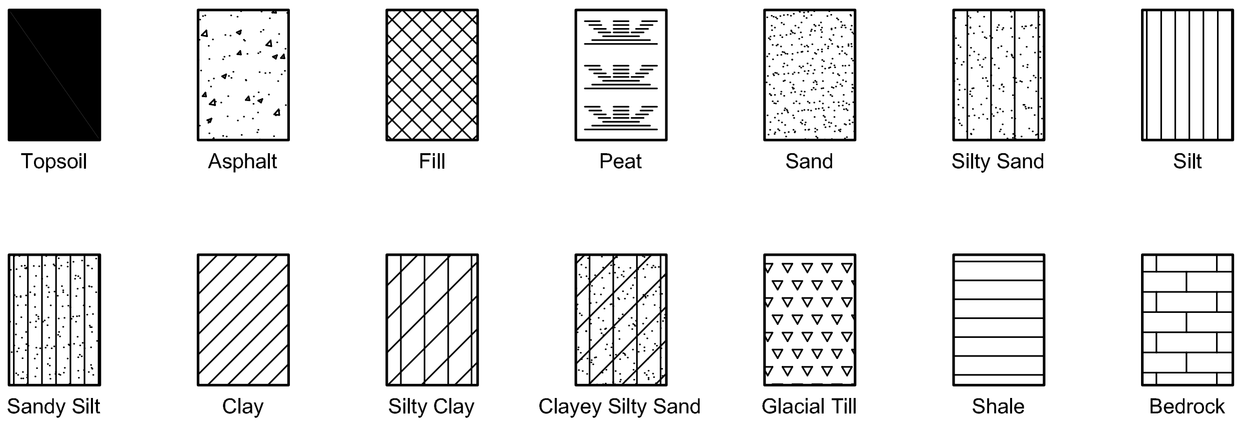
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

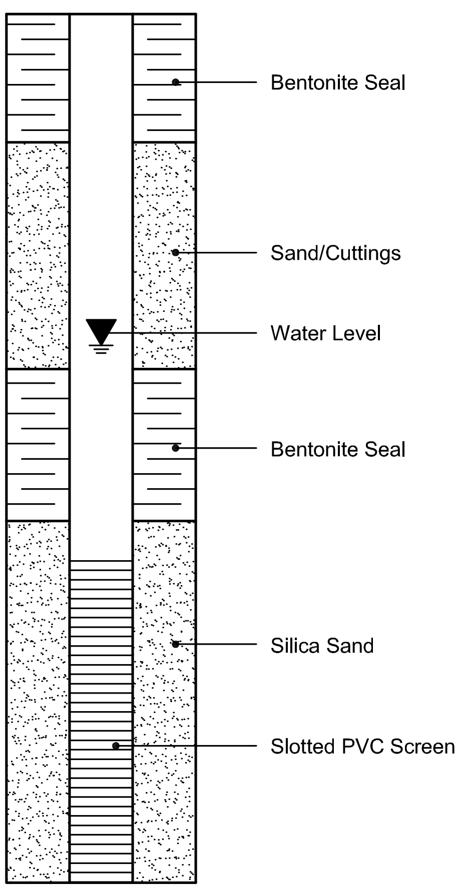
SYMBOLS AND TERMS (continued)

STRATA PLOT

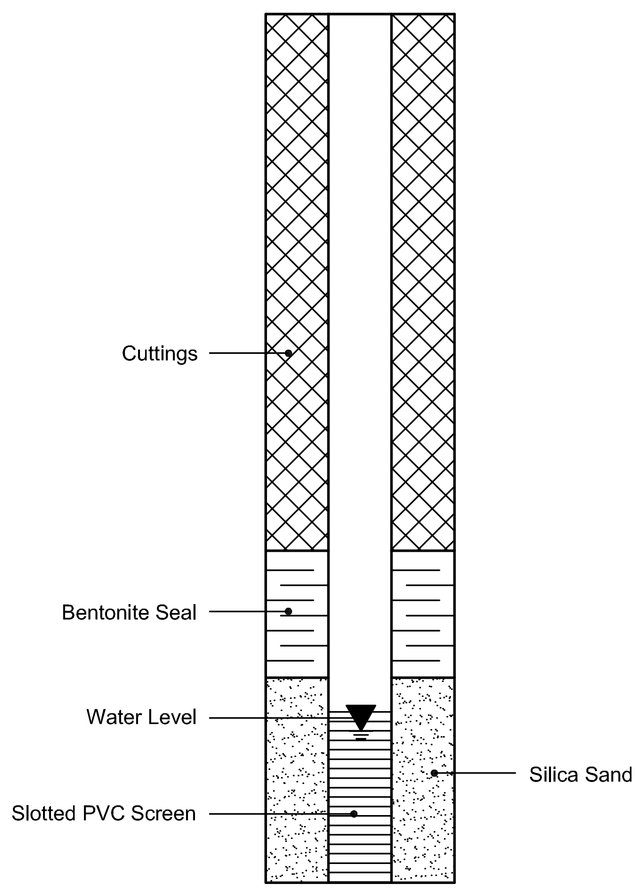


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 32182

Report Date: 21-Jun-2021

Order Date: 15-Jun-2021

Project Description: PG5858

	Client ID:	BH4-21 SS3	-	-	-
	Sample Date:	11-Jun-21 09:00	-	-	-
	Sample ID:	2125236-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	92.0	-	-	-
General Inorganics					
pH	0.05 pH Units	7.55	-	-	-
Resistivity	0.10 Ohm.m	69.8	-	-	-
Anions					
Chloride	5 ug/g dry	9	-	-	-
Sulphate	5 ug/g dry	41	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5858-1 - TEST HOLE LOCATION PLAN

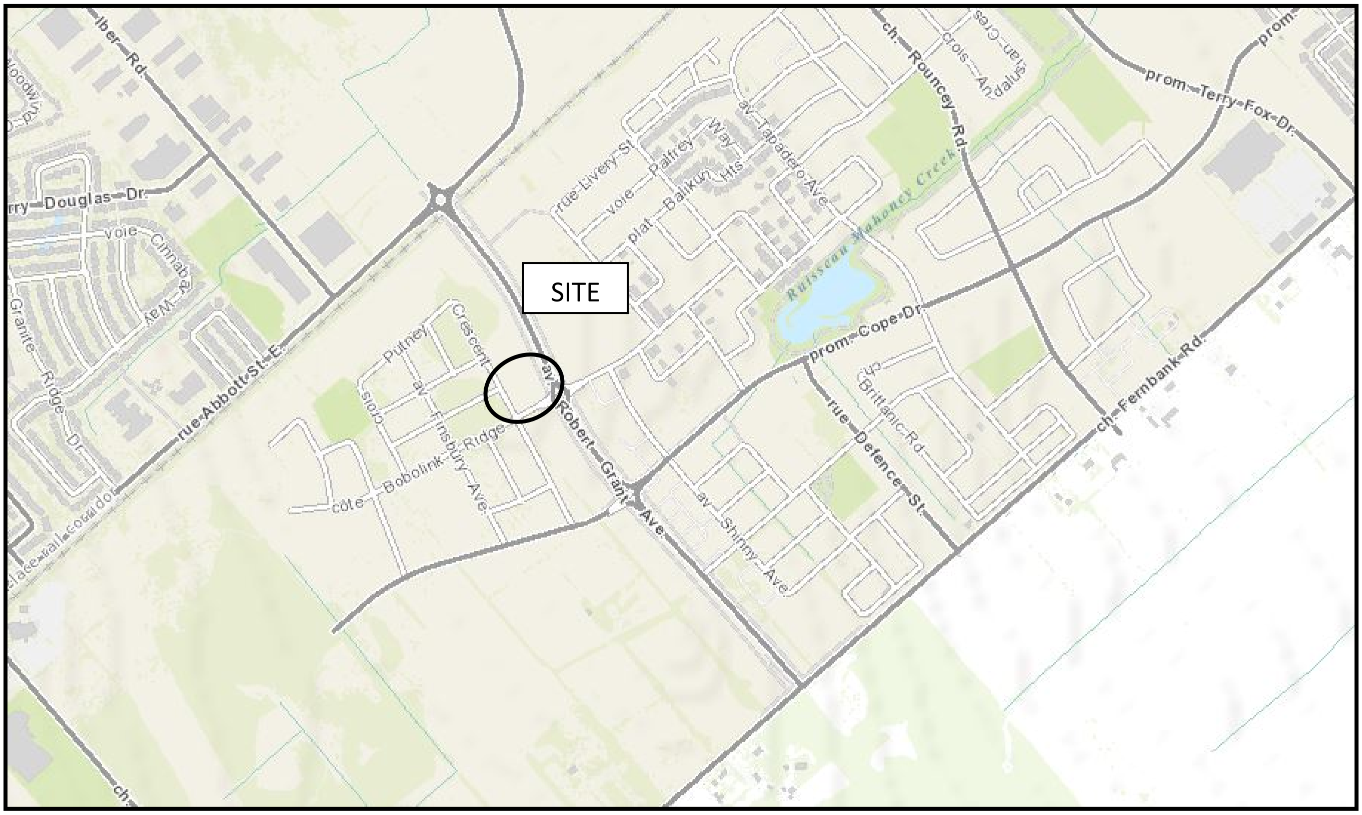
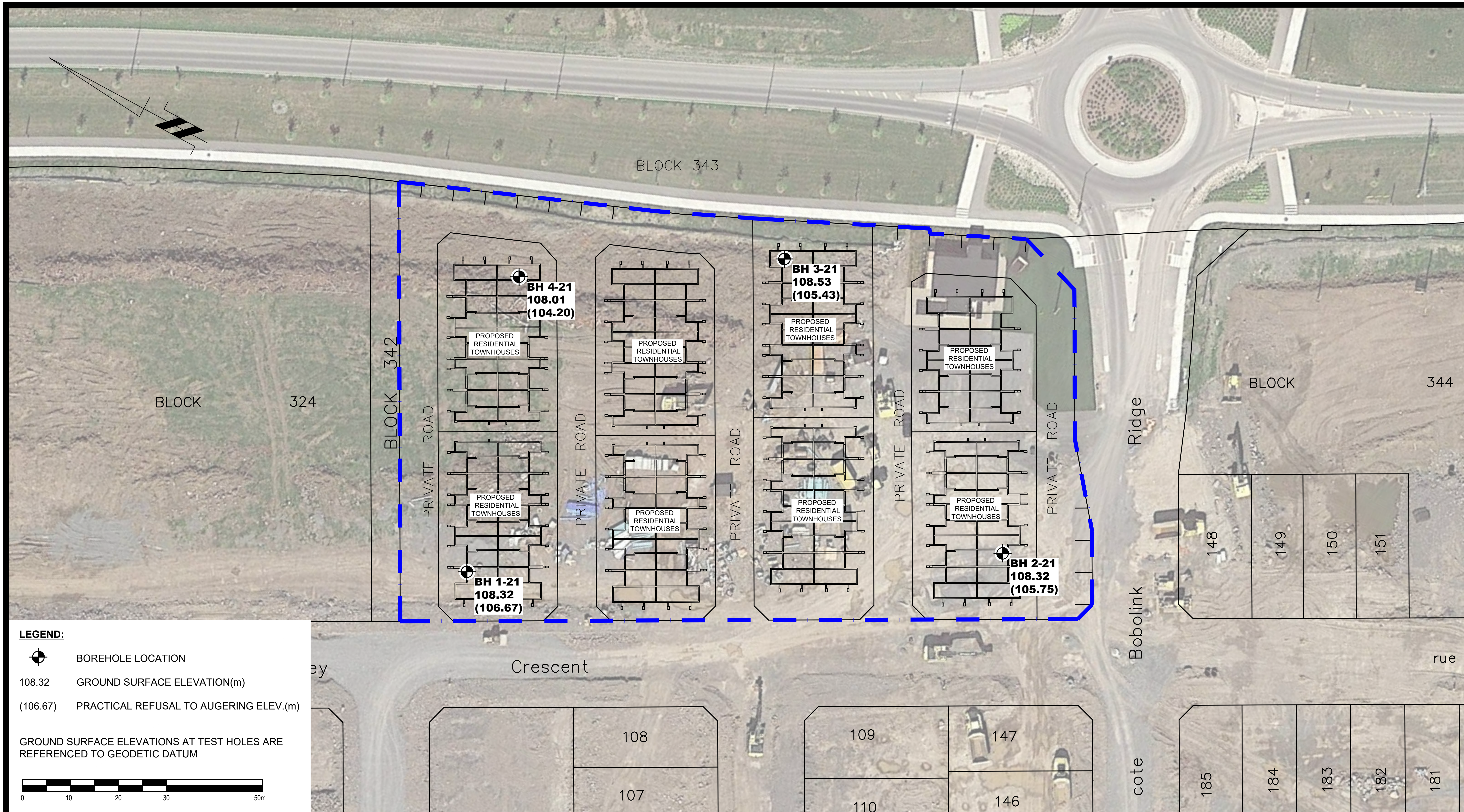


FIGURE 1

KEY PLAN



LEGEND:

⌖	BOREHOLE LOCATION
108.32	GROUND SURFACE ELEVATION(m)
(106.67)	PRACTICAL REFUSAL TO AUGERING ELEV.(m)

GROUND SURFACE ELEVATIONS AT TEST HOLES ARE REFERENCED TO GEODETIC DATUM

0 10 20 30 50m

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NO.	REVISIONS	DATE	INITIAL
0			

TAMARACK (FERNBANK) CORP. C/O HP URBAN INC.

GEOTECHNICAL INVESTIGATION - PROPOSED RESIDENTIAL DEVELOPMENT
585 BOBOLINK RIDGE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 06/2021
Drawn by: RCG	Report No.: PG5858-1
Checked by: NP	Dwg. No.: **PG5858-1**
Approved by: DJG	Revision No.: 0

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