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Geotechnical Engineering

Environmental Engineering

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Noise and Vibration Studies

Geotechnical Investigation
Proposed Multi-Storey Building
5497 Manotick Main Street
Ottawa, Ontario

Prepared For

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Report: PG5957-1

Table of Contents

Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Analytical Test Results
Appendix 2	Figure 1 - Key Plan Drawing PG5957-1 - Test Hole Location Plan Drawing PG5957-2 – Permissible Grade Raise Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by 12213559 Canada Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 5497 Manotick Main Street 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 boreholes.

- 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.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a three-storey residential building with one level of underground parking.

The remainder of the site will be developed with asphalt paved access lanes and parking areas with landscaped margins, hardscaped areas and an amenity area located at the east portion of the subject site.

It is anticipated that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried on September 3, 2021, consisting of three (3) boreholes which were advanced to a maximum depth of 6.1 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5957-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a low clearance drill rig operated by a two-person crew. 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. The samples were classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as SS and AU, 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.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at borehole 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.

Undrained shear strength testing was conducted in cohesive soils using a field vane apparatus.

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

Groundwater

All 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 borehole 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 using a handheld GPS and referenced to a geodetic datum. The locations of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG5957-1 - 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. 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

The subject site is currently occupied by a single-family residential dwelling with an associated asphalt paved driveway and at-grade parking areas located to the west of the building. The residential dwelling consists of a one-storey building with a basement. The east portion of the subject site consists of a grassed backyard followed by an area occupied by mature trees. A line of mature trees was also observed at the south boundary of the lot. The subject site slopes gradually downward towards the east, in the direction of Rideau River. A retaining wall was observed along the east side of the property.

The site is bordered by residential dwellings to the north, Rideau River to the east, commercial buildings to the south, and Manotick Main Street, followed by residential and commercial buildings to the west. The ground surface at the subject site was observed to be slightly lower than the adjacent roadway.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of a thin layer of asphaltic concrete and/or topsoil overlying a fill layer, followed by a silty clay deposit at BH 1-21 and a silty sand deposit at BH 2-21, which in turn was underlain by a glacial till deposit. Glacial till was encountered directly below at till at BH3-21.

The fill was generally observed to consist of brown silty sand, occasional organics, and a trace amount of gravel and crushed stone. The fill layer was observed to have a variable thickness, extending to approximate depths of 0.5 to 2.7 m below the existing ground surface.

The silty clay deposit generally consists of hard to very stiff brown silty clay crust. This deposit was observed to extend to an approximate depth of 3.8 m below the existing ground surface at the location of borehole BH 1-21.

The silty sand deposit was observed at the location of borehole BH 2-21. The deposit generally consisted of compact, brown silty sand with a trace amount of gravel and was observed to extend to an approximate depth of 3.7 m below the existing ground surface.

The glacial till deposit was observed at all borehole locations underlying the fill layer, silty clay, and/or silty sand deposit. The glacial till generally consisted of compact to very dense, brown silty sand with gravel, cobbles, and boulders. Practical refusal to augering was encountered at an approximate depth of 5.7 m below the existing ground surface at the location of borehole BH 1-21.

Practical refusal to DCPT was encountered at an approximate depth of 7.6 m below the existing ground surface at the location of borehole 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 Lower Ordovician Dolomite of the Oxford formation, with an overburden drift thickness of 5 to 15 m.

4.3 Groundwater

Groundwater levels were measured on September 8, 2021 within the installed piezometers. The measured groundwater levels are presented in Table 1 below:

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-21	87.45	5.33	82.12	September 8, 2021
BH 2-21	85.20	Damaged	-	
BH 3-21	84.56	Dry	-	

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

Based on these observations, the long-term groundwater table is anticipated to be below 5 m depth below existing surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1. 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

The subject site is considered suitable for the proposed development, from a geotechnical perspective. It is recommended that the proposed development be founded on conventional spread footings placed on hard to very stiff brown silty clay, compact silty sand, or compact glacial till bearing surface.

Where fill is encountered at the underside of footing elevation (such as in the vicinity of borehole BH 3-21), it should be sub-excavated to the surface of the undisturbed glacial till and replaced with engineered fill or lean concrete to the proposed founding elevation. The lateral limits of the engineered fill or lean concrete placement should be in accordance with our lateral support recommendations provided herein.

Due to the presence of the underlying silty clay layer within the west portion of the site, permissible grade raise recommendations are discussed in subsection 5.3. If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

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 building perimeter. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below grade.

Fill Placement

Fill used for grading beneath the proposed development 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 buildings and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill and beneath exterior parking where settlement of the ground surface is of minor concern. This material should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in 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.

Excess Soils

Excess soils generated by construction activities that will be transported off-site should be handled as per *Ontario Regulation 406/19: On-Site Excess Soil Management*. This can be handled by Paterson, upon request, under a separate cover.

Protection of Subgrade and Bearing Surfaces

It is expected that site grading and preparation will consist of stripping of the soils containing significant amounts of organic materials and existing topsoil piles above design underside of footing elevation. The contractor should take appropriate precautions to avoid disturbing the subgrade and bearing surfaces from construction and worker traffic. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional fill.

5.3 Foundation Design

Bearing Resistance Values

Based on the subsurface profile encountered, it is anticipated that the proposed multi-storey building will be founded on shallow foundations placed on hard to very stiff, brown silty clay, compact silty sand or compact to very dense glacial till. Footings for the proposed development can be designed using the bearing resistance values presented in Table 2.

Table 2 – Bearing Resistance Values

Bearing Surface	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Hard to very stiff, brown Silty Clay	150	225
Compact Silty Sand	120	200
Compact to very dense Glacial Till	200	300

Note: Strip footings up to 3 m wide, and pad footings up to 5 m wide can be designed for silty clay bearing mediums using the above noted bearing resistance values.
A geotechnical factor of 0.5 was used for the bearing resistance values at ULS.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces. 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 placement of concrete footings.

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 the in-situ bearing medium soils 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.

Lean Concrete Trenches

Where the building loads are such that the allowable bearing pressures previously provided are insufficient, consideration should be given to excavating vertical trenches to expose the underlying glacial till and backfilling with lean concrete (minimum 20 MPa 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying glacial till.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, a test pit should be undertaken to assess the water infiltration issues and stability of the excavation sidewalls extending to the glacial till surface.

The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by the geotechnical consultant, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to the glacial till can be designed using a factored bearing resistance value at serviceability limit state (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical factor of 0.5 was used for the bearing resistance value specified under ultimate limit states.

Settlement

Footings placed on an undisturbed soil bearing surface or lean concrete trenches placed directly on an undisturbed glacial till bearing surface and designed using the above noted bearing resistance values at SLS will be subject to potential post-construction total and differential settlements of 25 to 20 mm, respectively.

Permissible Grade Raise Recommendations

Due to the presence of the underlying silty clay within the west portion of the site, a permissible grade raise restriction of **2.0 m** is recommended for design purposes within the east portion of the subject site, in the immediate area of settlement sensitive structures. Reference should be made to Drawing PG5957-2 – Permissible Grade Raise Plan in Appendix 2.

If greater permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements of the soils surrounding the buildings.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for foundations constructed at the subject site. The soils underlying the subject site 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 the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the native soil surface, approved by the geotechnical consultant at the time of excavation, will be considered an acceptable subgrade surface on which to commence backfilling for slab-on-grade construction.

The upper 300 mm below the basement slab should consist of OPSS Granular A crushed stone. However, if storage or other uses of the lowest level will involve the construction of a concrete floor slab, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm 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 at least 98% of its SPMDD.

Where silty sand or glacial till subgrade is encountered below the basement slab, provisions should be made to proof-rolling the soil subgrade using heavy vibratory compaction equipment prior to placing any fill.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed basement space. 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, must be reviewed for design calculations. The parameters for design calculations for the two conditions 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 material
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 = $(1.45 - a_{max}/g)a_{max}$
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. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, 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

The pavement structures presented in the Tables 3 and 4 below are recommended for the design of car parking areas, access lanes and heavy truck loading/parking areas.

Table 3 – Recommended Pavement Structure – Car Only Parking Areas

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

Table 4 – Recommended Pavement Structure – Access Lanes and Heavy Truck Loading and Parking Areas

Thickness (mm)	Material Description
40	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course –** HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
450	**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 bedrock.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

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

Foundation Backfill

Backfill against the exterior sides of the basement 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 placement as backfill against the foundation walls unless used in conjunction with a composite drainage system, such as Delta Drain 6000 or Miradrain G100N. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be placed for this purpose.

6.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as those for isolated exterior piers and loading docks, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

The foundations for the underground parking levels are expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided at entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided to those areas.

6.3 Excavation Side Slopes

The side slopes of shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by temporary shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

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

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer.

Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should 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. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary shoring system could consist of a soldier pile and lagging system or steel sheet piles. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

The earth pressures acting on the temporary shoring system may be calculated with the following parameters.

Table 5 – Soils Parameter for Shoring System Design

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	20
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.

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 soil subgrade. 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 standard Proctor maximum dry density.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if 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 finish grade) should match the soils exposed at the trench walls to reduce the potential differential frost having. 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

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. 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.

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.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

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

6.7 Corrosion Potential and Sulphate

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

6.8 Slope Stability Assessment

The east boundary of the site is adjacent to the Rideau River. The existing slope is generally grass covered with some mature trees and an existing retaining wall structure. Based on current plans, there is an approximate proposed distance of 32 m from the proposed residential building to the normal high water mark, as identified on the drawing. However, it should be noted that a slope stability analysis be completed to define the Limit of Hazard Lands setbacks from the top of slope. It is understood that a slope stability assessment will be completed by others.

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 detailed grading plan(s) from a geotechnical perspective.
- Review of the proposed damp-proofing, drainage system and elevator waterproofing prior to commencement of construction.
- 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.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

The recommendations provided 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 12213559 Canada 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.

Patey

LICENSED PROFESSIONAL ENGINEER
Sept. 29, 2021
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Nicole Patey, B. Eng.

Faisal Abou-Seido, P. Eng.

Report Distribution:

☐ 12213559 Canada Inc. (email copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

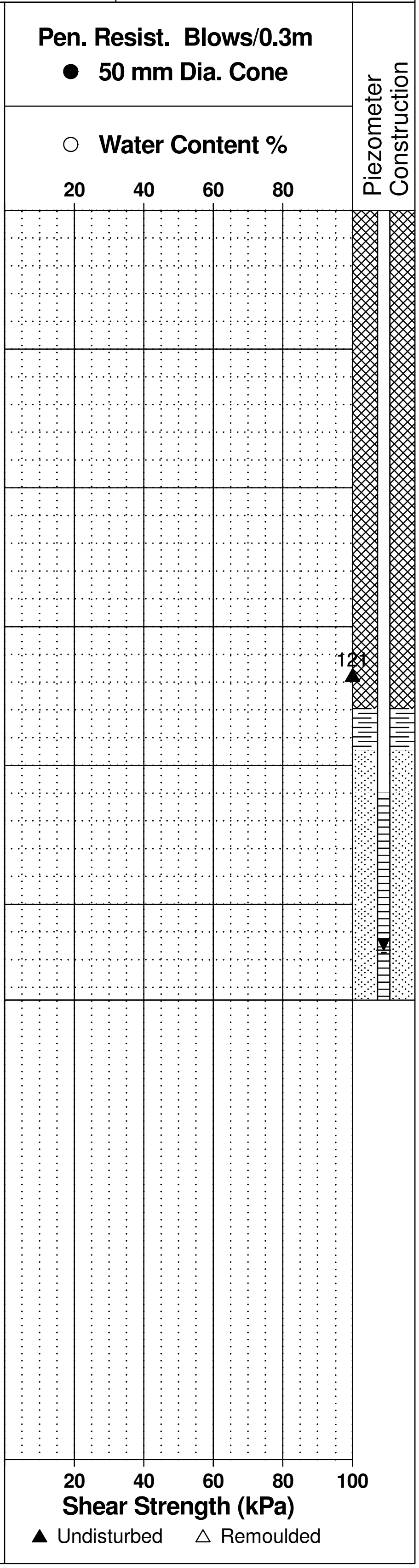
patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 5497 Manotick Main St.
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE September 3, 2021
FILE NO. PG5957
HOLE NO. BH 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	87.45
Asphaltic concrete 0.08							
FILL: Brown silty sand, trace gravel and crushed stone 0.46		AU	1				
		SS	2	92	8	1	86.45
Hard to very stiff, brown **SILTY CLAY**		SS	3	83	8		
						2	85.45
		SS	4	83	5		
						3	84.45
3.81		SS	5	100			
		SS	6	78	62	4	83.45
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders		SS	7	42	47	5	82.45
5.69		SS	8	45	50+		
End of Borehole Practical refusal to augering at 5.69m depth (GWL @ 5.33 - September 8, 2021)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 5497 Manotick Main St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 3, 2021

FILE NO. PG5957

HOLE NO. BH 2-21

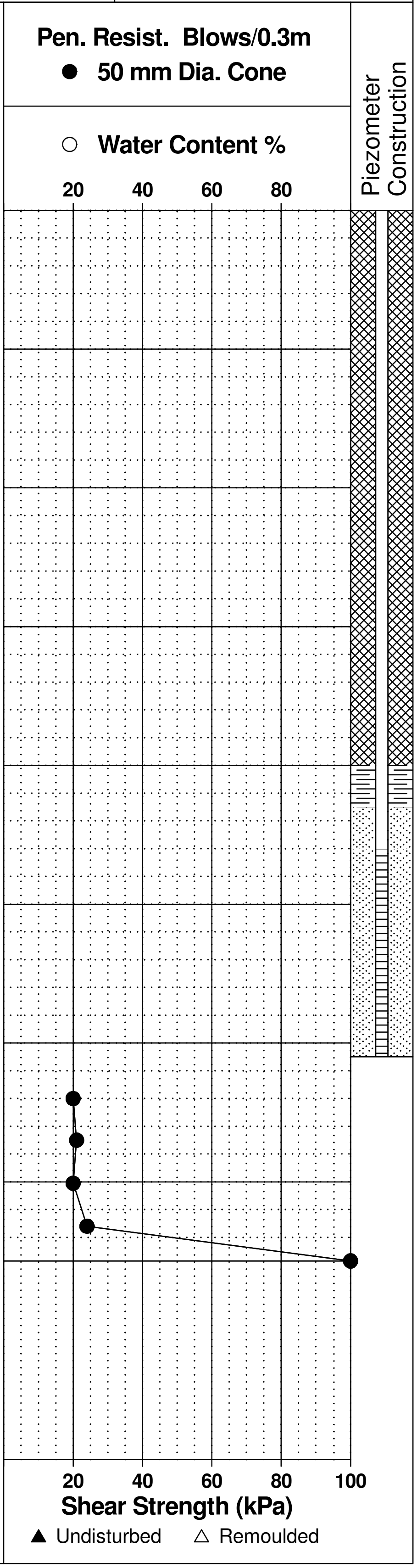
SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.13				
FILL: Brown silty sand, trace organics	0.46	AU	1		
Compact, brown **SILTY SAND,** trace gravel	3.66	SS	2	33	12
		SS	3	50	10
		SS	4	58	14
		SS	5	58	11
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders	6.10	SS	6	44	50+
		SS	7	50	28
		SS	8	58	53
Dynamic Cone Penetration Test commenced at 6.10m depth.	7.57				

End of Borehole

Practical DCPT refusal at 7.57m depth.

(Piezometer destroyed - Sept. 8, 2021)

DEPTH (m)	ELEV. (m)
0	85.20
1	84.20
2	83.20
3	82.20
4	81.20
5	80.20
6	79.20
7	78.20



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 5497 Manotick Main St.
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5957

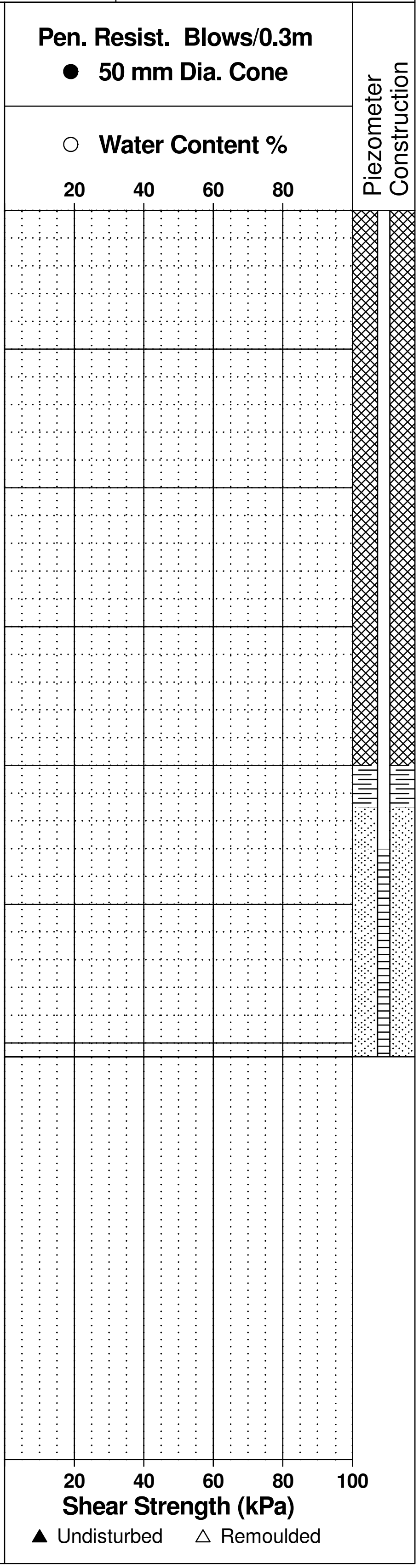
REMARKS

HOLE NO. BH 3-21

BORINGS BY CME-55 Low Clearance Drill

DATE September 3, 2021

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	84.56
TOPSOIL 0.10						
	AU	1				
					1	83.56
FILL: Brown silty sand, trace topsoil	SS	2	50	19		
- trace gravel by 1.2m depth						
	SS	3	67	8		
					2	82.56
2.74	SS	4	58	24		
					3	81.56
	SS	5	25	15		
					4	80.56
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders	SS	6	33	26		
- loose by 4.6m depth	SS	7	25	8		
					5	79.56
	SS	8	17	7		
6.10					6	78.56
End of Borehole						
(BH dry - September 8, 2021)						



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 = $(D30)^2 / (D10 \times D60)$

Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth

p'_c - Preconsolidation pressure of (maximum past pressure on) sample

Ccr - Recompression index (in effect at pressures below p'_c)

Cc - Compression index (in effect at pressures above p'_c)

OC Ratio Overconsolidaton ratio = p'_c / p'_o

Void Ratio Initial sample void ratio = volume of voids / volume of solids

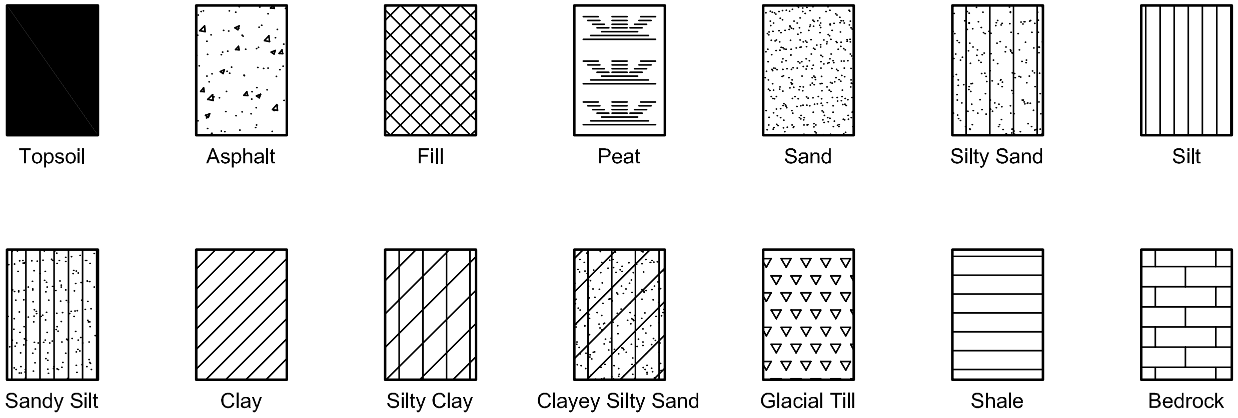
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

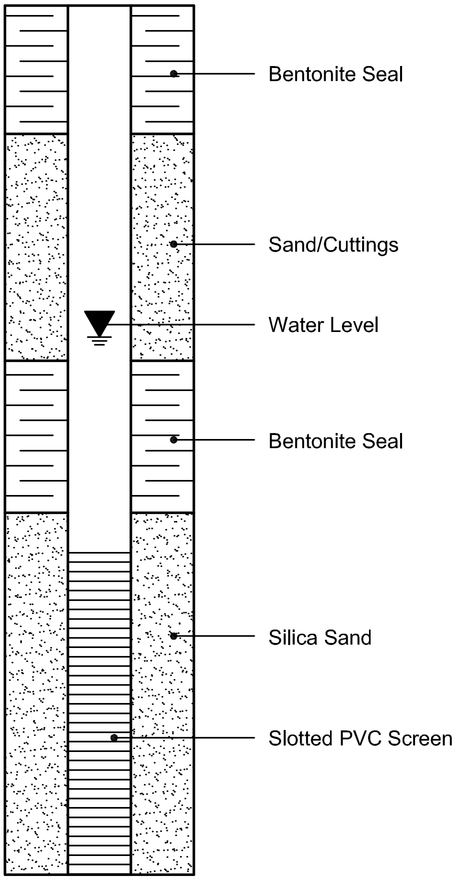
SYMBOLS AND TERMS (continued)

STRATA PLOT

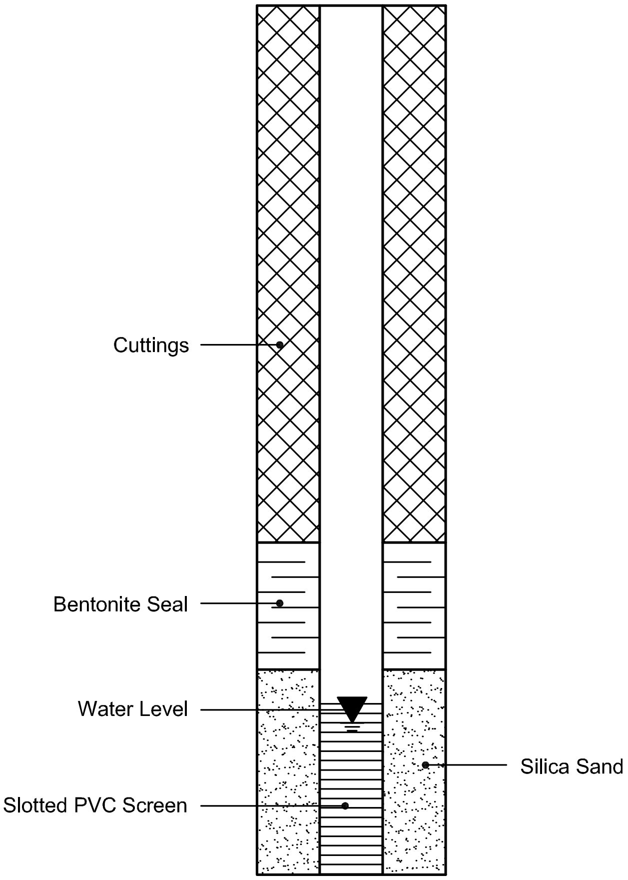


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 32746

Report Date: 16-Sep-2021

Order Date: 10-Sep-2021

Project Description: PG5957

	Client ID:	BH2-21 SS3	-	-	-
	Sample Date:	03-Sep-21 09:00	-	-	-
	Sample ID:	2137494-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	92.3	-	-	-
General Inorganics					
pH	0.05 pH Units	7.28	-	-	-
Resistivity	0.10 Ohm.m	38.0	-	-	-
Anions					
Chloride	5 ug/g dry	88	-	-	-
Sulphate	5 ug/g dry	55	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5957-1 – TEST HOLE LOCATION PLAN

DRAWING PG5957-2 – PREMISSIBLE GRADE RAISE PLAN

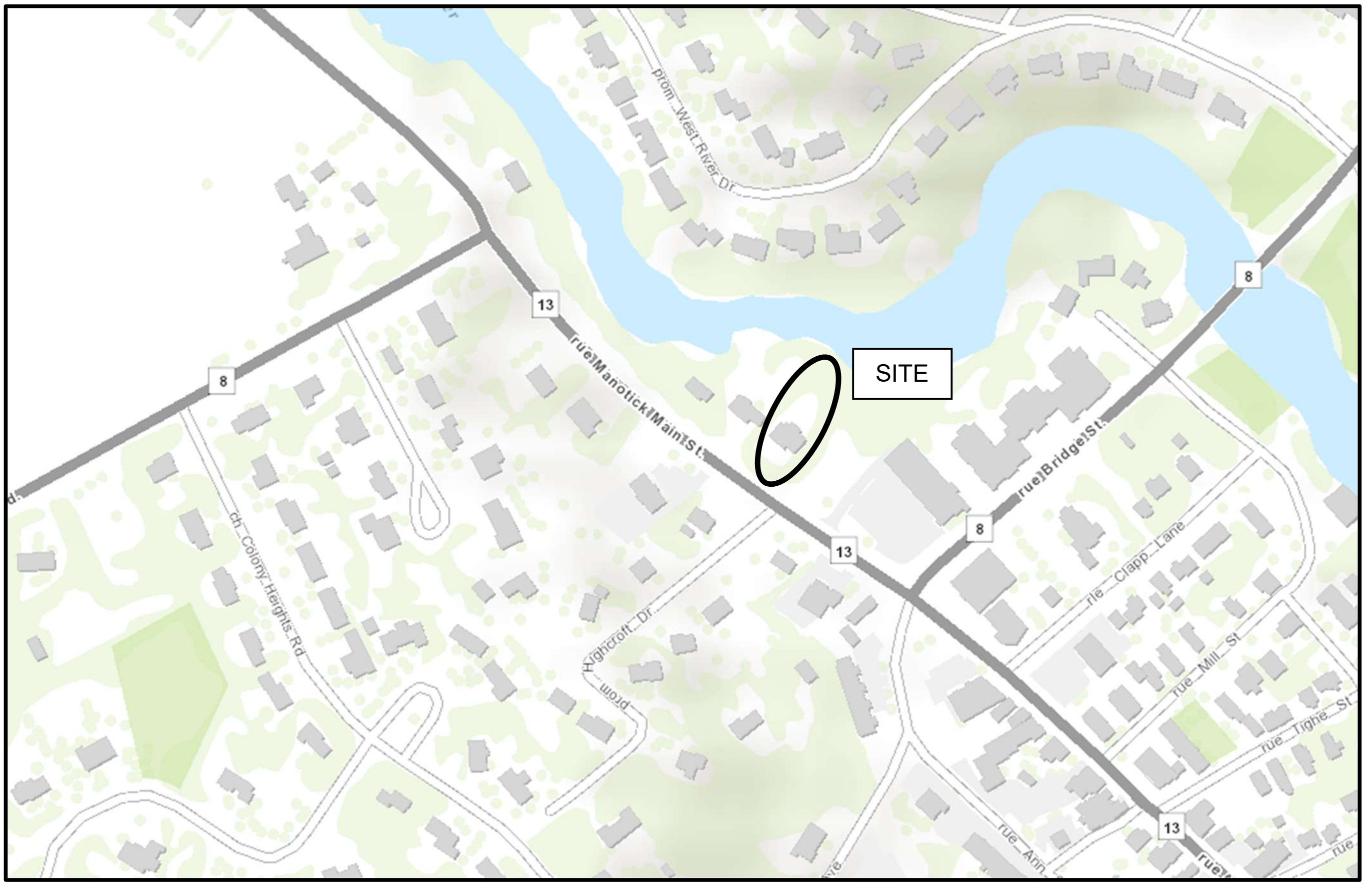
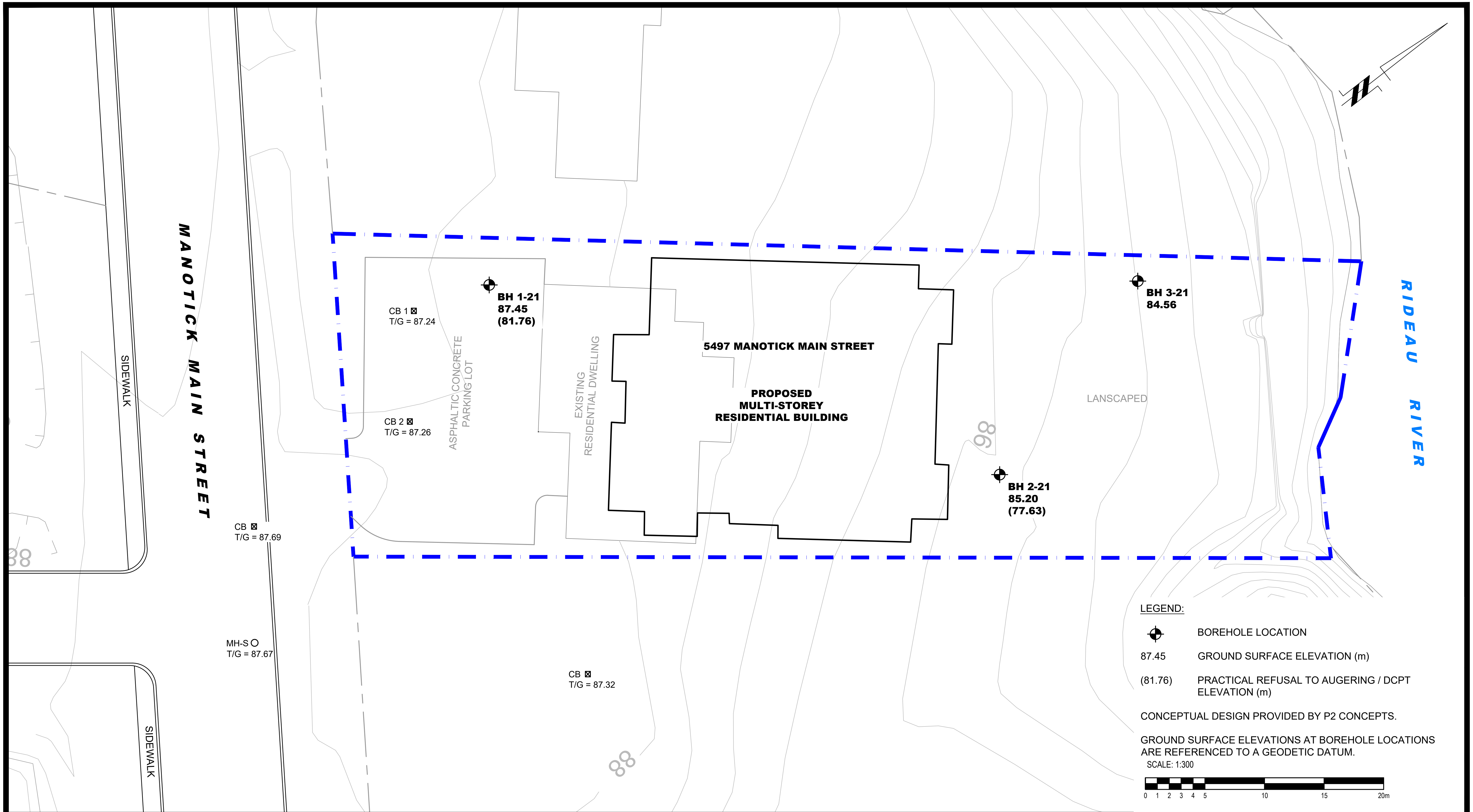


FIGURE 1

KEY PLAN



LEGEND:

⌖	BOREHOLE LOCATION
87.45	GROUND SURFACE ELEVATION (m)
(81.76)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

CONCEPTUAL DESIGN PROVIDED BY P2 CONCEPTS.

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

SCALE: 1:300

0 1 2 3 4 5 10 15 20m

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consulting engineers

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

NO.	REVISIONS	DATE	INITIAL

12213559 CANADA INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY RESIDENTIAL BUILDING
5497 MANITOCK MAIN STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:300
Date: 09/2021
Drawn by: JM
Report No.: PG5957-1
Checked by: NP
Dwg. No.: PG5957-1
Approved by: FAS
Revision No.:

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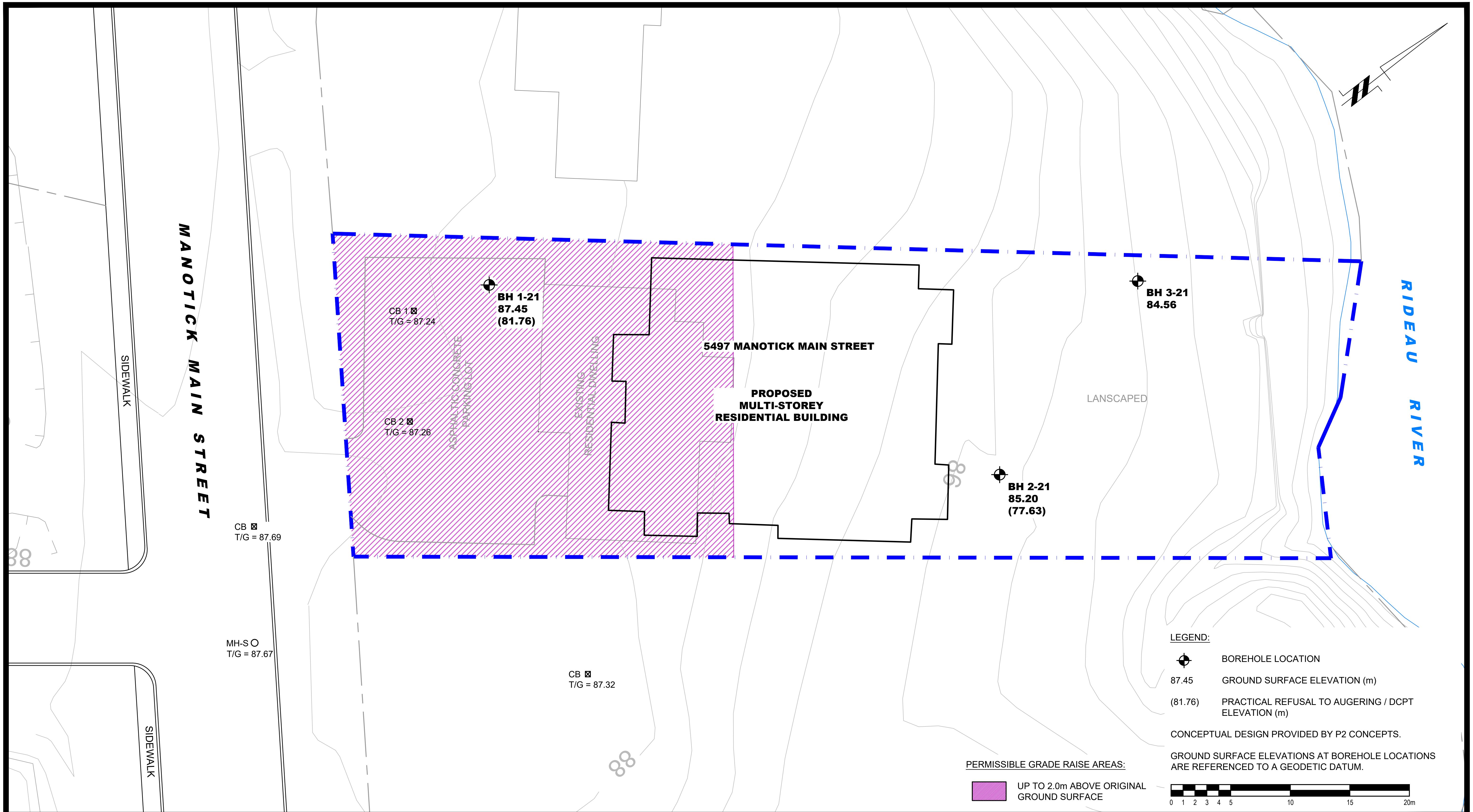
11x17



LEGEND:

⌖	BOREHOLE LOCATION
87.45	GROUND SURFACE ELEVATION (m)
(81.76)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

CONCEPTUAL DESIGN PROVIDED BY P2 CONCEPTS.

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

PERMISSIBLE GRADE RAISE AREAS:

▨ UP TO 2.0m ABOVE ORIGINAL GROUND SURFACE

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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

12213559 CANADA INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY RESIDENTIAL BUILDING
5497 MANOTICK MAIN STREET
OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE

Scale: 1:300
Date: 09/2021
Drawn by: JM
Report No.: PG5957-1
Checked by: NP
Dwg. No.: **PG5957-2**
Approved by: FAS
Revision No.:

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11x17