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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation

Proposed Multi-Storey Building
406-408 Bank Street
Ottawa, Ontario

Prepared For

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Report PG5538-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by 12291444 Canada Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 406-408 Bank Street, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

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 for the design of the proposed development based on the results of the test holes and other soil information available.

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.

2.0 Proposed Development

Based on the available conceptual plans, the proposed development will consist of a multi-storey, mixed-use building with one basement level. It is understood that the proposed building will occupy the entire site footprint. It is also anticipated that the subject site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on November 10, 2020. At that time, four (4) boreholes were advanced to a maximum depth of 9.75 m below existing ground surface. The test holes were located in the field by Paterson in a manner to provide general coverage of the subject site. A previous geotechnical investigation was also conducted at the subject site by others on December 30, 2005. The borehole locations are shown on Drawing PG5581-1 - Test Hole Location Plan in Appendix 2.

The boreholes were drilled with a track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department. The drilling procedures consisted of advancing each test hole to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples from the boreholes were recovered from the auger flights or using a 50 mm diameter split-spoon sampler. All soil samples were classified on site, placed in sealed plastic bags and transported to our laboratory for further review. The depths at which the auger and split spoon samples were recovered from the test hole are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Standard Penetration Testing (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.

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

The 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 its 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 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

Groundwater monitoring wells were installed within boreholes BH 1 and BH 2 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. In addition, groundwater in an existing monitoring well, identified as BHMW1 and installed during the previous geotechnical investigation by others, was measured as part of the current geotechnical investigation.

Sample Storage

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

3.2 Field Survey

The test hole locations were selected in the field by Paterson personnel in a manner to provide general coverage of the subject site taking into consideration existing site features. The test hole locations and ground surface elevations were measured in the field by Paterson personnel and referenced to a geodetic datum. The locations of the test holes are presented on Drawing PG5581-1 - 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 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 sample was submitted to determine the sulphate and chloride concentration, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently vacant, with the surface consisting of gravel and grassed areas. The subject site is bordered by multi-storey residential and commercial dwellings to the north and west, Bank Street to the east and Florence Street to the south. The ground surface across the subject site is generally flat and at grade with Bank Street and the surrounding properties.

It is understood that a building previously occupying the site was the subject of a structural fire and subsequently demolished. Reference should be made to the aerial photographs in Figure 2 - Aerial Photograph - 2002 and Figure 3 - Aerial Photograph - 2007 which illustrate the former and present site conditions, presented in Appendix 2.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consists of an approximate 1.4 to 3.1 m thickness of fill which is underlain by a silty clay deposit. The fill was generally observed to consist of a loose to dense, brown silty sand with some gravel, crushed stone, and occasional traces of organics.

A very stiff to stiff, brown to grey silty clay was encountered underlying the fill at all borehole locations, extending to approximate depths of 8.4 and 7.5 m at BH 1 and BH 2, respectively.

Glacial till was encountered underlying the silty clay, and was generally observed to consist of a silty clay with trace to some gravel and sand.

Practical refusal to DCPT was encountered at an approximate 14.5 m depth in BH 1. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the local bedrock consists of shale of the Billings formation with an anticipated overburden thickness of 10 to 15 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells installed at the borehole locations. The groundwater level observations are presented on the Soil Profile and Test Data sheets in Appendix 1 and in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1	71.50	3.57	67.93	November 18, 2020
BH 2	71.36	4.27	67.09	November 18, 2020
BHMW1	71.61	3.16	68.45	November 18, 2020

It should be noted that groundwater levels can be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater table can be expected at approximately 3 to 4 m below ground surface. It should also be noted that groundwater levels are subject to seasonal fluctuations, and therefore the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered suitable for the proposed development, from a geotechnical perspective. It is recommended that the proposed building be founded on one of the following:

❏ Conventional spread footings bearing on an undisturbed, stiff silty clay bearing surface, or

❏ A raft foundation bearing on an undisturbed, stiff silty clay bearing surface.

Due to the presence of a silty clay deposit, a permissible grade raise restriction will be required for the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings, pipe bedding and other settlement sensitive structures. Due to the anticipated depth of the basement excavation, and the subsurface conditions encountered in the boreholes, it is anticipated that all topsoil and fil will be excavated from within the proposed building footprint.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter and lateral support zones for the foundation. Existing foundation walls and other construction debris are not considered suitable for reuse at the site.

Protection of Subgrade (Raft Foundation)

Since the subgrade material will consist of a silty clay deposit, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay to potential disturbance due to drying.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, stiff silty clay 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 **180 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

An undisturbed soil bearing surface consists of one 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.

Footings designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Raft Foundation

As noted above, a raft foundation may be required to support the proposed multi-storey building. For our design calculations, one basement level was assumed which would extend to approximately 3 to 4 m below existing ground surface. The maximum SLS contact pressure is **150 kPa** for a raft foundation bearing on the undisturbed, stiff silty clay. It should be noted that the weight of the raft slab and everything above has to be included when designing with this value. 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. The factored bearing resistance (contact pressure) at ULS can be taken as **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **6 MPa/m** for a contact pressure of **150 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, stiff silty clay is not elastic and limits have to be placed on the stress ranges of a particular modulus.

The proposed building can be designed using the above parameters with 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 stiff silty clay above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passing through in situ soil of the same or higher bearing capacity as the bearing medium soil.

Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **1.5 m** is recommended for grading at the subject site.

If higher than permissible grade raises are required, lightweight fill and/or other measured should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** as defined in Table 4.1.8.4A of the Ontario Building Code (OBC) 2012 for foundations considered at this site. A higher seismic site class may be applicable, such as Class C, however, this would need to be confirmed by performing a seismic shear wave velocity test at the subject site. The soils underlying the proposed foundations are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the undisturbed, stiff silty clay subgrade approved by the geotechnical consultant at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for basement slab construction. It is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone.

If a raft slab is considered for the proposed multi-storey building, a granular layer of OPSS Granular A crushed stone will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level floor slab. The spacing of the sub-slab drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels. This is discussed further in Subsection 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, in our opinion, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3.

The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

Static Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

The earth force component (P_o) under seismic conditions could be calculated using:

$P_o = 0.5\ K_o \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions presented above.

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

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

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone and 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.

Where sufficient space is available for conventional wall construction, 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 board, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Where insufficient room is available for exterior backfill, it is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) be secured against the temporary shoring system or underpinning piers and extend to a series of drainage sleeve inlets through the building foundation walls, located above the footing or raft and below the lowest level floor slab. The drainage sleeves should be at least 150 mm diameter and be spaced 3 m along the perimeter foundation walls. An interior perimeter drainage pipe should be placed along the building perimeter along with the sub-slab drainage system. The perimeter drainage pipe and sub-slab drainage system should direct water to sump pit(s) within the lower basement area.

Foundation raft Slab Construction Joints

It is expected that the raft slab, where utilized, will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration below the lowest level floor slab. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres. The spacing of sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

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 an equivalent combination of soil cover and foundation insulation, should be provided in this regard.

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

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill 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. Given that the proposed building is anticipated to extend to the property lines, it is expected that a temporary shoring will be required to support the excavation on the east and south sides. Further, given the proximity of the adjacent buildings to the north and west, underpinning of these structures may be required prior to the basement construction of the proposed building. This is discussed further below.

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. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

As noted above, a temporary shoring system is anticipated to be required to support the overburden soils. The design and approval of the temporary shoring system will be the responsibility of the shoring contractor and the shoring designer who a is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 2 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight (γ), kN/m^3	13

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

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

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

Underpinning of Adjacent Structures

Should the excavation for the proposed basement level extend within the lateral support zones of the footings of the adjacent buildings to the north and west (downward and outward at 1.5H:1V), underpinning of the adjacent footings would be required.

Conventional timber lagged pits and concrete underpinning piers are considered to be suitable for this project. The depth of the underpinning, should it be required, will be dependent on the depth of the adjacent foundations relative to the foundation depths of the proposed building at the subject site.

It is recommended that test pits be completed prior to construction, or at the start of construction, in order to evaluate the foundation depths of the adjacent structures.

6.4 Pipe Bedding and Backfill

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

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the stiff silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. 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 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 300 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD.

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

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

6.5 Groundwater Control

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 low and controllable using conventional 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 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.

Impacts on Neighbouring Structures

It is understood that one basement level is planned for the proposed building. Based on the existing groundwater level, the extent of any groundwater lowering will be negligible and will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures.

It should be noted that no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

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

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

6.7 Corrosion Potential and Sulphate

The analytical testing results 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 moderately aggressive to aggressive corrosive environment.

7.0 Recommendations

For the foundation design data provided herein to be applicable, a materials testing and observation services program is required to be completed. The following aspects should be performed by the geotechnical consultant:

- ❏ A review of the site grading plan(s) from a geotechnical perspective, once available.
- ❏ Review of the Contractor's design of the temporary shoring system and underpinning.
- ❏ A review of architectural and structural drawings to ensure adequate frost protection is provided to the subsoil.
- ❏ 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.
- ❏ Field density tests to determine the level of compaction achieved.
- ❏ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The recommendations provided in this report are intended for the use of design professionals associated with this project. Contractors bidding on or undertaking the work should examine the factual information contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractors construction methods, equipment capabilities and schedules.

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 12291444 Canada Inc. or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Owen Canton, E.I.T.

LICENSED PROFESSIONAL ENGINEER
Dec. 7-2020
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Report Distribution:

- 12291444 Canada Inc.(1 digital copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA SHEET BY OTHERS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 406 Bank Street
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5581

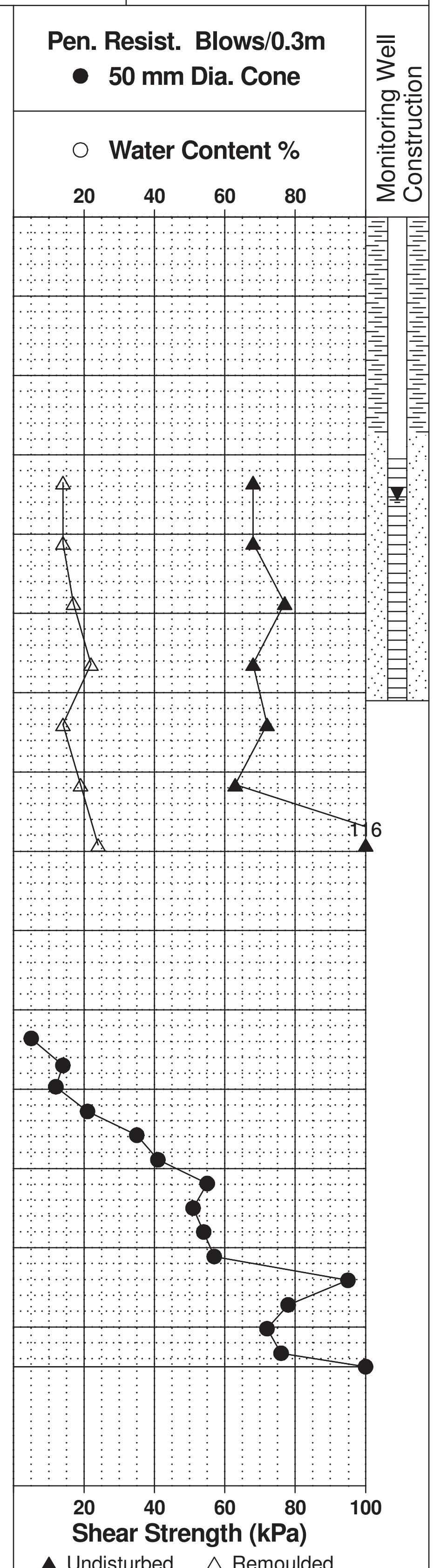
REMARKS

HOLE NO. BH 1

BORINGS BY CME-55 Low Clearance Drill **DATE** November 10, 2020

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.05	AU	1		
FILL: Brown silty sand with crushed stone	0.36				
FILL: Brown silty sand, trace clay and gravel	1.52	SS	2	21	10
Stiff to very stiff, brown **SILTY CLAY**		SS	3	12	14
		SS	4	92	4
		SS	5	100	P
- grey by 2.3m depth	8.38	SS	6	100	P
GLACIAL TILL: Grey silty clay, some gravel, trace sand	9.75	SS	7	42	5
		SS	8	58	4
Dynamic Cone Penetration Test commenced at 9.75m depth. Cone pushed to 10.0m depth.					
Inferred **GLACIAL TILL**	14.50				
End of Borehole					
Practical DCPT refusal at 14.50m depth.					
(GWL @ 3.57m - Nov. 18, 2020)					

DEPTH (m)	ELEV. (m)
0	71.50
1	70.50
2	69.50
3	68.50
4	67.50
5	66.50
6	65.50
7	64.50
8	63.50
9	62.50
10	61.50
11	60.50
12	59.50
13	58.50
14	57.50



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use Development - 406 Bank Street
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5581

REMARKS

HOLE NO. BH 2

BORINGS BY CME-55 Low Clearance Drill **DATE** November 10, 2020

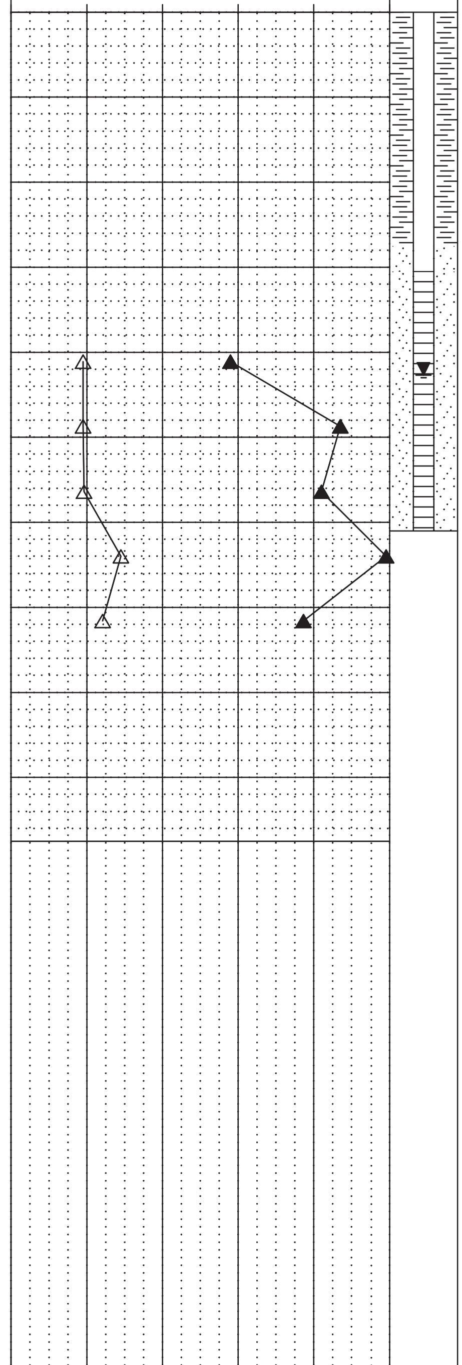
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.36
FILL: Brown silty sand with crushed stone (0.71)		AU	1				
FILL: Brown silty sand with gravel		SS	2	21	10	1	70.36
		SS	3	42	5	2	69.36
(3.05)		SS	4	33	36		
Stiff, grey **SILTY CLAY**		SS	5	100	2	3	68.36
						4	67.36
		SS	6	50	P	5	66.36
						6	65.36
(7.47)						7	64.36
GLACIAL TILL: Grey silty clay with sand, trace gravel		SS	7	17	5	8	63.36
		SS	8	71	8	9	62.36
(9.75)		SS	9		3		
End of Borehole							
(GWL @ 4.27m - Nov. 18, 2020)							

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

○ Water Content %

20 40 60 80

Monitoring Well 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
Proposed Mixed-Use Development - 406 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 10, 2020

FILE NO. PG5581

HOLE NO. BH 3

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)	Monitoring Well Construction
GROUND SURFACE						0	71.24		
FILL: Brown silty sand, some gravel, trace organics — 0.53		AU	1						
FILL: Brown silty clay, some sand — 1.37		SS	2	17	6	1	70.24		
Stiff, brown **SILTY CLAY**		SS	3	46	11	2	69.24		
		SS	4	38	5				
- grey by 3.0m depth		SS	5	100	2	3	68.24		∇
— 4.42		SS	6	100	2	4	67.24		
End of Borehole (GWL @ 3.0m depth based on field observations)									

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
Proposed Mixed-Use Development - 406 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 10, 2020

FILE NO. PG5581

HOLE NO. BH 4

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	71.36
FILL: Brown silty sand with crushed stone	AU	1				
	SS	2	17	6	1	70.36
	SS	3	12	5	2	69.36
2.74	SS	4	8	6		
Stiff, grey **SILTY CLAY**					3	68.36
3.66	SS	5	100	2		
End of Borehole (GWL @ 2.7m depth based on field observations)						

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

○ Water Content %
20 40 60 80

Monitoring Well 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 x 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

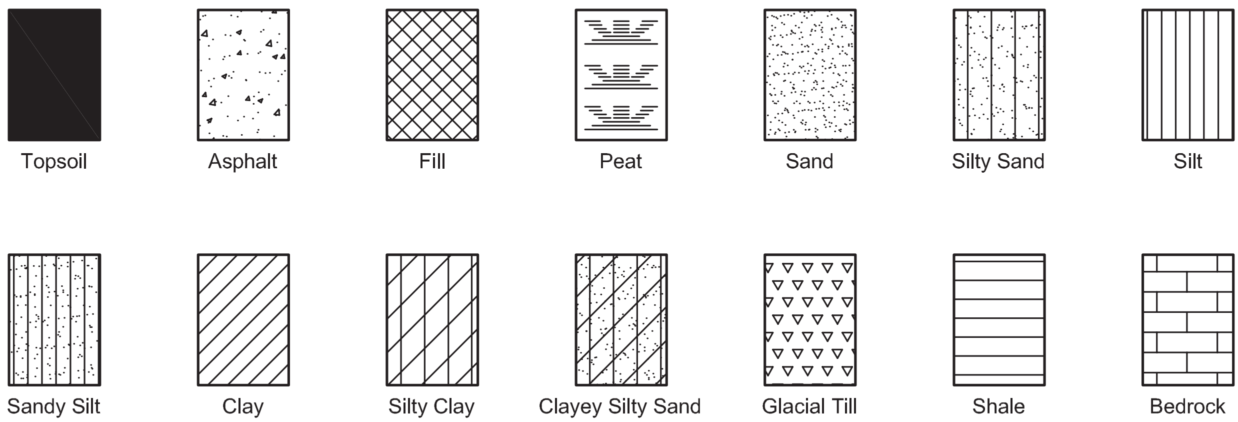
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

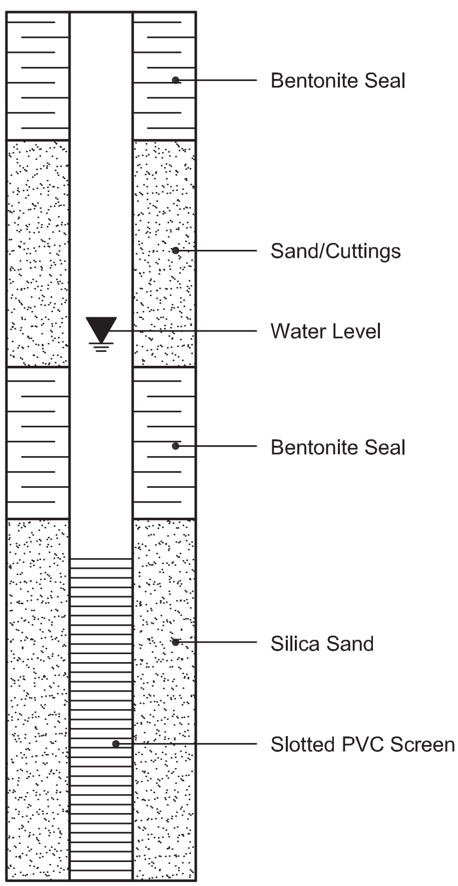
SYMBOLS AND TERMS (continued)

STRATA PLOT

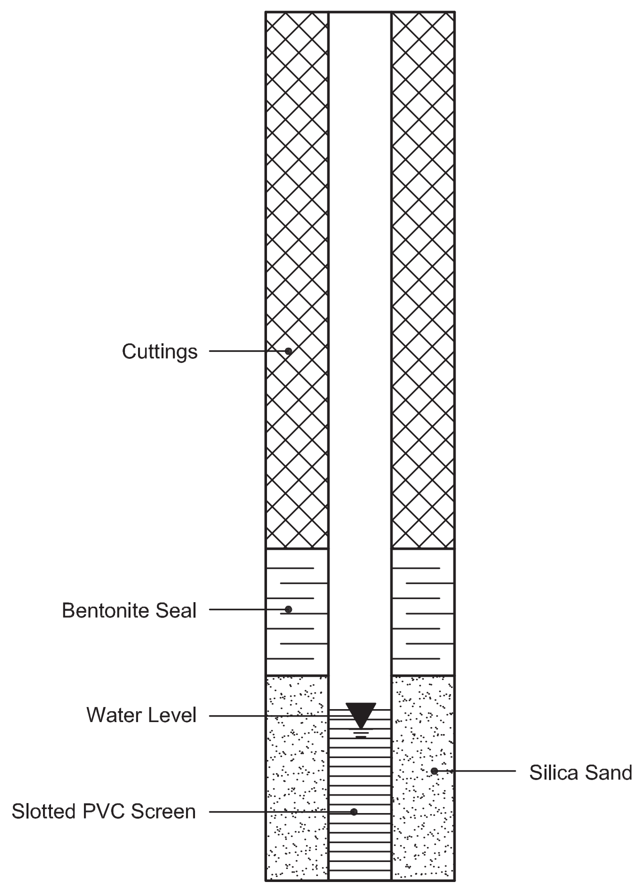


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



LOG OF BOREHOLE / MONITORING WELL BHMW1

DST REF. No.: **OG05373**
CLIENT: **Galaxy Camera**
PROJECT: **Preliminary Geotechnical Investigation**
LOCATION: **406 - 408 Bank Street, Ottawa, Ontario**
SURFACE ELEV.: **--/--**

Drilling Data
METHOD: **CME 75 Drill Rig**
DIAMETER: **100**

DATE: **December 09 2005**

DEPTH (m)	ELEV. (m)	Water Data	% MOISTURE W_p W W_l (20 40 60 80)	Symbol	MATERIAL DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	'N' VALUE	VANE (kPa) × 50 100 150 200 / SPT (N) □ Blows/0.3m DCPT ◆ 10 20 30 40	REMARKS
					SAND - compact, frozen, light brown (fill)					Monitoring well installed to 5.9 m depth on December 9, 2005.
					SAND - silty, trace gravel, loose, frozen, dark brown (fill)	SGS1				
1						SS2		7		
2					SAND - clayey, loose, moist (possible fill)	SS3		6		
					CLAY - silty, stiff, moist to wet, olive grey	SS4		3		
3						SS5		2		Groundwater level recorded at 2.9 m below grade on December 9, 2005.
4						SS6		1		
5						SS7		1		
					End of borehole at 5.2 m depth.					
6										
7										
8										
9										

BOREHOLE (STANDARD) OG05373.GPJ DST_MIN.GDT 1/24/06

DST CONSULTING ENGINEERS

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Web: www.dstgroup.com

SAMPLE TYPE LEGEND

- Auger Sample
- Split Spoon Sample
- Thin Wall Tube
- Rock Core
- Side Sampler
- Grab Sample
- Ponar Sample

APPENDIX C

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 31214

Report Date: 17-Nov-2020

Order Date: 11-Nov-2020

Project Description: PG5581

	Client ID:	BH3-SS4	-	-	-
	Sample Date:	10-Nov-20 15:00	-	-	-
	Sample ID:	2046283-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	67.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.72	-	-	-
Resistivity	0.10 Ohm.m	36.2	-	-	-
Anions					
Chloride	5 ug/g dry	11	-	-	-
Sulphate	5 ug/g dry	46	-	-	-

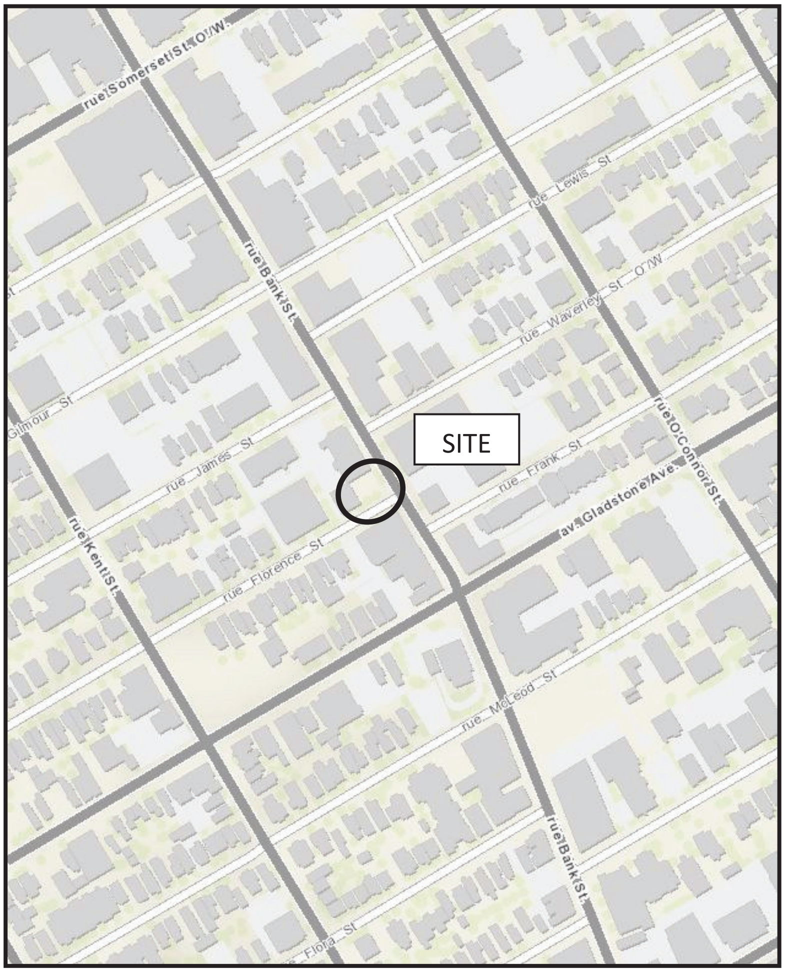
APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - AERIAL PHOTOGRAPH - 2002

FIGURE 3 - AERIAL PHOTOGRAPH - 2007

DRAWING PG5581-1 - TEST HOLE LOCATION PLAN



Source: GeoOttawa

FIGURE 1

KEY PLAN

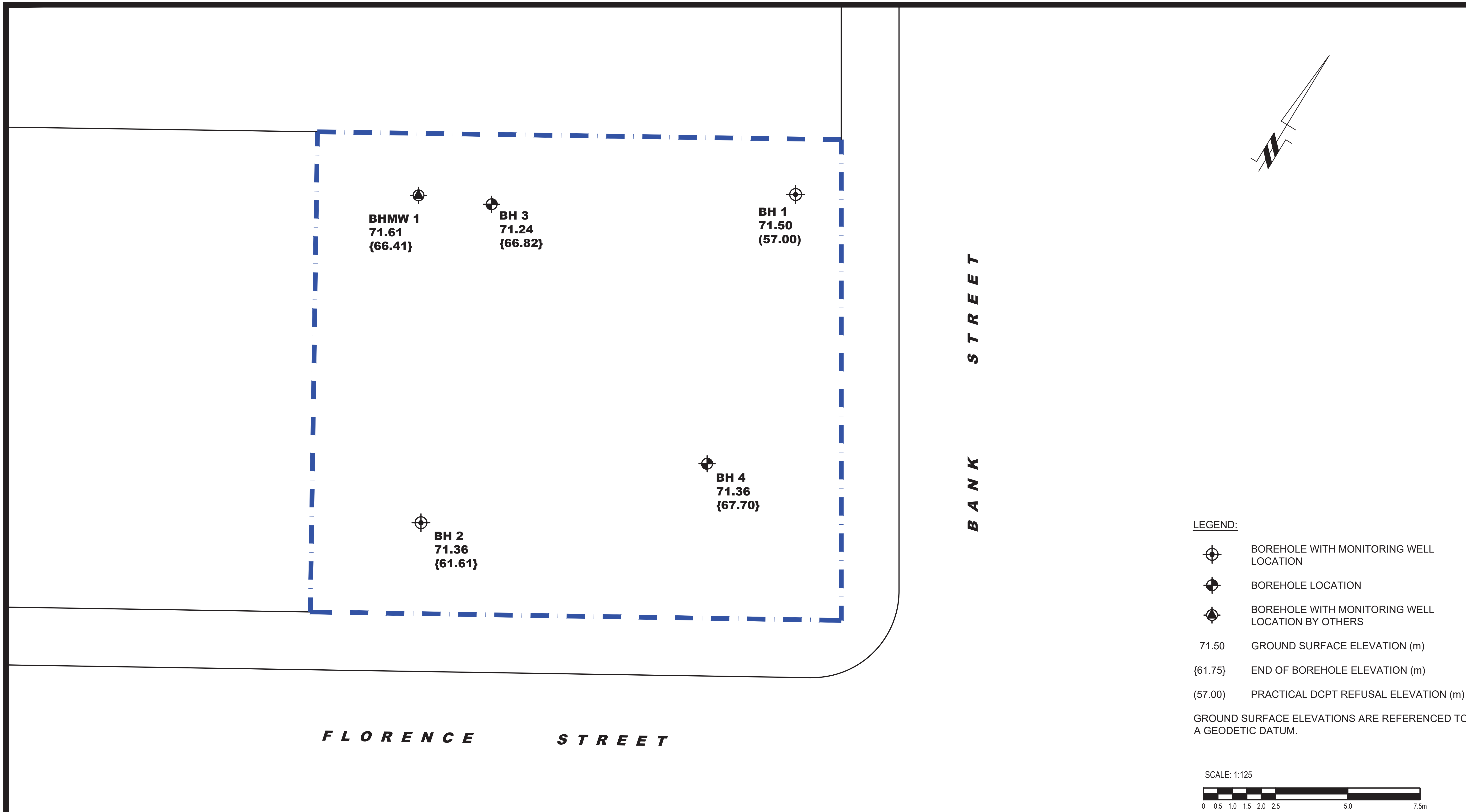
FIGURE 2

Aerial Photograph - 2002

FIGURE 3

Aerial Photograph - 2007

patersongroup



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consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

12291444 CANADA INC.
GEOTECHNICAL INVESTIGATION
406 BANK STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:125	Date: 11/2020
Drawn by: MPG	Report No.: PG5581-1
Checked by: OC	Dwg. No.: PG5581-1
Approved by: SD	Revision No.:

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