PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

214 Somerset Street East
Ottawa, Ontario

Ottawa Community Housing Corporation

Report PG6626-1 Revision 2 dated November 15, 2024

Table of Contents

Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Atterberg Limits Testing Results Grain-Size Distribution Analysis Analytical Testing Results
Appendix 2	Figure 1 - Key Plan Figures 2 & 3 - Slope Stability Cross Sections Drawing PG6626-1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Ottawa Community Housing Corporation to complete a geotechnical investigation for the proposed residential development to be located at 214 Somerset Street East, Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the geotechnical investigation was to:

- ❑ determine the subsoil and groundwater conditions at the 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 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 conceptual drawings, it is understood that the proposed development will consist of a multi-story residential building with one basement level that will occupy the north and northeast portion of the building footprint.

Associated hardscaped and landscaped areas, as well as retaining walls along Somerset Street East, are also anticipated as part of the proposed development. It is further understood that the proposed building will be municipally serviced.

It is expected the existing residential buildings will be demolished as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on October 28 and October 29, 2024, and consisted of advancing three (3) boreholes to a maximum depth of 8.2 m below the existing ground surface. A previous investigation was completed on March 31, 2023, and consisted of advancing one (1) borehole to a maximum depth of 8.4 m below the existing ground surface. The test holes were distributed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations are presented on Drawing PG6626-1 - Test Hole Location Plan included in Appendix 2.

The boreholes for the current investigation were advanced using a geoprobe 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 and sampling the overburden.

Sampling and In Situ Testing

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

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

The thickness of the overburden was evaluated by a dynamic cone penetration test (DCPT) completed at boreholes BH 1-24, BH2A-24, BH 3-24 and BH 1-23. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

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

Groundwater

Monitoring wells were installed at all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the field investigations.

Groundwater level observations are discussed in Section 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- Slotted 32 mm diameter PVC screen at the base of each borehole.
- 32 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No.3 silica sand backfill within annular space around screen.
- Bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the subject site, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevations were surveyed by Paterson using a handheld GPS, referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole are presented in Drawing PG6626-1 - 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. Two (2) samples were submitted for Atterberg Limits testing, one (1) sample for shrinkage limit testing, and two (2) samples for grain size distribution testing.

All test results are included in Appendix 1 and further discussed in Subsection 4.2 of the current report.

Sample Storage

All samples recovered during the current investigation 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.

3.4 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 by others. 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 discussed in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is partially occupied by three-storey residential dwellings with a basement level which are located along the norther portion of the property and along Somerset Street West. The remaining portion of the site consists of landscaped grassed area. A retaining wall was also observed along the western property boundary and south of the existing dwellings.

The existing ground surface gently slopes down from northwest to southeast between approximate geodetic elevations 65.4 to 60.0 m. The ground surface along the property boundary generally matched the ground surface of neighboring residential lots and roadways.

Due to the overall sloped ground surface, a slope stability assessment was completed for the subject site. The results of the slope stability assessment are discussed further in Section 6.8 of this report.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consisted of a thin layer of asphalt and/or fill material underlain by a deposit of silty clay, followed by a deposit of glacial till. The fill material was observed to generally consist of brown silty sand or silty clay with varying amounts of gravel and crushed stone. The fill layer extended to depths ranging between 0.9 and 2.3 m below the existing ground surface.

The silty clay deposit generally consisted of a hard to very stiff, brown weathered crust to depths ranging between 3.8 and 6.1 m below the existing ground surface. The brown silty clay was observed to be underlain by a stiff, grey silty clay deposit.

The glacial till deposit was observed at the location of BH 1-24 and BH 1-23, and generally consisted of compact grey silty clay or sand with gravel, cobbles, and boulders. The glacial till was observed at approximate depth of 7.5 and 6.8 m at BH 1-24 and BH 1-23, respectively.

A DCPT testing was completed at all borehole locations. Practical refusal to DCPT was encountered at the time of investigation at depths ranging between 8.4 and 12.0 m below ground surface.

Reference should be made to the Soil Profile in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Atterberg Limits Testing

Atterberg Limits testing, as well as associated moisture content testing, was completed on select silty clay samples. The results of the Atterberg limits testing are presented in Table 1 and on the Atterberg Limits Testing Results sheet in Appendix 1.

Table 1 - Atterberg Limits Test Results					
Test Hole	**Sample**	**LL (%)**	**PL (%)**	**PI (%)**	**Classification**
BH 3-24	SS7	56	24	32	CH
BH 1-23	SS5	65	30	35	CH
Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index. CH: Inorganic Clay of High Plasticity					

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

Grain Size Distribution and Hydrometer Test

Grain size distribution and hydrometer testing was completed on two (2) selected soil samples. The results of the grain size analysis are summarized in Table 2 and presented on the Grain-size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 2 - Grain Size Distribution and Hydrometer Testing					
Test Hole	**Sample**	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**
BH 1-24	SS6	0.0	0.4	30.6	69.0
BH 1-23	SS6	0.0	3.7	36.3	60.0

Shrinkage Test

Linear shrinkage testing was completed on sample SS3 recovered from borehole BH 1-23. The result of the shrinkage limit test indicates a shrinkage limit of 15.4 and shrinkage ratio of 14.42.

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of Ordovician shale of the Carlsbad formation, with overburden drift thickness of 10 to 15 m.

4.3 Groundwater

Groundwater levels were measured on November 4, 2024, within the installed monitoring wells. The measured groundwater levels are presented in Table 3 below and are shown on the Soil Profile and Test Data sheets in Appendix 1.

Table 3 – Summary of Groundwater Levels

Test Hole	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-24	61.12	3.47	57.65	November 4, 2024
BH 2A-24	64.13	4.14	59.99	November 4, 2024
BH 3A-24	62.78	Dry	NA	November 4, 2024
BH 1-23	60.01	3.47	56.54	April 6, 2023

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

It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level 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 residential development. The proposed building may be founded on conventional footings placed on an undisturbed, very stiff silty clay bearing medium.

Due to the presence of the silty clay layer, the subject site will have a permissible grade raise restriction. The permissible grade raise recommendations are discussed in Section 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

Asphalt, topsoil, and deleterious fill, such as those containing significant amounts of organic materials, should be stripped from under any building, paved areas or other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as CCW MiraDRAIN 2000 or Delta-Teraxx.

5.3 Foundation Design

Bearing Resistance Values – Conventional Spread Footings

Conventional strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, very stiff brown silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

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

Footings designed using the bearing resistance values at SLS given above 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 foundation levels. This includes areas where the basement level differs between a separation wall, stepped footings could be considered in these areas to mitigate potentially impacting lateral support for footings abutting these foundation walls.

Above the groundwater level, adequate lateral support is provided to the in-situ bearing medium soils when a plane extending 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Permissible Grade Raise Recommendations

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site, a permissible grade raise of **2.0 m** is recommended in the immediate area of settlement sensitive structures and where silty clay is encountered at underside of footing elevations. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise restriction calculations.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post-construction total and differential settlements. Provided sufficient time is available to induce the required settlements, consideration could be given to surcharging the subject site.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Site Class D** for foundations constructed at the subject site, according to Table 4.1.8.4.A of the 2012 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

With the removal of all topsoil and deleterious fill, containing significant amounts of organic matter, within the footprint of the proposed buildings, the native soil, approved by Paterson personnel at the time of construction, will be considered acceptable subgrade on which to commence backfilling for floor slab construction.

For structures with slab-on-grade construction, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprints of the proposed buildings should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of its SPMDD.

For structures with basement slabs supported by strip and pad footings, the upper 200 mm of sub-floor fill may consist of 19 mm clear crushed stone.

All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

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 expected (i.e., below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be estimated as 13 kN/m^3. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

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

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

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

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Design

Although car parking areas are not anticipated as part of the proposed development based on the current plans, the hard landscaping walkway and car only parking pavement structures presented in the following tables could be used for design purposes, if required.

Table 4 – Recommended Hard Landscaping – Pedestrian Walkways

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

Table 5 - 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.	

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

Temporary construction access paths and working pads may be considered as 600 mm of OPSS Granular B Type I or II crushed stone, or blast-rock covered with a minimum 150 mm thick layer of OPSS Granular B Type II crushed stone over a subgrade surface reviewed and approved by Paterson field personnel.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

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 structure. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of minimum 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The drainage pipe may be installed in two separate segments and split at the wall separating the lower and higher basement levels provided the upper drainage pipe terminates in direct contact with the lower basement drainage board layer.

The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump system. If a sump system is considered, it is recommended that Paterson reviews and advises on a potential underfloor drainage system to collect water that may collect below the basement slab.

Foundation Backfilling

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite 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.

Sidewalks and Walkways

Backfill material below sidewalks and walkways subgrade areas or other settlement sensitive structures which are not adjacent to the building should consist of free-draining, non-frost susceptible material. It is recommended the backfill consists of a minimum of 150 mm of OPSS Granular A and 300 mm of OPSS Granular B Type II crushed stone compacted in 300 mm thick loose lifts and 98% of the materials SPMDD. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard. Alternatively, a combination of soil cover (a minimum of 600 mm) and rigid insulation may be provided to attain sufficient soil cover from frost migration.

It is recommended that Paterson review and advise on these details during the planning and design phases of the proposed structure, and specifically for the western and southwestern portions of the proposed building.

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

6.3 Excavation Side Slopes

Temporary Side Slopes

The side slopes of excavations in the overburden materials should either be 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). However, it is expected a shoring system, or alternative methods of excavation support may be required to support the overburden along the western portion of the excavation given the proximity of the structure located at 212 Somerset Street West, and potentially the retaining wall along the southwestern portion of the subject site. It is recommended to confirm the founding depth of the structure located at 212 Somerset Street West during the design and planning phases.

Unsupported Excavations

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff where shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction.

It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side slopes.

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 or secant piles where minor settlements cannot be tolerated by the supported structures. 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 using the parameters provided in Table 6.

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.

Table 6 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
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 hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil 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.

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular A. The bedding layer thickness should be increased to a minimum of 300 mm where the subgrade will consist of grey silty clay. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

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

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

Clay Seals

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. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub bedding and cover material. The barriers should consist of relatively dry and compatible 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.

Impacts on Neighboring Properties

A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings. Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures.

6.6 Winter Construction

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

The subsurface conditions mostly 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 particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract documents to protect the walls of the excavations from freezing, if and where applicable.

Similarly, where insufficient soil cover is present to existing foundations in close proximity to the excavation sidewalls (as is anticipated for 442 Nelson Street), temporary heating, insulated tarps and/or other methods should be explored to maintain sufficient protection against the migration of frost below the supporting subsoils.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and/or glycol lines and tarpaulins or other suitable means.

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 foundation is protected with sufficient soil cover to prevent freezing at founding level.

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

Under winter conditions, if snow and ice is present within imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

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.

6.8 Slope Stability Assessment

The subject site is generally observed to slope gently from northwest to southeast, following the regional topography of the area. The existing slope conditions were reviewed by Paterson field personnel as part of the geotechnical investigation completed in October 2024. Two (2) slope cross-sections were studied as the worst-case scenarios. The cross-section locations are presented on Drawing PG6626-1 – Test Hole Location Plan in Appendix 2.

Field Observations

The existing slope was observed throughout the subject site and neighboring lots. Low-rise residential buildings and dwellings were observed bordering the subject property and within the sloped existing ground surface. Retaining walls were observed throughout the southwest boundary of the subject site.

A slope stability assessment was therefore carried out to evaluate the stability of the slope under proposed conditions and taking into consideration existing features.

Slope Stability Analysis

The analysis of the stability of the slope was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method.

The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable.

However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

Subsoil conditions at the cross-sections were inferred based on the boreholes completed throughout the subject site and general knowledge of the geology of the area. For a conservative review of the groundwater conditions, the silty clay deposit was considered to be fully saturated for our analysis. The effective strength soil parameters used for static analysis are presented in Table 7 below.

Table 7 – Effective Soil and Material Parameters (Static Analysis)			
Soil Layer	**Unit Weight (kN/m^3)**	**Friction Angle (Degrees)**	**Cohesion (kPa)**
Fill	18	33	1
Brown Silty Clay Crust	17	33	5
Grey Silty Clay	16	33	10
Glacial Till	19	35	1
Bedrock	24	-	Infinite

Static Loading Analysis

The results are shown in Figures 2A and 3A in Appendix 2. The results indicate a slope with factors of safety exceeding 1.5 beyond the top of slope at all analyzed sections. Therefore, the slopes are considered stable under static loading conditions.

Seismic Loading Analysis

An analysis considering seismic loading and the groundwater at ground surface was also completed.

A horizontal acceleration of 0.16g was considered for all slopes. This acceleration is considered as half of the peak (horizontal) ground acceleration (PGA) of 0.300g, specified in the National Building Code of Canada (NBCC 2015) Seismic Calculator for the subject site. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The total strength parameters for seismic analyses were chosen based on our observations and in-situ testing carried out on site, nearby boreholes, and our general knowledge of the geology in the area of the subject site. The strength parameters used for seismic analysis at the cross-sections are presented in Table 8 below.

Table 8 – Total Strees Soil and Material Parameters (Seismic Analysis)			
Soil Layer	**Unit Weight (kN/m^3)**	**Friction Angle (Degrees)**	**Undrained Shear Strength (kPa)**
Fill	18	33	-
Brown Silty Clay Crust	17	-	125
Grey Silty Clay	16	-	70
Glacial Till	19	35	-
Bedrock	24	-	-

The results of the analyses including seismic loading are shown in Figures 2B and 3B in Appendix 2. The results indicate a slope with a factor of safety greater than 1.1 beyond the top of slope for Section A and B.

The results indicate that the factor of safety is greater than 1.1 under seismic conditions for the cross sections. Based on these results, the slope and proposed structures are considered stable under seismic conditions.

Toe Erosion Setback

Based on the review completed at the subject site, geotechnical hazard lands, as a watercourse or other sources or erosional activity, are not present at the subject site or its vicinity. Therefore, toe erosion allowance is not applicable for the subject site.

Limit of Hazard Lands

Based on the above, a suitable factor of safety is present under static and seismic conditions such that the proposed residential development is not considered to negatively impact the existing slope and/or surrounding topography. Therefore, a limit of hazard lands is not applicable for the subject site and the existing slopes are stable from a geotechnical and slope stability perspective.

6.9 Landscaping Considerations

Tree Planting Restrictions

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. Sieve analysis testing was also completed on selected samples. The results of our testing are presented in Table 1 and Table 2 in Subsection 4.2 and in Appendix 1.

Based on the results of our testing, the plasticity index of the silty clay deposit at the subject site does not exceed 40%. Therefore, the following tree planting setbacks are recommended for the subject site:

Large trees (mature height over 14 m) can be planted within the silty clay areas 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).

Tree planting setback limits may be reduced to **4.5 m** for small (mature height up to 7.5 m) and medium size trees (mature tree height 7.5 to 14 m), provided that the conditions noted below 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 center of the tree trunk and verified by means of the Grading Plan.
- ❑ 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 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 facing trees 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), as noted on the subdivision Grading Plan.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows, and some maples (i.e., Manitoba Maples) and, as such, they should not be considered in the landscaping design.

Retaining Wall Design

It is expected that retaining walls will be required to support the grading along Somerset Street West. Retaining walls higher than 1.0 m should be designed by a Licensed Professional Engineer in the Province of Ontario. The bearing resistance values provided in Section 5.3 are applicable to the proposed retaining walls.

The soil parameters presented in Table 9 should be used for the design of the retaining walls. The design should also include a global stability analysis of the system. Global stability analysis should include static and seismic analysis of the system and present the minimum factor of safety. The system should be design for a factor of safety of 1.5 under static conditions and 1.1 for seismic conditions.

Backfill Material

The retaining wall should be backfilled with free-draining granular backfill materials and incorporate longitudinal drains and weep holes to provide positive drainage of the backfill. For the purpose of this report, it is recommended that the wall be backfilled with either OPSS Granular B Type II or Granular A materials. The backfill should be placed within a wedge-shaped zone defined by a line drawn up and back from the back edge of the base block of the wall at an inclination of 1H:1V or a minimum of 1 m behind the back of the blocks. All material should be compacted to a minimum of 98% of the material’s SPMDD.

Lateral Earth Pressures

It is recommended that a minimum of 500 mm of the backfill material to consist of clean imported engineered crushed stone such as OPSS Granular A or Granular B Type II. The soil parameters presented in Table 9 should be used for the design of the retaining wall.

Table 9 – Geotechnical Parameters for Backfill and Bedding Materials

Material Description	Unit Weight (kN/m^3)		Friction Angle ()	Friction Factor,	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'	φ'	tan δ	Active K_a	At-Rest K_o	Passive K_P
OPSS Granular A (Crushed Stone)	22	13.7	36	0.6	0.26	0.41	3.85
OPSS Granular B Type II (Crushed Stone)	22	13.7	36	0.6	0.26	0.41	3.85
OPSS Granular B Type I (Sand-Gravel)	21	13	32	0.52	0.31	0.47	3.25

Notes:
1. Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
2. The earth pressure coefficients provided are for horizontal backfill profile.
3. For soil above the groundwater level the "drained" unit weight should be used and below groundwater level the "effective" unit weight should be used.

Retaining Wall Types

Where the retaining wall is to be higher than 1 m and or support a roadway or slope consideration can be given to using large precast concrete segmental block retaining wall system, such as Redi-Rock and Stone Strong, if the wall will not consist of a cast-in-place wall incorporated into the buildings foundation structure.

Quality precast products are designed to resist large load under gravity and may not require as much excavation or reinforcement. Typical products vary in size from 0.6 to over 2.4 m in depth depending on the total height of the wall. The size of these supporting structures should be considered when drafting site plans and grading plans, especially where they will be located between structures.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- Review preliminary and detailed grading, servicing and structural plan(s) from a geotechnical perspective.

- Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction, if applicable.

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

- 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.

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

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

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Ottawa Community Housing Corporation or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
November 15, 2024
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Fernanda Carozzi, PhD.Geoph.

Drew Petahtegoose, P.Eng.

Report Distribution:

- ❑ Ottawa Community Housing Corporation (email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMIT TESTING RESULTS

GRAIN SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

214 Somerset Street E, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 369080.51 **NORTHING:** 5031718.09 **ELEVATION:** 61.12

PROJECT: Proposed Residential Development

BORINGS BY: Geoprobe

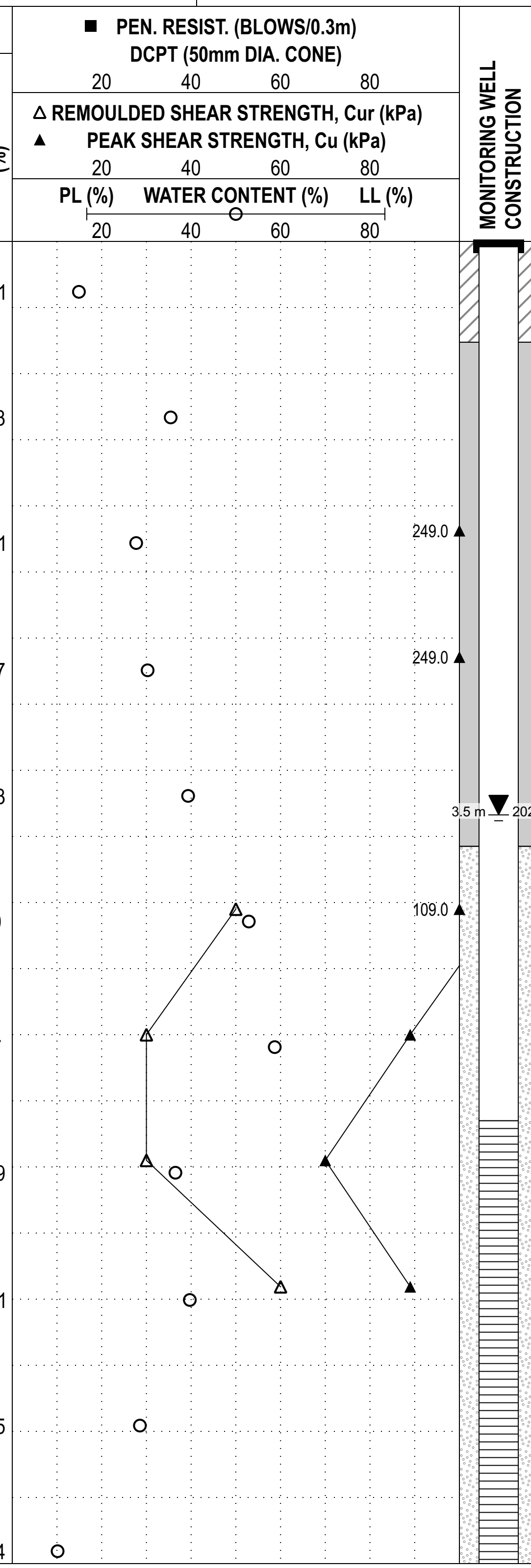
REMARKS:

DATE: October 28, 2024

FILE NO. : PG6626

HOLE NO. : BH1-24

SAMPLE DESCRIPTION (GROUND SURFACE)	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	ELEVATION (m)
FILL: Brown silty sand, trace clay, gravel and organics	0	SS 1	54	2-2-2-8 4	14.91	61
0.91m [60.21m]						
Hard to very stiff brown **SILTY CLAY**	1	SS 2	58	2-3-4-5 7	35.43	60
		SS 3	100	P	27.71	
	2	SS 4		P	30.27	59
	3	SS 5		P	39.33	58
	4	SS 6	100	P	52.9	57
- Stiff and grey by 4.5 m depth		SS 7	100	P	58.7	
	5	SS 8	100	P	36.49	56
	6					55
- Trace of sand and gravel by 6.4 m depth		SS 9	100	P	39.71	
	7	SS 10	100	1-1-1-2 2	28.55	54
7.54m [53.58m]						
GLACIAL TILL Loose grey silty clay with sand, gravel, cobbles and boulders	8	SS 11		5-3-5-5 8	10.14	



PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

214 Somerset Street E, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369080.51 **NORTHING:** 5031718.09 **ELEVATION:** 61.12

PROJECT: Proposed Residential Development

BORINGS BY: Geoprobe

REMARKS:

DATE: October 28, 2024

FILE NO. : PG6626

HOLE NO. : BH1-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH, Cur (kPa) ▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
8.23m [52.89m]		8	SS 11						53
Dynamic Cone Penetration Test commenced at 8.23 m depth		9							52
9.58m [51.54m]									
End of Borehole Practical refusal to DCPT at 9.58 m depth (GWL at 3.47 m depth - November 04, 2024)		10 11 12 13 14 15 16							51 50 49 48 47 46

DISCLAIMER: THE DATA PRESENTED IN THIS LOG IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHO IT WAS PRODUCED. THIS LOG SHOULD BE READ IN CONJUNCTION WITH ITS COORESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

214 Somerset Street E, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 369079.77 **NORTHING:** 5031738.46 **ELEVATION:** 64.13

PROJECT: Proposed Residential Development

BORINGS BY: Geoprobe

REMARKS:

DATE: October 28, 2024

FILE NO. : PG6626

HOLE NO. : BH2-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / △ REMOULDED SHEAR STRENGTH, Cur (kPa) ▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE		0							64
*Refer To BH 2A-24 for soil profile information			SS 1	25	4-2-4-4 6				
		1							63
			SS 2	67	1-1-1-2 2				
1.83m [62.30m]									
End of Borehole		2							62
		3							61
		4							60
		5							59
		6							58
		7							57
		8							

DISCLAIMER: THE DATA PRESENTED IN THIS LOG IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHO IT WAS PRODUCED. THIS LOG SHOULD BE READ IN CONJUNCTION WITH ITS COORESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

214 Somerset Street E, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369076.76 **NORTHING:** 5031736.66 **ELEVATION:** 64.13

PROJECT: Proposed Residential Development

BORINGS BY: Geoprobe

REMARKS:

DATE: October 29, 2024

FILE NO. : PG6626

HOLE NO. : BH2A-24

SAMPLE DESCRIPTION	
GROUND SURFACE	
ASPHALT	0.05m [64.08m]
FILL: Brown silty sand or silty clay with gravel and crushed stone	1.45m [62.68m]
Hard to very stiff brown **SILTY CLAY**	6.10m [58.03m]
Dynamic Cone Penetration Test commenced at 6.10 m depth	

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	PEAK SHEAR STRENGTH, Cu (kPa)
AU 1			16.34	
SS 2	58	1-2-1-1 3	31.1	
SS 3	79	2-2-3-4 5	43.05	
SS 4	100	P	23.06	219.0
SS 5	100	P	27.82	229.0
SS 6	100	P	30.07	229.0
SS 7	100	P	34.2	199.0
SS 8	100	P	33.85	149.0

■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

△ REMOULDED SHEAR STRENGTH, Cur (kPa)

▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

MONITORING WELL CONSTRUCTION

4.1 m 2024-11-04

DEPTH (m): 0 1 2 3 4 5 6 7 8

ELEVATION (m): 64 63 62 61 60 59 58 57

DISCLAIMER: THE DATA PRESENTED IN THIS LOG IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHO IT WAS PRODUCED. THIS LOG SHOULD BE READ IN CONJUNCTION WITH ITS COORESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

214 Somerset Street E, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 369076.76 **NORTHING:** 5031736.66 **ELEVATION:** 64.13

PROJECT: Proposed Residential Development

BORINGS BY: Geoprobe

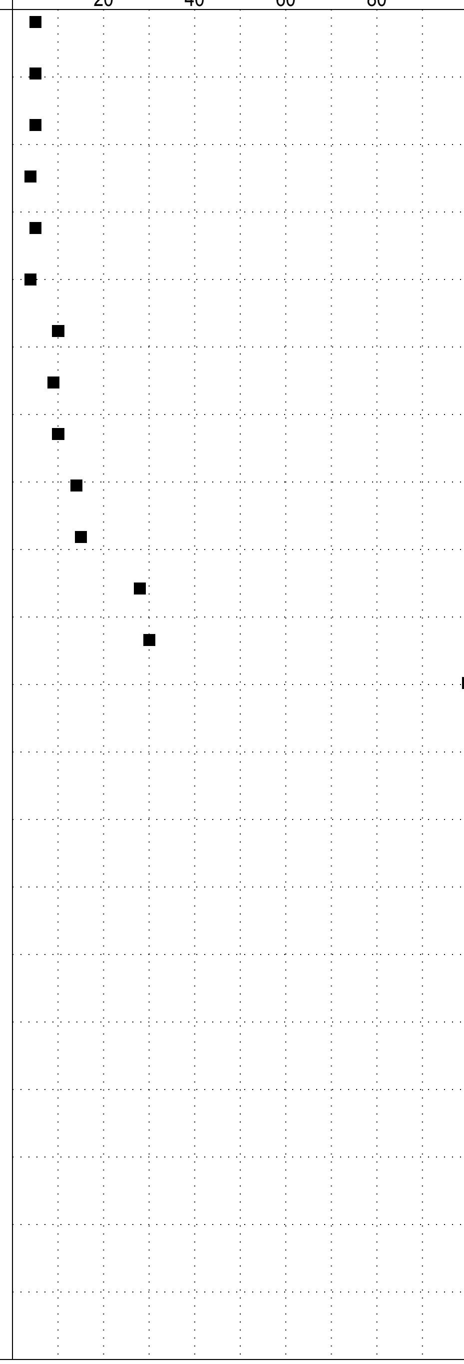
REMARKS:

DATE: October 29, 2024

FILE NO. : PG6626

HOLE NO. : BH2A-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / △ REMOULDED SHEAR STRENGTH, Cur (kPa) ▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
12.09m [52.04m]		8 – 12							56 – 52
End of Borehole Practical refusal to DCPT at 12.09 m depth (GWL at 4.14 m depth - November 04, 2024)		12 – 16							52 – 48



DISCLAIMER: THE DATA PRESENTED IN THIS LOG IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHO IT WAS PRODUCED. THIS LOG SHOULD BE READ IN CONJUNCTION WITH ITS COORESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SOIL PROFILE AND TEST DATA

PATERSON GROUP

Geotechnical Investigation
214 Somerset Street E, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369086.47 **NORTHING:** 5031743.07 **ELEVATION:** 62.78

PROJECT: Proposed Residential Development
BORINGS BY: Geoprobe
REMARKS:
DATE: October 29, 2024

FILE NO. : PG6626
HOLE NO. : BH3-24

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE	0					
ASPHALT 0.05m [62.73m]						
Fill: Brown silty sand or silty clay with gravel		AU 1			9.01	62
	1	SS 2		1-1-1-1 2	49.8	
	2	SS 3	79	1-1-1-1 2	30.92	61
2.36m [60.42m]						
Hard to very stiff, brown **SILTY CLAY**		SS 4	83	1-1-4-6 5	21.47	60
	3	SS 5	100	2-5-6-8 11	26.78	59
	4	SS 6	100	P	31.96	
	5	SS 7	100	P	36.77	58
		SS 8		P	38.65	57
- Stiff and grey by 6.0 m depth	6	SS 9		P	50.34	56
	7	SS 10		P	40.07	55
	8	SS 11		P	25.36	

■ PEN. RESIST. (BLOWS/0.3m)
DCPT (50mm DIA. CONE)
20 40 60 80

△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa)
20 40 60 80

PL (%) WATER CONTENT (%) LL (%)
20 40 60 80

249.0
189.0
149.0
149.0

STRATA PLOT
PIEZOMETER CONSTRUCTION

DISCLAIMER: THE DATA PRESENTED IN THIS LOG IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHO IT WAS PRODUCED. THIS LOG SHOULD BE READ IN CONJUNCTION WITH ITS COORESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SOIL PROFILE AND TEST DATA

PATERSON GROUP

Geotechnical Investigation

214 Somerset Street E, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 369086.47 **NORTHING:** 5031743.07 **ELEVATION:** 62.78

PROJECT: Proposed Residential Development

BORINGS BY: Geoprobe

REMARKS:

DATE: October 29, 2024

FILE NO. : PG6626

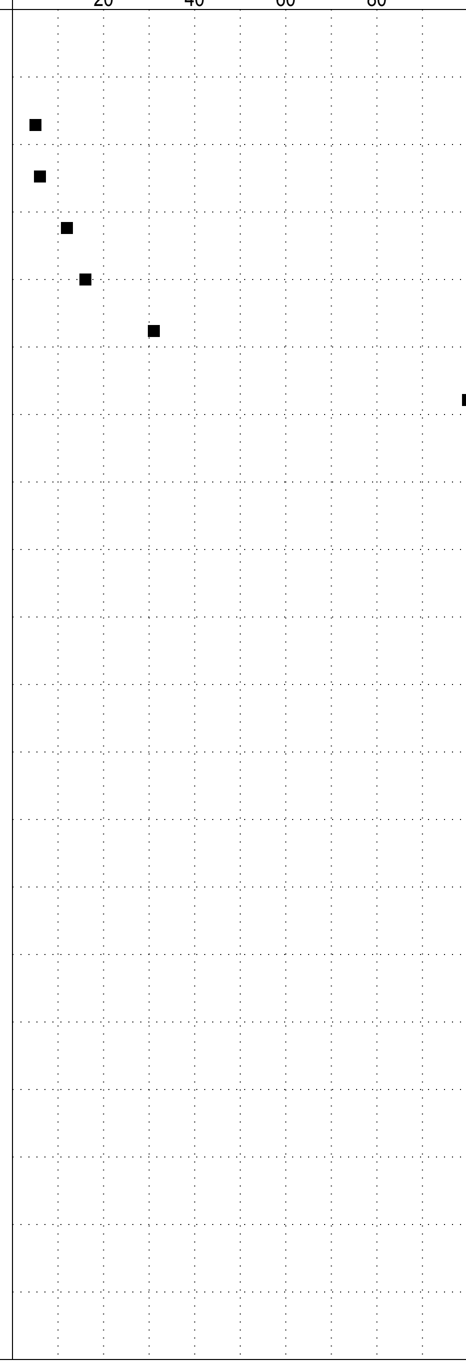
HOLE NO. : BH3-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: WATER CONTENT (%)	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE								
- Trace of sand by 8.0 m depth 8.23m [54.55m]		8	SS 11					
Dynamic Cone Penetration Test commenced at 8.23 m depth 10.57m [52.21m]		9 10						54 53
End of Borehole Practical refusal to DCPT at 10.57 m depth		11 12 13 14 15 16						52 51 50 49 48 47

■ PEN. RESIST. (BLOWS/0.3m)
DCPT (50mm DIA. CONE)
20 40 60 80

△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa)
20 40 60 80

PL (%) WATER CONTENT (%) LL (%)
20 40 60 80



DISCLAIMER: THE DATA PRESENTED IN THIS LOG IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHO IT WAS PRODUCED. THIS LOG SHOULD BE READ IN CONJUNCTION WITH ITS COORESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

214 Somerset Street E, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 369086.21 **NORTHING:** 5031742.81 **ELEVATION:** 62.78

PROJECT: Proposed Residential Development

BORINGS BY: Geoprobe

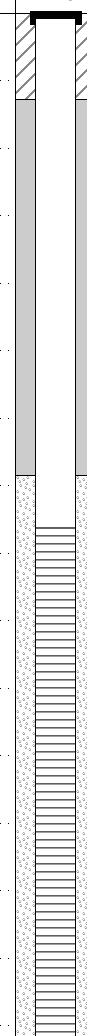
REMARKS:

DATE: October 29, 2024

FILE NO. : PG6626

HOLE NO. : BH3A-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 / △ REMOULDED SHEAR STRENGTH, Cur (kPa) ▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80 / PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE		0							
*Refer To BH 3-24 for soil profile information									62
		1							61
		2							60
		3							59
		4							58
		5							57
6.10m [56.68m]		6							
End of Borehole (Borehole dry - November 04, 2024)									56
		7							55
		8							



DISCLAIMER: THE DATA PRESENTED IN THIS LOG IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHO IT WAS PRODUCED. THIS LOG SHOULD BE READ IN CONJUNCTION WITH ITS COORESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
214 Somerset Street East
Ottawa, Ontario

DATUM Elevations are referenced from a geodetic datum

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 30, 2023

FILE NO. PG6626

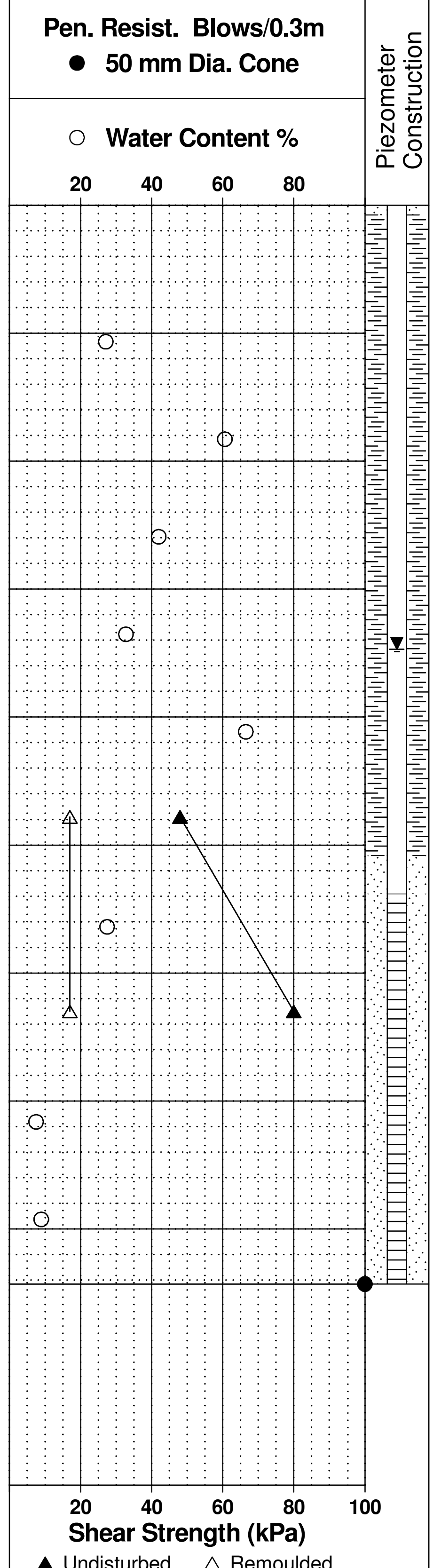
HOLE NO. BH 1-23

SOIL DESCRIPTION	Strata Depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	60.01
Asphaltic Concrete	0.05						
FILL: Loose, brown silty sand with some gravel	0.74	SS	1	33	5		
FILL: Loose, brown silty clay with sand and gravel	1.83	SS	2	67	3	1	59.01
Stiff, brown **SILTY CLAY** - Grey by 3.8m depth	6.83	SS	3	50	4	2	58.01
		SS	4	100	7	3	57.01
		SS	5	100	3	4	56.01
		SS	6	100	2	5	55.01
		SS	7	100	2	6	54.01
GLACIAL TILL: Compact, grey silty clay with sand, gravel, cobbles and boulders	7.47	SS	8	25	14	7	53.01
GLACIAL TILL: Very dense, grey silty sand with clay, cobbles and boulders	8.43	SS	9	58	54	8	52.01

Dynamic Cone Penetration test commenced at 8.23m depth

Practical Refusal to DCPT at 8.43m depth

(GWL @ 3.47m depth on April 6, 2023)



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their "sensitivity". The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

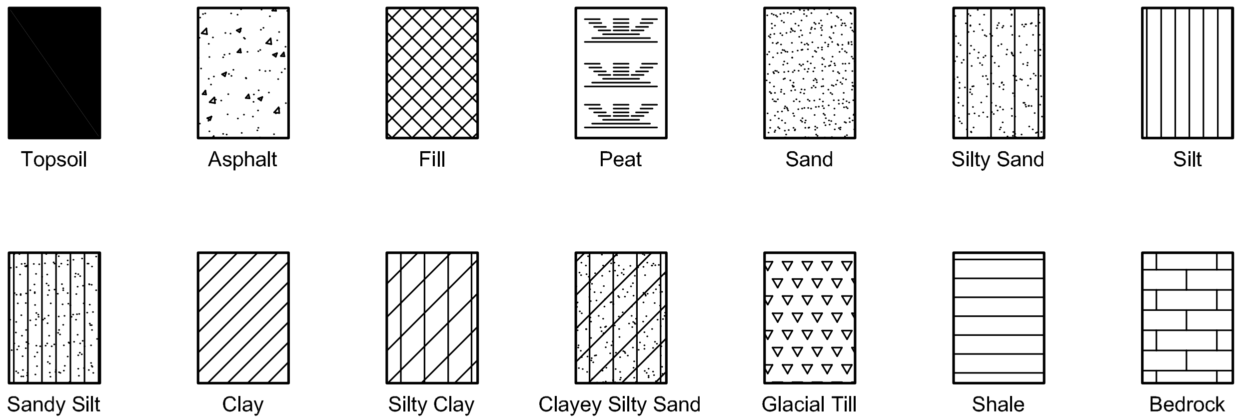
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

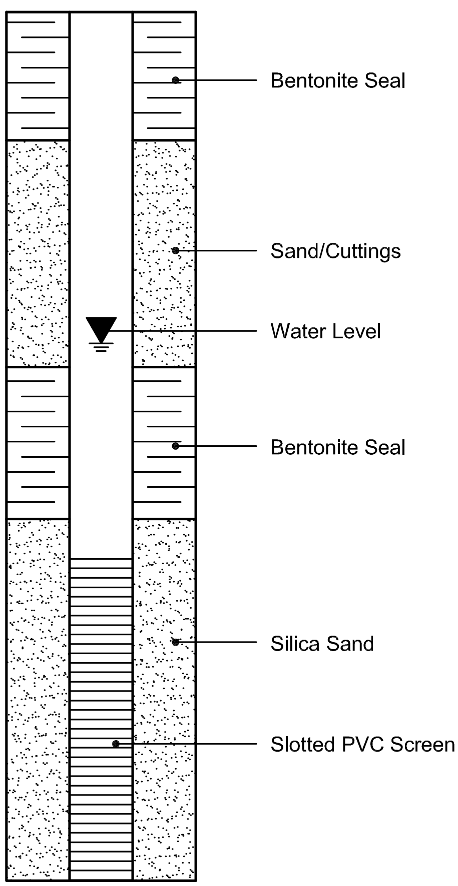
SYMBOLS AND TERMS (continued)

STRATA PLOT

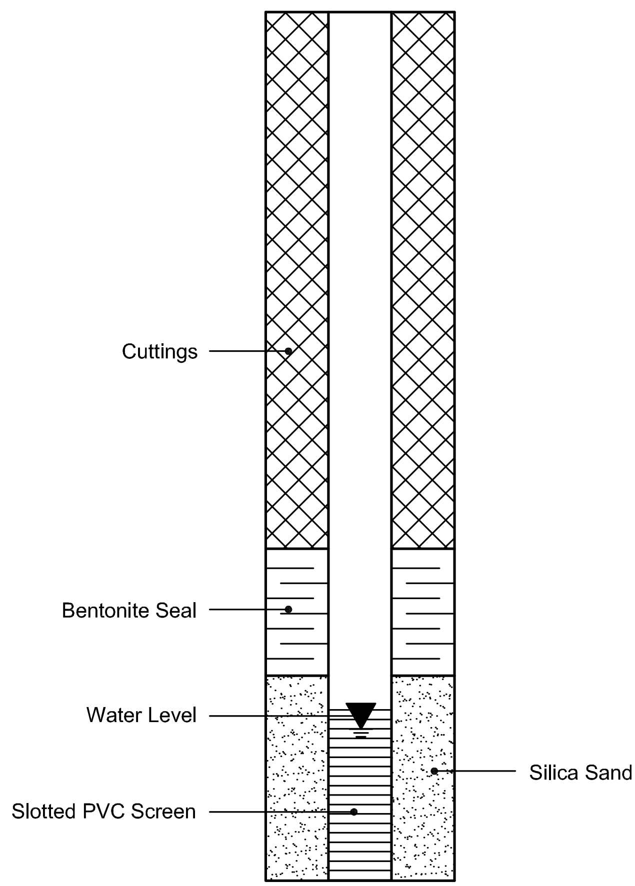


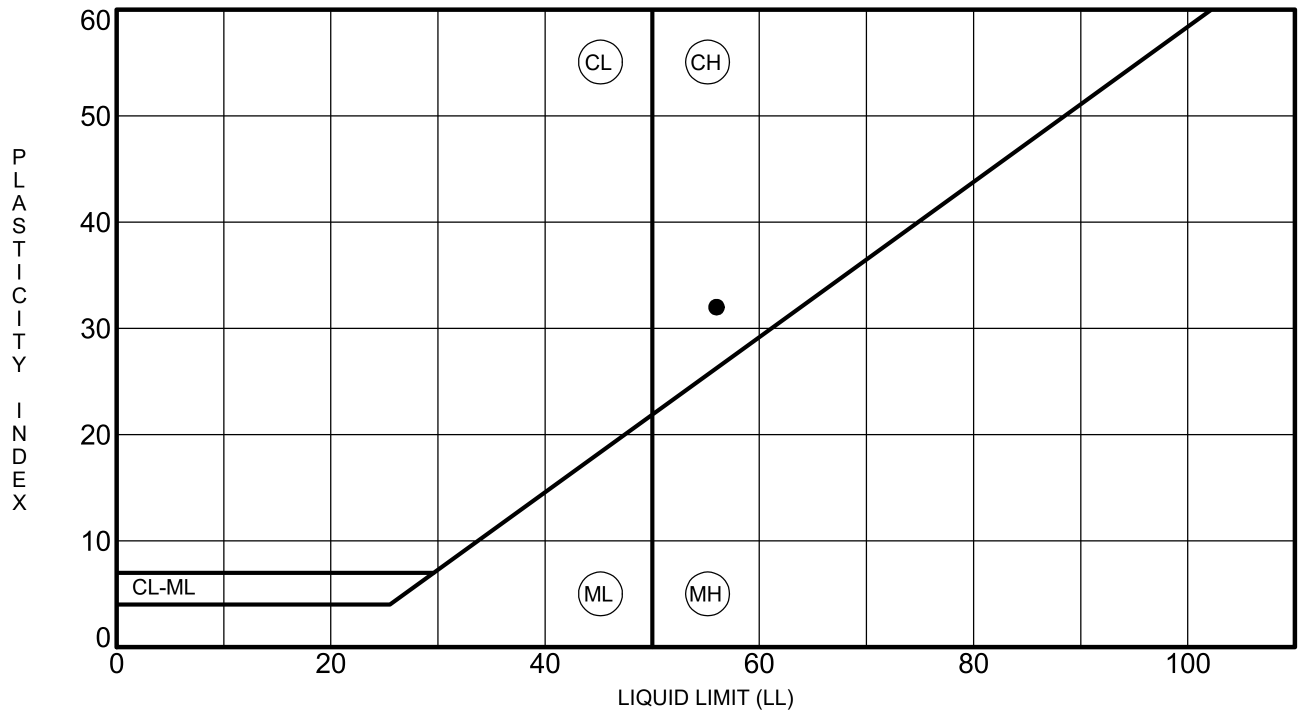
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



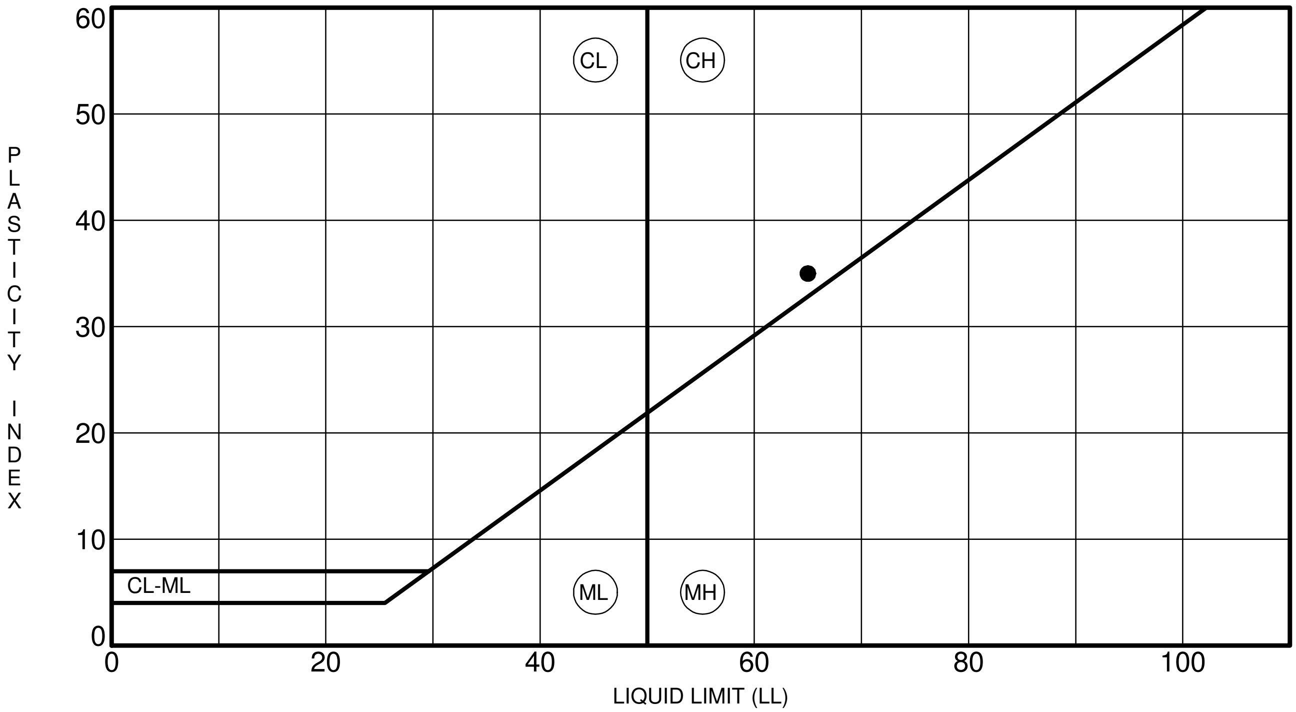


	Specimen Identification		LL	PL	PI	Fines	Classification
●	BH 3-24	SS7	56	24	32		CH - Inorganic clays of high plasticity

CLIENT	Ottawa Community Housing Corp.	FILE NO.	PG6626
PROJECT	Geotechnical Investigation - 214 Somerset Street East	DATE	29 Oct 24

PATERSON GROUP
9 Auriga Drive
Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

ATTERBERG LIMITS' RESULTS

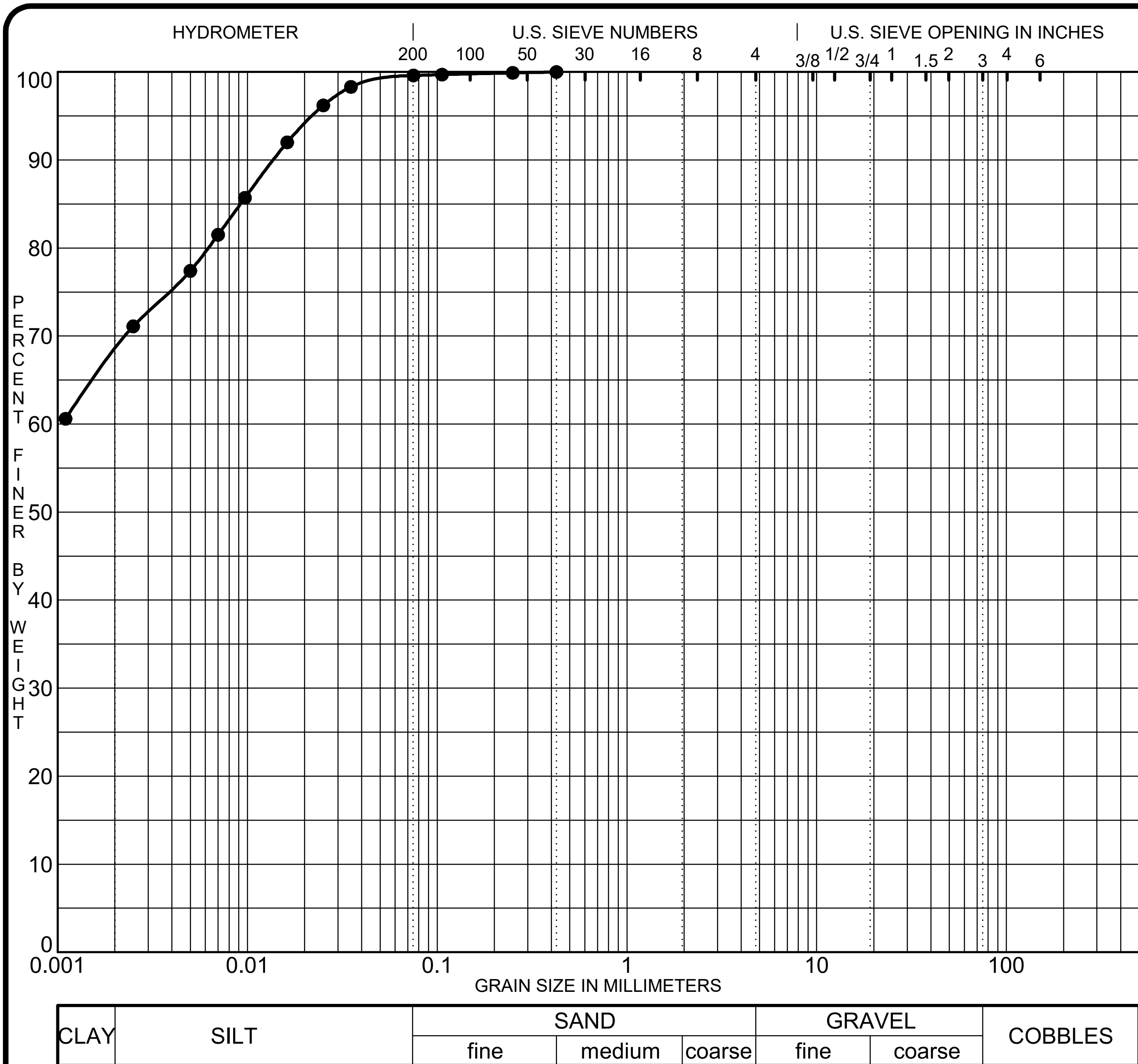


	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1-23**	**SS5**	**65**	**30**	**35**		**CH - Inorganic clay of high plasticity**

CLIENT **Ottawa Community Housing Corp.**
FILE NO. **PG6626**
PROJECT **Geotechnical Investigation - 214 Somerset Street East**
DATE **30 Mar 23**

patersongroup **Consulting Engineers**
9 Auriga Drive, Ottawa, Ontario K2E 7T9

ATTERBERG LIMITS' RESULTS



Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 1-24 SS6**		**51.7**					
⊠								
▲								
★								

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 1-24 SS6**	**0.43**				**0.0**	**0.4**	**30.6**	**69.0**
⊠									
▲									
★									

CLIENT **Ottawa Community Housing Corp.**

FILE NO. **PG6626**

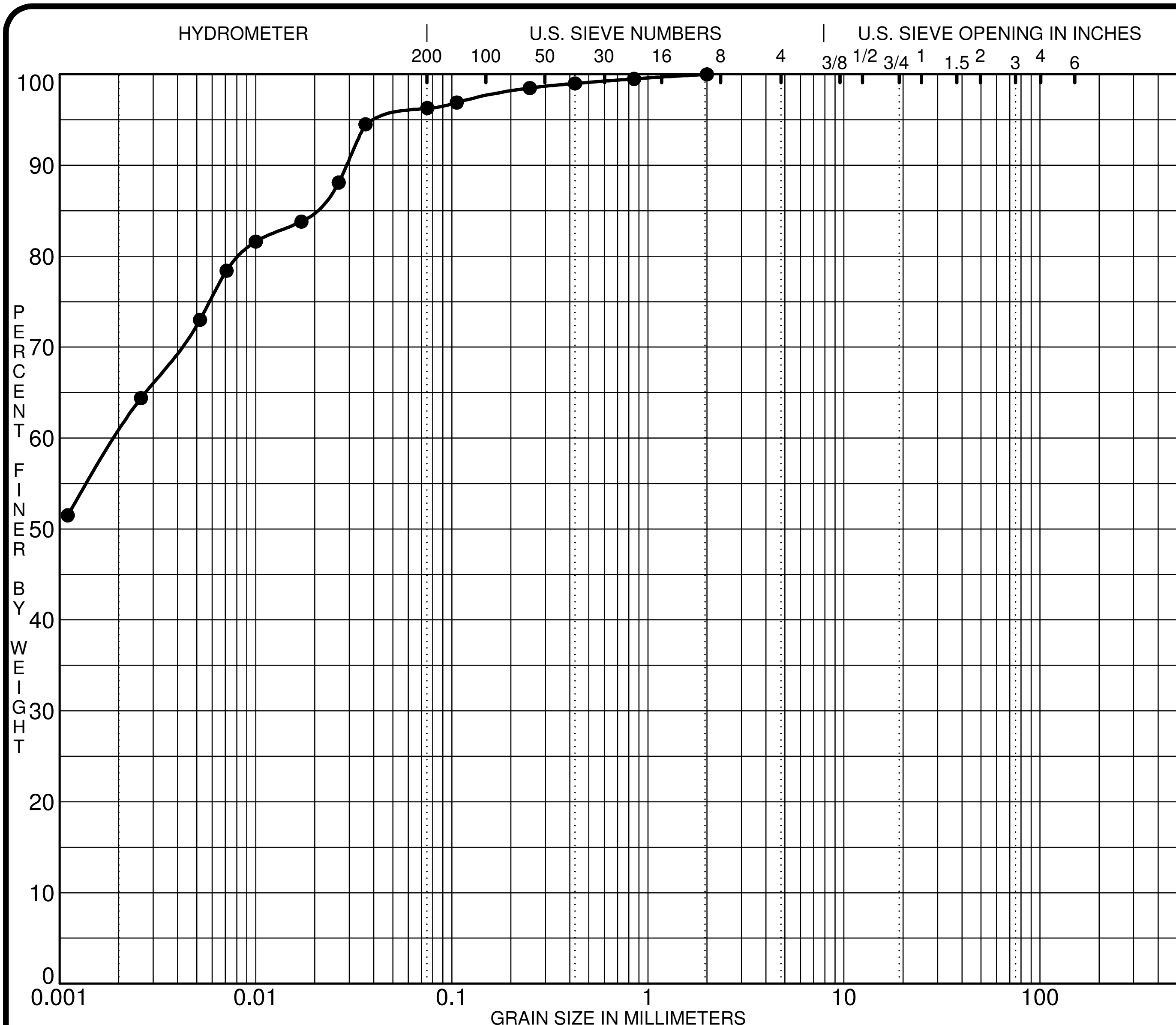
PROJECT **Geotechnical Investigation - 214 Somerset Street East**

DATE **29 Oct 24**

PATERSON GROUP

9 Auriga Drive
Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

GRAIN SIZE DISTRIBUTION



CLAY	SILT	SAND			GRAVEL		COBBLES
		fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 1-23**	**SS6**	**CH - Inorganic clay of high plasticity**						
⊠									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 1-23**	**SS6**	**2.00**	**0.00**			**0.0**	**3.7**	**36.3**	**60.0**
⊠										
▲										
★										

CLIENT **Ottawa Community Housing Corp.** FILE NO. **PG6626**

PROJECT **Geotechnical Investigation - 214 Somerset Street East** DATE **30 Mar 23**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 57176

Report Date: 11-Apr-2023

Order Date: 4-Apr-2023

Project Description: PG6626

Client ID:		BH1-23 - SS5 [10'-12']	-	-	-
Sample Date:		30-Mar-23 09:00	-	-	-
Sample ID:		2314185-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	70.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.21	-	-	-
Resistivity	0.1 Ohm.m	19.3	-	-	-
Anions					
Chloride	10 ug/g dry	196	-	-	-
Sulphate	10 ug/g dry	44	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

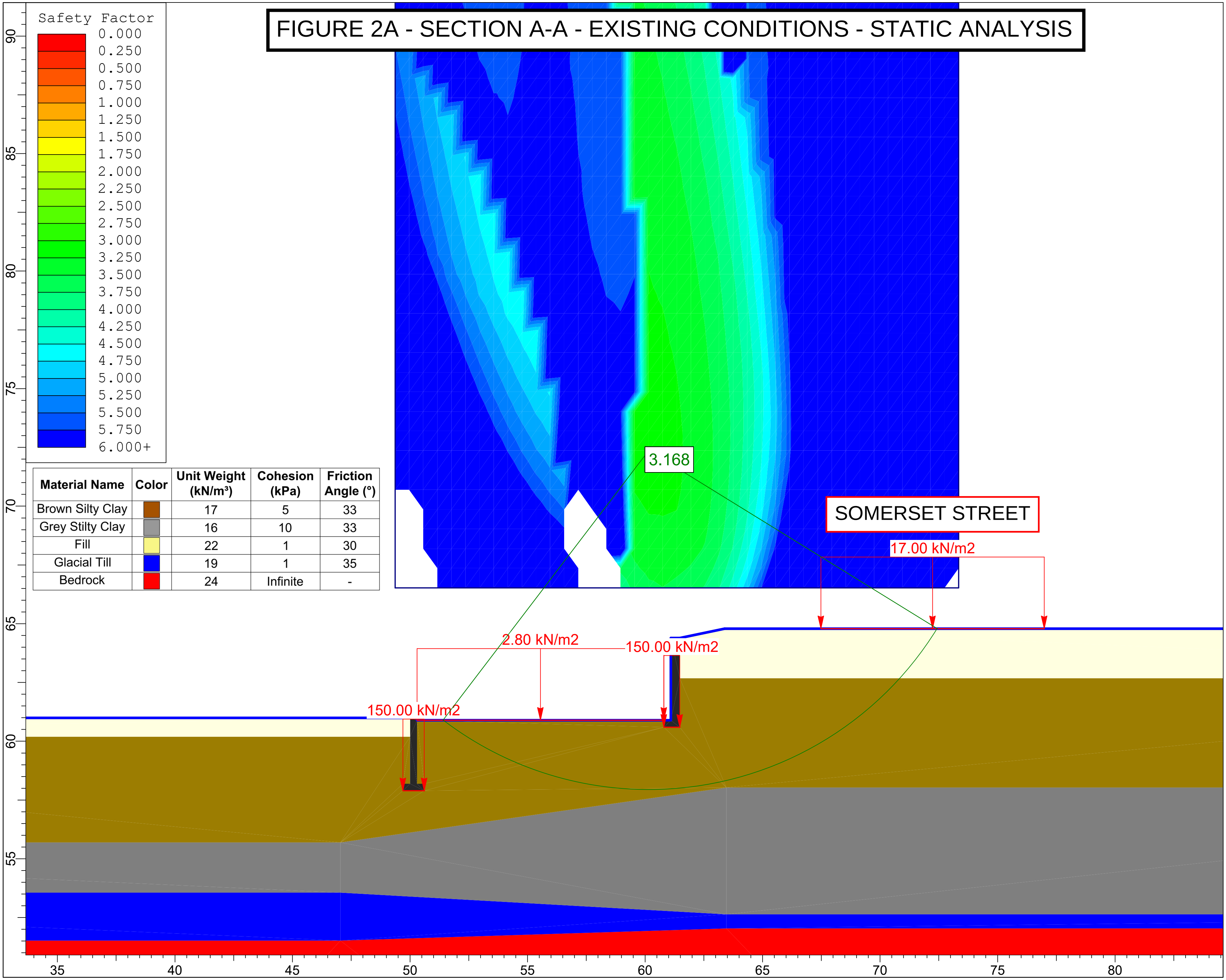
FIGURES 2 & 3 – SLOPE STABILITY CROSS SECTIONS

DRAWING PG6626-1 – TEST HOLE LOCATION PLAN

FIGURE 1

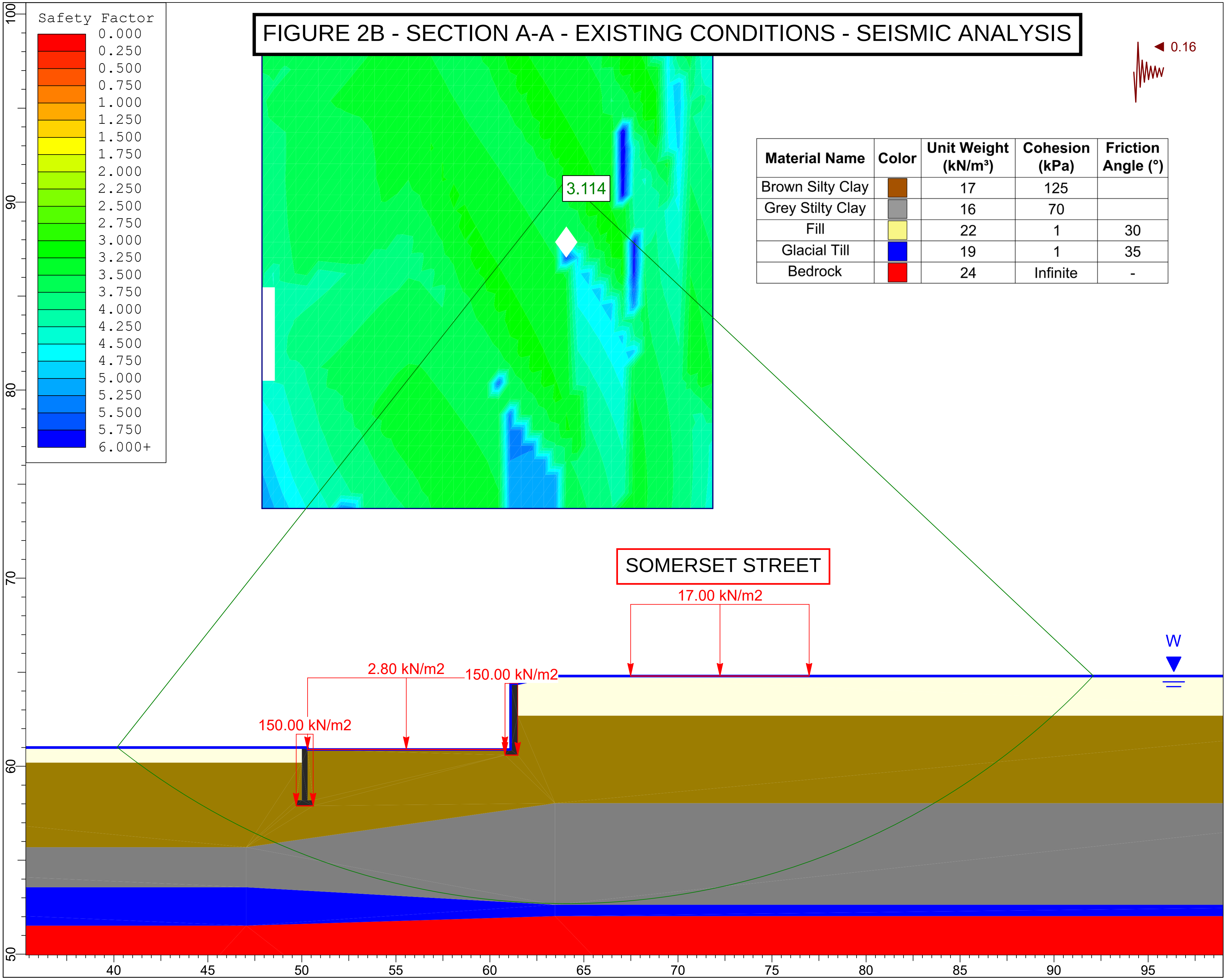
KEY PLAN

PATERSON GROUP



Material Name	Color	Unit Weight (kN/m³)	Cohesion (kPa)	Friction Angle (°)
Brown Silty Clay		17	5	33
Grey Stilty Clay		16	10	33
Fill		22	1	30
Glacial Till		19	1	35
Bedrock		24	Infinite	-

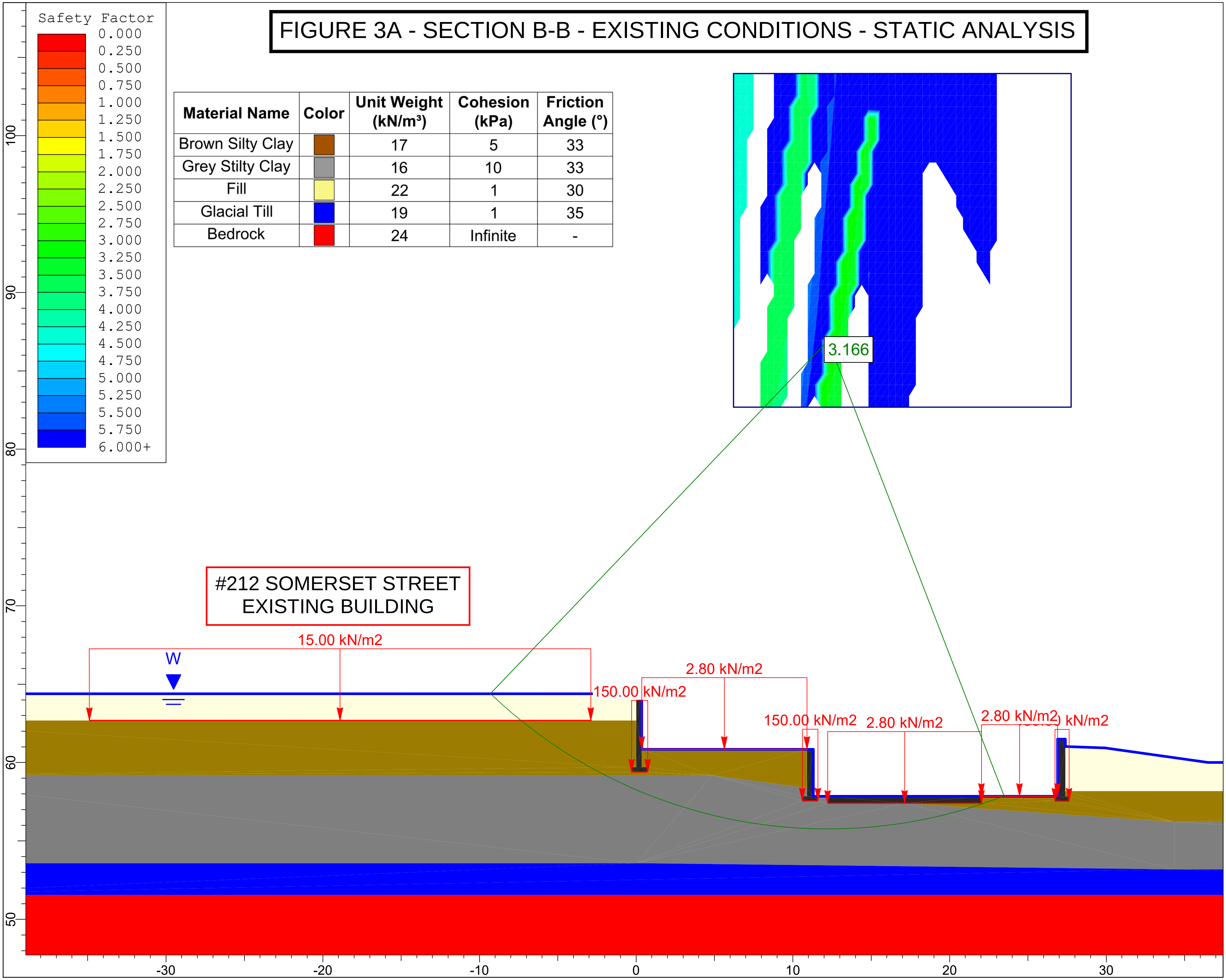
FIGURE 2B - SECTION A-A - EXISTING CONDITIONS - SEISMIC ANALYSIS

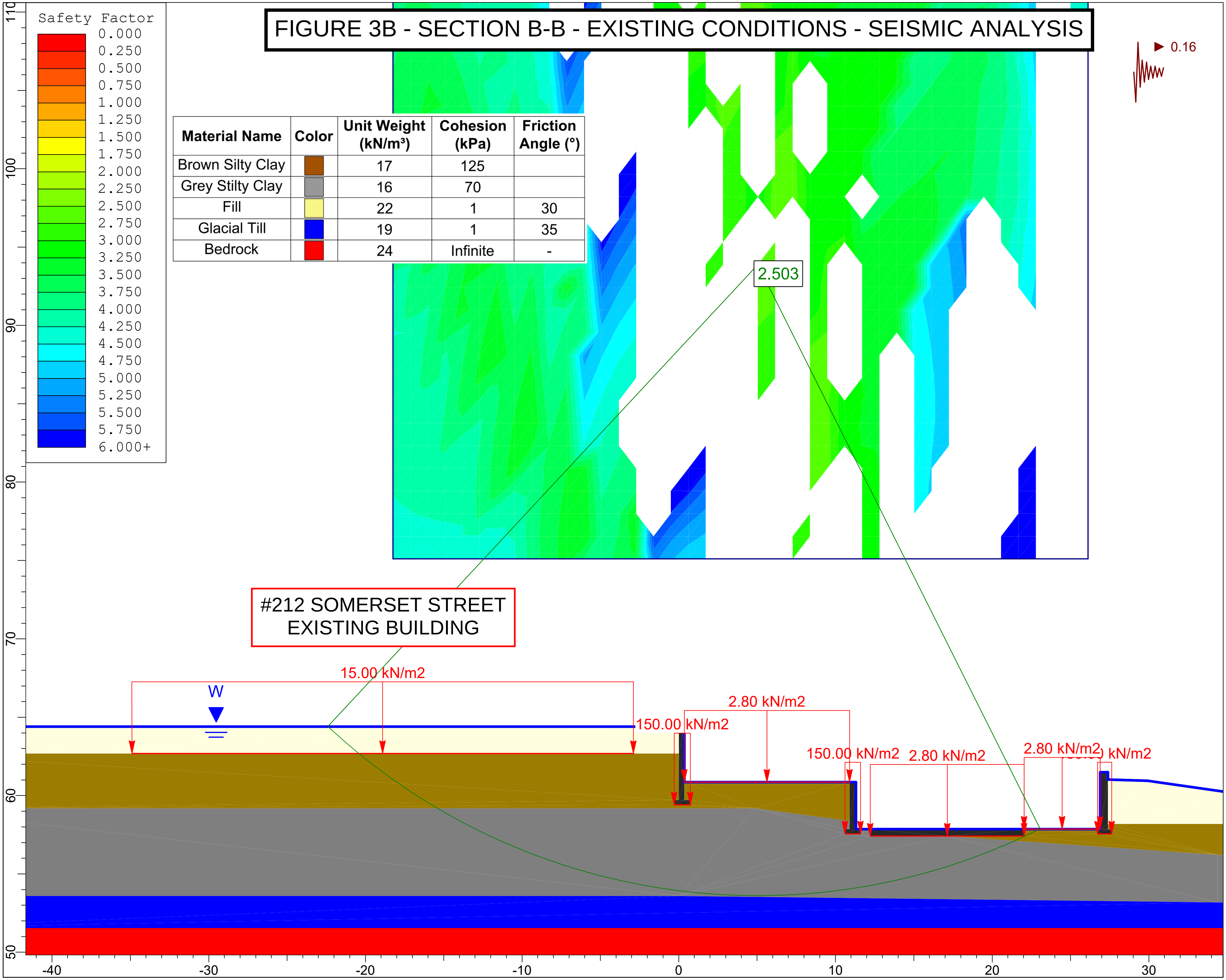


Material Name	Color	Unit Weight (kN/m³)	Cohesion (kPa)	Friction Angle (°)
Brown Silty Clay		17	125	
Grey Stilty Clay		16	70	
Fill		22	1	30
Glacial Till		19	1	35
Bedrock		24	Infinite	-

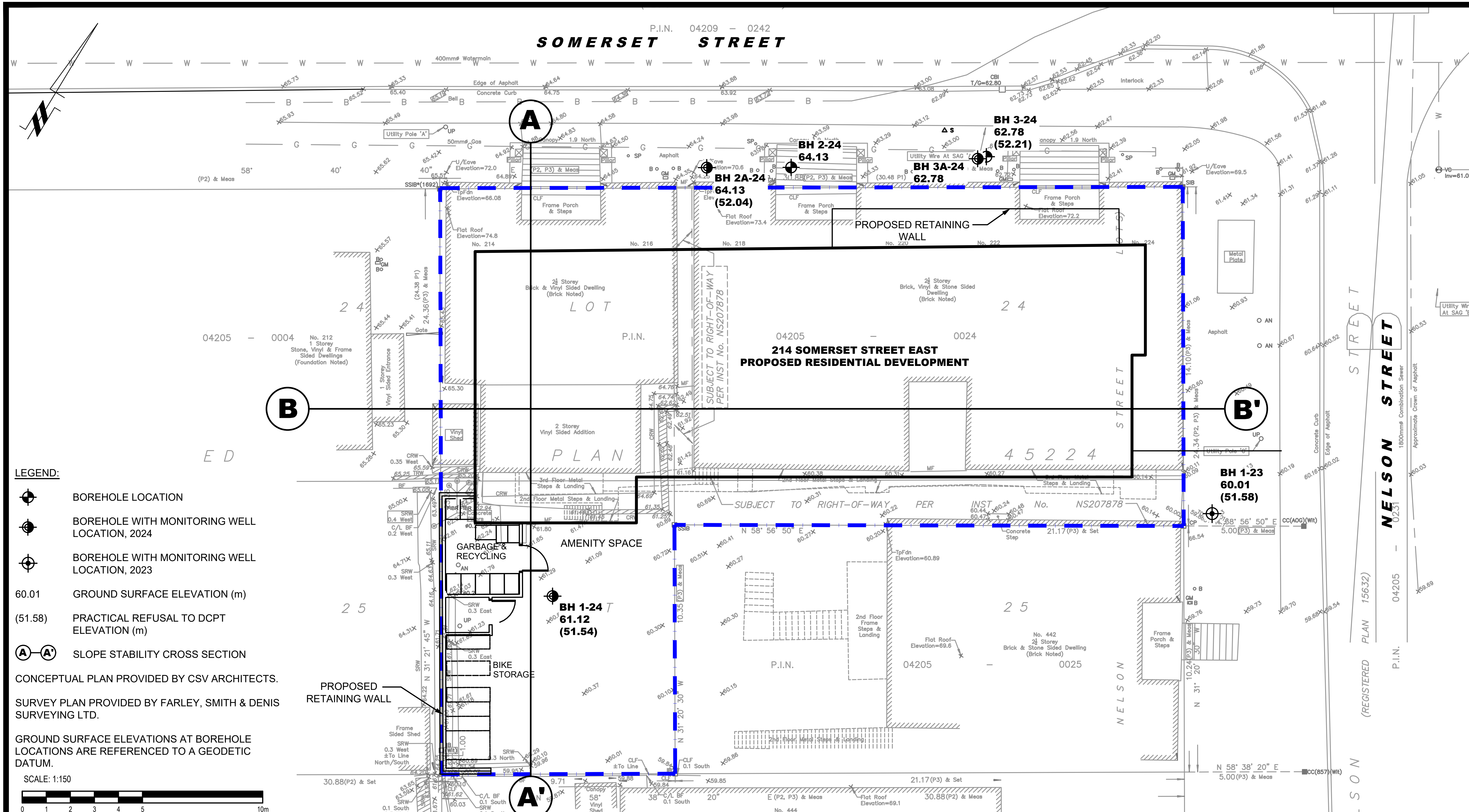
FIGURE 3A - SECTION B-B - EXISTING CONDITIONS - STATIC ANALYSIS

Material Name	Color	Unit Weight (kN/m³)	Cohesion (kPa)	Friction Angle (°)
Brown Silty Clay		17	5	33
Grey Stilty Clay		16	10	33
Fill		22	1	30
Glacial Till		19	1	35
Bedrock		24	Infinite	-





Material Name	Color	Unit Weight (kN/m³)	Cohesion (kPa)	Friction Angle (°)
Brown Silty Clay		17	125	
Grey Stilty Clay		16	70	
Fill		22	1	30
Glacial Till		19	1	35
Bedrock		24	Infinite	-



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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED CONCEPTUAL PLAN AND ADDED 2024 BOREHOLE BH 1-24 TO BH 3A-24	06/11/2024	DP

OTTAWA COMMUNITY HOUSING CORPORATION

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
214 SOMERSET STREET EAST

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:150	Date: 07/2023
Drawn by: NFRV	Report No.: PG6626-1
Checked by: FC	Dwg. No.: PG6626-1
Approved by: DP	Revision No.: 1

11x17

p:\autocad drawings\geotechnical\pg66xx\pg6626\pg6626-1 test hole location plan (rev.01).dwg