PATERSON GROUP

Geotechnical Investigation Update
Proposed Residential Building

2928 Bank Street
Ottawa, Ontario

Prepared for VIP Construction & Engineering

Report PG7073-1 dated April 1, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by VIP Construction & Engineering to conduct a geotechnical investigation for the proposed residential building to be located at 2928 Bank Street in the City of Ottawa (reference should be made to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

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

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

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

2.0 Proposed Development

Based on available preliminary architectural drawings, it is understood that the future development will generally consist of a four-storey apartment building with one basement level. Associated access lanes, parking areas, landscaped and hardscaped areas are also anticipated as part of the development. It is further understood that the subject site is municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was conducted on April 4, 2012, and consisted of advancing six (6) boreholes to a maximum depth of 6.7 m spaced across the subject site to provide general coverage of the proposed development. The locations of the test holes are shown on Drawing PG7073-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of Paterson's geotechnical division under the direction of a senior engineer. The drilling procedure consisted of excavating or augering to the required depths at the selected locations and sampling the overburden.

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. All samples were visually inspected and initially classified on site. 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. Further, photographs of the rock core are presented in Appendix 1.

The 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.

Overburden thickness was also evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at BH 1. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

Flexible standpipes were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected in the field by Paterson personnel to provide general coverage of the proposed development taking into consideration site features and underground utilities. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top of spindle of a fire hydrant located at the southwest corner of Bank Street and Kingsdale Avenue. An arbitrary elevation of 100.00 was assigned to the TBM. The borehole locations, TBM and the ground surface elevations of the test hole locations are presented on Drawing PG7073-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

Based on historical aerial photographs, the subject site was occupied by a single storey commercial building surrounded by asphaltic car parking and landscaping areas until 2017. The site was relatively flat and at grade with the surrounding properties to the south and west of the subject site. The adjacent property to the north and Bank Street bordering the subject site to the east are slightly elevated above the subject site. However, during the previous construction phase, it is understood that the asphaltic area has been removed and the half west portion of the site has been excavated to a depth of 1.5 to 2 m below the ground surface. It is further understood that the existing single-storey commercial building is not demolished.

Historical aerial photographs of the subject site and its surroundings are provided in Figures 2, and 3 – Aerial Photographs, in Appendix 2.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of a thin layer of asphaltic concrete and/or granular crushed stone fill overlying a compact brown silty sand deposit. A brown silty sand crust was observed overlaying a compact to dense, grey silty sand deposit at approximate depth of 2.2 below the ground surface.

Practical refusal to DCPT testing was encountered at depth of 8.4 m below the existing ground surface at the location of BH 1.

Specific details of the soil profile encountered at the test hole locations are presented on the Soil Profile and Test Data sheets provided in Appendix 1.

Bedrock

Based on available bedrock mapping, Shale of the Carlsbad formation is present in the area with an overburden drift thickness of approximately 15 to 25 m.

4.3 Groundwater

Groundwater levels were measured in the standpipes installed in the boreholes on April 16, 2012, and are presented in Table 1.

Table 1 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1	98.64	Blocked	N/A	April 16, 2012
BH 2	98.33	3.20	95.13	April 16, 2012
BH 3	97.98	3.05	94.93	April 16, 2012
BH 4	98.41	3.27	95.14	April 16, 2012
BH 5	98.26	Blocked	N/A	April 16, 2012
BH 6	98.05	Blocked	N/A	April 16, 2012

Note: The ground surface elevations at the borehole locations were referenced to a TBM, consisting of the top of spindle of a fire hydrant located at the southwest corner of Bank Street and Kingsdale Avenue. An arbitrary elevation of 100.00 m was assigned to the TBM.

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations.

The Long-term groundwater levels can also be estimated based on the observed color, consistency, and moisture content of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at an approximate depth of **2.0 to 2.5 m** below the ground surface. Groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

Based on our findings, the subject site is adequate for the proposed residential building from a geotechnical perspective. It is anticipated that the proposed building will be founded on conventional style footings placed on an undisturbed compact silty sand bearing surface.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, asphalt, and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials should be spread in a maximum of 300 mm thick loose lifts and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in maximum 300 mm thick loose lifts to at least 98% of the material's SPMDD. The placement of subgrade material should be reviewed at the time of placement by Paterson personnel. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved by Paterson prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 100% of its SPMDD.

5.3 Foundation Design

Bearing Resistance Value (Conventional Shallow Foundation)

Footings, placed directly over an undisturbed, compact silty sand bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **175 kPa**, incorporating a geotechnical resistance factor of 0.5.

Footings, placed directly over an undisturbed, dense silty sand bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **120 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **200 kPa**, incorporating a geotechnical resistance factor of 0.5.

Where the silty sand bearing surface is found to be in a loose state of compactness, the area should be proof rolled using a vibratory compactor and approved by the geotechnical consultant prior to placing footings. The compaction efforts should be completed under dry conditions and above freezing temperatures.

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.

Settlement

Footings bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided above will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Adequate lateral support is provided to a compact to very dense glacial till above the groundwater table or engineered fill placed over glacial till when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil of the same or higher capacity as that of the bearing medium.

Raft Foundation

Based on the expected loads from the proposed structure, a raft foundation bearing on an undisturbed, compact to dense silty sand bearing surface may be considered for foundation support for the subject proposed building. For design purposes, it was assumed that the base of the raft foundation would be located at an approximate depth of 3 to 4 m since there would be one basement level provided to the subject building.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

For the raft slab foundation, a bearing resistance value at SLS (contact pressure) of **175 kPa** will be considered acceptable for a raft supported on the undisturbed, compact silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **250 kPa**. For this case, the modulus of subgrade reaction was calculated to be **7 MPa/m** for a contact pressure of 165 kPa.

For the raft slab foundation, a bearing resistance value at SLS (contact pressure) of **195 kPa** will be considered acceptable for a raft supported on the undisturbed, dense silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **275 kPa**. For this case, the modulus of subgrade reaction was calculated to be **9 MPa/m** for a contact pressure of 185 kPa.

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS. Based on the following assumptions for the raft foundation, the proposed structure can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

End Bearing Pile Foundation

If the raft slab bearing resistance values are insufficient for the proposed building, a deep foundation system driven to refusal in the bedrock will be recommended for foundation support of the proposed building. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance values at SLS and ULS are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	940	1130	10	29
245	11	1175	1410	10	35
245	13	1375	1650	10	42

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

The minimum centre-to-centre pile spacing is 2.5 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Buildings founded on piles driven to refusal in the bedrock will have negligible post-construction settlement.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the foundations considered at this site. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab/Slab on Grade Construction

With the removal of all topsoil and deleterious materials within the footprint of the proposed residential building, the native soil surface or approved engineered fill surface will be considered an acceptable subgrade on which to commence backfilling for slab on grade construction. Where the subgrade consists of existing fill, a vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as Granular B Type II.

Where existing fill, free of deleterious material and significant organic content, is encountered below the floor slab, provisions should be made to removing the existing fill from within the building footprint and replacing the fill with OPSS Granular A or Granular B Type II compacted to a minimum 98% of the material's SPMDD.

All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD. An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings. Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

5.6 Pavement Design

For design purposes, the following pavement structures, presented below, are recommended for the design of car only parking areas, and access lanes at the subject site.

Table 3 - Recommended Pavement Structure - Car-Only Parking Areas	
Thickness (mm)	**Material Description**
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

Table 4 - Recommended Pavement Structure - Access Lanes	
Thickness (mm)	**Material Description**
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terratrack 200 or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

The following recommendations may be considered for the architectural design of the building's foundation drainage systems. It is recommended that Paterson be engaged at the design stage of the future building (and prior to tender) to review and provide supplemental information for the building foundation drainage system design.

Supplemental details, review of architectural design drawings, and additional information may be provided by Paterson for these items for incorporation in the building design packages and associated tender documents. It is recommended that Paterson review all details associated with the foundation drainage system prior to tender.

Groundwater Suppression System

It is recommended that a groundwater suppression system be provided for the proposed structure. It is expected that insufficient room will be available for exterior backfill and the foundation wall will be cast as a blind-sided pour against a shoring system and the bedrock surface. It is recommended that the groundwater suppression system consist of the following:

- A waterproofing membrane should be placed against the shoring system between the underside of footings and 1 m below the existing ground surface (1 m above long-term groundwater elevation). Where the membrane will extend against the shoring system, it is recommended to consist of a membrane with a bentonite-lined face for being paced against the shoring system. The membrane is recommended to overlap below the overlying perimeter foundation footprint by a minimum of 1 m inwards towards the building footprint and from the face of the overlying foundation. This will allow construction to proceed without imposing groundwater lowering within the surrounding area of the proposed building in the short and long term conditions.

- A composite drainage membrane (Delta Terraxx, MiraDrain G100N or equivalent) should be placed against the HDPE face of the waterproofing membrane with the geotextile layer facing the waterproofing layer from finished ground surface to the top of the footing.

- The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by the geotechnical consultant. It is highly recommended that the drainage board rolls be installed horizontally rather than vertically to minimize the number of vertical joints forming between the rolls.

- It is recommended that 150 mm diameter PVC sleeves at 6 m centers be cast in the foundation wall at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe system.

The top end lap of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and foundation wall.

Interior Perimeter and Underfloor Drainage

An interior perimeter and underfloor drainage system will be required to redirect water from the building's foundation drainage system to the building's sump pit(s) if it will not discharge to an exterior catch basin structure. For preliminary design purposes, it is recommended that the interior perimeter and underfloor drainage pipes should consist of 100 or 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock, placed at approximately 6 m.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves.

The spacing of the underfloor drainage should be confirmed by Paterson at the time of excavation when water infiltration can be better assessed and once the foundation layout and sump system location has been finalized.

Foundation Backfill

Where applicable, backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials.

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

Foundation backfill material should be compacted in maximum 300 mm thick loose lifts and with suitably sized vibratory compaction equipment (smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill).

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized structural and architectural drawings for the proposed building to provide a building-specific waterproofing and drainage design which includes the above-noted recommendations. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double sided pours (if applicable), etc.

6.2 Protection of Footings Against Frost Action

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

Other 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 proper structure. These footings should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either 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 excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

Unsupported Side Slopes

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff where shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction.

It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

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

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

6.4 Pipe Bedding and Backfill

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

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

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

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

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

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. Provisions should be carried out for using higher capacity open sump systems for excavations undertaken below the bedrock surface. The contractor should be prepared to direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Persons 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.

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

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

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Due to the absence of sensitive marine clays within the subject site, no tree planting restrictions are required for the proposed building. However, it is highly recommended that Paterson reviews the landscaping plans once available from a geotechnical perspective.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ❑ Review preliminary and detailed grading, servicing, landscaping and structural plan(s) from a geotechnical perspective.
- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if not design by Paterson, prior to construction, if applicable.
- ❑ Review of architectural plans pertaining to groundwater suppression system, underfloor drainage systems and waterproofing details for elevator shafts.

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

- ❑ Review and inspection of the installation of the foundation drainage systems.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Observation of driving and re-striking of all pile foundations.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ❑ 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.

All excess soil must be handled as per ***Ontario Regulation 406/19: On-Site and Excess Soil Management*.**

8.0 Statement of Limitations

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

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

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

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

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
April 1, 2024
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Yashar Ziaeimehr, M.A.Sc. Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ❑ VIP Construction & Engineering (Email Copy)
- ❑ Paterson Group (1 Copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2928 Bank Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Kingsdale Avenue and Bank Street. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

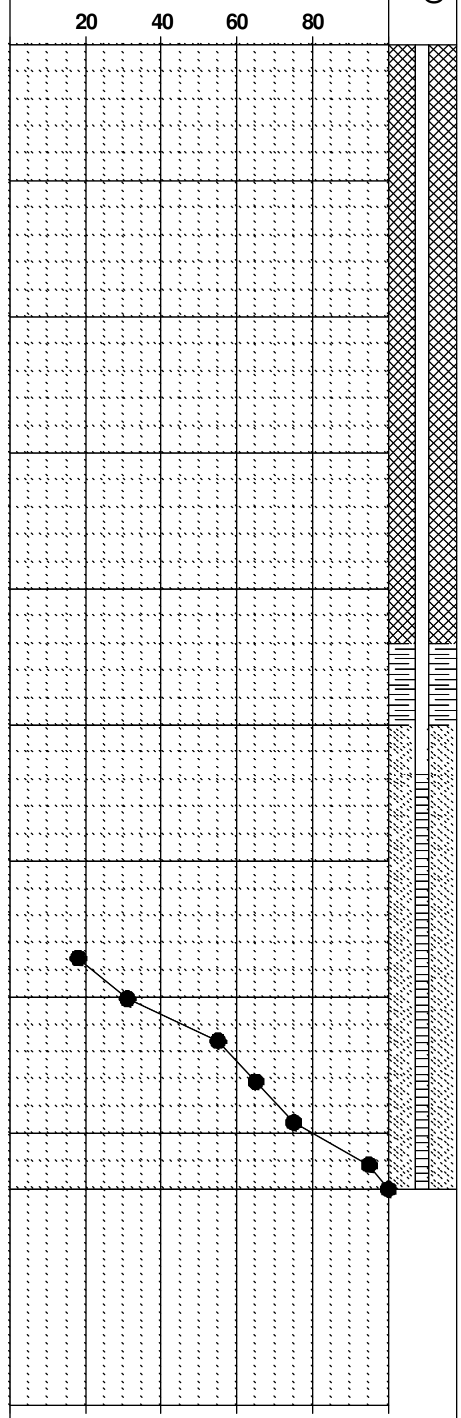
DATE April 4, 2012

FILE NO. PG2653

HOLE NO. BH 1

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	98.64
Asphaltic concrete 0.05						
FILL: Brown silty sand with gravel 0.41						
Loose to compact, brown **SILTY SAND**	SS	1	58	7	1	97.64
	SS	2	67	24	2	96.64
- compact to dense and grey by 2.2m depth	SS	3	67	25	3	95.64
	SS	4	67	31	4	94.64
	SS	5	67	39	5	93.64
	SS	6	75	40	6	92.64
	SS	7	100	15		
6.71	SS	8	100	28		
Dynamic Cone Penetration Test commenced @ 6.71m depth.					7	91.64
8.41					8	90.64
End of Borehole						
Practical refusal to DCPT at 8.41m depth						
(Piezometer blocked at 3.20m depth - April 16, 2012)						

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2928 Bank Street
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DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Kingsdale Avenue and Bank Street. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 4, 2012

FILE NO. PG2653

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.33
Asphaltic concrete 0.05							
FILL: Brown silty sand with gravel 0.60							
Loose to compact, brown **SILTY SAND**		SS	1	33	9	1	97.33
		SS	2	58	19	2	96.33
- compact to dense and grey by 2.2m depth		SS	3	67	29	3	95.33
		SS	4	58	25	4	94.33
		SS	5	33	45	5	93.33
		SS	6	42	40	6	92.33
6.22		SS	7	58	26		
End of Borehole							
(GWL @ 3.20m-April 16, 2012)							

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

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2928 Bank Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Kingsdale Avenue and Bank Street. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 4, 2012

FILE NO. PG2653

HOLE NO. BH 3

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.98
Asphaltic concrete	0.05						
FILL: Brown silty sand with gravel	0.60						
Compact, brown **SILTY SAND**		SS	1	83	12	1	96.98
		SS	2	50	18	2	95.98
- dense to very dense and grey by 2.2m depth		SS	3	75	39	3	94.98
		SS	4	58	50	4	93.98
		SS	5	50	45	5	92.98
		SS	6	17	83	6	91.98
	6.22	SS	7	50	38		
End of Borehole							
(GWL @ 3.05m-April 16, 2012)							

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2928 Bank Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Kingsdale Avenue and Bank Street. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 4, 2012

FILE NO. PG2653

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.41
Asphaltic concrete — 0.05							
FILL: Brown silty sand with gravel — 0.60							
		SS	1	50	6	1	97.41
Loose to compact, brown **SILTY SAND**		SS	2	75	14	2	96.41
- grey by 2.2m depth		SS	3	67	21	3	95.41
		SS	4	75	21	4	94.41
		SS	5	75	27	5	93.41
		SS	6	75	24	6	92.41
— 6.22		SS	7	67	14		
End of Borehole							
(GWL @ 3.27m-April 16, 2012)							

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

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2928 Bank Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Kingsdale Avenue and Bank Street. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 4, 2012

FILE NO. PG2653

HOLE NO. BH 5

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	98.26		
Asphaltic concrete 0.05									
FILL: Brown silty sand with gravel 0.48									
		SS	1	50	5	1	97.26		
Loose, brown **SILTY SAND**		SS	2	58	8	2	96.26		
- compact and grey by 2.2m depth		SS	3	67	26	3	95.26		
		SS	4	58	29				
3.73									
End of Borehole									
(Piezometer blocked at 2.86m depth - April 16, 2012)									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2928 Bank Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southwest corner of Kingsdale Avenue and Bank Street. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 4, 2012

FILE NO. PG2653

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	98.05		
Asphaltic concrete 0.05									
FILL: Brown silty sand with styrofoam		SS	1	33	10	1	97.05		
1.45									
Compact, brown **SILTY SAND** with gravel		SS	2	67	13				
2.20						2	96.05		
Compact, grey **SILTY SAND**		SS	3	42	17				
						3	95.05		
		SS	4	67	19				
3.73									
End of Borehole									
(Piezometer blocked at 2.88m depth - April 16, 2012)									

Water Content %: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ 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 strength of cohesionless soils is the relative density, 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.

Relative Density	'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 vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

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 NXL size core. However, it can be used on smaller core sizes, such as BX, 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
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture 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 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

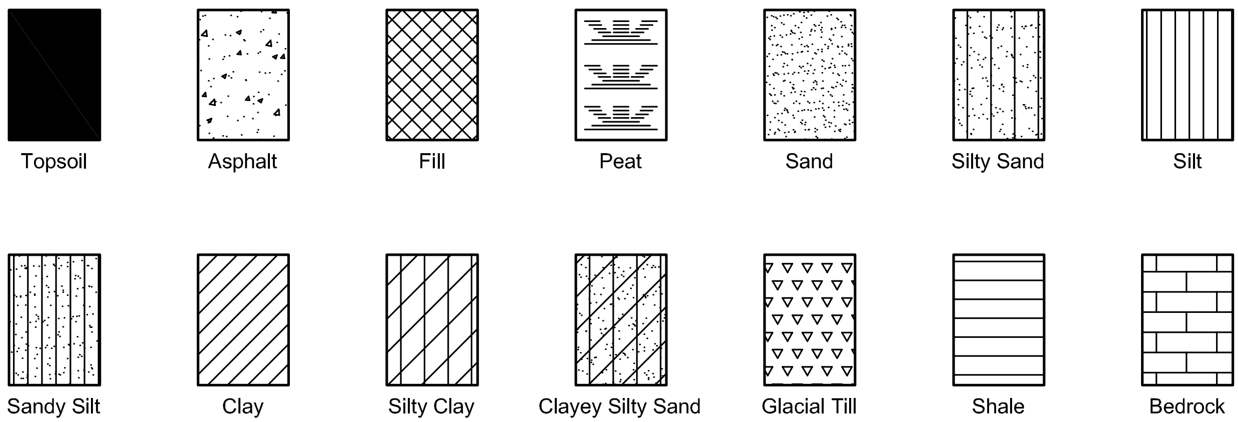
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

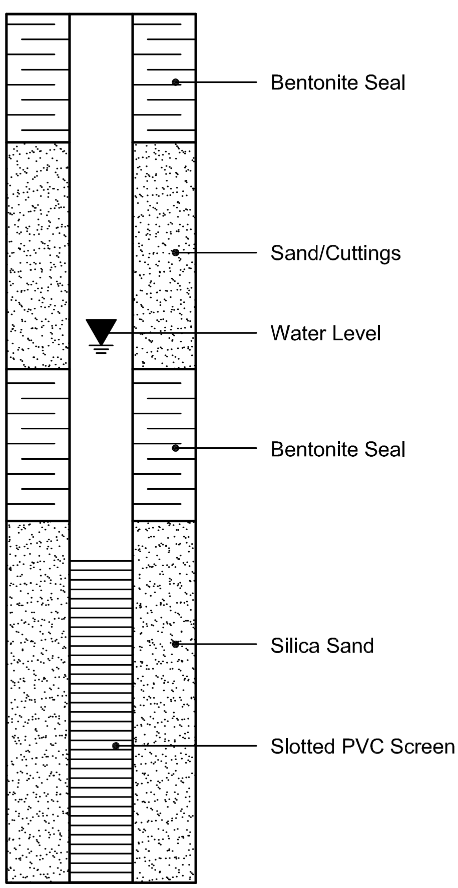
SYMBOLS AND TERMS (continued)

STRATA PLOT

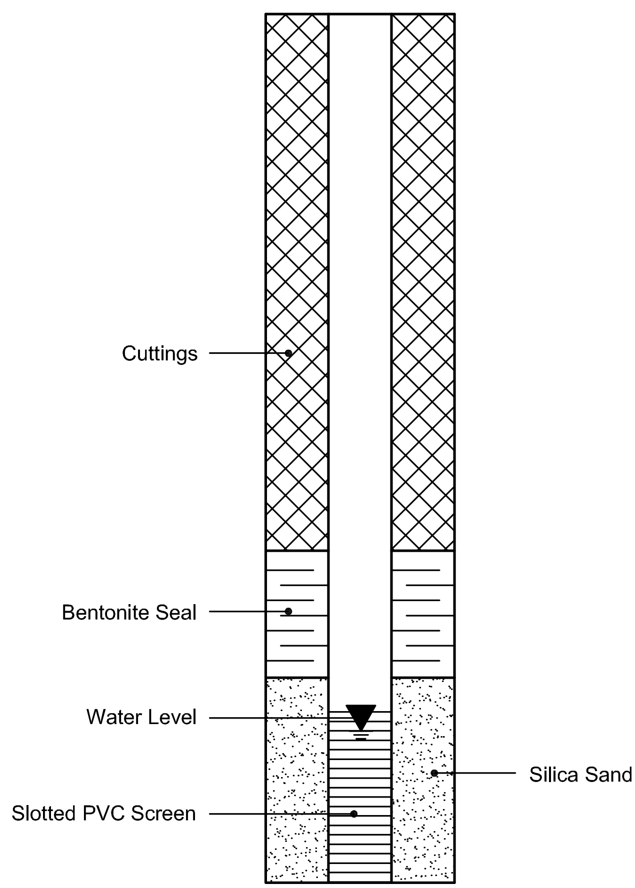


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 1215074

Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 12151

Project Description: PG2653

Report Date: 16-Apr-2012

Order Date:10-Apr-2012

	MDL/Units				
Client ID:		BH3 SS2	-	-	-
Sample Date:		04-Apr-12	-	-	-
Sample ID:		1215074-01	-	-	-
		Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	83.3	-	-	-
General Inorganics					
pH	0.05 pH Units	8.02	-	-	-
Resistivity	0.10 Ohm.m	32.9	-	-	-
Anions					
Chloride	5 ug/g dry	73	-	-	-
Sulphate	5 ug/g dry	106	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - AERIAL PHOTOGRAPH - 2017

FIGURE 3 - AERIAL PHOTOGRAPH - 2022

DRAWING PG7073-1 - TEST HOLE LOCATION PLAN

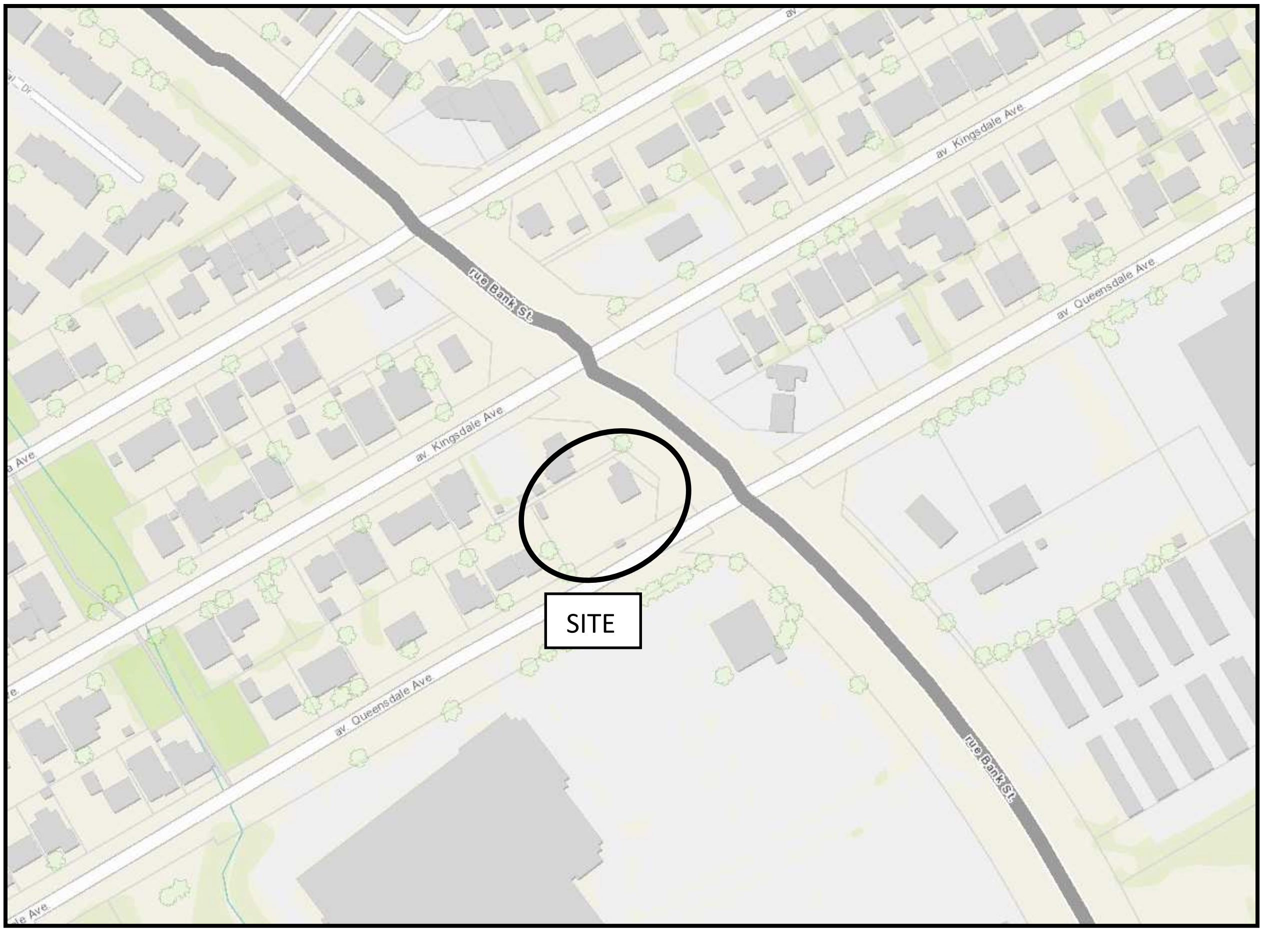


FIGURE 1

KEY PLAN

PATERSON GROUP

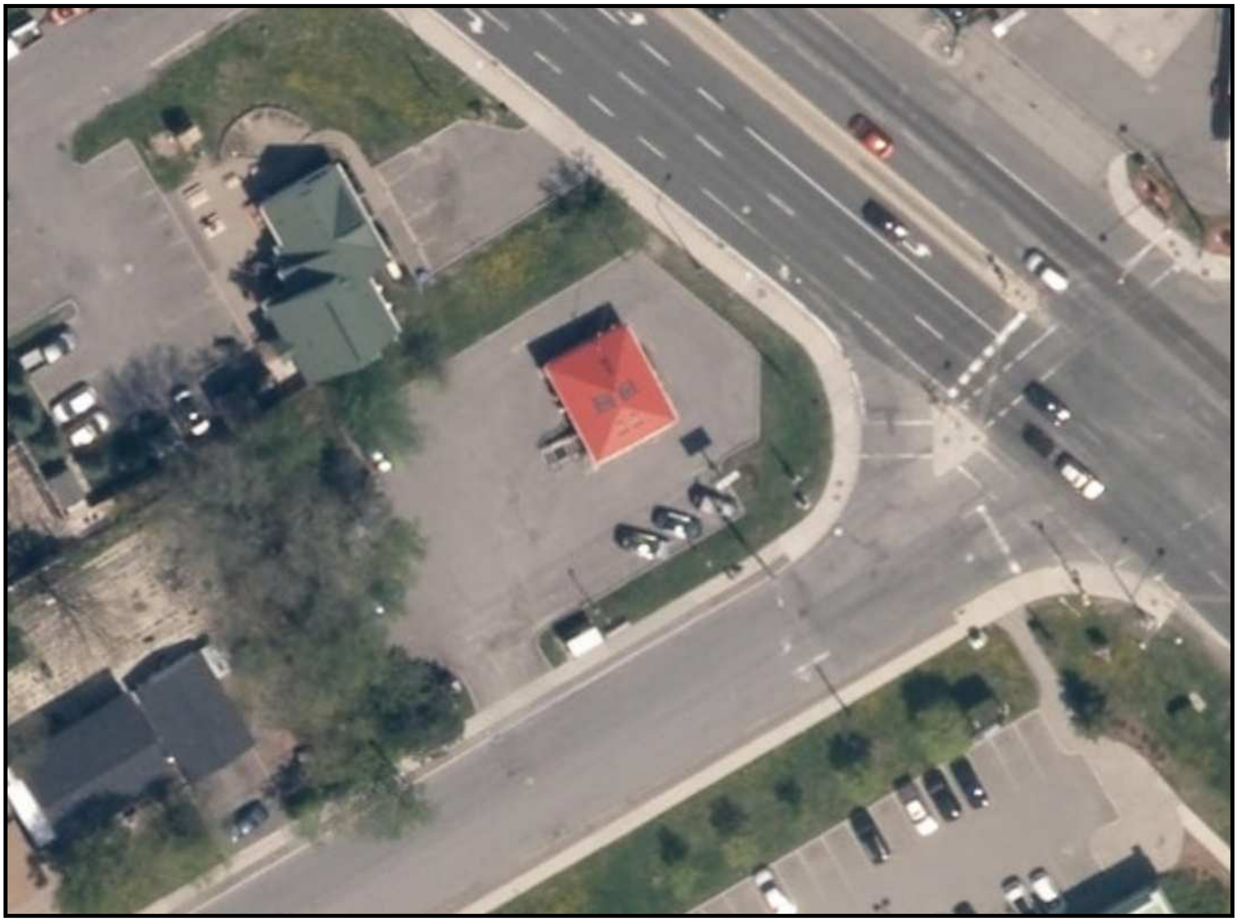


FIGURE 2

Aerial Photograph - 2017

PATERSON GROUP

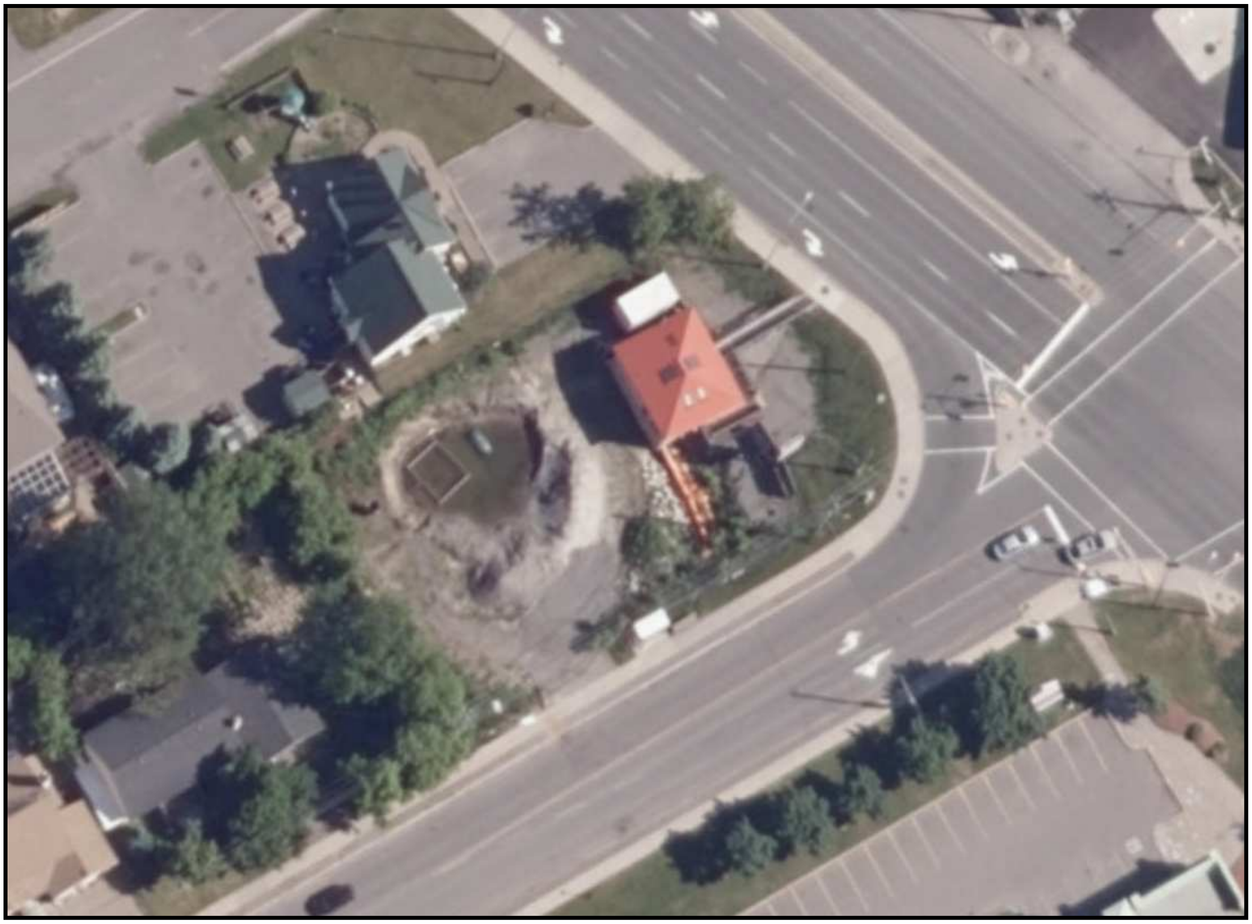
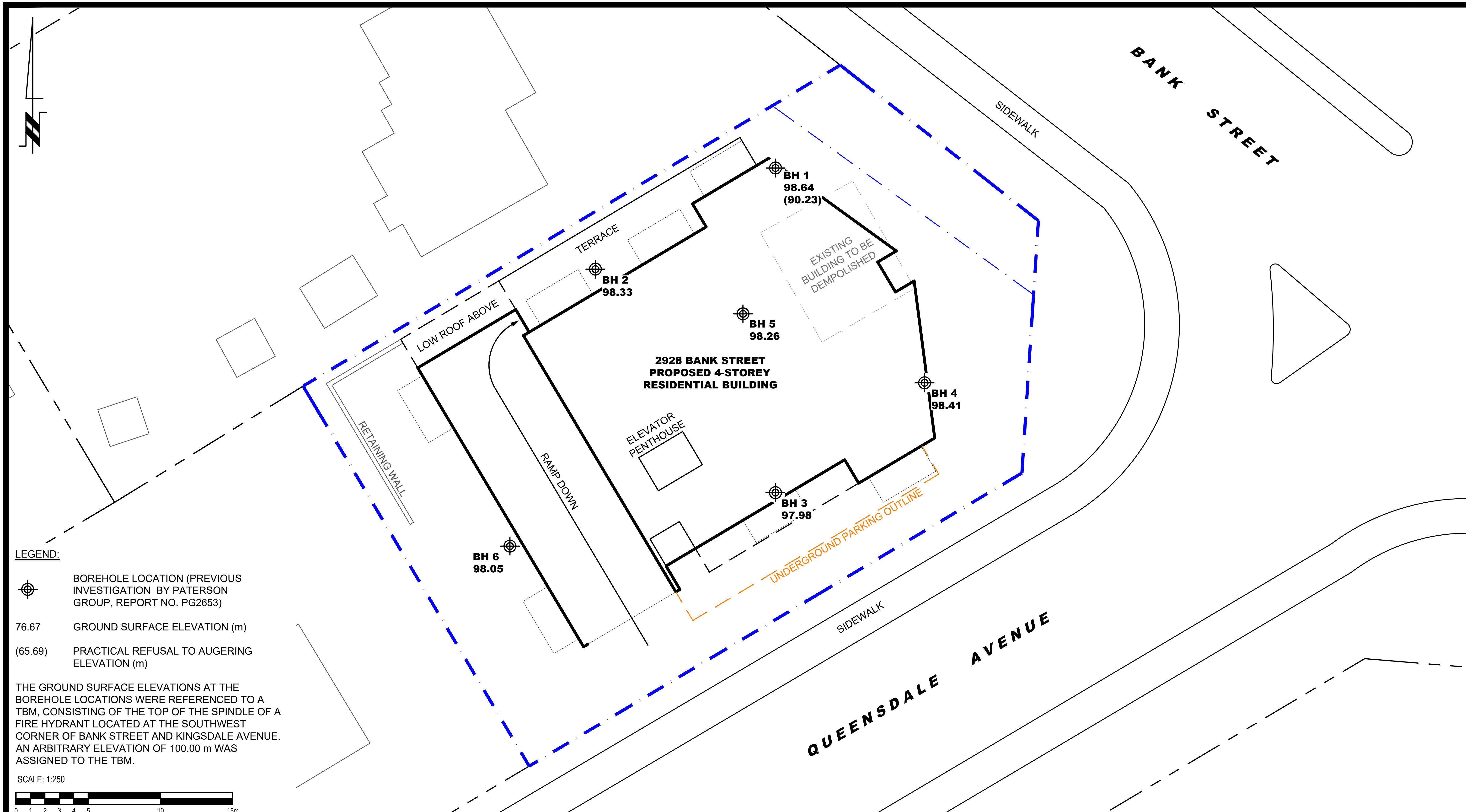


FIGURE 3

Aerial Photograph - 2022

PATERSON GROUP



LEGEND:

⌖	BOREHOLE LOCATION (PREVIOUS INVESTIGATION BY PATERSON GROUP, REPORT NO. PG2653)
76.67	GROUND SURFACE ELEVATION (m)
(65.69)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

THE GROUND SURFACE ELEVATIONS AT THE BOREHOLE LOCATIONS WERE REFERENCED TO A TBM, CONSISTING OF THE TOP OF THE SPINDLE OF A FIRE HYDRANT LOCATED AT THE SOUTHWEST CORNER OF BANK STREET AND KINGSDALE AVENUE. AN ARBITRARY ELEVATION OF 100.00 m WAS ASSIGNED TO THE TBM.

PATERSON GROUP
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NO.	REVISIONS	DATE	INITIAL

VIP CONSTRUCTION & ENGINEERING
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
2928 BANK STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 03/2024
Drawn by: ZS	Report No.: PG7073-1
Checked by: YZ	Dwg. No.: PG7073-1
Approved by: FA	Revision No.:

11x17

p:\autocad drawings\geotechnical\pg70xx\pg7073\pg7073-1-test hole location plan.dwg