Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

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Geotechnical Investigation

Proposed Multi-Storey Building
91,93 Holland Avenue
Ottawa, Ontario

Prepared For

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Report PG5696-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Nicholson Gluckstein to conduct a geotechnical investigation for the proposed multi-storey building to be located at 91 and 93 Holland Avenue, in the city of Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objectives of the geotechnical investigation were to:

- ❏ determine the subsoil and groundwater conditions at this site by means of test holes.
- ❏ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

The following report has been prepared specifically and solely for the aforementioned project. This report contains geotechnical findings and includes recommendations pertaining to the design and construction of the proposed 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 for this current geotechnical investigation. Therefore the current report does not address environmental concerns.

2.0 Proposed Development

It is understood that the proposed project will consist of a multi-storey building with one underground parking level. 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 investigation was carried out on March 8, 2021. At that time, three (3) boreholes were advanced to a maximum depth of 5.3 m below the existing ground surface. The test hole locations were distributed across the site in a manner to provide general coverage of the subject site. The locations of the test holes are shown on Drawing PG5696-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a low-clearance track mounted auger 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 augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter splitspoon (SS) sampler or from the auger flights. The split-spoon and auger samples were classified on site, placed in sealed plastic bags, and transported to the laboratory for further review. The depths at which the split-spoon and auger 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.

Diamond drilling was carried out to extend the boreholes and confirm bedrock when practical refusal of the drilling equipment was encountered. The depth at which the rock core samples are recovered from the test holes are shown as RC on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Groundwater

Monitoring wells were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Ground observations are discussed in Subsection 4.3 and 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 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 personnel to provide general coverage of the subject site. The boreholes were located in the field by Paterson. The ground surface elevations at the test hole locations were determined by Paterson and are referenced to a geodetic datum. The test hole locations and the ground surface elevation at each test hole location are presented on Drawing PG5696-1 - Test Hole Location Plan included in Appendix 2.

3.3 Laboratory Review

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site consists of two commercial and residential dwellings. A fenced gravel parking and asphalt driveway were observed to the eastern portion of the site. The subject site is bordered by a mid-rise building to the east, and a high-rise residential development to the south. To the north, there is a two storey residential building and a health care center. To the west of the subject site is Holland Avenue. The existing ground surface across the subject site is relatively flat and at-grade with Holland Avenue.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile consists of a fill layer of brown silty sand with gravel. The fill layer is underlain by a very stiff to stiff brown silty clay layer which in turn is followed by a compact glacial till deposit. The glacial till deposit consists of brown silty sand with gravel, cobbles and boulders extending to a maximum depth of 4.2 m.

Bedrock

Bedrock was encountered in all boreholes at depths ranging from 3.2 m to 4.3 m. The bedrock consisted of grey limestone of good to excellent quality.

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic interbedded limestone and dolomite from the Gull River formation, with an overburden drift thickness of 3 to 5 m depth.

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

4.3 Groundwater

The groundwater levels measured in the boreholes are presented in Table 1. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, it is estimated that groundwater can be expected between 4 and 5 m depth below existing ground surface. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-21	63.48	4.44	59.04	March 8, 2021
BH 2-21	64.15	4.83	59.32	March 8, 2021
BH 3-21	63.53	2.73	60.80	March 8, 2021

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed project. It is anticipated that the proposed multi-storey building will be founded on conventional spread footings placed on a clean, bedrock bearing surface. Where bedrock is not encountered at the design underside of footing elevation, consideration should be taken to transferring the building loads to the bedrock surface by means of a lean concrete in-filled trench. Our foundation design recommendations are further detailed in Subsection 5.3.

It is expected that the underground parking level will occupy the majority of the site. Therefore, a temporary shoring system, comprised of soldier piles and wood lagging will be required to establish the construction of the underground parking level. This is discussed further in section 6.3.

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

5.2 Site Grading and Preparation

Stripping Depth

It is expected that the majority of the overburden will be removed based on the anticipated coverage of the underground parking level.

Fill Placement

Fill placed for grading beneath the structure(s) or other settlement sensitive areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to the delivery to the site. The engineered fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the materials Standard Proctor Maximum Dry Density (SPMDD).

Non-specified existing fill along with site-excavate soil can be placed as general landscaping fill where surface settlement is a minor concern. The backfill should be spread in thin lifts and, at minimum, compacted by the tracks of the spreading equipment to minimize voids. If the non-specified fill is to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 95% of the material's SPMDD. Non-specified existing fill and site excavated soils are not suitable for placement as backfill against foundations walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

If excavated rock is to be placed as exterior backfill, the bedrock should be crushed to produce a well-graded material, similar to a 150 mm minus crushed stone material and approved by the geotechnical consultant. Where the crushed bedrock is open-graded a layer of woven geotextile, such as Terrafix 200W or equivalent, is recommended to prevent adjacent finer materials from migrating into the voids that are associated with loss of ground and settlements.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling, controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards.

Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a preconstruction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

5.3 Foundation Design

Bearing Resistance Values

Footings placed directly on clean, surface sounded bedrock, can be designed using a factored bearing resistance value at Ultimate Limit States (ULS) of **2,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Lean Concrete Trenches

Where bedrock is encountered below the design underside of footing elevation, consideration should be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (**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 bedrock.

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 bedrock 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 bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa.**

Shallow Footings on Glacial Till Bearing Surface

Footings placed on an undisturbed glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

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

Settlement

Footings placed on a soil bearing surface and designed using the bearing resistance values at SLS given for the soil bearing surface will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Footings bearing on directly on a sound bedrock or lean concrete trenches placed directly on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible post-construction total and differential settlements.

Soil/Bedrock Transition Areas

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the subexcavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

Lateral Support

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

Adequate lateral support is provided to a compact to dense silty sand and glacial till bearing surface above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher bearing capacity as the bearing medium soil.

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

5.4 Design for Earthquakes

For design purposes, the site class for seismic site response can be taken as **Class C** for the foundations considered at this site. A higher site class, such as Class A or B, is possible for footings placed within 3 m of the bedrock surface. However, the higher seismic site class would need to be confirmed by site-specific shear wave velocity testing. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed building, the native soil surface will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. The recommended pavement structures noted in Subsection 5.7 will be applicable for the founding level of the proposed parking garage structure. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist 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 a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions encountered at the time of the fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a sump pit, should be provided in the clear stone under the lower parking level. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed (discussed further in Subsection 6.1).

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, 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 parameters for design calculations for the two conditions are presented below.

Static Conditions

The static horizontal earth pressure (p_o) could be calculated with 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 with 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 Conditions

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}) could 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. The vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions could be calculated using $P_o = 0.5\ K_o \gamma H^2$, where $K_o = 0.5$ for the soil conditions presented above. The total earth force (P_{AE}) is considered to act at a height, h(m), from the base of the wall, where:
$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

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

5.7 Pavement Structure

Pavement Design

For design purposes, the pavement structure presented in the following tables could be used for the design of the pavement structure for the car only parking areas and access lanes.

Table 2 - 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 fill	

Table 3 - Recommended Pavement Structure - Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
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 fill	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

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 SPMDD with suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed structure. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The clear stone should be wrapped in a non-woven geotextile. The pipes should have a positive outlet, such as a gravity connection to a sump pit located in the lowest basement level or storm sewer.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm in perforated pipes be placed in each bay. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. 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 composite drainage blanket, such as Miradrain G100N or Delta Drain 6000.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

At this site, temporary shoring will be required to complete the majority of the required excavations. However, it is recommended that where sufficient room is available open cut excavation in combination with temporary shoring can be used.

Temporary Side Slopes

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. 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 the groundwater level. It may be possible that in localized areas, where Type 3 soils are present, a 1.5H:1V excavation side slope may be required.

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

It is anticipated that temporary shoring is required to complete the required excavation 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 system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 4 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	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 & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. If the bedding is placed on bedrock, 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 a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the 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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be 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.

Groundwater Control for Building Construction

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 an aggressive to very aggressive corrosive environment.

7.0 Recommendations

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

- ❑ Review of the geotechnical aspects of the excavating contractor’s shoring design, prior to construction.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of the 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 its 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 Nickolson Gluckstein or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
April 1, 2021
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100116130
PROVINCE OF ONTARIO

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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
Proposed Multi-Storey Residential Development
91, 93 Holland Avenue - Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 March 8

FILE NO. PG5696

HOLE NO. BH 1-21

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	63.48
FILL: Brown silty sand with gravel, some topsoil	AU	1				
0.69						
Very stiff to stiff brown **SILTY CLAY**	SS	2	75	11	1	62.48
	SS	3	67	9	2	61.48
2.34						
GLACIAL TILL: Very dense brown silty sand with gravel, cobbles and boulders	SS	4	50	73		
	SS	5	33	62	3	60.48
	SS	6	33	+50	4	59.48
4.27						
BEDROCK: Excellent to fair quality grey limestone	RC	1	100	92	5	58.48
- Vertical seams from 6.10 m to 6.40 m and 6.71 m to 7.37 m	RC	2	100	57	6	57.48
7.37					7	56.48
End of Borehole (GWL @ 4.44 m depth - March 17, 2021)						

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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FILE NO. PG5696

REMARKS

HOLE NO. BH 2-21

BORINGS BY CME 55 Power Auger

DATE 2021 March 8

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic Concrete	0.05				
FILL: Brown silty sand with crushed stone trace clay	0.69	AU	1		
FILL: Brown silty sand with gravel, coal and construction debris	1.68	SS	2	33	45
Very stiff brown **SILTY CLAY**	2.29	SS	3	50	17
GLACIAL TILL: Compact brown silty sand with gravel, cobbles and boulders - Cored boulder from 3.68m to 4.04 m	4.04	SS	4	17	20
		SS	5	13	+50
BEDROCK: Good to excellent quality grey limestone	7.04	RC	1	100	88
		RC	2	100	100
End of Borehole (GWL @ 4.83 m depth - March 17, 2021)					

DEPTH (m)	ELEV. (m)
0	64.15
1	63.15
2	62.15
3	61.15
4	60.15
5	59.15
6	58.15
7	57.15

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Residential Development
91, 93 Holland Avenue - Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 March 8

FILE NO. PG5696

HOLE NO. BH 3-21

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with crushed stone, topsoil and organics	0.05	AU	1		
FILL: Brown silty sand trace topsoil, organics and construction debris	1.52	SS	2	33	10
Very stiff brown **SILTY CLAY**	2.29	SS	3	50	13
GLACIAL TILL: Very dense brown silty sand with gravel, cobbles and boulders	3.28	SS	4	33	76
		SS	5	87	+50
BEDROCK: Good to excellent quality grey limestone	6.86	RC	1	100	70
		RC	2	100	98
End of Borehole					
(GWL @ 2.73 m depth - March 17, 2021)					

DEPTH (m)	ELEV. (m)
0	63.53
1	62.53
2	61.53
3	60.53
4	59.53
5	58.53
6	57.53

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = $D60 / D10$

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

CONSOLIDATION TEST

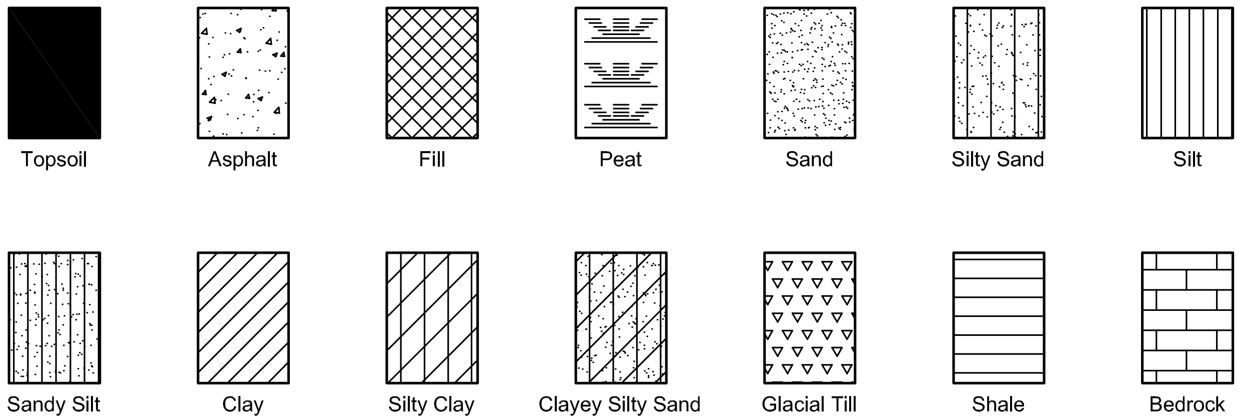
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

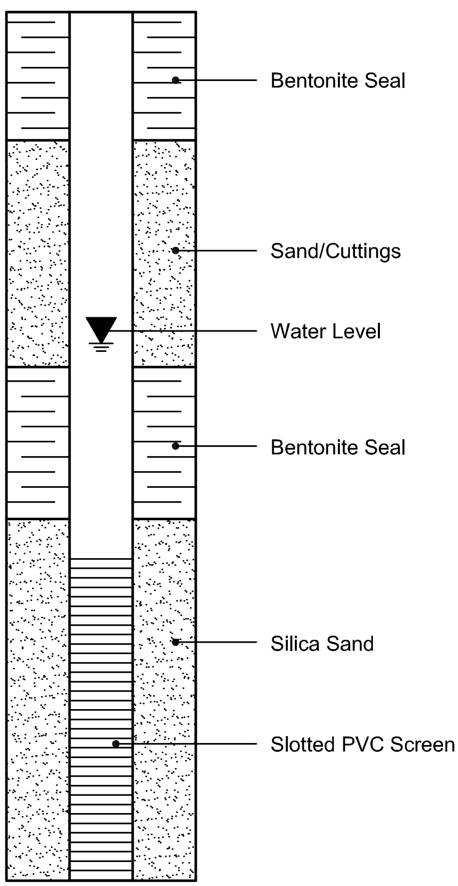
SYMBOLS AND TERMS (continued)

STRATA PLOT

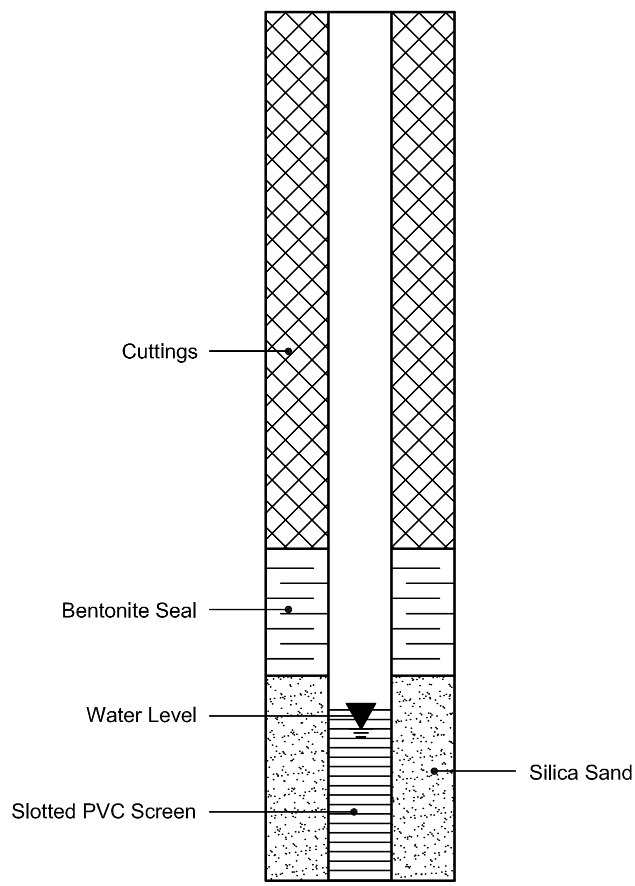


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 31974

Report Date: 15-Mar-2021

Order Date: 10-Mar-2021

Project Description: PG5696

	Client ID:	BH2.SS3	-	-	-
	Sample Date:	08-Mar-21 12:00	-	-	-
	Sample ID:	2111396-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	76.0	-	-	-
General Inorganics					
pH	0.05 pH Units	7.70	-	-	-
Resistivity	0.10 Ohm.m	9.97	-	-	-
Anions					
Chloride	5 ug/g dry	161	-	-	-
Sulphate	5 ug/g dry	665	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5696-1 - TEST HOLE LOCATION PLAN

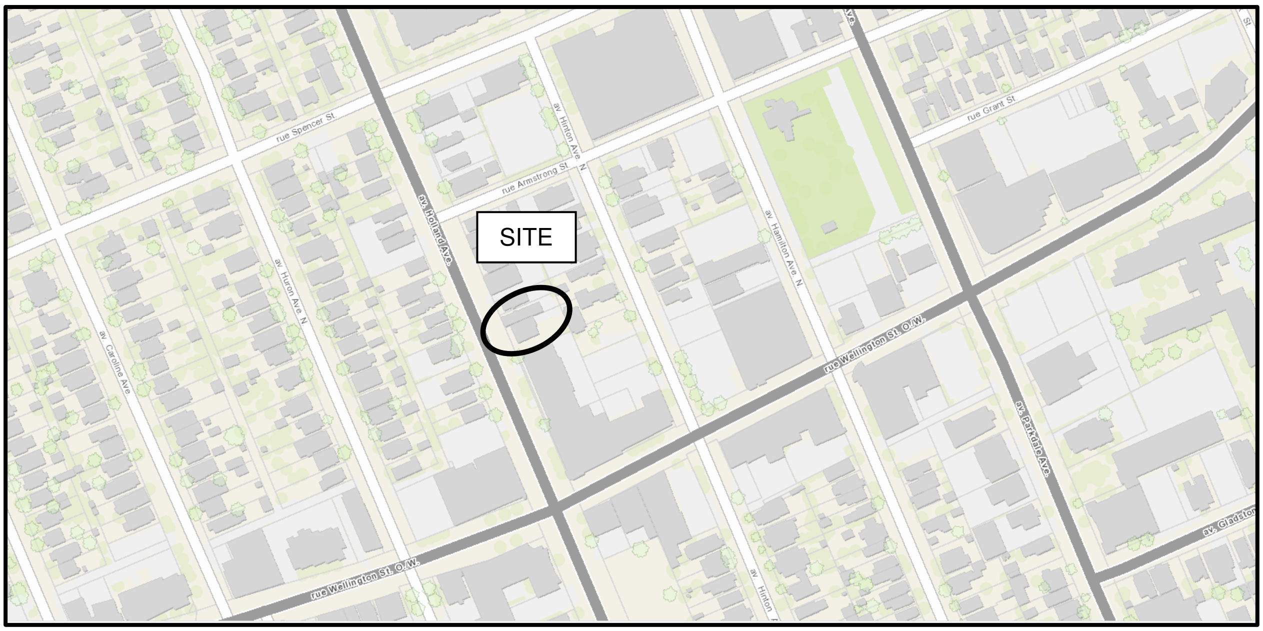


FIGURE 1

KEY PLAN

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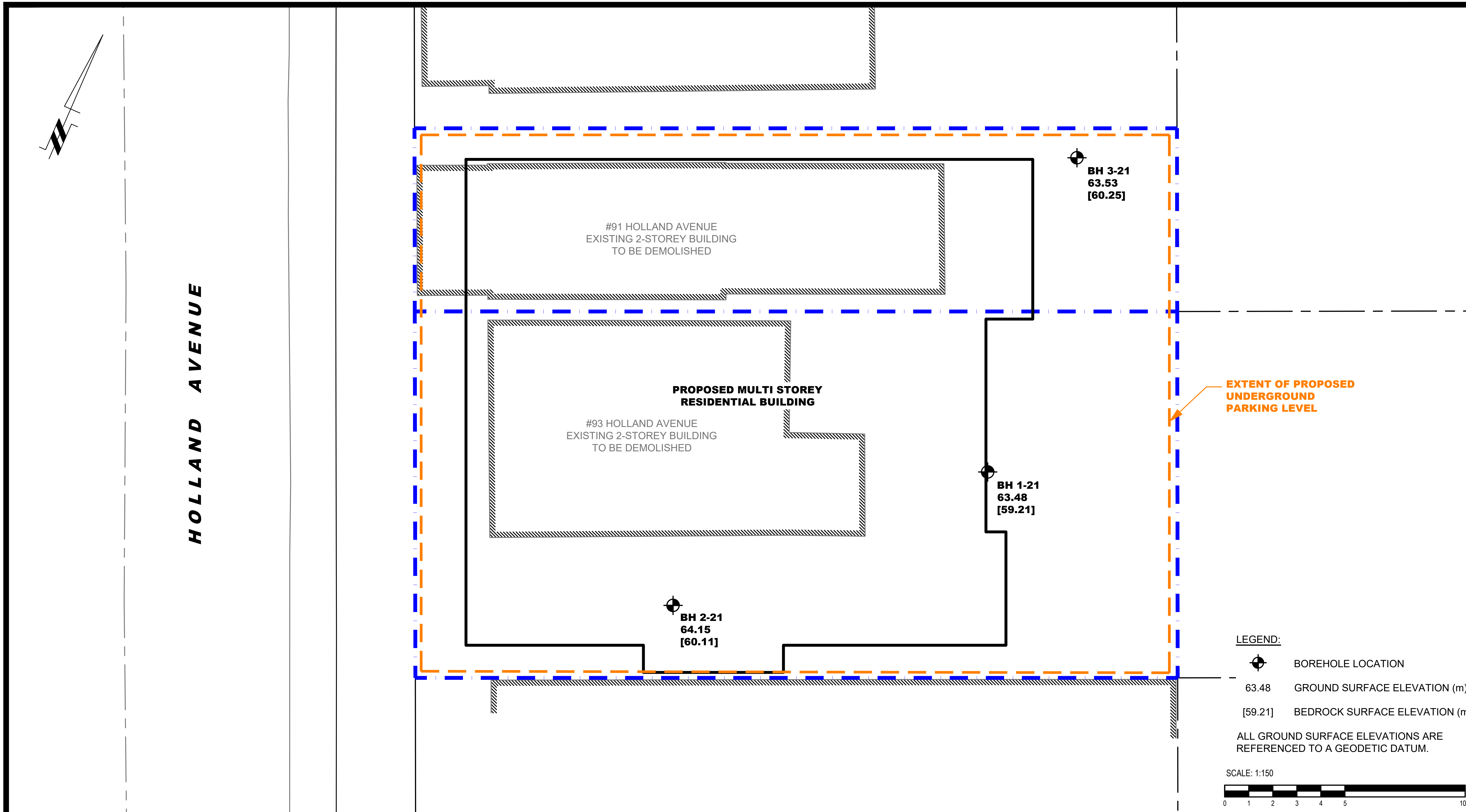

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NO.	REVISIONS	DATE	INITIAL

NICHOLSON GLUCKSTEIN

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI STOREY RESIDENTIAL BUILDING
91 & 93 HOLLAND AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:150	Date: 02/2021
Drawn by: NFRV	Report No.: PG5696-1
Checked by: FC	Dwg. No.: PG5696-1
Approved by: DJG	Revision No.:

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