PATERSON GROUP

Geotechnical Investigation

Proposed Residential Building

197-201 Wilbrod Street
Ottawa, Ontario

Prepared for Empire Holdings

Report PG5955-1 Revision 2 dated January 26, 2023

Table of Contents

Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Grain Size Distribution and Hydrometer Testing Results Atterberg Limit Testing Results Analytical Testing Results
Appendix 2	Figure 1 - Key Plan Drawing PG5955-1 - Test Hole Location Plan
Appendix 3	Relevant Memorandums

1.0 Introduction

Paterson Group (Paterson) was commissioned by Empire Holdings to conduct a geotechnical investigation for the proposed residential development site to be located at 197-201 Wilbrod Street in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

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

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

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

2.0 Proposed Development

Based on the available drawings and correspondence with the client, it is understood that the proposed development will consist of a mid-rise residential building with one basement level. Associated access lanes, parking areas, walkways, and landscaped areas are also anticipated as part of the development. It is expected that the proposed buildings will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on September 21, 2021 and consisted of advancing a total of 2 boreholes to a maximum depth of 7.5 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5955-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a low-clearance, rubber track-mounted 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 collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler. The samples were initially 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 AU, and SS respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils using a 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

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

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5955-1 - 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. A total of 1 shrinkage test, 1 grain size distribution analyses, and 2 Atterberg limits tests were completed on selected soil samples. The results are presented in Subsection 4.2 and on Grain Size Distribution and Hydrometer Testing, and Atterberg Limit's Results and Shrinkage Test Results sheets presented in Appendix 1.

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 Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The ground surface across the subject site is relatively flat and at grade with the surrounding roadways. The subject site consists of two residential lots currently occupied by residential dwellings with associated landscaped areas, fences, and driveways.

The site is bordered by residential dwellings to the north, east and west and by Wilbrod Street and a multi-purpose building to the south. The existing ground surface across the site is relatively level with an approximate geodetic elevation of 70 m.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of topsoil underlain by fill. The fill was generally observed to consist of brown silty sand and brown silty clay with some gravel and topsoil. A hard to stiff, brown silty clay layer was encountered underlying the fill, transitioning into a grey silty clay. A compact, grey silty sand with trace clay was observed underlying the silty clay, followed by a thin layer of glacial till, consisting of grey sandy silt with clay, gravel cobbles and boulders.

Practical refusal to augering was encountered on inferred bedrock surface at 7.52 m depth at BH 1-21.

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 interbedded limestone and shale of the Verulam formation, with an overburden drift thickness of 5 to 10 m depth.

Atterberg Limit and Shrinkage Tests

Atterberg limits testing, as well as associated moisture content testing, was completed on the recovered silty clay samples at selected locations throughout the subject site. The results of the Atterberg limits are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results					
Sample	**Depth (m)**	**LL (%)**	**PL (%)**	**PI (%)**	**Classification**
BH 1-21 SS4	2.3 – 2.9	69	38	31	CH
BH 2-21 SS4	2.3 – 2.9	75	39	36	CH
Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; CH: Inorganic Clay of High Plasticity MH: Inorganic Silt of High Plasticity					

The results of the moisture contest test are presented in Table 2 and on the Soil Profile and Test Data Sheet in Appendix 1.

The results of the shrinkage limit test indicate a shrinkage limit of 21.0% and a shrinkage ratio of 1.69.

Table 2 – Moisture Content Results			
Borehole	**Sample**	**Depth (m)**	**Water Content (%)**
BH 1-21	SS2	1.1	30.8
BH 1-21	SS3	1.8	38.1
BH 1-21	SS4	2.6	47.6
BH 1-21	SS5	3.4	63.6
BH 1-21	SS6	4.1	68.8
BH 1-21	SS7	4.9	65.1
BH 1-21	SS8	5.6	53.4
BH 1-21	SS9	6.4	25.7
BH 1-21	SS10	7.2	8.0
BH 2-21	SS2	1.1	32.8
BH 2-21	SS3	1.8	36.3
BH 2-21	SS4	2.6	49.8
BH 2-21	SS5	3.4	57.0
BH 2-21	SS6	4.2	66.2
BH 2-21	SS7	4.9	61.2

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) was also completed on one soil sample. The results of the grain size analysis are summarized in Table 3 and presented on the Grain-size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 3 - Summary of Grain Size Distribution Analysis

Test Hole	Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 2-21	SS3	0.0	1.9	21.6	76.5

4.3 Groundwater

Groundwater levels were measured on September 1, 2021 within the installed polytube piezometers. The measured groundwater levels are presented in Table 4 below.

Table 4 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-21	69.53	Dry	N/A	September 28, 2021
BH 2-21	69.67	5.01	64.66	September 28, 2021

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

It should be noted that long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 4.5 to 5.0 m below ground 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

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed development will be founded on conventional spread footings placed on an undisturbed, hard to stiff, brown silty clay.

Due to the presence of a silty clay deposit, a permissible grade raise restriction is required for the subject site for all footings founded on a silty clay bearing surface.

The above and other considerations are discussed in the following paragraphs.

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. It is anticipated that existing fill within the proposed building footprint, free of deleterious material and significant amounts of organics, and approved by the geotechnical consultant at the time of construction can be left in place outside of the proposed building footprint and within landscaped areas. However, it is recommended that the existing fill layer be proof-rolled by a vibratory roller making several passes under dry and above freezing conditions and approved by the geotechnical consultant at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

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

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 99% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials 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 excavated brown very stiff to stiff brown silty clay, free of organics and deleterious materials, is to be used to build up the subgrade level for areas to be paved, the silty clay, under dry conditions, should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD using a sheepsfoot roller. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000, connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Foundation)

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, stiff brown silty clay can be designed using a bearing resistance value at serviceability limit states (SLS) of **180 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa** incorporating a geotechnical factor of 0.5.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed, in the dry, prior to the placement of concrete footings.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to silty clay and engineered fill bearing media 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 or engineered fill of the same or higher capacity as that of the bearing medium.

Permissible Grade Raise

A permissible grade raise restriction of **2 m** is recommended for the subject site. 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.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for foundations constructed at the subject site. A higher site class may be achievable, such as Class C. However, a site-specific shear-wave velocity test is required to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.6.4.A of the Ontario Building Code (OBC) 2012. 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 Construction

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the native silty clay will be considered an acceptable subgrade upon which to commence backfilling for slab-on-grade or basement slab construction.

All backfill material within the footprint of the proposed buildings (but outside the zones of influence of the footings) should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD. Within the zones of influence of the footings, the backfill material should be compacted to a minimum of 98% of its SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material. A clear crushed stone fill is recommended for backfilling below the floor slab for limited span slab-on-grade areas, such as front porch or garage footprints. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone below basement floor slabs.

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.

Lateral Earth Pressure

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

K_o = At-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the basement 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 willonly 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 exercisedduring the compaction of the backfill materials to stay at least 0.3 m away 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 $0.375 \cdot a \cdot \gamma \cdot H^2/g$ 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 earthforce component (P_o) under seismic conditions can be calculated using:

$P = 0.5\ K \cdot \gamma \cdot H^2$, where $K = 0.5$ 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 thewall, 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 shouldbe factored as live loads, as per OBC 2012.

If the basement walls are to be poured against a waterproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3).

Where soil is retained, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees anda 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 as13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total staticearth pressure when using the effective unit weight.

5.7 Pavement Design

Car only parking areas and heavy traffic access areas are expected at this site. The subgrade material will consist of native soil, fill and possibly bedrock. The proposed pavement structures are presented in Tables 5 to 7.

Table 5 – Recommended Pavement Structure – Pedestrian Pathways

Thickness (mm)	Material Description
50	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
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, bedrock or concrete fill.	

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 compaction equipment.

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 I or II material.

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

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Due to the impervious nature of the silty clay deposit, where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction. The subdrain inverts should be approximately 300 mm below subgrade level and run longitudinal along the curb lines. The subgrade surface should be crowned to promote water flow to the drainage lines.

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 structures. The system should consist of a 150 mm diameter perforated, corrugated plastic pipe which is surrounded on all sides by 150 mm of 19 mm clear crushed stone and is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pit.

Foundation Backfill

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

6.2 Protection of Footings Against Frost Action

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

Other exterior unheated footings, such as those for isolated exterior piers and retaining walls, are more prone to deleterious movement associated with frost action. A minimum of 2.1 m thick soil cover (or equivalent) should be provided for all exterior unheated footings, such as canopy footings.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations). Where space restrictions exist, or to reduce the trench width, the excavation can be carried out within the confines of a fully braced steel trench box.

Unsupported Side Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The 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 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 shoring system may be calculated with the following parameters.

Table 6 - 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
Dry Unit Weight (γ), kN/m^3	20
Effective 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. 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 upper portion of the dry to moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

The backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. The seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material.

The seals should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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 the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, 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 non-aggressive to slightly aggressive corrosive environment.

6.8 Landscaping Considerations

The proposed development is located in an area of low to medium sensitive silty clay deposits for tree planting. Based on our review of the subsurface profile below the subject site, the underlying silty clay deposit is relatively dry and designated as a very stiff to firm silty clay. Therefore, the proposed development is considered to be located within an area of low sensitive silty clay deposits for tree planting.

Tree Planting Restrictions

Based on the results of the representative soil samples, the subject site is considered as a **low/medium** sensitivity area for tree planting according to the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines)

Since the modified plasticity limit (PI) generally does not exceed 40%, large trees (mature height over 14 m) can be planted at the subject site provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space).

Based on our testing results, tree planting setback limits should be 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the following conditions are met:

- ❑ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.

- ❑ A small tree must be provided with a minimum of 25 m^3 of available soil volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

- ❑ The The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

- ❑ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

- ❑ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree).

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.

- ❑ Grading plan review from a geotechnical perspective, once the final grading plan is available.
- ❑ 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 Empire Holdings or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Jan.26-2023
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Owen Canton, E.I.T.

David J. Gilbert, P.Eng.

Report Distribution:

- ❏ Empire Holdings (Digital copy)
- ❏ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN-SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ATTERBERG LIMIT TESTING RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 197-201 Wilbrod St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 21, 2021

FILE NO. PG5955

HOLE NO. BH 1-21

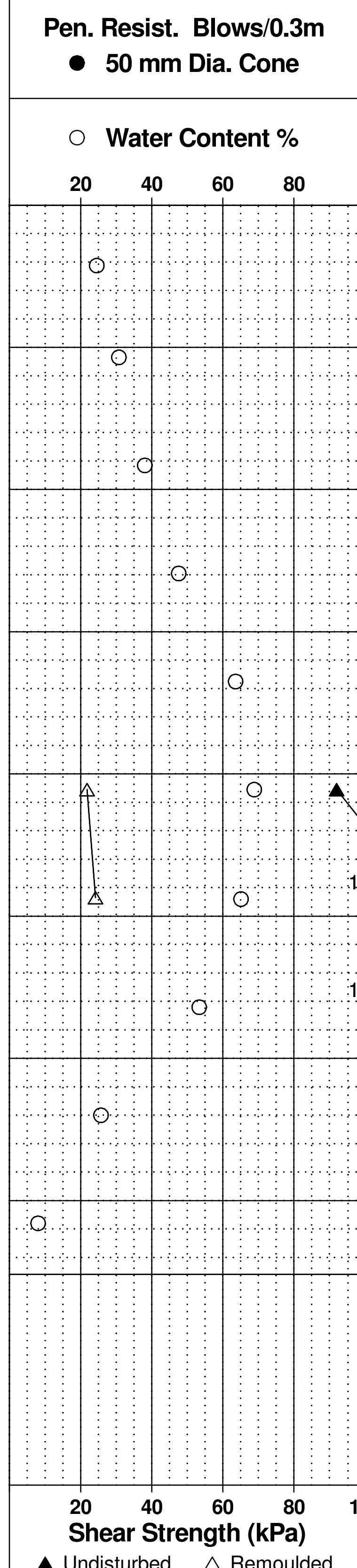
SOIL DESCRIPTION	Layer bottom depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.25				
FILL: Brown silty clay with sand and topsoil	0.91	AU	1		
Hard to very stiff, brown **SILTY CLAY**		SS	2	25	21
		SS	3	50	26
		SS	4	58	14
		SS	5	83	6
		SS	6	75	P
		SS	7	62	P
- grey by 5.3m depth	6.02	SS	8	33	P
Compact, grey **SANDY SILT,** tace clay	7.00	SS	9	50	13
GLACIAL TILL: Grey sandy silt with clay, gravel, cobbles and boulders	7.52	SS	10	42	52

End of Borehole

Practical refusal to augering at 7.52m depth

(BH dry - September 28, 2021)

DEPTH (m)	ELEV. (m)
0	69.53
1	68.53
2	67.53
3	66.53
4	65.53
5	64.53
6	63.53
7	62.53



Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 197-201 Wilbrod St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

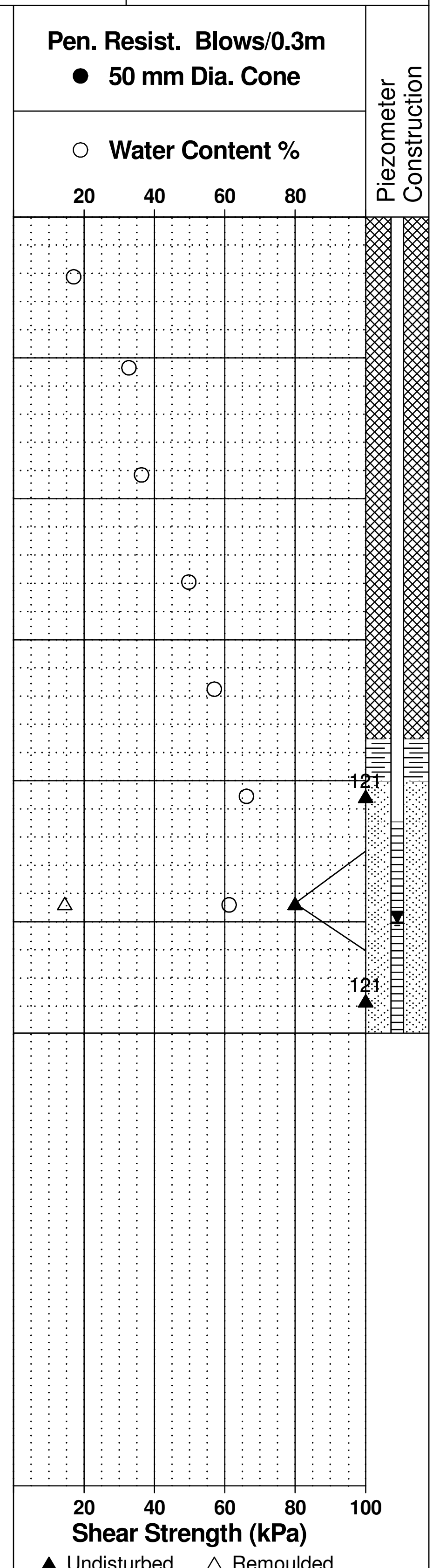
BORINGS BY CME-55 Low Clearance Drill

DATE September 21, 2021

FILE NO. PG5955

HOLE NO. BH 2-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.67
TOPSOIL 0.25							
FILL: Brown silty sand with clay, some gravel		AU	1				
		SS	2	25	18	1	68.67
1.37							
Hard to stiff, brown **SILTY CLAY**		SS	3	83	25	2	67.67
		SS	4	75	14		
		SS	5		8	3	66.67
- grey by 4.5m depth		SS	6		P	4	65.67
		SS	7		P	5	64.67
5.79							
End of Borehole							
(GWL @ 5.01m - Sept. 28, 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 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

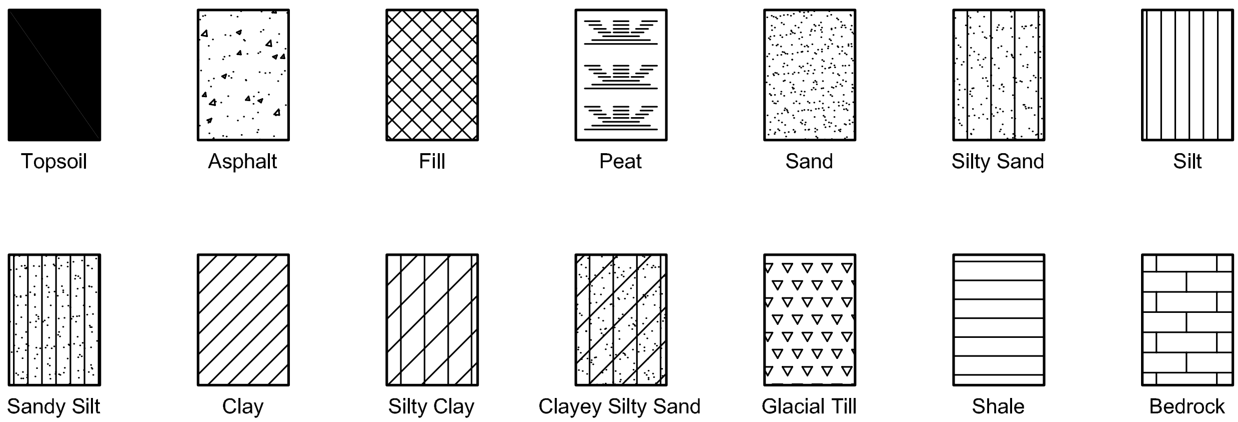
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

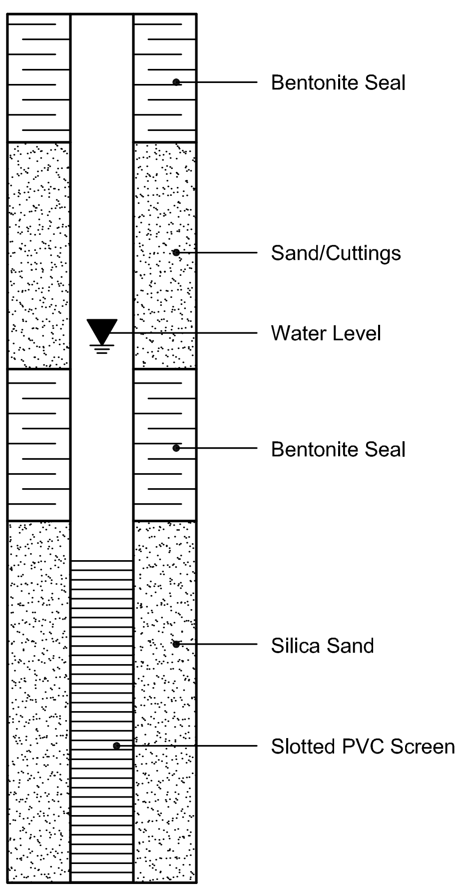
SYMBOLS AND TERMS (continued)

STRATA PLOT

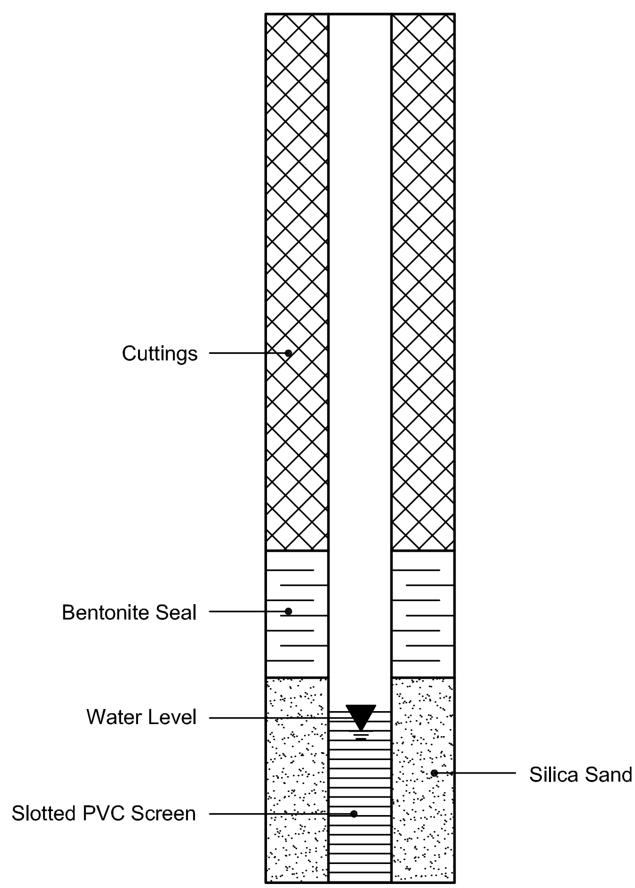


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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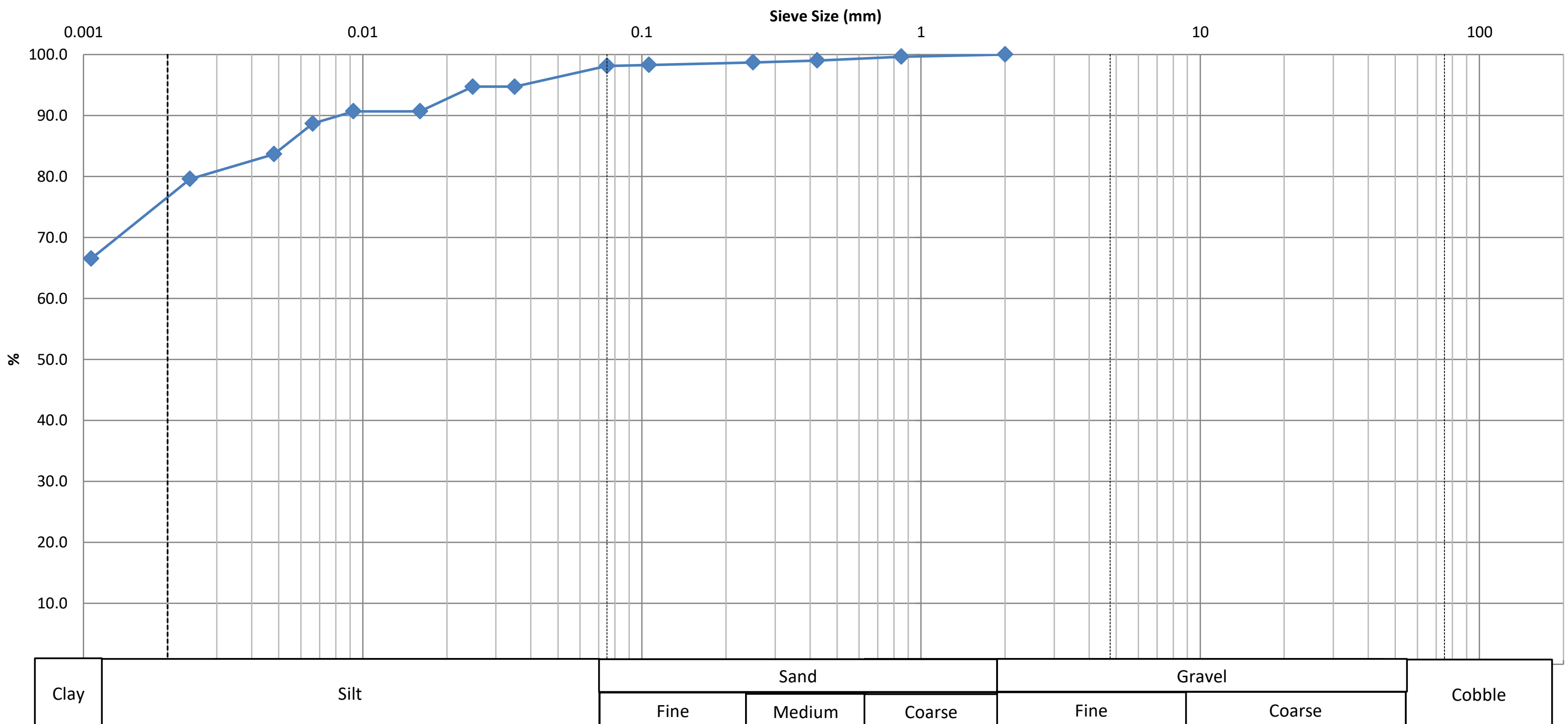
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Empire Holdings	DEPTH:	5' - 7'	FILE NO:	PG5955
CONTRACT NO.:		BH OR TP No.:	BH2-21 SS3	LAB NO:	29048
PROJECT:	197-201 Wilbrod			DATE RECEIVED:	21-Sep-21
				DATE TESTED:	27-Sep-21
DATE SAMPLED:	21-Sep-21			DATE REPORTED:	1-Oct-21
SAMPLED BY:	P. Bandi			TESTED BY:	CS



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					34.0					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)
					0.0	1.9		21.6		76.5

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

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consulting engineers

HYDROMETER
LS-702 ASTM-422

CLIENT:	Empire Holdings	DEPTH:	5' - 7'	FILE NO.:	PG5955
PROJECT:	197-201 Wilbrod	BH OR TP No.:	BH2-21 SS3	DATE SAMPLED	21-Sep-21
LAB No. :	29048	TESTED BY:	CS	DATE RECEIVE	21-Sep-21
SAMPLED BY:	P. Bandi	DATE REPT'D:	1-Oct-21	DATE TESTED:	27-Sep-21

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
112.8	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.00			
WEIGHT CORRECTED	39.70	TARE WEIGHT	50.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	0.95	AIR DRY	129.10	79.10
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	112.80	62.80
		CORRECTED	0.794	

GRAIN SIZE ANALYSIS

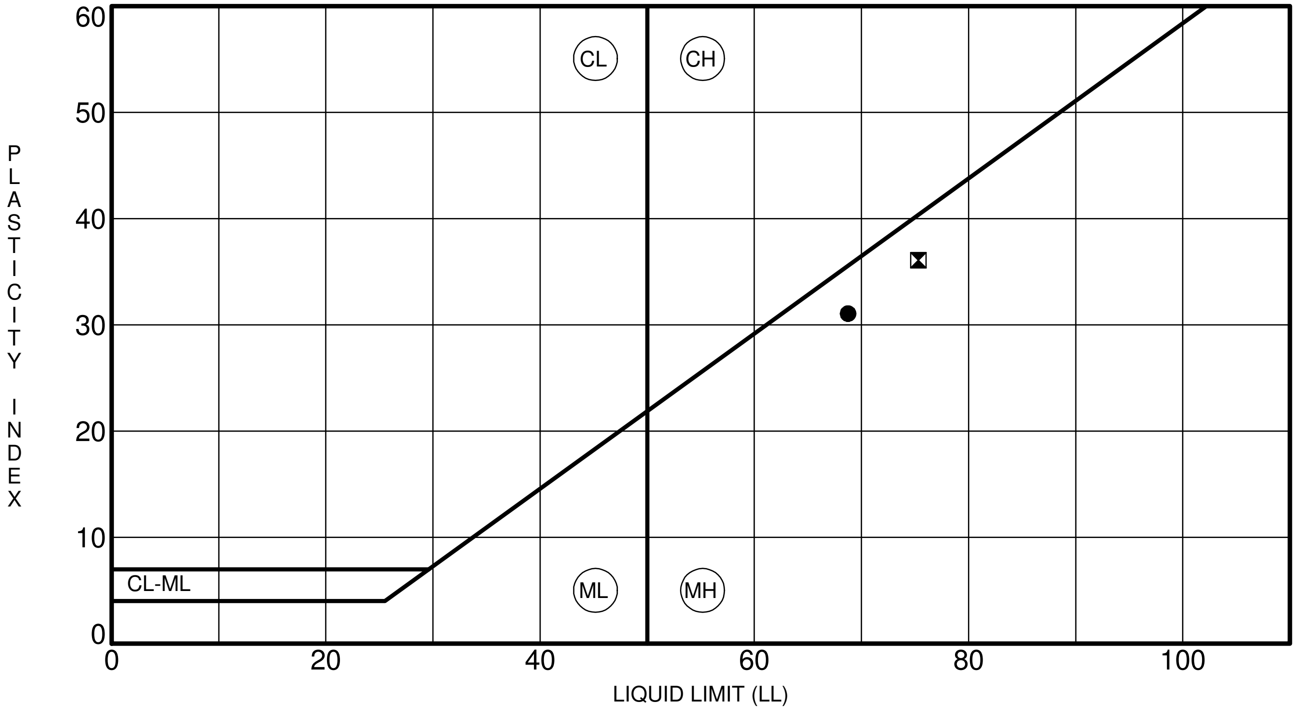
SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75			
2.0	**0.0**	0.0	**100.0**
Pan	**112.8**		
0.850	**0.18**	0.4	**99.6**
0.425	**0.49**	1.0	**99.0**
0.250	**0.65**	1.3	**98.7**
0.106	**0.85**	1.7	**98.3**
0.075	**0.93**	1.9	**98.1**
Pan	**0.95**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	7:44	53.0	6.0	23.0	0.0351	94.7	**94.7**
2	7:45	53.0	6.0	23.0	0.0248	94.7	**94.7**
5	7:48	51.0	6.0	23.0	0.0160	90.7	**90.7**
15	7:58	51.0	6.0	23.0	0.0093	90.7	**90.7**
30	8:13	50.0	6.0	23.0	0.0066	88.7	**88.7**
60	8:45	47.5	6.0	23.0	0.0048	83.6	**83.6**
250	11:53	45.5	6.0	23.0	0.0024	79.6	**79.6**
1440	7:43	39.0	6.0	23.0	0.0011	66.5	**66.5**

Moisture = 34.04%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1-21**	**SS4**	**69**	**38**	**31**		**MH - Inorganic silts of high plasticity**
⧗	**BH 2-21**	**SS4**	**75**	**39**	**36**		**MH - Inorganic silts of high plasticity**

CLIENT	**Empire Holdings**	FILE NO.	**PG5955**
PROJECT	**Geotechnical Investigation - Prop. Residential Development - 197-201 Wilbrod St.**	DATE	**21 Sep 21**

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: **32992**

Report Date: 30-Sep-2021

Order Date: 24-Sep-2021

Project Description: PG5955

	Client ID:	BH 2-21 SS3	-	-	-
	Sample Date:	21-Sep-21 09:00	-	-	-
	Sample ID:	2139519-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	72.3	-	-	-
General Inorganics					
pH	0.05 pH Units	5.67	-	-	-
Resistivity	0.10 Ohm.m	59.3	-	-	-
Anions					
Chloride	5 ug/g dry	13	-	-	-
Sulphate	5 ug/g dry	63	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5955-1 – TEST HOLE LOCATION PLAN

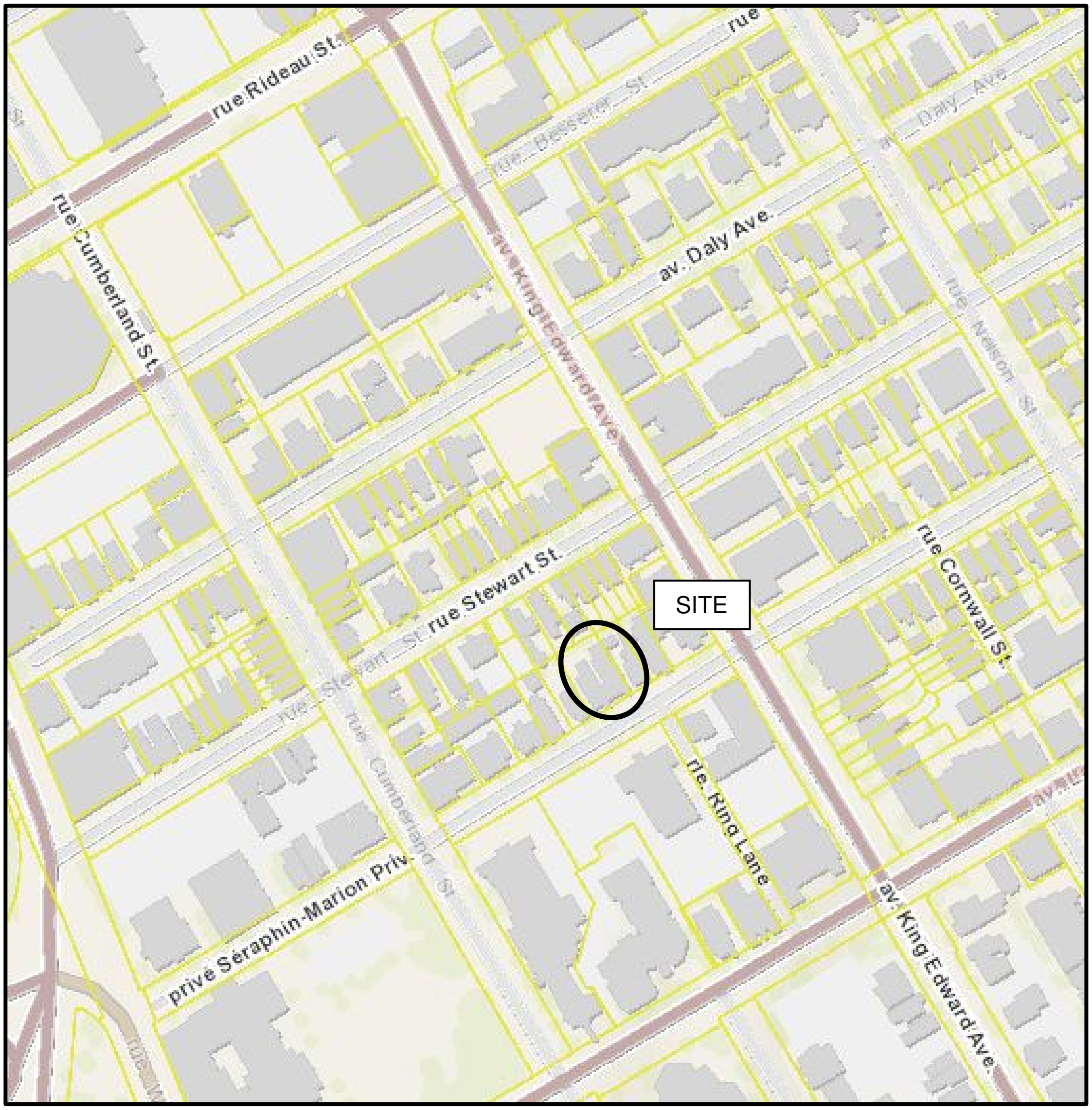
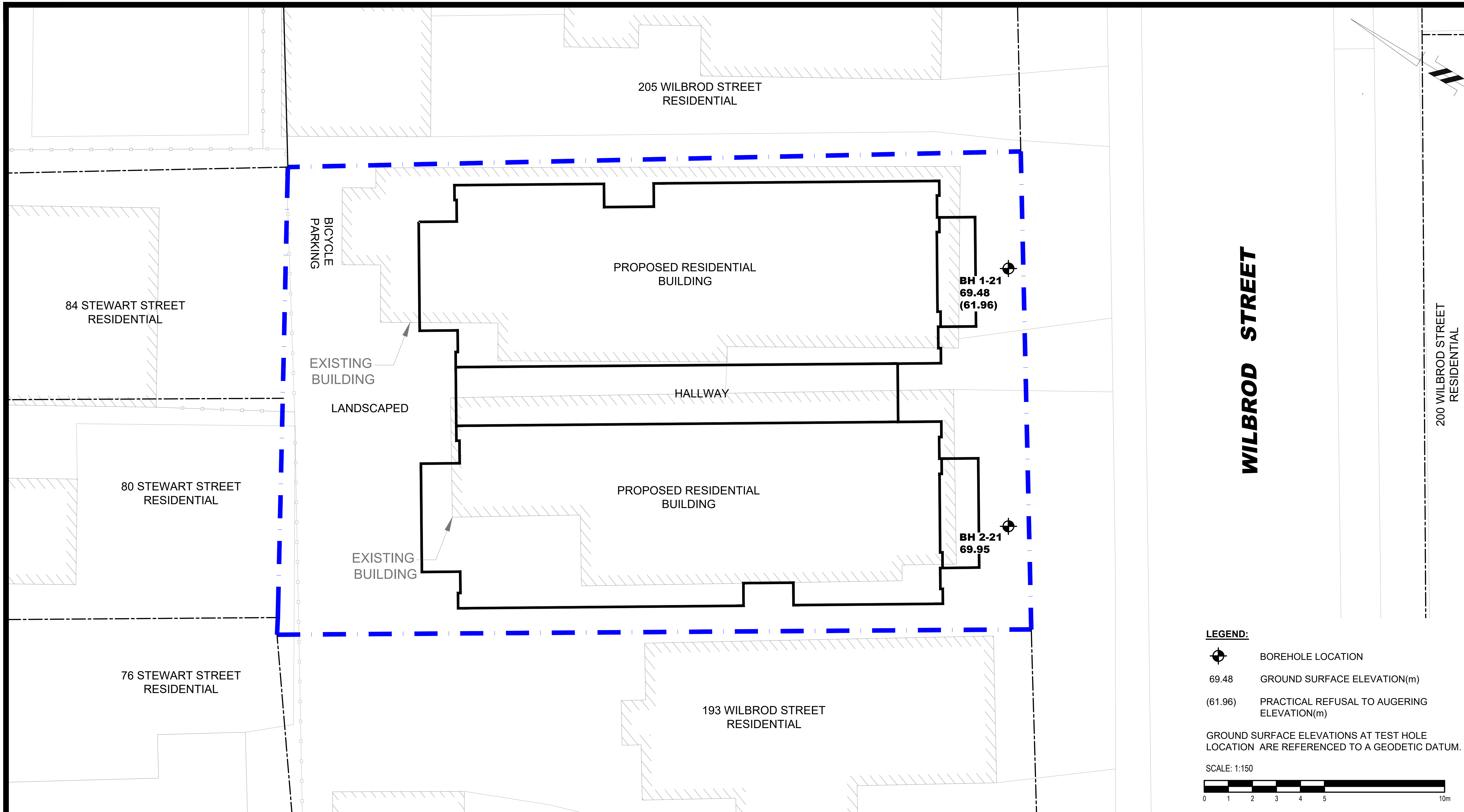


FIGURE 1

KEY PLAN

PATERSON GROUP



PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

EMPIRE HOLDINGS
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
197-201 WILBROD STREET
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:150	Date: 01/2023
Drawn by: RCG	Report No.: PG5955-1
Checked by: OC	Dwg. No.: PG5955-1
Approved by: DJG	Revision No.:

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APPENDIX 3

RELEVANT MEMORANDUMS

re:	**Geotechnical Review of Grading and Site Servicing Plans** **Proposed Residential Development** **197-201 Wilbrod Street - Ottawa**
to:	Empire Holdings - **Ms. Cynthia Blumenthal** - cyblumenthal@rogers.com
date:	June 22, 2022
file:	PG5955-MEMO.01

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide a geotechnical review of the grading and site servicing plans for the proposed residential development to be located at the subject site. This memorandum should be read in conjunction with Paterson Geotechnical Report PG5955-1 Revision 1 dated November 9, 2021.

1.0 Grading Plan Review

Paterson reviewed the following grading plan prepared by D.B. Gray Engineering Inc. for the proposed residential development:

- ❑ Grading Plan, Erosion & Sediment Control Plan and Existing Conditions – Drawing No. C-2 – Job No. 21057 – Revision 2 dated December 21, 2021.

Based on the review of the grading plan, the proposed grading for the residential building does not exceed the permissible grade raise elevation provided by Paterson and is considered acceptable from a geotechnical perspective. Therefore, no lightweight fill is required for the proposed building.

Additional grading around proposed decks, patios, and/or entryway structures does not exceed the permissible grade raise recommendation. Standard construction practices are considered acceptable.

Based on the current proposed grading, adequate soil cover is available against frost protection for the proposed building foundations. Therefore, rigid insulation will not be required for the proposed foundations from a geotechnical perspective. Furthermore, the grading plan is in general conformance with the recommendations provided by Paterson Group in the aforementioned geotechnical report.

2.0 Site Servicing Plan Review

Paterson reviewed the following site servicing plan prepared by D.B. Gray Engineering Inc. for the proposed residential development:

- ❑ Site Servicing Plan – Drawing No. C-1 – Job No. 21057 – Revision 2 dated December 21, 2021.

Based on our review of the above noted site service plan, the majority of the design details are considered to be acceptable from a geotechnical perspective. However, additional geotechnical precautions are recommended for the following item:

Proposed 150 PVC Sanitary Pipe – Existing Watermain Crossing

The proposed 150mm PVC sanitary pipe was observed to be crossing below the existing 300mm watermain running along Wilbrod Street. The invert level for the existing watermain along Wilbrod Street is at approximate geodetic elevation 66.8m and that of the proposed storm pipe is at approximate geodetic elevation 66m. Therefore, the existing watermain shall be protected during construction as per the geotechnical recommendations provided in our memorandum PG5955-MEMO.03 – Protection Plan of Existing Services, dated June 22, 2022.

We trust that this information satisfies your immediate requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 22-2022
M. SALEH
100507739
PROVINCE OF ONTARIO

Owen Canton, EIT

Maha K.Saleh, M.A,Sc., P. Eng.

re: **Geotechnical Response to City Comments**
Proposed Residential Development
197-201 Wilbrod Street - Ottawa

to: Empire Holdings - **Ms. Cynthia Blumenthal** - cyblumenthal@rogers.com

date: June 22, 2022

file: PG5955-MEMO.02

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide geotechnical responses to comments from the City of Ottawa regarding the proposed residential development to be located at the aforementioned site. This memorandum should be read in conjunction with Paterson Geotechnical Report PG5955-1 Revision 1 dated November 9, 2021, memorandum PG5955-MEMO.01 dated June 22, 2022, and PG5955-MEMO.03 dated June 22, 2022.

Geotechnical Comment 1: *Test hole location Plan is missing in the appendix 2. Please note that at least 1 borehole is required in the building area.*

Response: The geotechnical investigation report has been resubmitted with the test hole location plan. Test hole locations were distributed in a manner to provide general coverage of the site, taking into consideration underground utilities and current site features (i.e. existing buildings).

Comment 2: *Please submit a letter stating that the latest Grading and Servicing Plan has been reviewed and that it complies with the recommendations and statements of the latest Geotechnical Investigation.*

Response: Paterson completed a review of the grading and site servicing plans for the proposed development. The proposed grading and site servicing is considered acceptable from a geotechnical perspective, provided our recommendations for proportion of the existing watermain are implemented and approved by Paterson. Reference should be made to Paterson Report PG5955-MEMO.01 dated June 22, 2022, and PG5955-MEMO.03 dated June 22, 2022.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 22-2022
M. SALEH
100507739
PROVINCE OF ONTARIO

Owen Canton, EIT

Maha K. Saleh, M.A.Sc., P.Eng.

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63 Gibson Street
North bay – Ontario – P1B 8Z4
Tel: (705) 472-5331

patersongroup memorandum

consulting engineers

re: **Protection Plan for Existing Services –Watermain Crossing**
Proposed Residential Development
197-201 Wilbrod street - Ottawa

to: Empire Holdings - **Ms. Cynthia Blumenthal** - cyblumenthal@rogers.com

date: June 22, 2022

file: PG5955-MEMO.03

Further to your request and authorization, Paterson Group (Paterson) prepared the current memorandum to provide geotechnical requirements for a protection plan for the existing watermain pipe along Wilbrod Street, during the installation of the proposed sanitary service pipe for the proposed development at the aforementioned site. The following memorandum should be read in conjunction with Paterson Report PG5955-1 Revision 1 dated November 9, 2021 and PG5955-MEMO.01 dated June 22, 2022.

Paterson reviewed the following site servicing plan prepared by D.B. Gray Engineering Inc. for the proposed residential development:

- ❑ Site Servicing Plan – Drawing No. C-1 – Job No. 21057 – Revision 2 dated December 21, 2021.

Based on our review of the above noted plan and profile, the proposed 150mm PVC sanitary pipe was observed to be crossing below the existing 300mm watermain running along Wilbrod Street. The invert level for the existing watermain along Wilbrod Street is at approximate geodetic elevation 66.8m and that of the proposed storm pipe is at approximate geodetic elevation 66.0m. Therefore, the existing watermain shall be protected during construction. The following service protection plan section provides details of our recommendations for the service alignment crossing. The proposed sanitary sewer connection work will not impact the existing watermain provided the service protection plan is properly completed by the contractor and approved in the field by Paterson at the time of excavation.

Service Protection Plan

Based on our review of the above noted plan and profile, the following items are recommended for the Service Protection Plan:

- ➢ The side slopes of the excavation above the invert level of the existing service pipe can be cut back at acceptable slopes. Unsupported excavation side slopes, extending to a maximum depth of 3 m, should be cut back at 1H:1V, or shallower. For excavation below the invert level of the existing service pipes, an engineered trench box approved by a structural engineer is recommended to be in place at all times to support the sides of the excavation. A vertical cut could be completed safely in this manner.

- The placement of the trench box should be reviewed by Paterson personnel to confirm the supported slope face is stable until the sanitary sewer and watermain connection works are completed.
- It should be noted that if signs of soil sloughing are observed within the supported soil, unshrinkable concrete may be required to replace the impacted area prior to removing the trench box from the excavation. In these events, Paterson should be notified immediately to provide a detail to rehabilitate the impacted areas.
- Extra precautions should be taken during excavation around and below the existing services where the proposed sanitary pipe will be crossing below the watermain. It is suggested that hand digging or using a 'hydrovac' technique be used when excavating in close proximity to the existing storm sewer and watermain to avoid any potential damage to the existing mainline pipes. It is expected that pipe bedding below the proposed service alignments will be in accordance with the recommendations provided in Paterson Report PG5955-1 Revision 1 dated November 9, 2021.
- The excavated area below the existing watermain should be reviewed by the geotechnical consultant and backfilled with an unshrinkable fill (0.3 to 0.5 MPa - compressive strength). The unshrinkable fill should in-fill the excavated area adjacent to the installed sanitary sewer and extend vertically to the invert level of the existing watermain pipe and horizontally to the supported sides of the excavation. Furthermore, the unshrinkable fill shall extended horizontally beyond the face of the existing service pipe a minimum distance of 300 mm. The unshrinkable fill should be placed against a clean, excavation sidewall face to eliminate any voids that may cause settlement issues.
- Figures 1 attached present the service protection plan details for the unshrinkable fill at the service crossing areas. The remainder of the excavated trench outside the vicinity of the pipe crossing points can be backfilled with typical pipe backfill materials.
- Temporary support for the existing 300 mm watermain will be required to protect the pipe from hanging without support for too long. Refer to the attached City of Ottawa Drawing No. W28- Temporary Support for Existing Watermain.
- A diesel plate compactor should be used for compaction of granular fill above 300 mm of the existing service obvert level at the service crossing location. It is recommended that loose lift thickness not exceed 225 mm to ensure adequate compaction is met. Granular fill, such as Granular A or Granular B Type II, can be placed for subgrade backfill and compacted to at least 95% of its SPMDD

Reinstatement of the Pavement Structure

The reinstatement of the pavement structure should be completed as follows:

- A 300 mm wide section of the existing asphalt surrounding the excavation should be saw-cut from the existing pavement edge to provide a sound surface to abut the proposed reinstated excavated area.
- It is recommended to mill a 300 mm wide and 40 mm deep section of the existing asphalt.

- The new pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD.
- 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 A or Granular B Type II material.
- Clean existing granular road subbase materials can be reused upon assessment by the geotechnical consultant at the time of excavation (construction) as to its suitability.
- The minimum requirement for pavement structure for the subject reinstatement is presented in Table 1. Performance Graded (PG) 64-28 asphalt cement should be used for this project.
- A tack coat should be provided between the asphalt lifts to create an adequate bond.
- The above noted program abides with the City of Ottawa Drawing R10 - Standard Road Cut Reinstatement Attached.

Table 1 – Wilbrod Street

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 in situ soil or OPSS Granular B Type II material placed over in situ soil.	

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 22, 2022
M. SALEH
100507739
PROVINCE OF ONTARIO

Owen Canton, EIT

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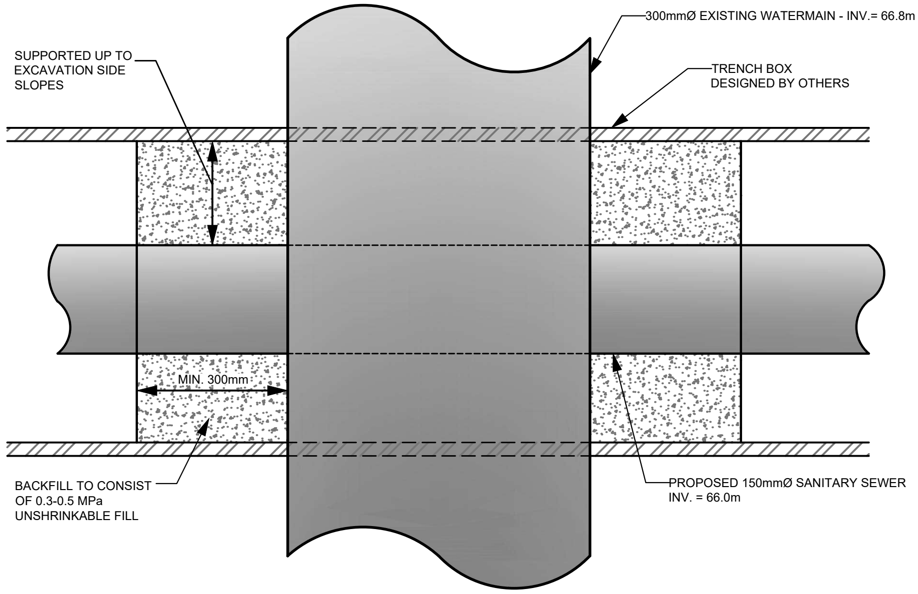
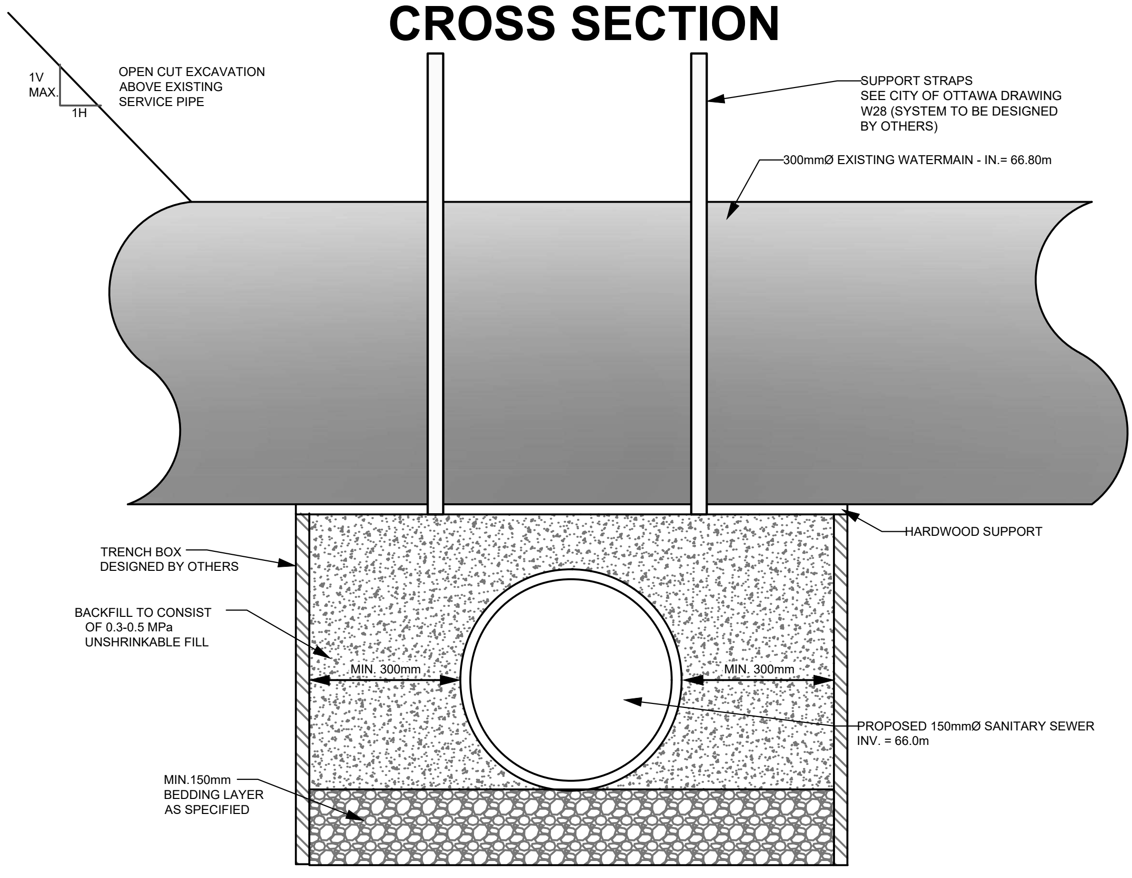
PLAN VIEW



CROSS SECTION



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EMPIRE HOLDINGS

PROPOSED RESIDENTIAL DEVELOPMENT

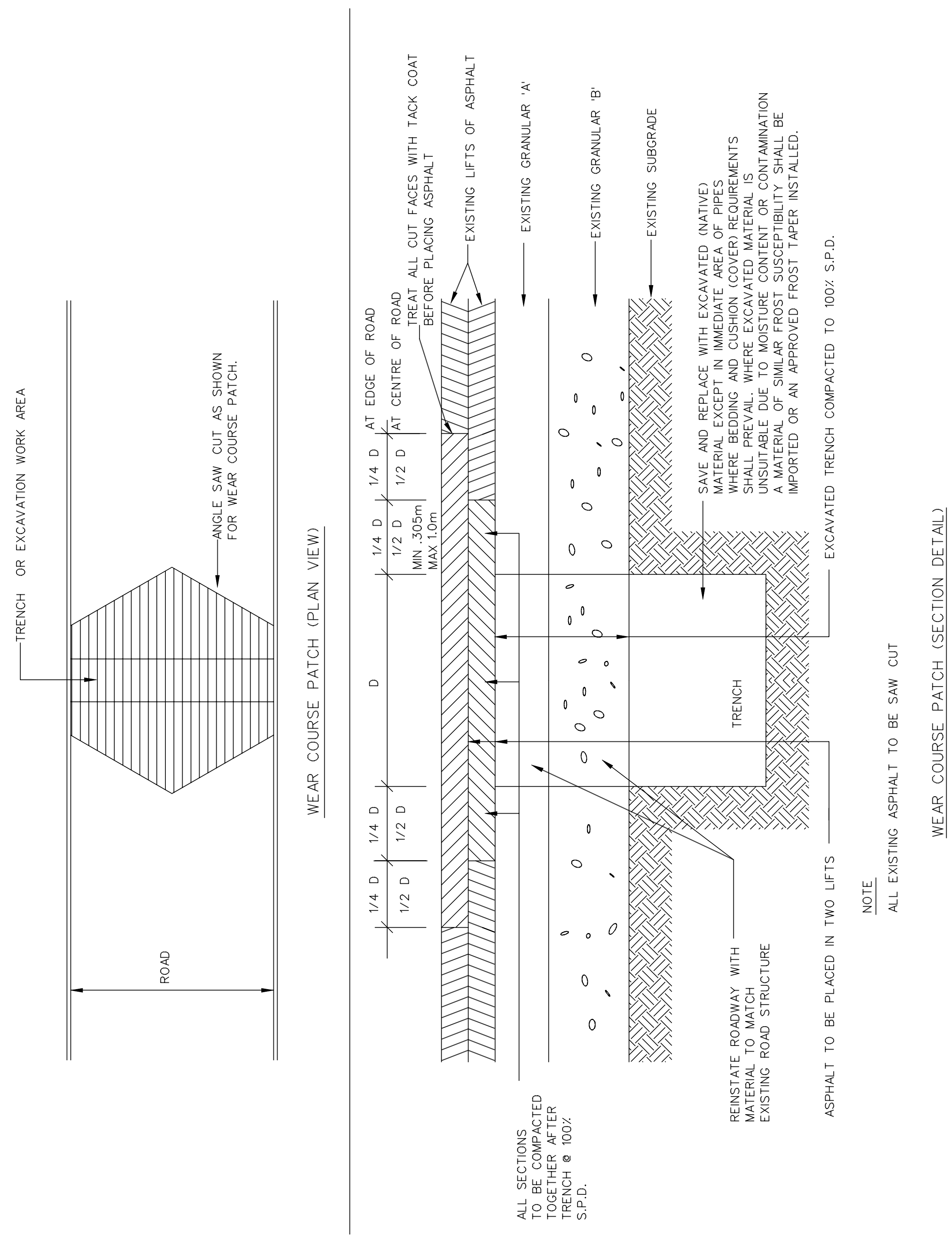
197-201 WILBROD STREET

Title:

SERVICE PROTECTION PLAN

Date: 06/2022	Report No.: PG5955-MEMO.03
Scale: 1:15	Drawing No.: PG5955-FIG.1
Drawn by: RCG	Checked by: MS

p:\autocad drawings\geotechnical\pg50xx\pg5083\pg5083 details_recover.dwg

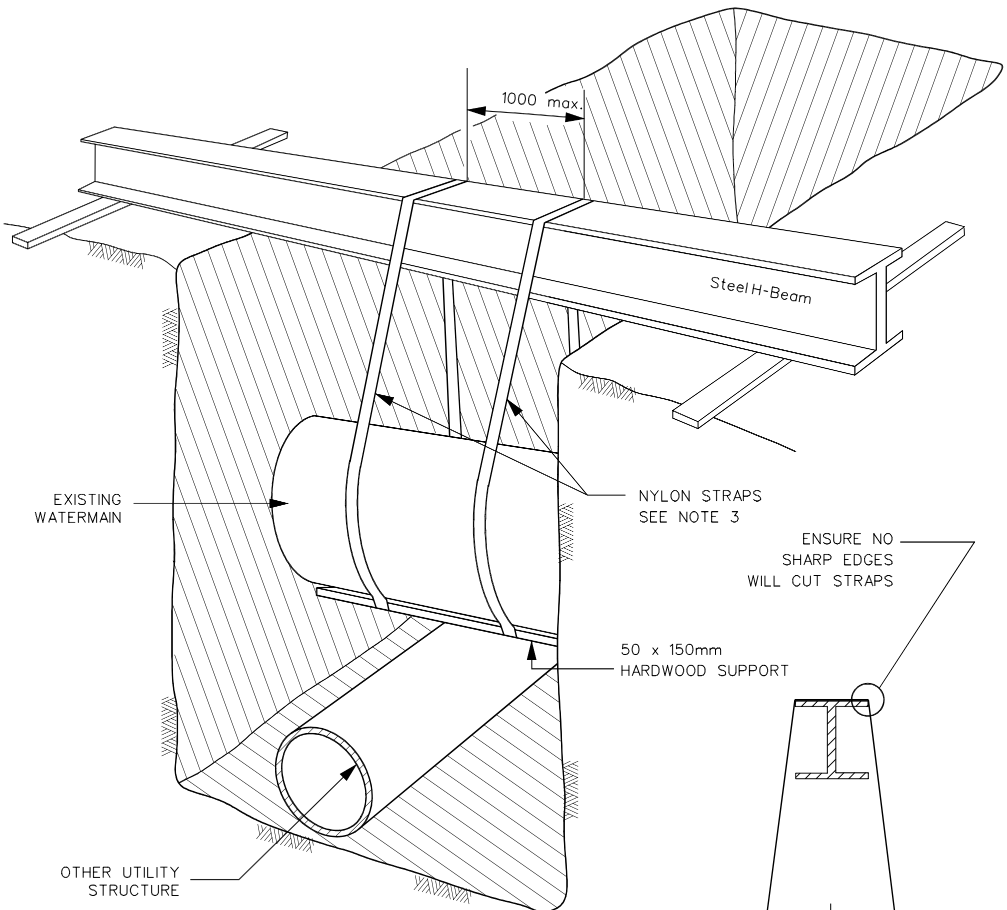


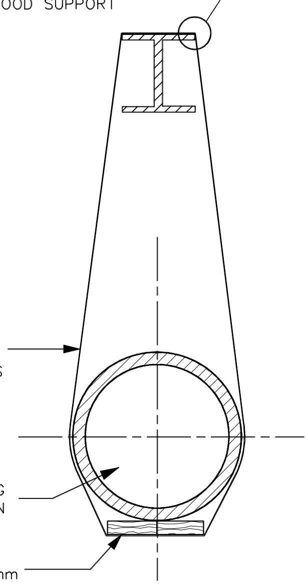
STANDARD ROAD CUT REINSTATEMENT

DATE: MAY 2001

REV. DATE: NONE

DWG. No.: R10





SECTION

NOTES

FOR WATERMAINS 400mm DIA. OR LESS.

1. THE UNSUPPORTED LENGTH OF THE WATERMAIN SHALL NOT EXCEED 1000mm.
2. TEMPORARY SUPPORT FOR WATERMAINS 600mm DIA. AND GREATER, REQUIRE SPECIAL DESIGN.
3. STRAPS MUST MEET THE REQUIREMENTS OF THE O.H.S.A. CHAINS ARE NOT PERMITTED.
4. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS SHOWN OTHERWISE.

Ottawa	TEMPORARY SUPPORT FOR EXISTING WATERMAIN	DATE: MAY 2001
		REV. DATE: NONE
		DWG. No.: W28

PATERSON GROUP

memorandum

re: **Geotechnical Response to City Comments**
Proposed Residential Development
197-201 Wilbrod Street - Ottawa

to: Empire Holdings – **Ms. Cynthia Blumenthal** - cyblumenthal@rogers.com

cc: GRC Architects – **Mr. Randall Rowat** – rrowat@grcarchitects.com

date: December 12, 2022

file: PG5955-MEMO.04

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide geotechnical response to comments from the City of Ottawa regarding the proposed residential development to be located at the aforementioned site. This memorandum should be read in conjunction with Paterson Geotechnical Report PG5955-1 Revision 1 dated November 9, 2021.

Geotechnical Comment 1: *As per Geotechnical Investigation and Reporting Guidelines for Development Applications in the City of Ottawa, a Borehole is required within the building area. Please note that the requirement of a revised Geotechnical report with a borehole investigation within the proposed building area will be added as a condition of Site plan approval.*

Response: Paterson completed two boreholes within the area of the proposed building. Test hole locations were distributed in a manner to provide general coverage of the site, taking into consideration underground utilities and current site features (i.e. existing buildings). In view of the development size, the consistency in the borehole findings, and our knowledge of the area, the existing coverage is considered acceptable for the proposed building, from a geotechnical perspective.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Dec.12-2022
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