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Geotechnical Engineering

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Noise and Vibration Studies

Geotechnical Investigation
Proposed Building Addition
1250 and 1252 Wellington Street West
Ottawa, Ontario

Prepared For

Domicile Developments Inc.

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Report: PG5972-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Domicile Developments Inc. to conduct a geotechnical investigation for the proposed building addition to the existing building currently located at 1250 and 1252 Wellington Street West in the City of Ottawa (refer 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 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

Detailed plans of the proposed development were not available at the time of writing this report. However, it is anticipated that the proposed development will consist of a three-storey wood-framed building addition to the existing building located at 1250 to 1252 Wellington Street West. It is understood consideration is being given to providing an approximately 1.8 m deep crawl-space area for the proposed addition. It is also understood the existing building will remain as part of the proposed development and will also receive an additional building storey.

Associated access lanes and walkways are anticipated as part of the development. It is expected that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on September 17, 2021. At that time, two (2) boreholes and two (2) test pits were advanced to maximum depth of 6.5 m and 3.0 m below the existing ground surface, respectively. Two (2) additional test pits were undertaken to a maximum depth of 2.55 m along the southern face of the existing structure located at 1250-1252 Wellington Street West on January 12, 2022. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration the location of underground utilities and site features. The test hole locations are shown on Drawing PG5972-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a low-clearance drill rig operated by a two-person crew. The test pits were excavated using a rubber-tired back-hoe. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of advancing each test hole to the required depths at the selected locations and sampling the overburden.

Sampling and In-Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. Grab samples were collected from the test pit sidewalls and by hand-auger recovery at selected intervals. The samples were classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger, split spoon and grab samples were recovered from the boreholes are shown as SS, AU and G, respectively, on the Soil Profile and Test Data sheets 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.

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 2-21. 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 test holes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

The boreholes were fitted with flexible standpipe piezometers to allow for groundwater level monitoring. Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

All samples will be stored in the laboratory for a period of one (1) 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 by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson personnel using a handheld GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5972-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

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 sample was 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

Existing Conditions

The subject site is currently occupied by a two-storey brick building fronting onto Wellington Street West and an asphalt surfaced parking area. The parking area is located between the rear side of the building and the north and west faces of the existing building located at 1244 Wellington Street West. The ground surface throughout the parking area is relatively flat and at geodetic elevations of approximately 67.0 m.

The site is bordered to the east and south by a one-storey commercial building, to the west by Huron Avenue North, and to the north by Wellington Street West. The site is at grade with surrounding roadways and properties.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consisted of an asphalt pavement structure underlain by a variable layer of fill. The fill was observed to generally consist of crushed stone and/or brown silty sand with gravel, cobbles and boulders. The fill was observed to extend to depths ranging between of 0.1 m to 0.8 m below the existing ground surface.

The fill material was observed to be underlain by a layer of silty sand. This layer was generally observed to consist of a very dense to dense brown silty sand with gravel, cobbles and boulders. The silty sand layer was observed to extend to depths ranging between 2.9 and 3.0 m.

The silty sand layer was further underlain by a deposit of silty clay. This deposit was generally observed to consist of a stiff grey silty clay. Trace amounts of sand and gravel were observed within the deposit.

Practical refusal to DCPT was encountered at an approximate depth of 8.9 m at the location of BH 2-21.

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

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic interbedded limestone and dolomite of the Gull River formation, with an overburden drift thickness of 3 to 10 m depth.

Existing Neighboring Building Foundation (1244 Wellington Street West)

The existing building was accessed at its rear and along Huron Street via an ingress door located at the northwest corner of the neighboring property and approximately 3 m west of TP 1-21. The space at the entrance consists of a stairwell at an approximate finished floor elevation (FFE) of 69.30 m and heading downwards to a basement level in the east direction.

Two test pits were advanced against the portions of the existing building that will be adjacent to the proposed buildings excavation. The foundation wall was generally observed to consist of bare concrete and had been backfilled against by fill containing variable amounts of sand, gravel, cobbles and inorganic debris.

The footing encountered at TP 1-21 consisted of an approximately 100 mm wide and 220 mm deep footing founded at an approximate geodetic elevation of 65.88 m. The footing appeared to be bearing upon the compact, in-situ silty sand deposit at the time of our review. Based on the construction of the existing building, it was interpreted that this footing steps down in elevation across the length of the basement stairwell and towards the eastern property boundary.

The footing and foundation wall were not encountered at TP 2-21 due to a buried ground floor slab encountered within the test pit and extending from the neighboring building footprint. The extended slab prevented the test pit excavation from extending laterally towards the building to expose the foundation. Based on our review and field measurements, the basement floor slab for the neighboring building and directly adjacent to the area of TP 2-21 was measured as being approximately 64.00 m. Based on this and the results of our investigation, it is anticipated the footings for the basement level may be inferred as being founded as high as an approximate geodetic elevation of 63.6 m and upon the silty clay deposit.

Existing Building (1250-1252 Wellington Street West)

Two test pits, TP1-22 and TP 2-22, were advanced against portions of the existing building that will be adjacent to the proposed buildings excavation. The foundation wall was generally observed to consist of bare concrete and had been backfilled against by fill containing variable amounts of sand, gravel, cobbles and inorganic debris.

TP 1-22 and TP 2-22 were advanced to the founding level of the structure which was interpreted as being at an approximate geodetic elevation of 65.03 m. The foundations did not appear to be provided a conventional spread footing at the bottom of the foundation wall structure. Based on our review, the foundations were founded upon the silty sand deposit.

Perimeter clay drainage pipe was encountered at the bottom of the foundation in TP1-22, however, had not been observed in TP 2-22. It is possible the pipe had been removed in TP2-22 by the sub-excavation process and evidence of the pipe in sidewalls had been covered by sidewall slough. Additional components pertaining to a foundation drainage system, such as composite foundation board and/or damp-proofing membranes, were not observed at the time of the investigation.

4.3 Groundwater

Groundwater levels were measured on September 22, 2021 within the installed piezometers. Further, groundwater infiltration levels were recorded within the open holes during the excavation of the test pits. The measured groundwater levels and observed depth of infiltration are presented in Table 1 below.

Table 1 – Summary of Groundwater Levels

Test Hole Number	Groundwater Measuring Medium	Ground Surface Elevation (m)	Measured Groundwater Level / Groundwater Infiltration for Test Pits		Dated Recorded
			Depth (m)	Elevation (m)	
BH 1-21	Piezometer	67.23	2.90	64.33	Sept 22, 2021
BH 2-21	Piezometer	67.09	Dry	-	Sept 22, 2021
TP 1-21	Sidewall Infiltration	67.32	Dry	-	Sept 17, 2021
TP 2-21	Sidewall Infiltration	67.15	Dry	-	Sept 17, 2021

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.

It should be noted that long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximate depths of 3.0 to 4.0 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is anticipated the proposed building will have a crawl space basement. Based on the results of the field program, the proposed building addition may be founded upon conventional spread footings placed on an undisturbed, very dense to dense silty sand or a stiff, grey silty clay bearing surface.

Detailed plans for founding depths were not available at the time of preparation of this report. However, it is expected that the excavation for the proposed structure will be within the lateral support zone of the footings for the existing buildings. Therefore, it is recommended that the existing building foundations be supported from becoming undermined by the construction of the proposed building.

This may be accomplished by carrying out underpinning of the pertinent portions of the existing buildings in advance of or at the time of the excavation of the proposed building. An underpinning program such as a series of 1 to 1.2 m wide excavations completed with a piano key style excavation technique and extending to the proposed depth of excavation of the proposed structure would be sufficient to transfer the footing’s lateral support zone sufficiently to mitigate being negatively impacted by the proposed buildings excavations. It is anticipated that these underpinning panels will be terminated upon a compact silty sand bearing surface.

Due to the presence of the silty clay deposit, a permissible grade raise restriction will be required for the proposed grading throughout 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 deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures.

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

Fill Placement

Fill used for grading beneath the proposed development should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness.

Fill placed beneath the building and potential paved areas should be compacted to at least 98% of the material’s standard Proctor maximum dry density (SPMDD), unless noted otherwise throughout this report.

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. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material’s SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Bearing Resistance Values

Conventional spread footings and underpinning panels placed on an undisturbed, very dense to dense silty sand bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**.

Provisions should be carried to proof-roll the silty sand subgrade using heavy vibratory compaction equipment where the subgrade is encountered in a relatively loose state and prior to the construction of footings. Proof-rolling of the existing sand deposit should be reviewed and approved by Paterson personnel prior to the construction of footings, if considered.

Strip footings, up to 2 m wide, and pad footings, up to 4 m wide placed on an undisturbed, stiff grey silty clay bearing surface can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS. The bearing resistance value at SLS will be subjected to potential postconstruction 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 silty sand or silty clay bearing medium 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 soil.

Permissible Grade Raise

Based on the existing borehole coverage and results of the undrained shear strength testing completed within the underlying cohesive soils, a permissible grade raise restriction of **2.0 m** may be considered for design purposes for the subject site.

Underpinning Program

In areas where the proposed buildings excavation will be within the lateral support zone of the existing buildings footings, it is recommended that an underpinning program be carried out to safely transfer the existing building loads down to a lower founding elevation. This would generally consist of excavating a series of 1 to 1.2 m wide panels in a piano key style excavation technique and extending a minimum of 150 mm below the depth of the excavation for the proposed building. The underpinning panels should be in-filled with a lean concrete (minimum **17 MPa**, 28 day compressive strength) between the underside of the existing structures' footings to the depth of the panel. Provisions should be carried to allow the hardening of freshly placed panels for a minimum of 24 hours prior to excavating adjacent sections.

Once details of the proposed building are finalized, it is recommended that details of the underpinning program be prepared in conjunction with the projects structural engineer. It is recommended that the geotechnical engineer be consulted at the time of design to assess the suitability of the proposed underpinning program. Further, the geotechnical consultant should carry out routine inspections of the implementation of the underpinning program at the time of construction.

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. If a higher seismic site is required, such as a Site Class C, a site-specific shear wave velocity testing may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented on Table 4.1.8.4.A of the Ontario Building Code 2012.

The soils underlying the proposed shallow foundations are not susceptible to liquefaction. Reference should be made to the latest revision of the 2021 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Crawl Space and Basement Slab

It is understood the proposed building will be provided an unoccupied crawl space below the ground floor level. Recommendations are provided herein if consideration is given to providing an occupied basement floor level. All crawl spaces should be adequately ventilated and sloped to drain to a suitable outlet.

With the removal of all topsoil and deleterious materials within the footprint of the proposed building, the native soil surface, approved by the Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of Granular A clear crushed stone.

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.

5.6 Basement Wall

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

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented below.

Static Earth Pressures

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

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

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

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

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

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

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

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

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

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

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

5.7 Pavement Design

Although car parking areas are not anticipated as part of the proposed development, hard landscaping walkways and car-only parking areas may be designed considering the proposed pavement structures presented in the following tables:

Table 2 – Recommended Flexible Pavement Structure – Car Only Parking Areas

Thickness (mm)	Material Description
Specified by Others	**Wear Course –** Interlocking Stones and/or Brick Pavers
25 to 40	**Leveling Course** – Stone Dust or SAnd
300	**SUBBASE –** OPSS Granular A
SUBGRADE – Either fill, in-situ soil, or OPSS Granular B Type **I** or **II** material placed over in-situ soil or raft slab.	

Table 3 – Recommended Flexible Pavement Structure – Car-Only Parking Areas

Thickness (mm)	Material Description
30	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type **II**
SUBGRADE – Either fill, in-situ soil, or OPSS Granular B Type **I** or **II** material placed over in-situ soil or raft slab.	

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

A perimeter foundation drainage system is recommended for the proposed structure. The system should consist of a 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. It is also recommended to connect the existing foundation drainage pipe observed at 1250-1252 Wellington Street West to the new drainage pipe to maintain continuity of the foundation drainage system. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

Where sufficient space is available for conventional backfilling, 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 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.

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 (or insulation equivalent) should be provided in this regard, or a minimum of 0.6 m of soil cover in conjunction with foundation insulation as recommended by the geotechnical consultant 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 structure proper and require additional protection. These should be provided with a minimum 2.1 m thick soil cover or a 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 either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

Given the proximity of the existing buildings basement footprint to the property lines, it is expected that a temporary shoring system or underpinning structure will be required to support the excavation along the portions of the excavation that may undermine the neighbouring foundation for the proposed development.

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. Excavation side slopes should also be protected from erosion by surface water and rainfall events by the use of secured tarpaulins spanning the length of the side slopes or other means of erosion protection along their footprint. Additional measures may be recommended at the time of construction by Paterson personnel.

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

6.4 Pipe Bedding and Backfill

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.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes when placed on a soil subgrade. If the bedding subgrade consists of grey silty clay the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe.

Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material’s SPMDD.

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

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) 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. The rate of flow of groundwater into the excavation through the overburden should be low to moderate for the conditions expected at this site. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

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 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 Neighboring Properties

Based on our observations, the groundwater level is anticipated to be located within the silty clay layer. Based on the depth of the proposed building excavation, building excavations will not extend into the silty clay deposit or below the long-term groundwater table. Therefore, no groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. Further, issues are not 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 should be carried in a manner to avoid the introduction of frozen materials, snow, or ice into the trenches.

Precautions must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring and underpinning system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soils. Provisions should be made in the contract documents to protect the founding soils for neighboring structures against the excavations from freezing, if applicable.

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 moderate to aggressive corrosive environment.

7.0 Recommendations

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

- Review of the grading plan from a geotechnical perspective.
- Review of the geotechnical aspects of the underpinning program design prior to construction.
- Review of the installation of the building's foundation drainage system.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling and placement of mud slabs.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews, if applicable.

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

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Domicile Developments Inc. 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.

Drew Petahtegoose, B.Eng.

LICENSED PROFESSIONAL ENGINEER
Jan. 17, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

Report Distribution:

☐ Domicile Developments Inc. (email copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Building Addition - 1250 to 1252 Wellington Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE January 12, 2022

FILE NO. PG5972

HOLE NO. TP 1-22

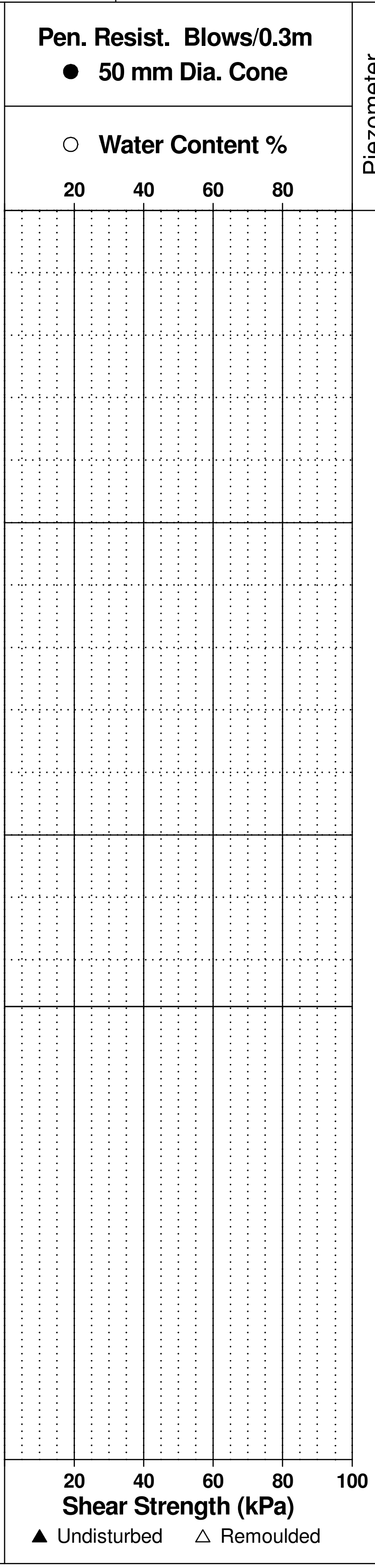
SOIL DESCRIPTION	Depth to base (m)	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	67.38
Asphaltic concrete	0.06							
FILL: Crushed stone	0.30							
FILL: Dark brown silty sand with gravel, cobbles and brick fragments	0.95						1	66.38
Loose to compact, brown **SILTY SAND** with gravel and cobbles	2.55						2	65.38

End of Test Pit

Concrete foundation wall extended to a depth of 2.35m. Concrete foundation cast in place with no footing.

Clay drainage pipe observed at bottom of foundation.

(TP dry upon completion)



Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Building Addition - 1250 to 1252 Wellington Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE January 12, 2022

FILE NO. PG5972

HOLE NO. TP 2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	67.17		
Concrete pad 0.26									
FILL: Brown silty sand with gravel, cobbles, brick fragments and roots 0.85									
						1	66.17		
Loose to compact, brown **SILTY SAND** with gravel and cobbles, trace roots									
						2	65.17		
2.25									

End of Test Pit

Concrete foundation wall extended to a depth of 2.15m. Concrete foundation cast in place with no footing.

No drainage system encountered.

(TP dry upon completion)

Pen. Resist. / Water Content % scale: 20 40 60 80

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Building Addition - 1250 to 1252 Wellington Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE September 17, 2021

FILE NO. PG5972

HOLE NO. TP 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	67.32		
Asphaltic concrete 0.05		G	1						
FILL: Crushed stone 0.41									
FILL: Brown silty sand with gravel, occasional cobbles and boulders 1.22		G	2			1	66.32		
Brown **SILTY SAND,** occasional cobbles and boulders 1.45									
End of Test Pit									

0.23m Thick concrete footing encountered at 1.22m below ground surface, extends 0.1m from foundation wall.

(TP dry upon completion)

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Building Addition - 1250 to 1252 Wellington Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE September 17, 2021

FILE NO. PG5972

HOLE NO. TP 2-21

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Crushed stone	0.15				
Concrete slab	0.46	G	1		
FILL: Brown silty sand with gravel, cobbles and boulders	1.83	G	2		
Brown **SILTY SAND,** occasional cobbles and boulders	2.84				
Stiff, grey **SILTY CLAY**	3.00	G	3		
End of Test Pit (TP dry upon completion)					

DEPTH (m)	ELEV. (m)
0	67.15
1	66.15
2	65.15
3	64.15

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

○ **Water Content %**

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Building Addition - 1250 to 1252 Wellington Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

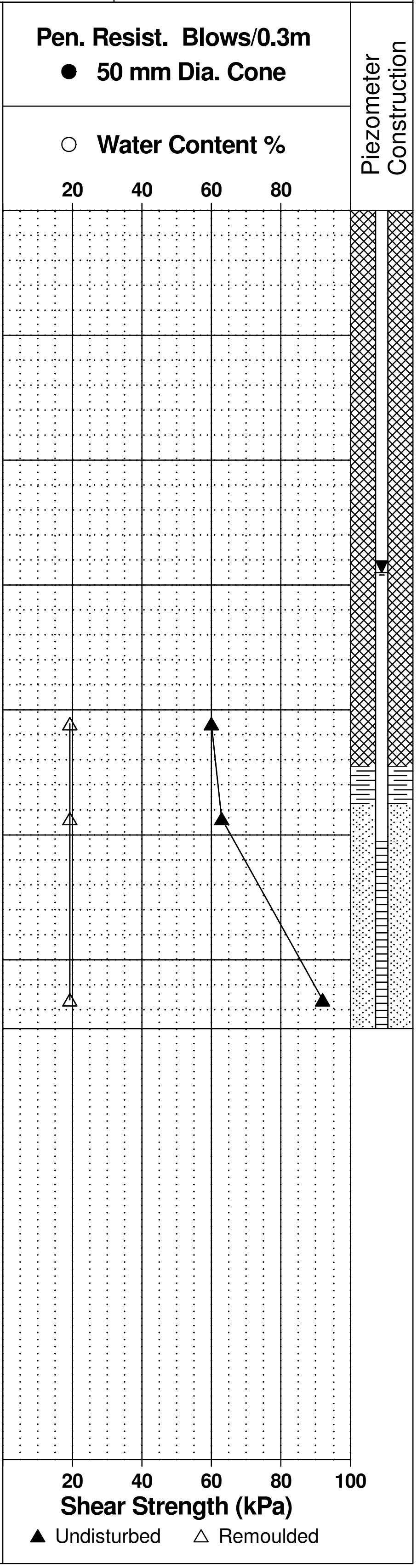
BORINGS BY CME-55 Low Clearance Drill

DATE September 17, 2021

FILE NO. PG5972

HOLE NO. BH 1-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	67.23
Asphaltic concrete 0.05	AU	1				
FILL: Brown silty sand with crushed stone, occasional cobbles, trace clay 0.84						
Very dense to dense, brown **SILTY SAND** with gravel, occasional cobbles and boulders	SS	2	33	11	1	66.23
	SS	3	62	79	2	65.23
2.97	SS	4	62	39	3	64.23
Stiff, grey **SILTY CLAY**	SS	5	83	P		
	SS	6	75		4	63.23
- trace sand and gravel from 4.1 to 5.3m depth	SS	7	75		5	62.23
6.55	SS	8	83		6	61.23
End of Borehole						
(GWL @ 2.90m - Sept. 22, 2021)						



patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

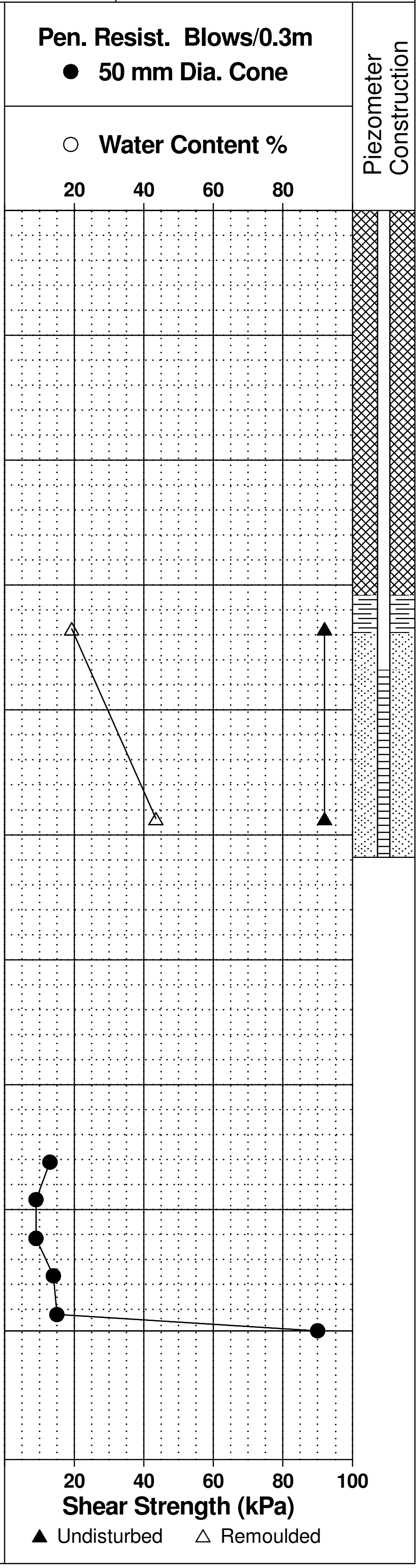
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Building Addition - 1250 to 1252 Wellington Street W.
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE September 17, 2021
FILE NO. PG5972
HOLE NO. BH 2-21

SOIL DESCRIPTION		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05	AU	1		
FILL: Crushed stone	0.25				
Very dense to dense, brown **SILTY SAND** with gravel, occasional cobbles and boulders		SS	2	42	66
		SS	3	63	50+
	3.05	SS	4	33	38
Stiff, grey **SILTY CLAY,** trace sand		SS	5	83	
- trace gravel by 4.5m depth		SS	6	83	
	5.18	SS	7	83	
Dynamic Cone Penetration Test commenced at 5.18m depth. Cone pushed to 7.5m depth.	8.97				
End of Borehole Practical DCPT refusal at 8.97m depth (BH dry - Sept. 22, 2021)					

DEPTH (m)	ELEV. (m)
0	67.09
1	66.09
2	65.09
3	64.09
4	63.09
5	62.09
6	61.09
7	60.09
8	59.09



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

SYMBOLS AND TERMS (continued)

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 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)

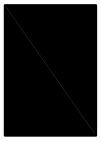
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

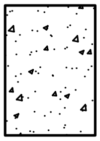
PERMEABILITY TEST

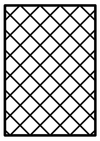
k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during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

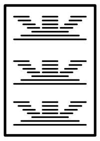
SYMBOLS AND TERMS (continued)

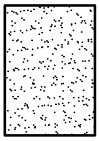
STRATA PLOT

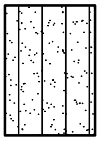

Topsoil

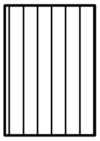

Asphalt

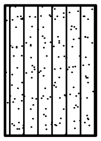

Fill

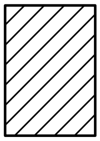

Peat

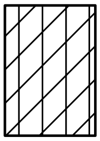

Sand

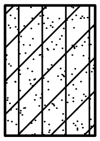

Silty Sand

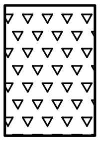

Silt

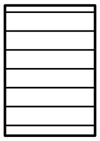

Sandy Silt

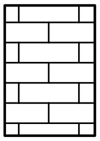

Clay


Silty Clay


Clayey Silty Sand

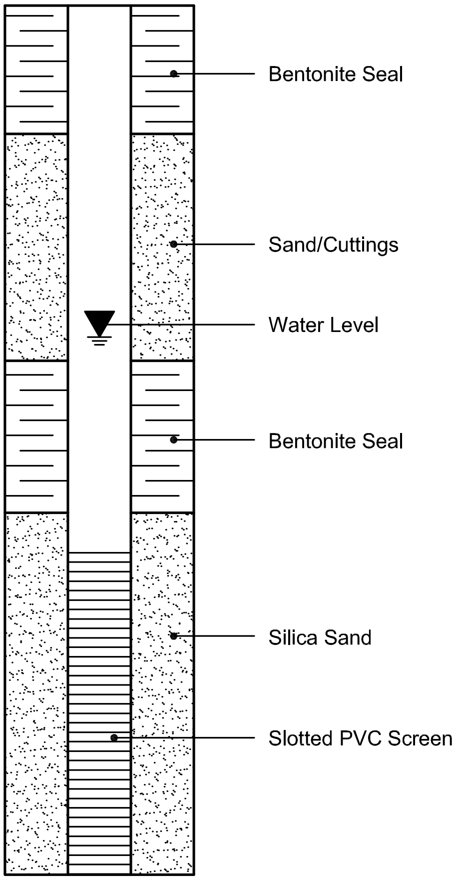

Glacial Till


Shale

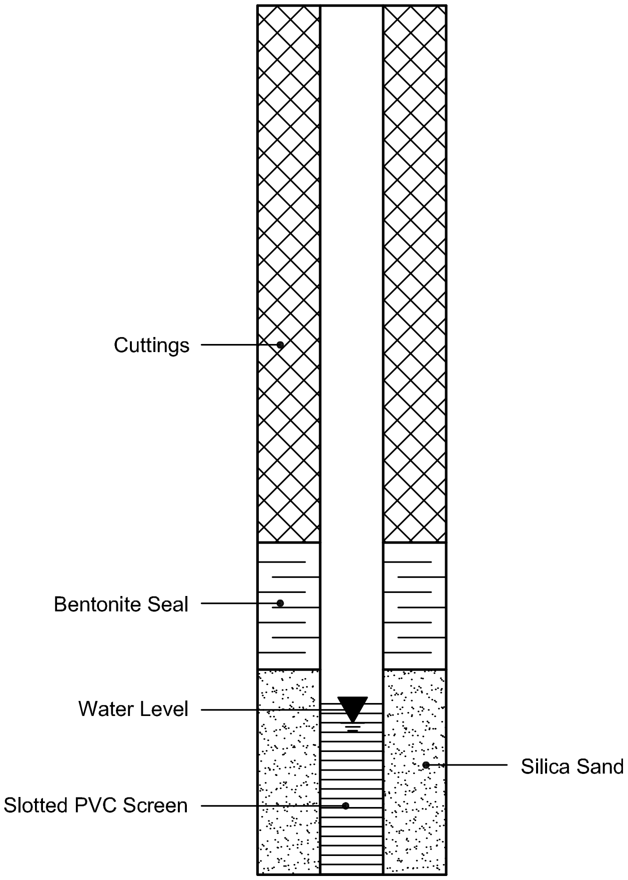

Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 32990

Report Date: 27-Sep-2021

Order Date: 21-Sep-2021

Project Description: PG5972

	Client ID:	BH1-21 SS3	-	-	-
	Sample Date:	17-Sep-21 09:00	-	-	-
	Sample ID:	2139228-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	96.2	-	-	-
General Inorganics					
pH	0.05 pH Units	8.00	-	-	-
Resistivity	0.10 Ohm.m	25.8	-	-	-
Anions					
Chloride	5 ug/g dry	152	-	-	-
Sulphate	5 ug/g dry	25	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5972-1 – TEST HOLE LOCATION PLAN

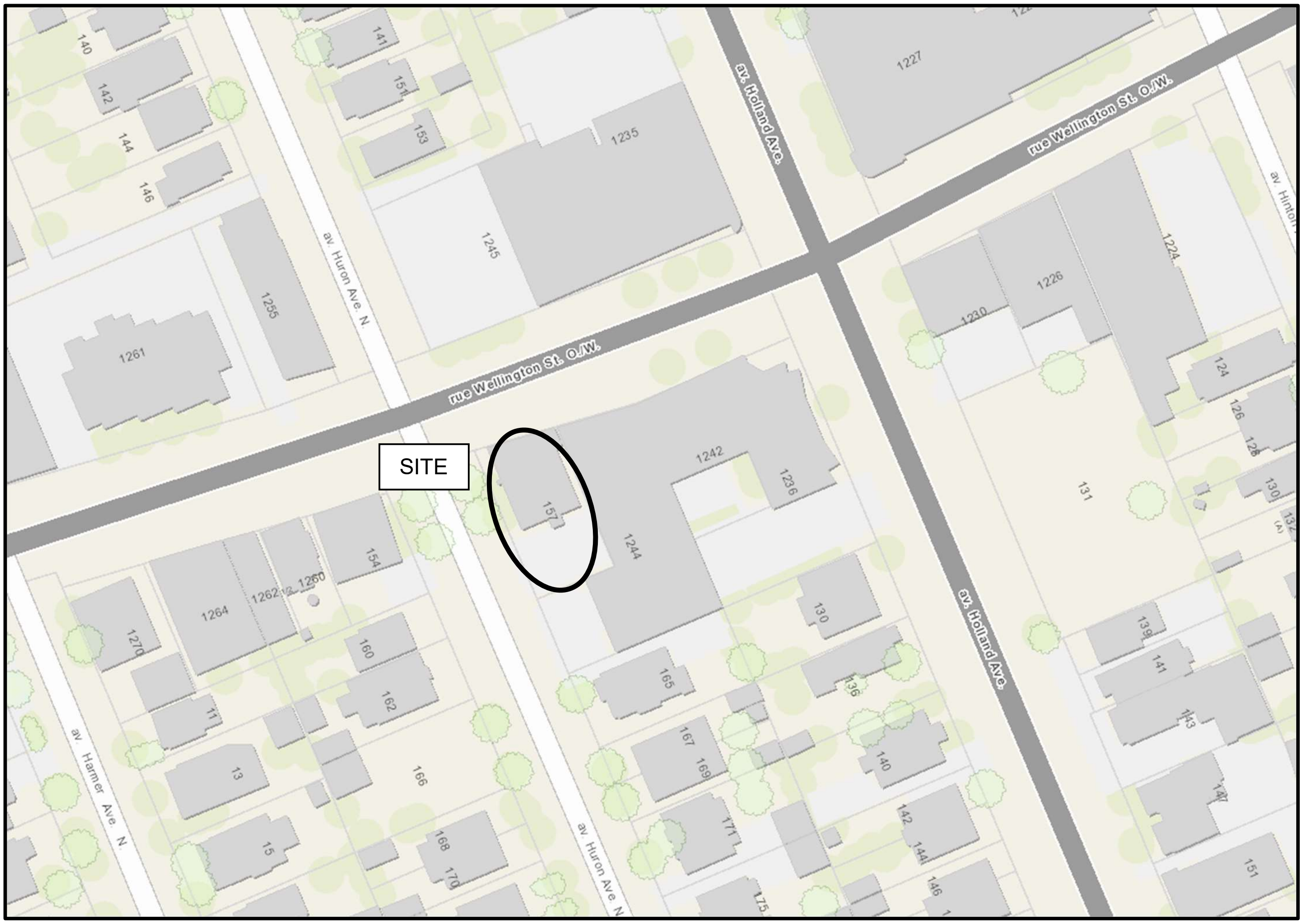
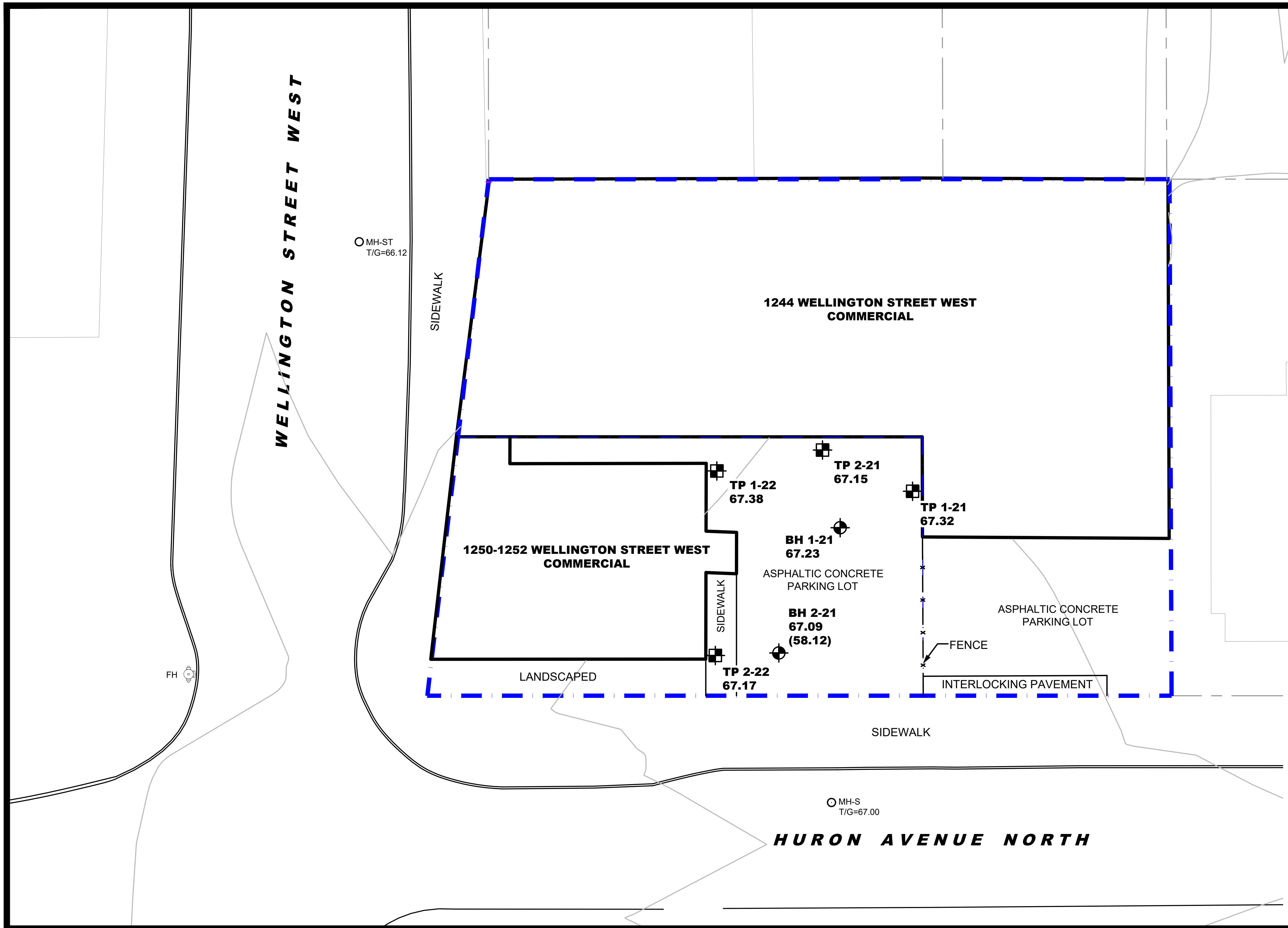


FIGURE 1

KEY PLAN



LEGEND:

⊞	TEST PIT LOCATION
⊕	BOREHOLE LOCATION
67.09	GROUND SURFACE ELEVATION (m)
(58.12)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)

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

SCALE: 1:250

0 1 2 3 4 5 10 15m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
1	TP 1-22 & TP 2-22 ADDED TO PLAN	01/14/2022	DP

DOMICILE HOLDINGS
GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING ADDITON
1250-1252 WELLINGTON STREET WEST

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 09/2021
Drawn by: JM
Report No.: PG5972-1
Checked by: DP
Dwg. No.: PG5972-1
Approved by: DJG
Revision No.: 1

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