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Geotechnical Engineering

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Noise and Vibration Monitoring

Geotechnical Investigation

Proposed Multi-Storey Buildings
Hillside Development
3277 St. Joseph Boulevard
Ottawa, Ontario

Prepared For

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c/o Landric Homes

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1.0 Introduction

Paterson Group (Paterson) was commissioned by DCR Phoenix Homes c/o Landric Homes to conduct a geotechnical investigation for the proposed multi-storey buildings to be located at 3277 St. Joseph Boulevard in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of test holes.
- Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.
- Provide geotechnical design and construction recommendations regarding the protection of the existing 1200 mm diameter Gloucester Cumberland Trunk Sewer which bisects the subject site.

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, the proposed development will consist of 2 multi-storey residential buildings, Buildings A and B, which will have underground parking levels extending to geodetic elevation 62.1 and 64.6 m, respectively.

Further, it is anticipated that the proposed buildings will be surrounded by asphalt paved access lanes and parking areas with landscaped margins. It is also anticipated 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 during the period of February 24, 2021 through March 3, 2021 and consisted of a total of 8 boreholes (BH 1-21 to BH 8-21) sampled to a maximum depth of 11.5 m below the existing ground surface. A previous geotechnical investigation was carried out at the subject site on May 20 through 22, 2020. At that time, a total of 12 boreholes (BH 1A-20, BH 1B-20, BH 2A-20, BH 2B-20, BH 2C-20, BH 3A-20, BH 3B-20, BH 3C-20, BH 3D-20, BH 4-20, BH 5-20, and BH 6-20) were sampled down to a maximum depth of 11 m below the existing ground surface. The borehole 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 PG5625-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a track-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The testing procedure for boreholes consisted of augering to the required depths and at the selected locations and sampling the overburden.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were 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.

Rock samples were recovered from boreholes BH 1-21, BH 2-21, BH 3-21, BH 4-21, BH 5-21, BH 8-21, BH 2C-20, and BH 3D-20 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory for further review.

The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

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.

Sample Storage

All samples from the current geotechnical 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.

3.2 Field Survey

The borehole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The borehole locations and ground surface elevation at each borehole 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 borehole location are presented on Drawing PG5625–1 – 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.

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 analyzed to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are discussed in Section 6.7 and shown in Appendix 1.

4.0 Observations

4.1 Surface Conditions

The subject site is bordered to the east by the existing Tenth Line Road overpass embankment, to the south by St. Joseph Boulevard, and to the west by 3-storey residential townhouses constructed over a single-level underground parking structure. A 5-storey residential apartment building occupies the neighbouring property to the north, which in turn fronts onto Eric Czapnick Way to the north.

The central portion of the subject site slopes gradually down toward the north, but is relatively low in comparison to the neighbouring property to the west and the adjacent roadways to the south and east. According to the available topographic information, the neighbouring property to the west and the adjacent roadways to the south and east vary between 6 and 9 m above the lower central portion of the site at approximate gradients of 2.7H:1V to 6H:1V.

Based on available information, a 10 m service easement bisects the central portion of the site into an east and west parcel, and is occupied by the 1200 mm diameter Class V Gloucester Cumberland Trunk Sewer.

The lower central portion of the site is generally grass covered while the surrounding embankments to the south, east and west are brush covered and sparsely forested.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of topsoil overlying varying thicknesses of fill, which in turn is overlying stiff, brown silty clay and/or glacial till, followed by bedrock.

The fill encountered at the site ranges in thickness from 1.5 to 8.7 m, and generally consists of silty sand to silty clay with gravel, cobbles, boulders, crushed stone and blast rock.

In the northeast portion of the site, at BH 6-21 through BH 8-21, BH 3D-20, BH 4-20, and BH 6-20, a hard to stiff, brown silty clay deposit was encountered underlying the fill.

A glacial till deposit was also encountered underlying the silty clay at BH 5-20, BH 6-20, BH 7-21, and BH 8-21 at approximate depths of 5.6 to 7.5 m below the existing ground surface. Where encountered, the glacial till deposit was observed to consist of a dense to very dense, grey silty sand to sandy silt with gravel, cobbles, and boulders.

Bedrock

Bedrock was cored at BH 1-21 through BH 5-21, and BH 8-21, starting at approximate depths of 1.5 to 9.2 m below existing ground surface, which was generally increasing in depth from southwest to northeast across the site. Refer to Drawing PG5625-2 – Bedrock Contour Plan which provides estimated contours of the bedrock surface, which are interpolated between the boreholes where bedrock was encountered.

Based on our observations of the recovered bedrock cores, the bedrock consists of a grey limestone. Further, the RQDs of the recovered bedrock core ranged from 30 to 100%, generally increasing with depth, and which is indicative of a poor to excellent quality bedrock.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Gull River formation.

4.3 Groundwater

The groundwater levels measured in the boreholes are presented in Table 1 on the next page. Groundwater conditions can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, it is estimated that groundwater can be expected between approximate geodetic elevation 57 to 59 m.

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

Table 1 - Summary of Groundwater Level Readings

(PG5349)

Borehole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 1A-20	74.57	Dry	< 73.22	May 29, 2020
BH 2C-20	70.93	Blocked	-	
BH 3D-20	69.36	Blocked	-	
BH 4-20	64.90	Blocked	-	
BH5-20	65.12	6.52	58.60	
BH6-20	64.58	6.82	57.76	

(PG5625)

Borehole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 1-21	72.40	7.78	64.62	March 8, 2021
BH 2-21	74.66	Blocked	-	
BH 3-21	73.50	Blocked	-	
BH 4-21	71.21	4.75	66.46	
BH 5-21	70.94	5.55	65.39	
BH 6-21	68.32	8.52	59.80	
BH 7-21	64.86	Damaged	57.15	
BH 8-21	65.52	6.95	58.57	

Note: The location of the boreholes and the ground surface elevation at each borehole location are referenced to a geodetic datum and presented on Drawing PG5625-1 - Test Hole Location Plan in Appendix 2.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed multi-storey buildings. It is recommended that the proposed buildings be supported on conventional spread footings which bear on the undisturbed, hard to stiff silty clay, dense glacial till, and/or clean, surface sounded bedrock.

Where fill is encountered at the proposed underside of footing elevation, it should be sub-excavated to the undisturbed, hard to stiff silty clay, dense glacial till, and/or clean, surface sounded bedrock, and replaced with a minimum 17 MPa lean-mix concrete.

The proposed building footings will also need to be located at a certain minimum elevation such that the lateral support zones of the footings do not intersect with the existing 1200 mm diameter Class V Gloucester Cumberland Trunk Sewer. These elevations are provided on the Figures 1 through 7 in Appendix 2.

Where portions of the proposed buildings are founded on soil and other portions are founded on bedrock, a control joint between the foundations of the buildings can be considered to avoid differential settlement. The structural design will dictate if this is required.

Bedrock removal is anticipated to be required to complete portions of the underground parking levels. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations. A vibration monitoring program should be implemented and monitored by the geotechnical consultant to confirm that the controlled blasting program does not negatively impact the existing structures and utilities at and/or in the vicinity of the site, including the 1200 mm diameter Class V Gloucester Cumberland Trunk Sewer which runs through the central portion of the site.

Due to the presence of a silty clay layer within the north portion of the site, this area of the site is subjected to a permissible grade restriction.

Protection of Existing Services

Due to the depth and proximity of the existing 1200 mm diameter Class V Gloucester Cumberland Trunk Sewer with respect to the proposed structure, it is recommended that the lateral support zone of the existing service pipe be protected over the course of the construction.

Furthermore, a sewer pipe monitoring program is recommended to ensure that excessive settlement and vibrations do not occur at the sewer pipe location.

It will also be important to ensure that the building loads of the proposed multi-storey structures are extended below the invert level of the existing service pipe in order to permit future repairs to the service pipe without resulting in temporary shoring or underpinning of the multi-storey structures.

Temporary Shoring Requirements

It is understood that a temporary shoring system will be in place during the excavation program for the proposed structures. For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. However, due to the blast rock and bouldery fill encountered within the embankment, the site may not be suitable for interlocking steel sheet piling.

The temporary shoring system will be required to support the adjacent roadways and neighboring properties surrounding the site from all sides. In addition, the temporary shoring system will be required to adequately support the soils below the southern portion of the existing sanitary trunk sewer within the middle of the site. Refer to Figure 1 - Cross-Section A provided in Appendix 2.

The design of the temporary shoring system should also take into consideration the sub-excavation and placement of lean concrete which will be required for foundation support in certain portions of the site.

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 significant organic materials, should be stripped from under any building, paved areas, pipe bedding and other settlement sensitive structures. Precautions should be taken to ensure that all bearing surfaces and subgrade soils remain undisturbed during site preparation activities.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter and within the lateral support zones of the foundations. Existing foundation walls and other construction debris are not considered suitable for reuse at the site. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is severely weathered or where only a small quantity of bedrock needs to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, building, and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in the proximity of the blasting operations should be carried out prior to commencing site works.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program in order to reduce the risks of damage to the existing surrounding structures and utilities. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

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

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people. A pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Rock Stabilization

Excavation side slopes in sound bedrock can be completed with almost vertical side walls. A minimum of 1 m horizontal bench should remain between the bottom of the overburden and the top of the bedrock surface to provide an area for potential sloughing or to provide a stable base for the overburden shoring system.

Horizontal rock anchors may be required at specific locations to stabilize the bedrock excavation face and to prevent pop-outs of the bedrock, especially in areas where bedrock fractures or fault lines are conducive to the failure of the bedrock surface.

The requirement for horizontal rock anchors will be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

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 fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be placed as general landscaping fill where settlement of the ground surface is a minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm thick lifts and to a minimum density of 95% of the respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 95% of its SPMDD.

Gloucester Cumberland Trunk Sewer Monitoring Program

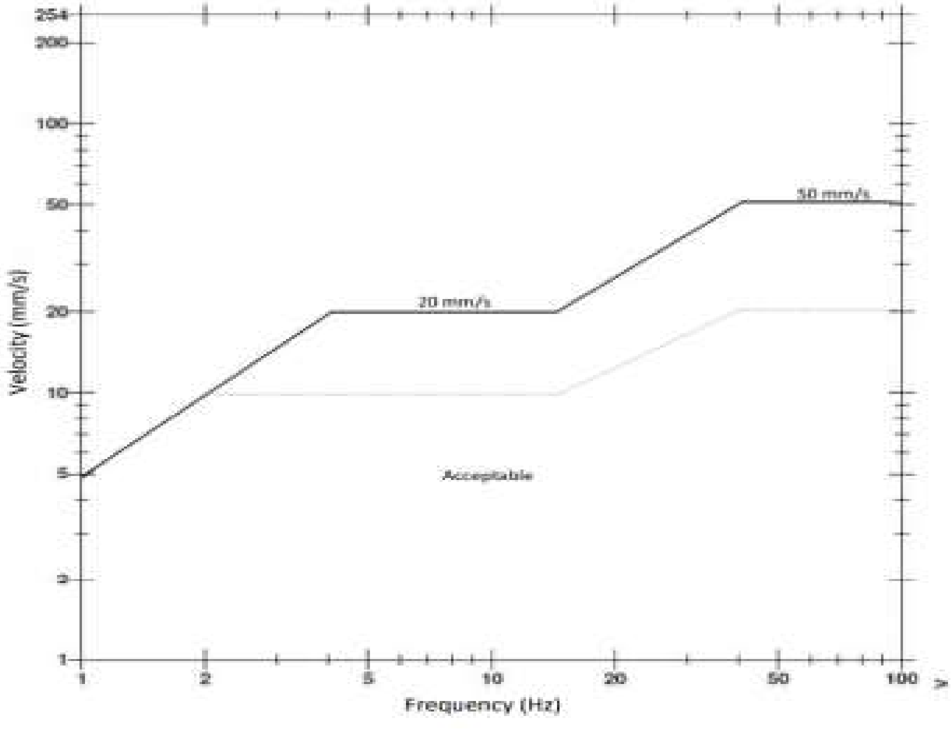
The following sewer pipe monitoring program is recommended to ensure that excessive settlement and vibrations do not occur at the sewer pipe location:

- Install 2 utility monitoring points and inclinometers directly on top of the 1200 mm diameter sewer. Further, it is recommended that two (2) inclinometers be installed adjacent to the sewer pipe and the adjacent shoring face for monitoring lateral deflection. Daily monitoring events should be completed during the excavation program until the tiebacks are stressed and then weekly during the construction program until the foundation extends above exterior finished grade. An alert level with 3 mm of movement will require an assessment. An action level with movement greater than 6 mm will require immediate attention and possible mitigation measures. A visual inspection will also be completed along with the monitoring events.

- Periodically monitor the vibration levels using 2 vibration monitors and inclinometers installed directly on the 1200 mm diameter sewer pipe.

- If the vibration limits provided on Vibration Criteria Figure are exceeded, the site superintendent will be notified by Paterson personnel of the exceedance and the shoring/excavation operation will be stopped. The project surveyor will survey the sewer pipe level to ensure pipe movement has not occurred. If pipe movement is not observed based on the survey results, the shoring/excavation operation will resume

The following vibration limits are recommended for the shoring/excavation operation to be completed adjacent to the 1200 mm diameter sewer pipe.



Vibration Criteria Figure - Proposed Vibration Limits at the Sewer Pipe

Weekly reporting of our findings and recommendations will be provided to the owner and the City of Ottawa. Any mitigation measures contemplated for implementation will be discussed with the owner and City of Ottawa personnel.

5.3 Foundation Design

Conventional Shallow Footings

Footings placed on clean, surface sounded limestone or dolomite bedrock, or on lean concrete which is placed directly over the clean, surface sounded limestone or dolomite bedrock, can be designed using a factored bearing resistance value at ULS of **1,500 kPa**.

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

A factored bearing resistance value at ULS of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5, could be used for the design of footings bearing on bedrock, or on lean concrete bearing on bedrock, which is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footing footprints. One drill hole should be completed per major footing. The drill hole inspection should be completed by the geotechnical consultant.

Conventional spread footings placed over an undisturbed, hard to stiff silty clay, dense glacial till, or on lean concrete which is placed directly over the undisturbed, hard to stiff silty clay or dense glacial till, can be designed using bearing resistance values at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

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 prior to the placement of concrete for footings.

Settlement

Footings bearing on the undisturbed, hard to stiff silty clay, dense glacial till, or engineered fill at the bearing resistance values at SLS provided above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively. Footings placed on clean, surface sounded bedrock will be subjected to negligible post-construction total and differential settlements.

Conventional Spread Footings Extended to Bedrock

Footings can be extended to bedrock by either:

- completing a mass excavation and placing the footings directly on bedrock and then backfilling under the basement slab with engineered fill, as per the recommendations provided Subsection 5.2 - Fill Placement and in Subsection 5.8 - Pavement Structure Design, or

- by means of trenching and in-filling with lean concrete. If a lean concrete in-filled trench is considered, it is recommended that a near vertical, zero entry trench extend at least 300 mm beyond the outside edge of the proposed footings.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Above the groundwater level, adequate lateral support is provided to a stiff silty clay when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:1V passes only through in situ soil or engineered fill.

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

Bedrock/Soil Transition

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

Permissible Grade Raise

Based on our review of the subsoil profile, a permissible grade raise restriction of **4.5 m** above existing ground surface will be assigned for the north portion of the site where a silty clay deposit was encountered.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. A higher site class, such as Class A or B, may be obtained for footings placed within 3 m of the bedrock surface. However, the higher seismic site class would need to be confirmed by site-specific seismic shear wave velocity testing.

The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

The basement areas for the proposed buildings will be mostly parking and the recommended pavement structure noted in Subsection 5.8 will be applicable. However, if storage or other uses of the lower level will involve the use of a concrete floor slab, then the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. The upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone for slab on grade construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building(s) should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structures. 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.

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.

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

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

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

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

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

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

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that centre to centre spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m in order to lower the group influence effects. It is also recommended that rock anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Based on compressive strength testing results completed for limestone and dolomite in the Ottawa area, the unconfined compressive strength of limestone and dolomite generally ranges between 100 to 150 MPa, which is stronger than most routine grouts. Conservatively, a compressive strength of 80 MPa can be used for bedrock at the subject site. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system.

Based on existing subsoils information, a conservative **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required.

For our calculations, the parameters in Table 2, provided on the next page, were used:

Table 2 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	65 m=.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	80 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the total anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 3.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.1	1.1	2.2	250
	2.2	2.2	4.4	500
	4.6	1.4	6.0	1000
	8.8	1.2	10.0	2000
125	0.8	1.3	2.1	250
	1.3	1.7	3.0	500
	2.6	1.9	4.5	1000
	5.3	2.2	7.5	2000

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Design

Car only parking, access lanes and heavy truck parking are expected at this site. The subgrade material will consist of native soil and fill. The proposed pavement structures are presented in Tables 4 and 5 below.

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.

Table 4 – 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, fill, or bedrock.	

Table 5 – Recommended Pavement Structure – Access Lanes and Heavy Truck Parking

Thickness (mm)	Material Description
40	**Wear Course –** Superpave 12.5 Asphaltic Concrete
50	**Wear Course –** Superpave 19 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
400	**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, fill, or bedrock.	

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. The 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 using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. It is expected that the foundation walls along the north, south, east and part of the west will be placed directly against the temporary shoring system and/or adequately prepared bedrock surface in conjunction with a foundation drainage system detailed below:

- The temporary shoring system and/or vertical bedrock surface should be suitably prepared to receive the foundation drainage system. The vertical bedrock surface will be prepared by grinding or using shotcrete to smooth out angular sections depending on the manufacturer's requirements of the composite drainage layer.

- The composite drainage layer, such as Miradrain G100N or Delta Drain 6000, will be securely fastened to the temporary shoring system and/or approved bedrock in an overlapping shingle fashion to direct groundwater down and away from the exterior concrete foundation wall to the 150 mm diameter sleeves cast in the foundation wall/footing interface.

- It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to the sump pit(s) within the lower basement area.

It is expected that the remaining foundation walls will be constructed using conventional double-sided formwork with a conventional perimeter foundation drainage system detailed below:

- The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pit through 150 mm sleeves at 3 m centres be cast in the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe.

- 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 Miradrain G100N or 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.

Underfloor Drainage

Underfloor drainage underlying the basement slabs will be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at approximate 6 to 9 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Adverse Effects of Dewatering on Adjacent Properties

Based on the expected foundation levels of the underground parking structures and the expected depth of the groundwater level, the proposed development will be founded above the long-term groundwater level. As a result, any minor dewatering effect from the foundation drainage system will not have adverse effects to the surrounding buildings or properties.

6.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as 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 an equivalency combination of soil cover and foundation insulation.

However, the foundations are generally not expected to require additional protection against frost action due to the founding depth. Unheated structures, such as the access ramp, may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

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 subsurface soil 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 maintain safe working distance 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.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

It is understood that a temporary shoring system will be in place during the excavation for the proposed structures. The temporary shoring system will be required to support the adjacent roadways and neighbouring properties to the north, south, and west. In addition, the temporary shoring system will most likely be required along a portion of the west exterior foundation wall of Building A to adequately support the soils below a portion of the existing sanitary trunk sewer within the south portion of the site. Refer to Figure 1 - Cross-Section A provided in Appendix 2.

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 also 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 design of the temporary shoring system should also take into consideration the sub-excavation and placement of lean concrete which will be required for foundation support in certain portions of the site.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. However, due to the boulders in the fill and glacial till, the site may not be suitable for 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 design of the rock anchors for temporary shoring can be based on the values provided in Subsection 5.7 of the present report.

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 & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the material's SPMDD.

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 reduce the potential 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 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. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding 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 stratigic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low through the sides of the excavation and controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

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, 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.

Long-term Groundwater Control

The recommendations for the proposed building long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building perimeter or sub-slab drainage system will be directed to the proposed building cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, the groundwater flow should be low (i.e.- less than 10,000 L/day per building) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. The groundwater flow should be controllable using conventional open sumps.

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 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 installation of straw, propane heaters 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 footings are protected with sufficient soil cover to prevent freezing at founding level.

The 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.

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 samples 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 highly aggressive corrosive environment.

6.8 Existing Gloucester Cumberland Trunk Sewer

According to the legal survey drawing prepared by Annis, O'Sullivan, Volebekk Ltd., in conjunction with the design drawings prepare by the Regional Municipality of Ottawa-Carleton, Drawing S-1472 - 2-3, the 1200 mm diameter C302, Class V concrete sewer pipe is centrally located within the 10 m wide service easement between the proposed Buildings A and B. The 10 m service easement bisects the central portion of the site in a north/south direction, subdividing the site into an east and west parcel.

Based on our review of the available information, it is our understanding that the ASTM C-76, Class V designation for a 1200 mm diameter concrete sewer pipe is considered an equivalent to the current CSA A257.2, Class 140-D designation standard for a 1200 mm concrete sewer pipe. It is recommended that the existing 1200 mm diameter trunk sewer be further evaluated with a CCTV and a structural review by the project's structural engineer.

Under conventional service pipe installation in the Ottawa area, it is considered acceptable to assume that the existing 1200 mm diameter ASTM C-76, Class V / CSA A257.2, 140D sewer pipe was placed in general accordance with Class B - Pipe Bedding Details similarly detailed above. According to OPSD 807.010 - Fill Height Table - Reinforced Concrete Pipe - Confined Trench - Class 50-D, Class 65-D, Class 100-D and Class 140-D, the maximum height of fill permitted above the 1200 mm diameter Class 140-D reinforced concrete pipe installed in accordance with Class B - Pipe Bedding is 13.1 m.

Based on our review of the currently as-built survey information provided by Annis, O'Sullivan, Vollebekk Ltd., the maximum height of existing fill over the subject alignment of the sewer pipe varies approximately between 2.5 and 8.5 m. However, it should be noted that the additional earth pressure from the nearby Tenth Line Road embankment which was constructed after the installation of the 1200 mm diameter trunk sewer should be taken into consideration.

Due to the location of the proposed structures with respect to the existing 1200 mm diameter trunk sewer and the Tenth Line Road embankment, it is expected that the construction of the underground parking levels of the proposed structure will unload the earth pressure that was previously added to the existing 1200 mm diameter trunk sewer during the placement of the roadway embankment. Although a significant volume of earth pressure will be removed from the existing 1200 mm diameter, it will be important to ensure that the proposed grade raises will not exceed the allowable fill height above the existing sewer pipe. Any areas that exceed the maximum allowable fill height will receive the equivalent volume of expanded polystyrene (EPS), Type 19 lightweight fill (LWF) as noted in Figure 5 - Cross-Section E included in Appendix 2.

Due to the depth and proximity of the existing trunk sewer with respect to the proposed structure, it is recommended that the lateral support zone of the existing service pipe be protected over the course of the construction program. Based on our cursory review of the available conceptual drawings, a temporary shoring system will most likely be required along a portion of the west exterior foundation wall of Building A to adequately support the soils below a portion of the existing sanitary trunk sewer within the south portion of the site. Refer to Figure 1 - Cross-Section A provided in Appendix 2.

As previously noted, it will be important to ensure that the building loads of the proposed multi-storey structures are extended below the invert level of the existing service pipe and to permit future repairs to the service pipe without resulting in requirements for temporary shoring or underpinning of the proposed buildings. Based on our review of the available conceptual drawings, it is recommended that the footings be lowered, or supported on lean concrete, to the minimum geodetic elevations provided on the attached Figures 1 through 7.

6.9 Slope Stability Assessment

The stability of the slope at the subject site was assessed for existing and proposed conditions at the subject site.

The analysis of slope stability was carried out using SLIDE, a computer program that 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 favouring 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 than 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.

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16 g was considered for the cross-sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

Based on the groundwater observations, the groundwater table will be located at approximate geodetic elevations of 57 to 59 m. However, as a conservative measure, the slope stability analysis assumes the subsoil profile to be fully saturated, with the exception of the vicinity of the proposed buildings where the foundation drainage will lower the groundwater to foundation level. Subsoil conditions at the cross-sections were inferred based on the findings of the test holes which were drilled at the subject site during the current and previous investigations and based on our general knowledge of the area's geology.

Static Conditions

The results of the stability analysis under static loading for the existing and proposed conditions at Sections H, I, and J are presented on Figures 9H, 10H, 9I, 10I, 9J, and 10J respectively enclosed in Appendix 2. The results indicate that the factor of safety for the sections is greater than 1.5 for these sections under static conditions. Therefore, the construction of the proposed buildings will not influence the stability of the slope and a stable slope setback is not required for the subject slope.

Seismic Loading

The results of the analyses for the seismic loading considering both existing and proposed conditions for Section 1 and 2 are shown on Figures 11H, 12H, 11I, 12I, 11J and 12J respectively and enclosed in Appendix 2. The results indicate that the factor of safety is greater than 1.1 under seismic conditions. Based on these results, the slopes are considered to be stable under seismic loading.

Geotechnical Setback - Limit of Hazard Lands

As the slopes have a factor of safety greater than 1.5 for existing and proposed conditions under static loading, and a factor of safety greater than 1.1 for existing and proposed conditions under seismic loading, a stable slope allowance is not required for this slope. Further, as a watercourse is not present at or in the vicinity of this site, toe erosion and erosion access allowances are not required. Therefore, the proposed development at the site is not subject to any geotechnical setback from the top of slope.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that the following recommendations be completed by the geotechnical consultant.

- Review of the structural foundation plan to review proposed footing elevations in relation to the 1200 mm Gloucester Cumberland Trunk Sewer.
- Review of the grading plan, once available.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Observation of the placement of foundation insulation, if applicable.
- 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 herein are in accordance with our present understanding of the project. Paterson requests permission to review our 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 DCR Phoenix Homes, Landric Homes, 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.

Maha Saleh, P.Eng.

Scott Dennis

Scott S. Dennis, P.Eng.

LICENSED PROFESSIONAL ENGINEER
April 28, 2022
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Hillside Development - 3277 St. Joseph Boulevard
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2020

FILE NO. PG5349

HOLE NO. BH 1A-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	74.57		
TOPSOIL 0.15		AU	1						
FILL: Brown silty sand with gravel, cobbles and inferred blast rock 1.35		SS	2	20	11	1	73.57		
End of Borehole Practical refusl to augering at 1.35m depth (BH dry upon completion)									

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Hillside Development - 3277 St. Joseph Boulevard
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2020

FILE NO. PG5349

HOLE NO. BH 1B-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	74.57		
TOPSOIL 0.15	■								
FILL: Brown silty sand with gravel, cobbles and inferred blast rock 1.19	▨					1	73.57		
End of Borehole Practical refusal to augering at 1.19mn depth (BH dry upon completion)									

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Hillside Development - 3277 St. Joseph Boulevard
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2020

FILE NO. PG5349

HOLE NO. BH 2A-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	70.93		
TOPSOIL 0.10									
FILL: Brown silty sand with gravel 0.76		AU	1						
FILL: Brown silty clay with sand, trace gravel, cobble, boulders and inferred blast rock 1.52		SS	2	52	22	1	69.93		
End of Borehole Practical refusal to augering at 1.52m depth (BH dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Hillside Development - 3277 St. Joseph Boulevard
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2020

FILE NO. PG5349

HOLE NO. BH 2B-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	70.93		
TOPSOIL 0.10									
FILL: Brown silty sand with gravel 0.76									
FILL: Brown silty clay with sand, trace gravel, cobbles, boulders and inferred blast rock						1	69.93		
2.19						2	68.93		
End of Borehole Practical refusal to augering at 2.19m depth (BH dry upon completion)									

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Hillside Development - 3277 St. Joseph Boulevard
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 21, 2020

FILE NO. PG5349

HOLE NO. BH 2C-20

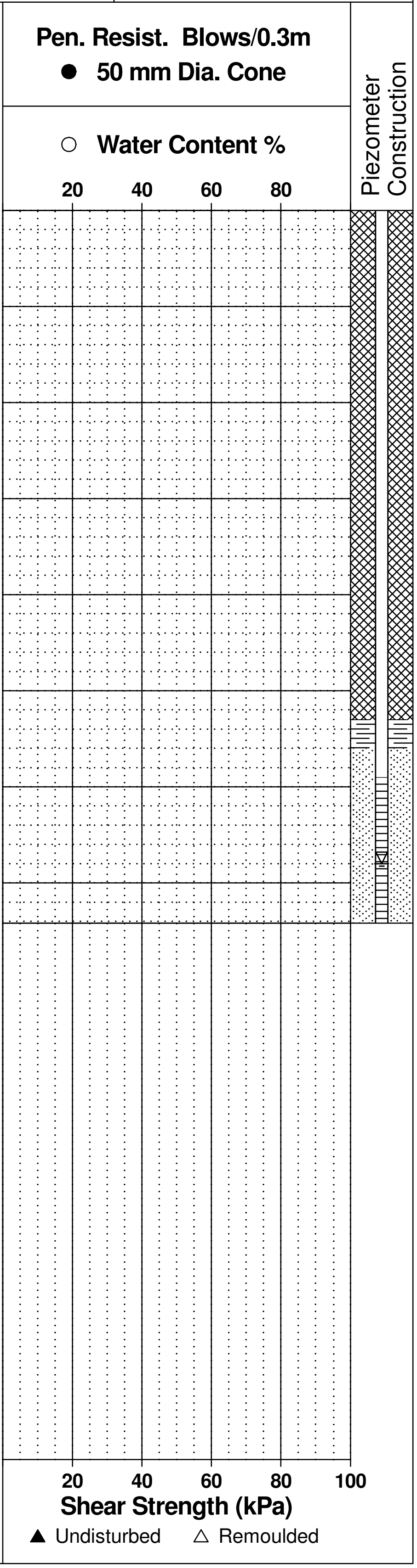
SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.10				
FILL: Brown silty sand with gravel 0.76				
FILL: Brown silty clay with sand, some boulders, trace gravel, cobbles and boulders 1.52				
FILL: Blast rock with crushed stone, some sand and clay	RC	1	100	
	SS	1	75	50+
	RC	2	59	
	SS	2	4	26
	SS	3	0	13
6.86				
FILL: Brown silty clay, some sand, trace gravel, topsoil and organics 7.42	SS	4	58	9

End of Borehole

Practical refusal to augering at 7.42m depth

(GWL @ 6.8m depth based on field observations)

DEPTH (m)	ELEV. (m)
0	70.93
1	69.93
2	68.93
3	67.93
4	66.93
5	65.93
6	64.93
7	63.93



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Hillside Development - 3277 St. Joseph Boulevard
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE May 20, 2020
FILE NO. PG5349
HOLE NO. BH 3A-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	69.36		
TOPSOIL 0.20	■	AU	1						
FILL: Brown silty clay, some sand, trace gravel and organics, occasional cobbles and boulders	▨	SS	2	38	10	1	68.36		
1.78	▨	SS	3	71	50+				
End of Borehole Practical refusal to augering at 1.78m depth. (BH dry upon completion)									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Hillside Development - 3277 St. Joseph Boulevard
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2020

FILE NO. PG5349

HOLE NO. BH 3B-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE									
TOPSOIL 0.20						0	69.36		
FILL: Brown silty clay, some sand, trace gravel and organics, occasional cobbles and boulders						1	68.36		
2.19						2	67.36		
End of Borehole Practical refusal to augering at 2.19m depth. (BH dry upon completion)									

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

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REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2020

FILE NO. PG5349

HOLE NO. BH 3C-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	69.36		
TOPSOIL 0.20									
FILL: Brown silty clay, some sand, trace gravel and organics, occasional cobbles and boulders 1.80						1	68.36		
End of Borehole Practical refusal to augering at 1.80m depth. (BH dry upon completion)									

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

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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 21, 2020

FILE NO. PG5349

HOLE NO. BH 3D-20

SOIL DESCRIPTION	
GROUND SURFACE	
TOPSOIL	0.20
FILL: Brown silty clay, some sand, trace gravel and organics, occasional cobbles and boulders	1.52
FILL: Blast rock with crushed stone, some sand	4.57
FILL: Crushed stone with sand and clay, some blast rock	5.33
FILL: Brown silty clay, trace to some gravel	8.08
Hard, brown **SILTY CLAY**	11.00

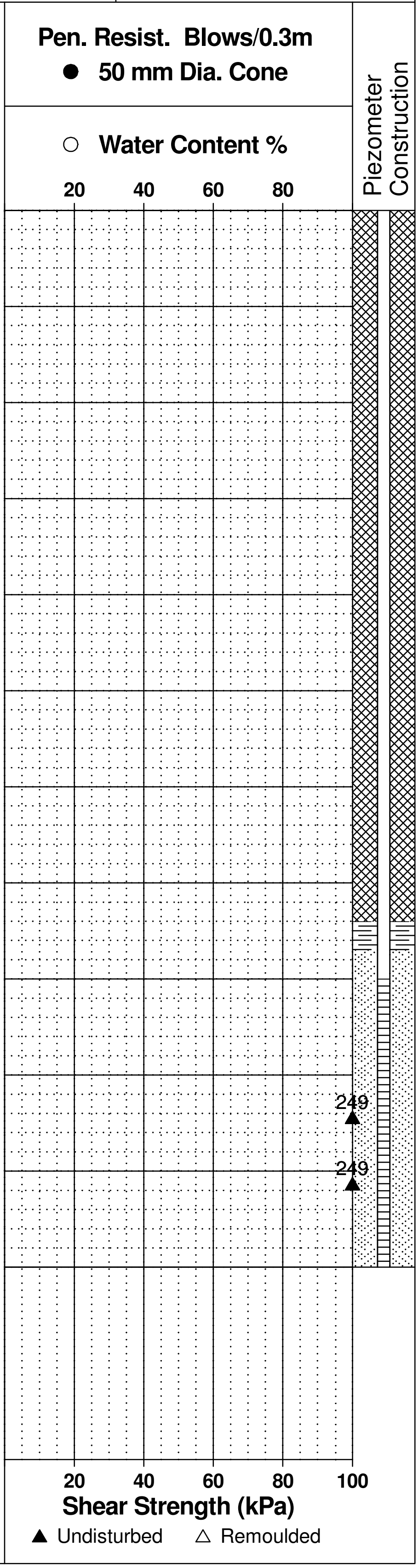
End of Borehole

Practical refusal to augering at 11.00m depth.

(BH dry upon completion)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
RC	1	100	
RC	2	20	
SS	1	42	16
SS	2	54	7
SS	3	21	14
SS	4	62	19
SS	5	46	21
SS	6	100	15
SS	7	100	P
SS	8		50+

DEPTH (m)	ELEV. (m)
0	69.36
1	68.36
2	67.36
3	66.36
4	65.36
5	64.36
6	63.36
7	62.36
8	61.36
9	60.36
10	59.36
11	58.36



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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY Track-Mount Power Auger

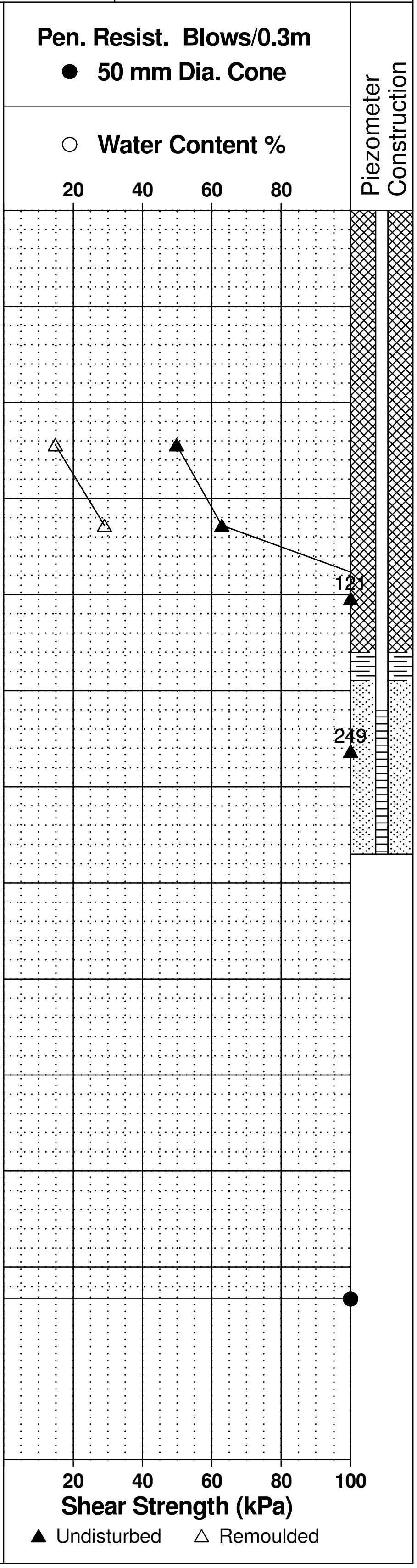
DATE May 20, 2020

FILE NO. PG5349

HOLE NO. BH 4-20

SOIL DESCRIPTION GROUND SURFACE		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
TOPSOIL	0.15	AU	1		
FILL: Brown silty sand with clay, some gravel, crushed stone and topsoil	1.22	SS	2	75	13
FILL: Brown silty clay, trace to some topsoil - trace gravel by 2.7m depth	4.27	SS	3	67	4
Hard, brown **SILTY CLAY**	6.70	SS SS SS	4 5 6	88 100 100	13 P 4
Dynamic Cone Penetration Test commenced at 6.70m depth. Cone pushed to 11.3m depth.	11.33				
End of Borehole Practical DCPT refusal at 11.33m depth. (BH dry upon completion)					

DEPTH (m)	ELEV. (m)
0	64.90
1	63.90
2	62.90
3	61.90
4	60.90
5	59.90
6	58.90
7	57.90
8	56.90
9	55.90
10	54.90
11	53.90



SOIL PROFILE AND TEST DATA

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Prop. Hillside Development - 3277 St. Joseph Boulevard
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REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2020

FILE NO. PG5349

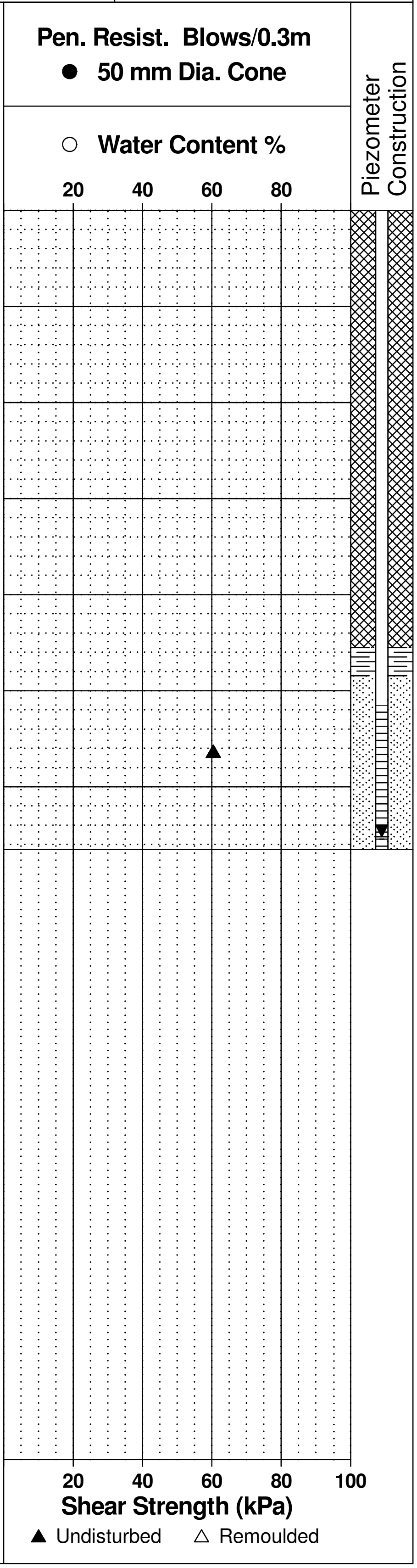
HOLE NO. BH 5-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	65.12
TOPSOIL 0.13		AU	1				
FILL: Brown silty clay, some topsoil, trace sand and gravel		SS	2	25	5	1	64.12
1.88		SS	3	79	10	2	63.12
FILL: Topsoil with sand 2.13							
FILL: Dark grey to brown silty clay, trace gravel and topsoil		SS	4	54	10		
		SS	5	42	5	3	62.12
		SS	6	33	3	4	61.12
		SS	7	25	W	5	60.12
5.79		SS	8	21	P		
GLACIAL TILL: Dense, brown sandy silt to silty fine sand with gravel, cobbles and boulders 6.65		SS	9	81	43	6	59.12

End of Borehole

Practical refusal to augering at 6.65m depth.

(GWL @ 6.52m - May 29, 2020)



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SOIL PROFILE AND TEST DATA

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Prop. Hillside Development - 3277 St. Joseph Boulevard
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REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 22, 2020

FILE NO. PG5349

HOLE NO. BH 6-20

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.30
FILL: Brown silty clay, some organics, trace clay	3.20
Hard to very stiff, brown **SILTY CLAY** - grey with trace sand and gravel by 6.9m depth	7.47
GLACIAL TILL: Grey clayey silt with sand, gravel, cobbles and boulders	8.74

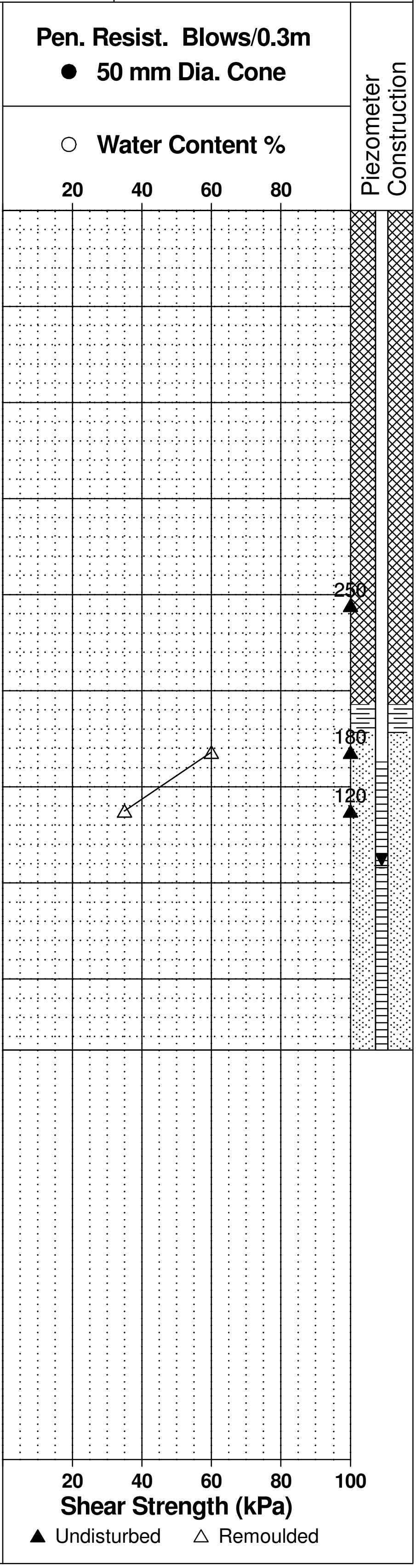
End of Borehole

Practical refusal to augering at 8.74m depth.

(GWL @ 6.82m - May 29, 2020)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	92	10
SS	3	33	12
SS	4	25	14
SS	5	92	16
SS	6	100	P
SS	7	100	8
SS	8	58	104
SS	9	75	50+
SS	10	33	50+

DEPTH (m)	ELEV. (m)
0	64.58
1	63.58
2	62.58
3	61.58
4	60.58
5	59.58
6	58.58
7	57.58
8	56.58



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Hillside Development
3277 St. Joseph Boulevard - Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 February 24

FILE NO. PG5625

HOLE NO. BH 1-21

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with crushed stone, gravel and clay — 0.61	AU	1		
FILL: Brown silty clay, with sand and gravel — 2.21	SS	2	33	5
	SS	3	33	6
FILL: Brown silty clay trace sand and gravel	SS	4	58	8
	SS	5	58	7
	SS	6	58	5
	SS	7	67	5
- Some topsoil and organics by 6.5 m depth	SS	8	50	5
— 6.71	SS	9	58	6
FILL: Brown silty clay with topsoil, sand, gravel, trace organics - Topsoil content decreasing with depth	SS	10	25	10
	SS	11	33	12
— 8.61	SS	12	0	+50
BEDROCK: Excellent quality Grey Limestone	RC	1	100	93
— 10.44	RC	2	100	100
End of Borehole (GWL @ 7.78 m depth - March 8, 2021)				

DEPTH (m)	ELEV. (m)
0	72.40
1	71.40
2	70.40
3	69.40
4	68.40
5	67.40
6	66.40
7	65.40
8	64.40
9	63.40
10	62.40

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

○ Water Content %

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Piezometer Construction

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REMARKS

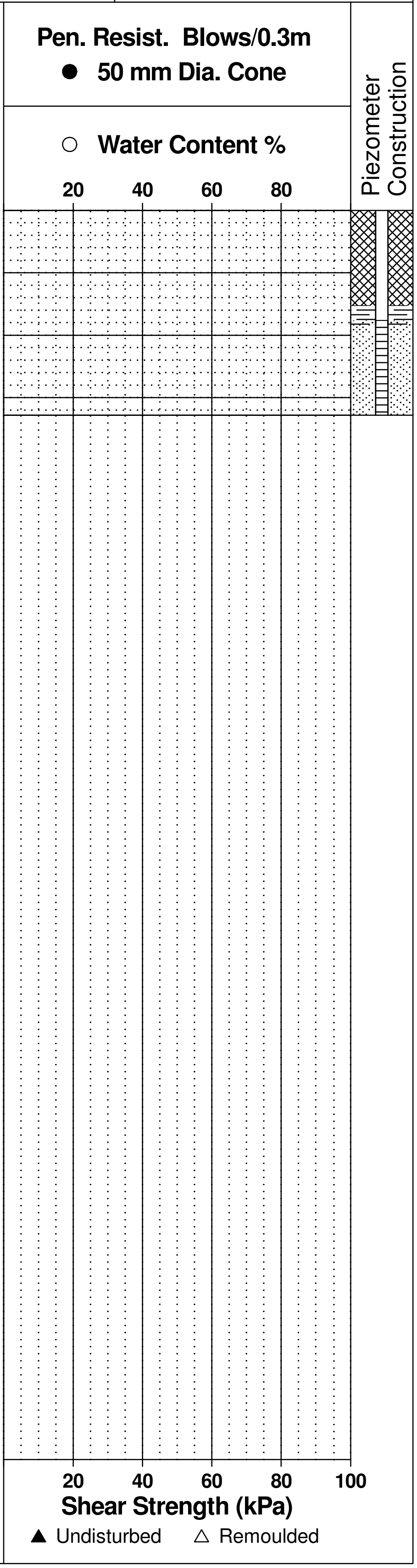
BORINGS BY CME 55 Power Auger

DATE 2021 February 25

FILE NO. PG5625

HOLE NO. BH 2-21

SOIL DESCRIPTION GROUND SURFACE	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
TOPSOIL 0.15	AU	1			0	74.66
FILL: Brown silty clay with sand and gravel - Clay content decreasing with depth 1.47	SS	2	17	+50	1	73.66
BEDROCK: Fair to excellent quality Grey Limestone	RC	1	100	62	2	72.66
3.28	RC	2	100	100	3	71.66
End of Borehole (Piezometer dry/blocked - March 8, 2021)						



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SOIL PROFILE AND TEST DATA

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REMARKS

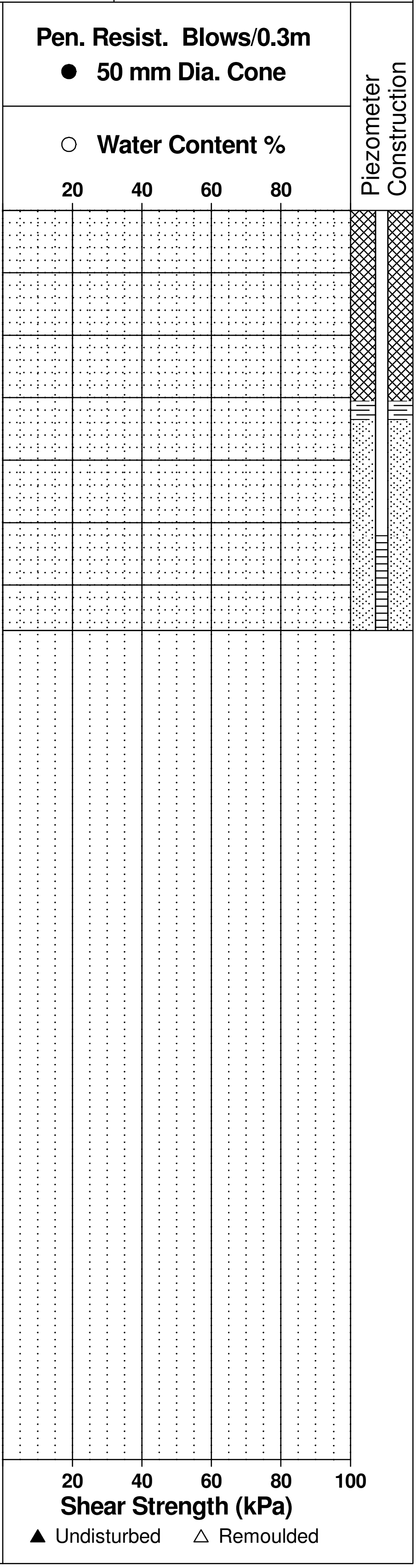
BORINGS BY CME 55 Power Auger

DATE 2021 February 25

FILE NO. PG5625

HOLE NO. BH 3-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	73.50
TOPSOIL with brown silty sand and gravel 0.15	AU	1				
FILL: Brown silty sand with crushed stone, trace clay	SS	2	8	12	1	72.50
2.21					2	71.50
FILL: Granular crushed stone with silty clay some blast rock 2.92	RC	1	100	63		
					3	70.50
BEDROCK: Fair to excellent quality Grey Limestone	RC	2	100	51		
					4	69.50
	RC	3	100	73	5	68.50
					6	67.50
6.73	RC	4	100	86		
End of Borehole (Piezometer dry/blocked - March 8, 2021)						



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SOIL PROFILE AND TEST DATA

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FILE NO. PG5625

REMARKS

HOLE NO. BH 4-21

BORINGS BY CME 55 Power Auger

DATE 2021 February 24

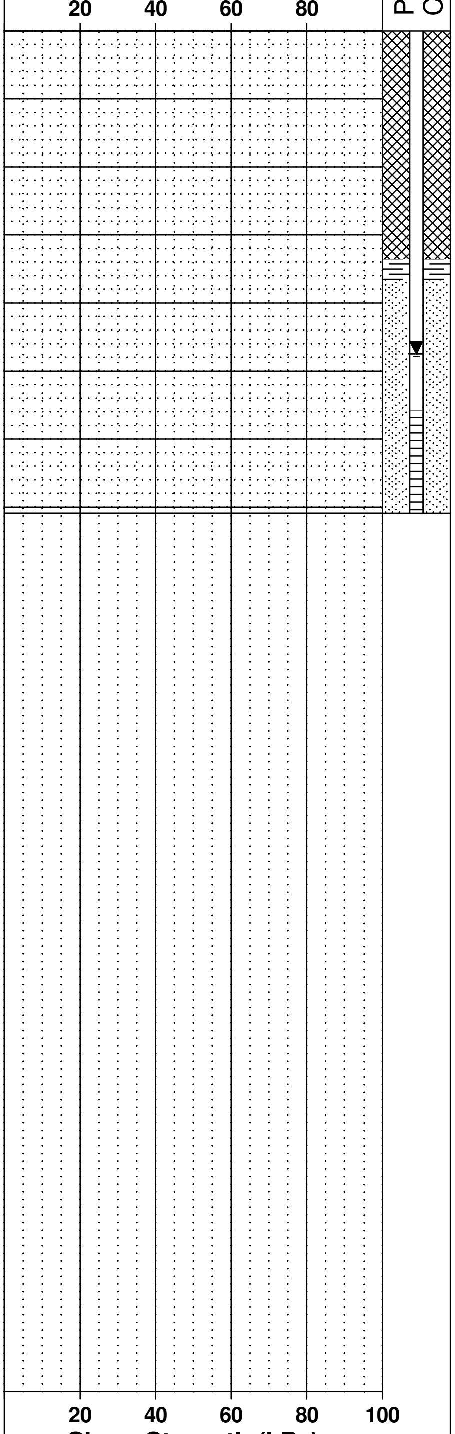
SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with gravel, topsoil, organics, some clay	0.61	AU	1		
FILL: Brown silty clay with sand, gravel, crushed stone, trace topsoil		SS	2	42	11
- Crushed stone and rock fragments increasing at 1.8 m depth		SS	3	58	24
	2.97	SS	4	25	38
FILL: Brown silty sand with gravel, cobbles trace clay - Some blast rock rock by 3.5 m depth		SS	5	33	0
		SS	6	0	0
	4.90	SS	7	0	0
BEDROCK: Fair to good quality Grey Limestone		RC	1	100	65
	7.09	RC	2	100	82
End of Borehole (GWL @ 4.75 m depth - March 8, 2021)					

DEPTH (m)	ELEV. (m)
0	71.21
1	70.21
2	69.21
3	68.21
4	67.21
5	66.21
6	65.21
7	64.21

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Hillside Development
3277 St. Joseph Boulevard - Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 February 26

FILE NO. PG5625

HOLE NO. BH 5-21

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
TOPSOIL	0.10
FILL: Brown silty sand with gravel	0.76
	1.52
FILL: Brown silty clay with sand trace gravel, cobbles, boulders, some blast rock	
FILL: Blast rock with crushed stone, some sand and clay	6.86
FILL: Brown silty clay, some sand, trace gravel, topsoil and organics	7.29
BEDROCK: Good to excellent quality Grey Limestone	9.53
End of Borehole	
(GWL @ 5.55 m depth - March 8, 2021)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	12
SS	3	47	+50
RC	1	22	
SS	4	42	38
RC	2	15	
SS	5	4	21
RC	3	100	79
RC	4	100	80

DEPTH (m)	ELEV. (m)
0	70.94
1	69.94
2	68.94
3	67.94
4	66.94
5	65.94
6	64.94
7	63.94
8	62.94
9	61.94

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

○ **Water Content %**

20 40 60 80

Piezometer Construction

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

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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 February 26

FILE NO. PG5625

HOLE NO. BH 6-21

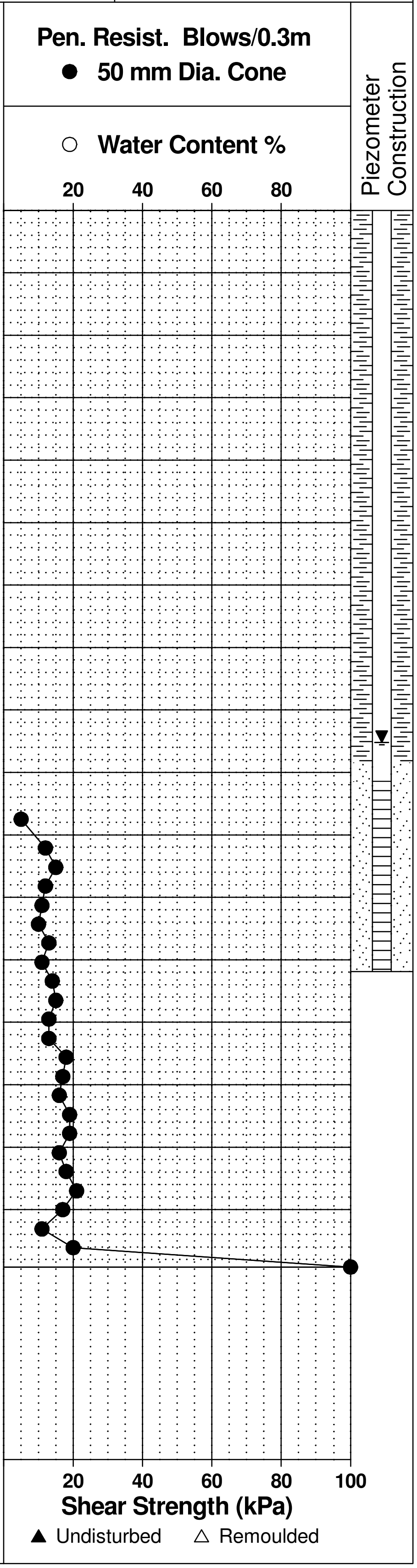
SOIL DESCRIPTION	Depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.15	AU	1		
FILL: Brown silty clay with sand, gravel, some topsoil, trace asphalt and cobbles		SS	2	67	8
		SS	3	33	12
		SS	4	50	9
		SS	5	42	22
	4.57	SS	6	42	16
FILL: Brown silty clay trace sand, gravel, and organics		SS	7	75	5
		SS	8	50	17
		SS	9	100	19
		SS	10	100	21
	8.69	SS	11	75	37
Very stiff brown **SILTY CLAY**		SS	12	100	19
	9.75	SS	13	100	6
Dynamic Cone Penetration Test commenced at 9.75 m depth.	16.92				

End of Borehole

Practical refusal to DCPT at 16.92 m depth

(GWL @ 8.52 m depth - March 8, 2021)

DEPTH (m)	ELEV. (m)
0	68.32
1	67.32
2	66.32
3	65.32
4	64.32
5	63.32
6	62.32
7	61.32
8	60.32
9	59.32
10	58.32
11	57.32
12	56.32
13	55.32
14	54.32
15	53.32
16	52.32



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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 2021 March 3

FILE NO. PG5625

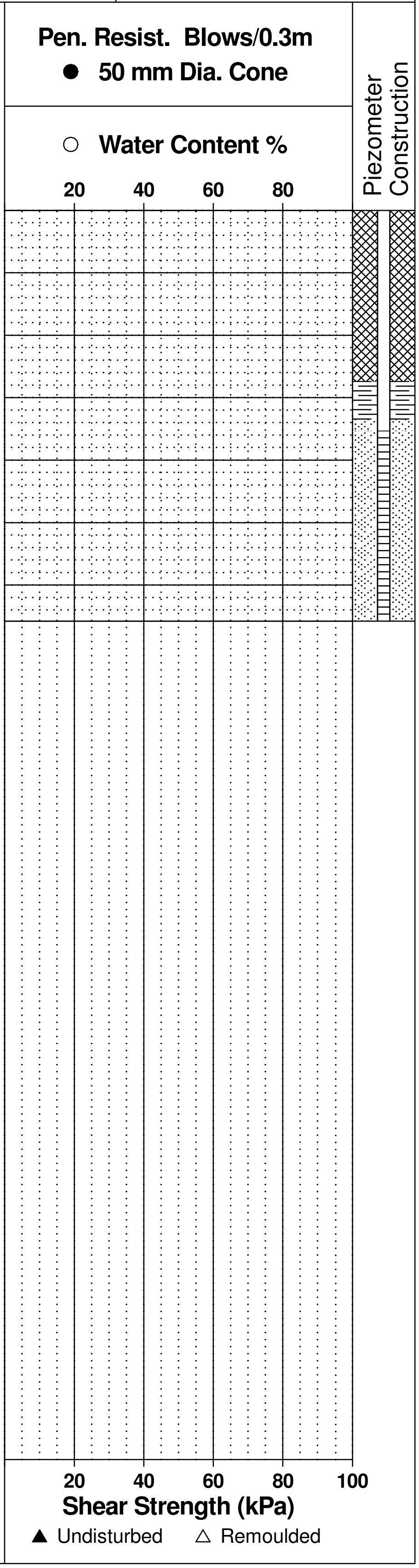
HOLE NO. BH 7-21

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	64.86
TOPSOIL 0.10	AU	1				
FILL: Brown silty clay trace sand, gravel, organics, cobbles and topsoil	SS	2	67	7	1	63.86
	SS	3	42	4	2	62.86
	SS	4	13	6	3	61.86
3.73	SS	5	71	0		
Hard to very stiff brown **SILTY CLAY**	SS	6	92	13	4	60.86
5.64	SS	7	79	15	5	59.86
GLACIAL TILL: Brown sandy silt to silty sand with clay, gravel, cobbles and boulders 6.58	SS	8	42	+50	6	58.86

End of Borehole

Practical refusal to augering at 6.58 m depth

(Piezometer destroyed - March 8, 2021)



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FILE NO. PG5625

REMARKS

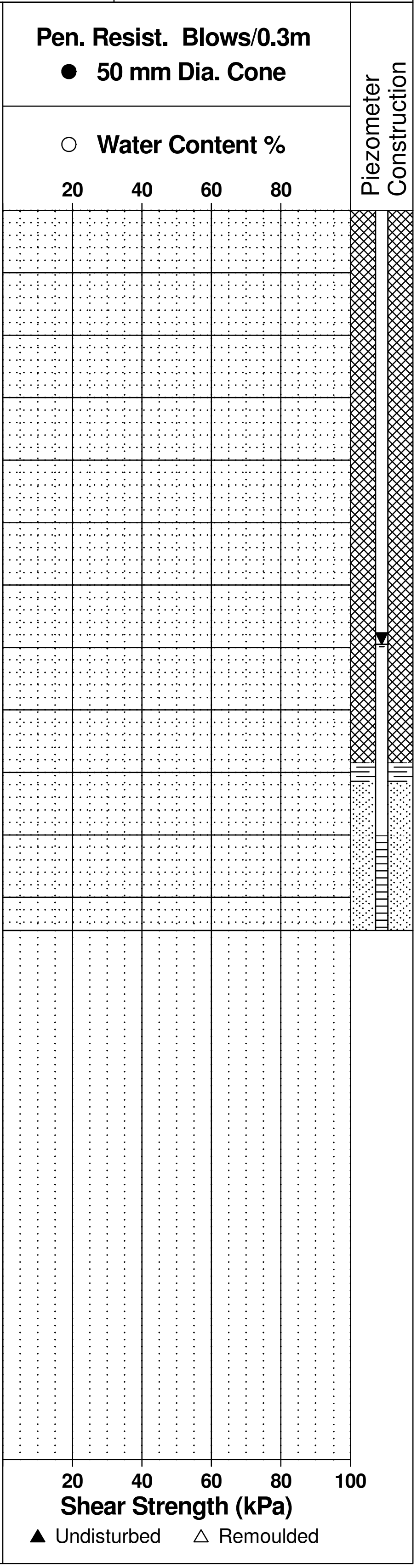
HOLE NO. BH 8-21

BORINGS BY CME 55 Power Auger

DATE 2021 March 3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.15	AU	1		
FILL: Brown silty clay trace sand, gravel and organics	SS	2	42	6
	SS	3	33	4
	SS	4	67	8
3.45	SS	5	92	11
Hard to very stiff brown **SILTY CLAY**	SS	6	100	12
	SS	7	100	10
	SS	8	100	9
- Grey by 6.7 m depth	SS	9	100	3
7.01				
GLACIAL TILL: Grey clayey silt with sand, gravel, cobbles and boulders	SS	10	25	20
	SS	11	54	50
	SS	12	46	+50
9.19				
BEDROCK: Poor to fair quality Grey Limestone	RC	1	79	30
	RC	2	100	41
11.53				
End of Borehole				
(GWL @ 6.95 m depth - March 8, 2021)				

DEPTH (m)	ELEV. (m)
0	65.52
1	64.52
2	63.52
3	62.52
4	61.52
5	60.52
6	59.52
7	58.52
8	57.52
9	56.52
10	55.52
11	54.52



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))

TW - Thin wall tube or Shelby tube

PS - Piston sample

AU - Auger sample or bulk sample

WS - Wash sample

RC - Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

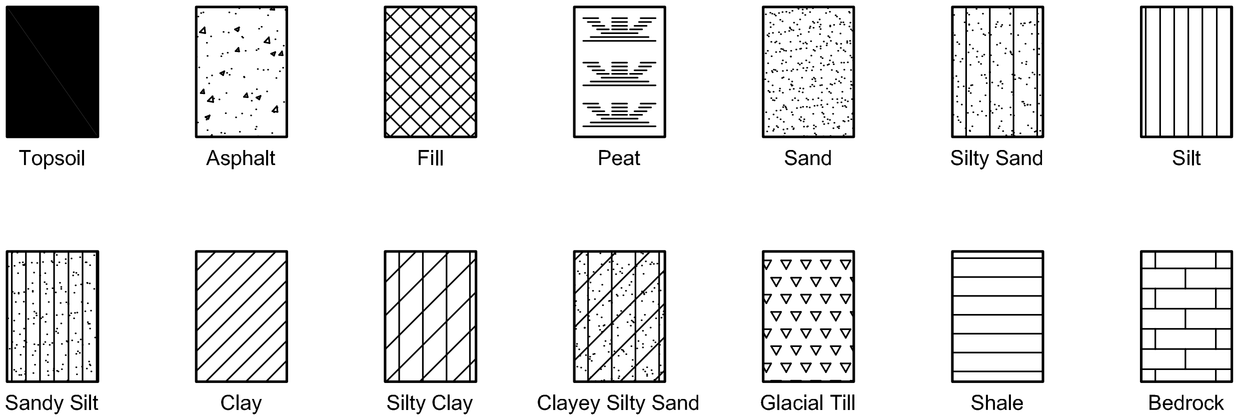
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

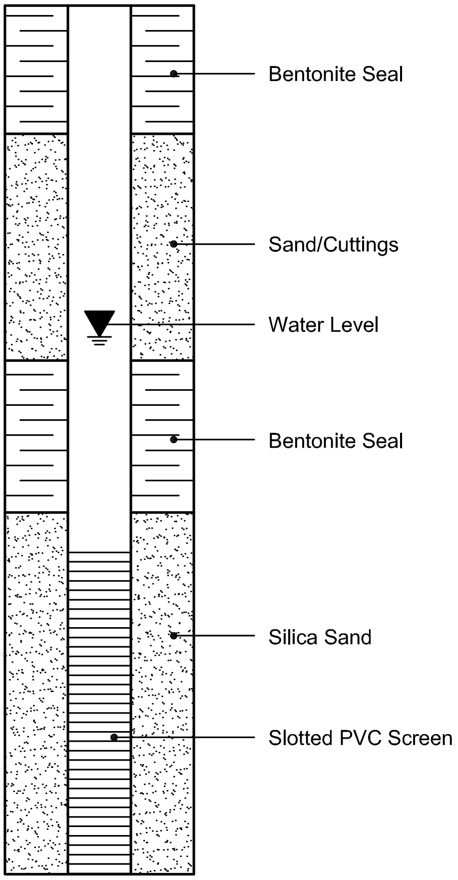
SYMBOLS AND TERMS (continued)

STRATA PLOT

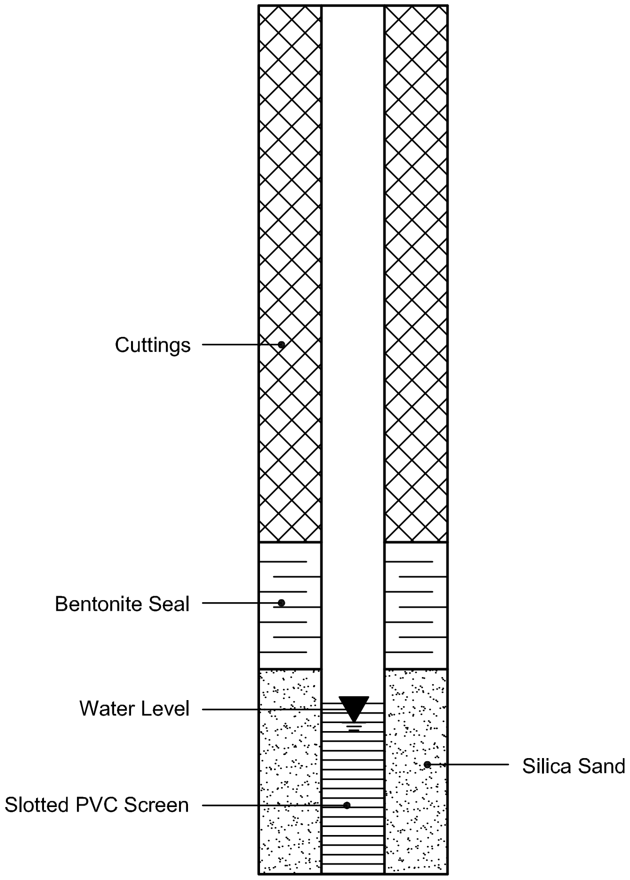


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 30149

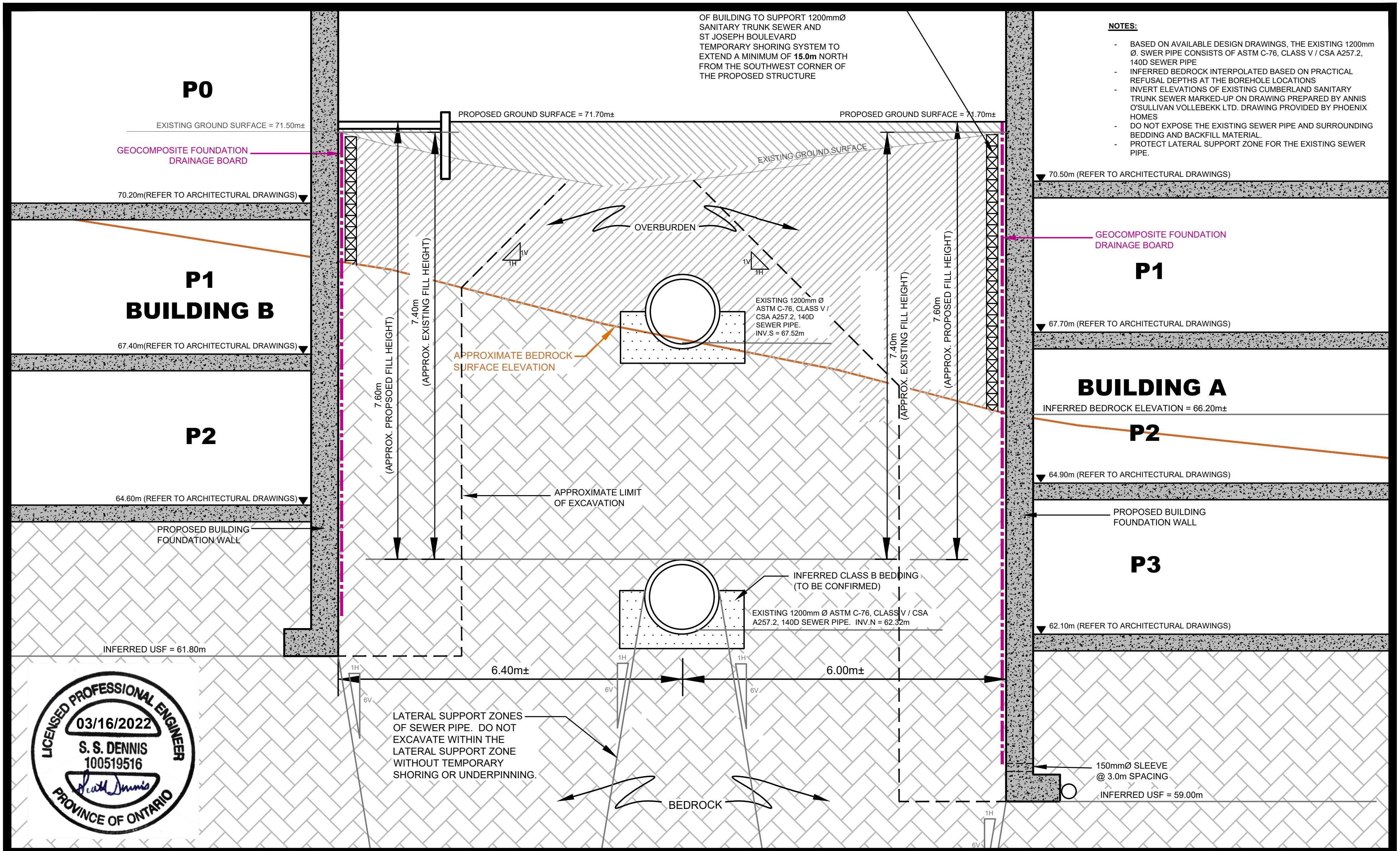
Report Date: 27-May-2020

Order Date: 21-May-2020

Project Description: PG5349

Client ID:		BH5-SS4	-	-	-
Sample Date:		20-May-20 13:00	-	-	-
Sample ID:		2021245-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	79.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.30	-	-	-
Resistivity	0.10 Ohm.m	20.2	-	-	-
Anions					
Chloride	5 ug/g dry	221	-	-	-
Sulphate	5 ug/g dry	78	-	-	-

APPENDIX 2



NOTES:

- BASED ON AVAILABLE DESIGN DRAWINGS, THE EXISTING 1200mm Ø. SWER PIPE CONSISTS OF ASTM C-76, CLASS V / CSA A257.2, 140D SEWER PIPE
- INFERRED BEDROCK INTERPOLATED BASED ON PRACTICAL REFUSAL DEPTHS AT THE BOREHOLE LOCATIONS
- INVERT ELEVATIONS OF EXISTING CUMBERLAND SANITARY TRUNK SEWER MARKED-UP ON DRAWING PREPARED BY ANNIS O'SULLIVAN VOLLEBEKK LTD. DRAWING PROVIDED BY PHOENIX HOMES
- DO NOT EXPOSE THE EXISTING SEWER PIPE AND SURROUNDING BEDDING AND BACKFILL MATERIAL.
- PROTECT LATERAL SUPPORT ZONE FOR THE EXISTING SEWER PIPE.

LICENSED PROFESSIONAL ENGINEER
03/16/2022
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DCR PHOENIX C/O LANDRIC HOMES

GEOTECHNICAL INVESTIGATION - HILLSIDE DEV.
PROPOSED MULTI-STOREY BUILDING - 3277 ST.JOSEPH BOULVARD

OTTAWA, ONTARIO

Title:

CROSS SECTION A-A

Scale: 1:100
Date: 03/2021
Drawn by: RCG
Report No.: PG5625
Checked by: MS
Drawing No.: FIG. 1
Approved by: SD
Revision No.:

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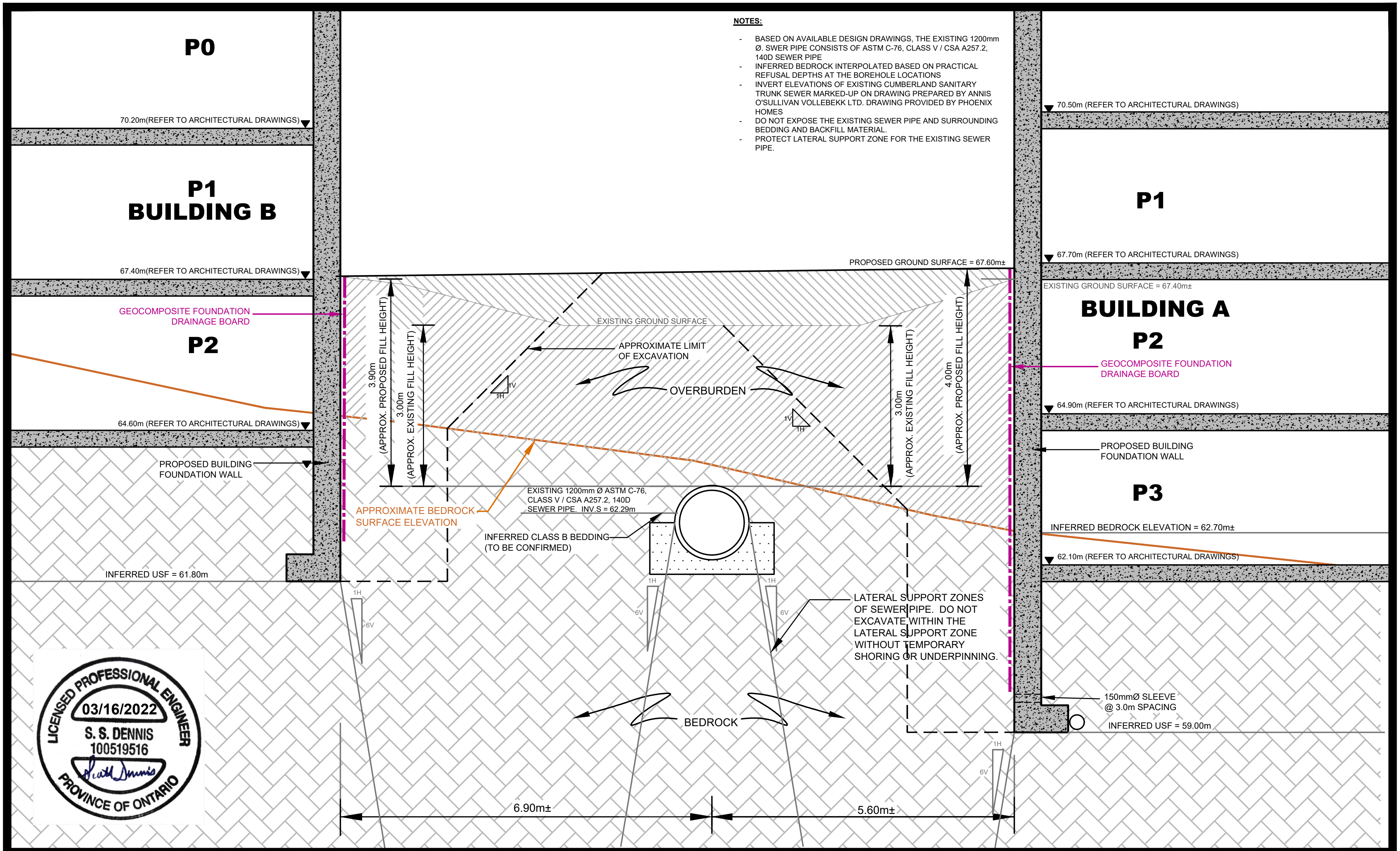

NOTES:

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PROPOSED MULTI-STOREY BUILDING - 3277 ST.JOSEPH BOULVARD

OTTAWA, ONTARIO

Title:

CROSS SECTION B-B

Scale: 1:100	Date: 03/2021
Drawn by: RCG	Report No.: PG5625
Checked by: RG	Drawing No.: FIG. 2
Approved by: SD	Revision No.:

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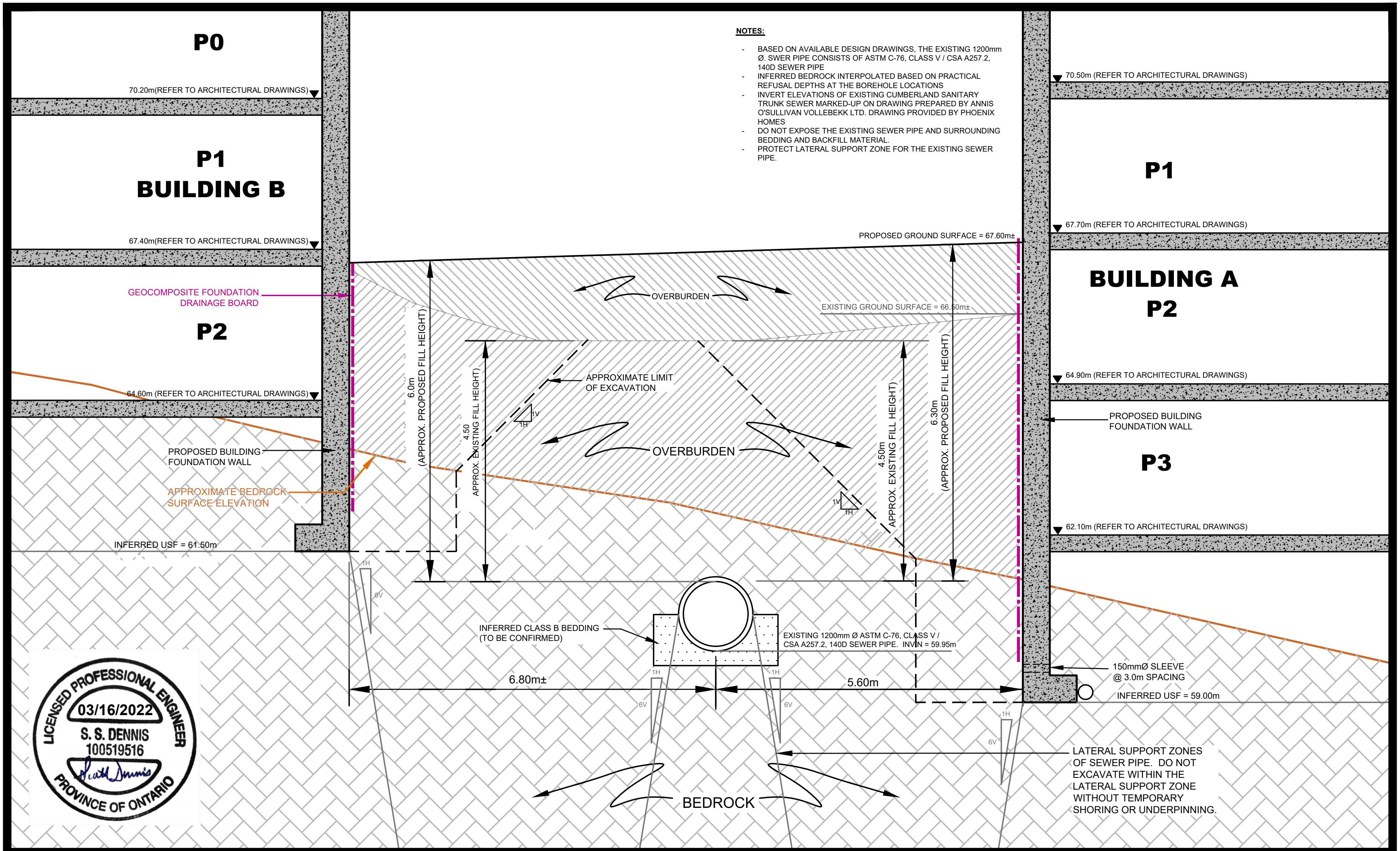

NOTES:

- BASED ON AVAILABLE DESIGN DRAWINGS, THE EXISTING 1200mm Ø. SWER PIPE CONSISTS OF ASTM C-76, CLASS V / CSA A257.2, 140D SEWER PIPE
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PROPOSED MULTI-STOREY BUILDING - 3277 ST.JOSEPH BOULVARD
OTTAWA, ONTARIO

Title:

CROSS SECTION C-C

Scale: 1:100
Date: 03/2021
Drawn by: RCG
Report No.: PG5625
Checked by: RG
Drawing No.: FIG. 3
Approved by: SD
Revision No.:

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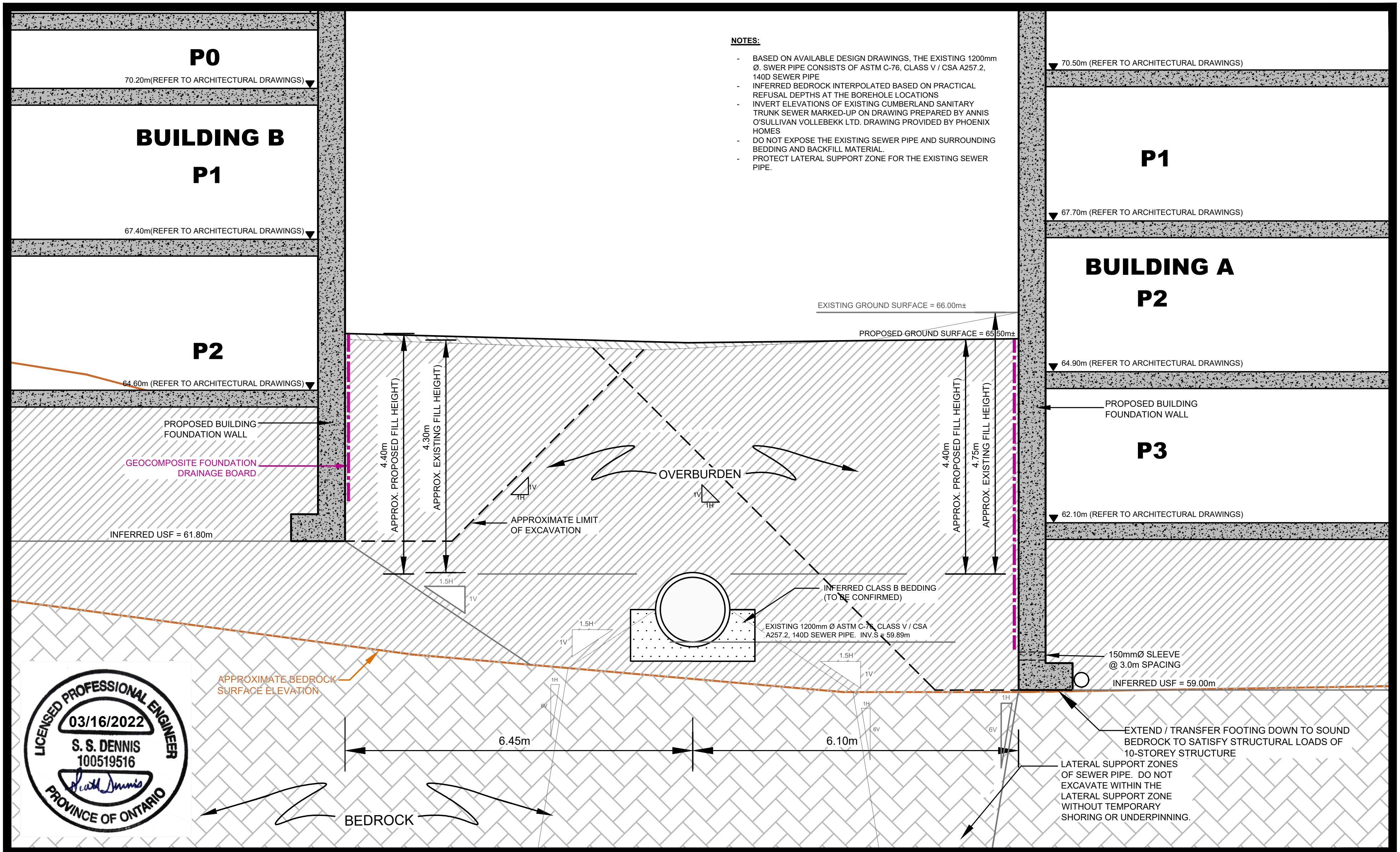

NOTES:

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OTTAWA, ONTARIO

Title:

CROSS SECTION D-D

Scale: 1:100
Date: 03/2021
Drawn by: RCG
Report No.: PG5625
Checked by: RG
Drawing No.: FIG. 4
Approved by: SD
Revision No.:

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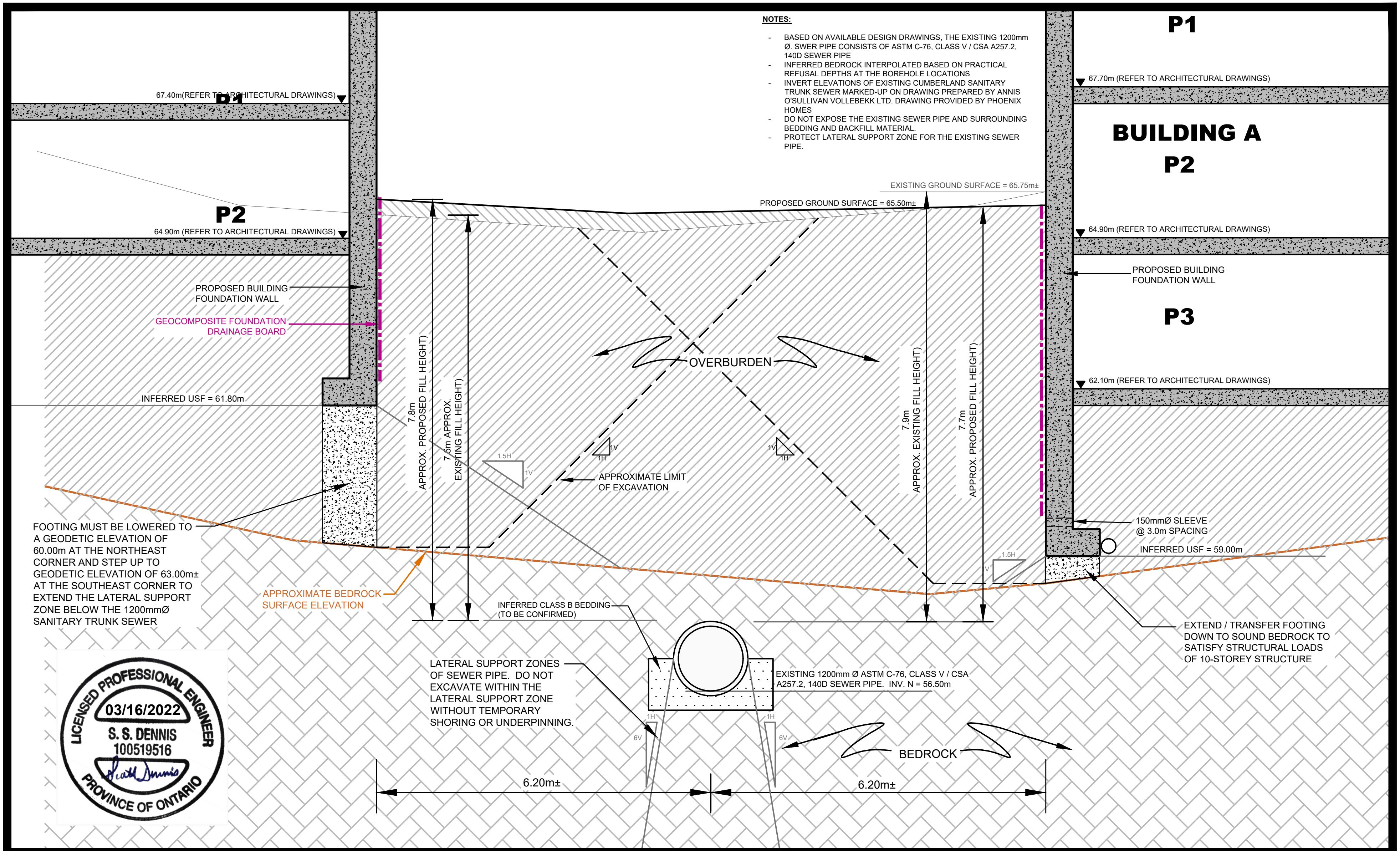

NOTES:

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PROPOSED MULTI-STOREY BUILDING - 3277 ST.JOSEPH BOULVARD
OTTAWA, ONTARIO

Title:

CROSS SECTION E-E

Scale: 1:100	Date: 03/2022
Drawn by: RCG	Report No.: PG5625
Checked by: RG	Drawing No.: FIG. 5
Approved by: SD	Revision No.:

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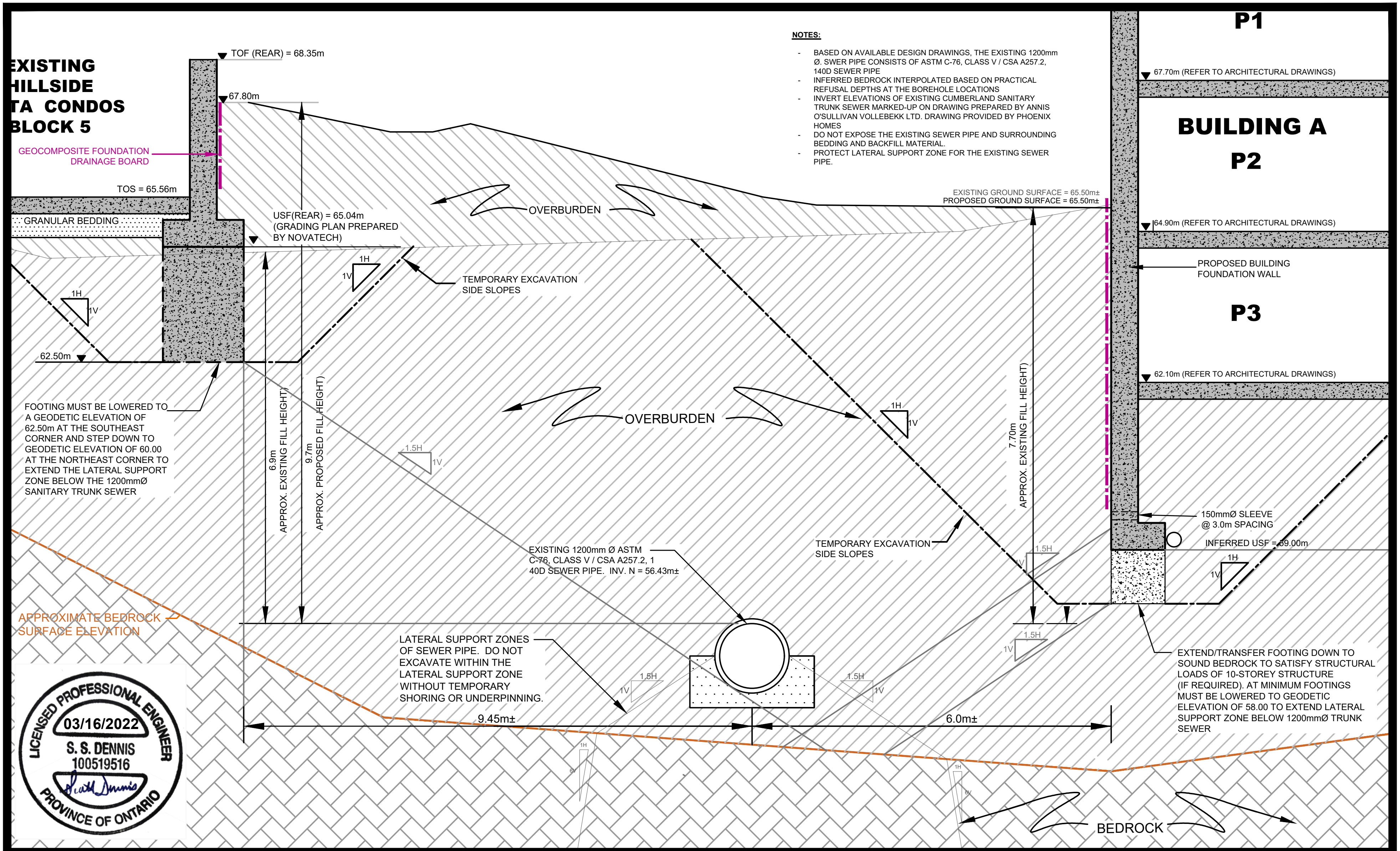

FOOTING MUST BE LOWERED TO A GEODETIC ELEVATION OF 62.50m AT THE SOUTHEAST CORNER AND STEP DOWN TO GEODETIC ELEVATION OF 60.00 AT THE NORTHEAST CORNER TO EXTEND THE LATERAL SUPPORT ZONE BELOW THE 1200mmØ SANITARY TRUNK SEWER

EXISTING 1200mm Ø ASTM C-76, CLASS V / CSA A257.2, 140D SEWER PIPE. INV. N = 56.43m±

LATERAL SUPPORT ZONES OF SEWER PIPE. DO NOT EXCAVATE WITHIN THE LATERAL SUPPORT ZONE WITHOUT TEMPORARY SHORING OR UNDERPINNING.

EXTEND/TRANSFER FOOTING DOWN TO SOUND BEDROCK TO SATISFY STRUCTURAL LOADS OF 10-STOREY STRUCTURE (IF REQUIRED). AT MINIMUM FOOTINGS MUST BE LOWERED TO GEODETIC ELEVATION OF 58.00 TO EXTEND LATERAL SUPPORT ZONE BELOW 1200mmØ TRUNK SEWER

NOTES:

- BASED ON AVAILABLE DESIGN DRAWINGS, THE EXISTING 1200mm Ø. SWER PIPE CONSISTS OF ASTM C-76, CLASS V / CSA A257.2, 140D SEWER PIPE
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GEOTECHNICAL INVESTIGATION - HILLSIDE DEV.
PROPOSED MULTI-STOREY BUILDING - 3277 ST.JOSEPH BOULVARD
OTTAWA, ONTARIO

Title:

CROSS SECTION F-F

Scale: 1:100	Date: 03/2021
Drawn by: RCG	Report No.: PG5625
Checked by: RG	Drawing No.: FIG. 6
Approved by: CDS	Revision No.:

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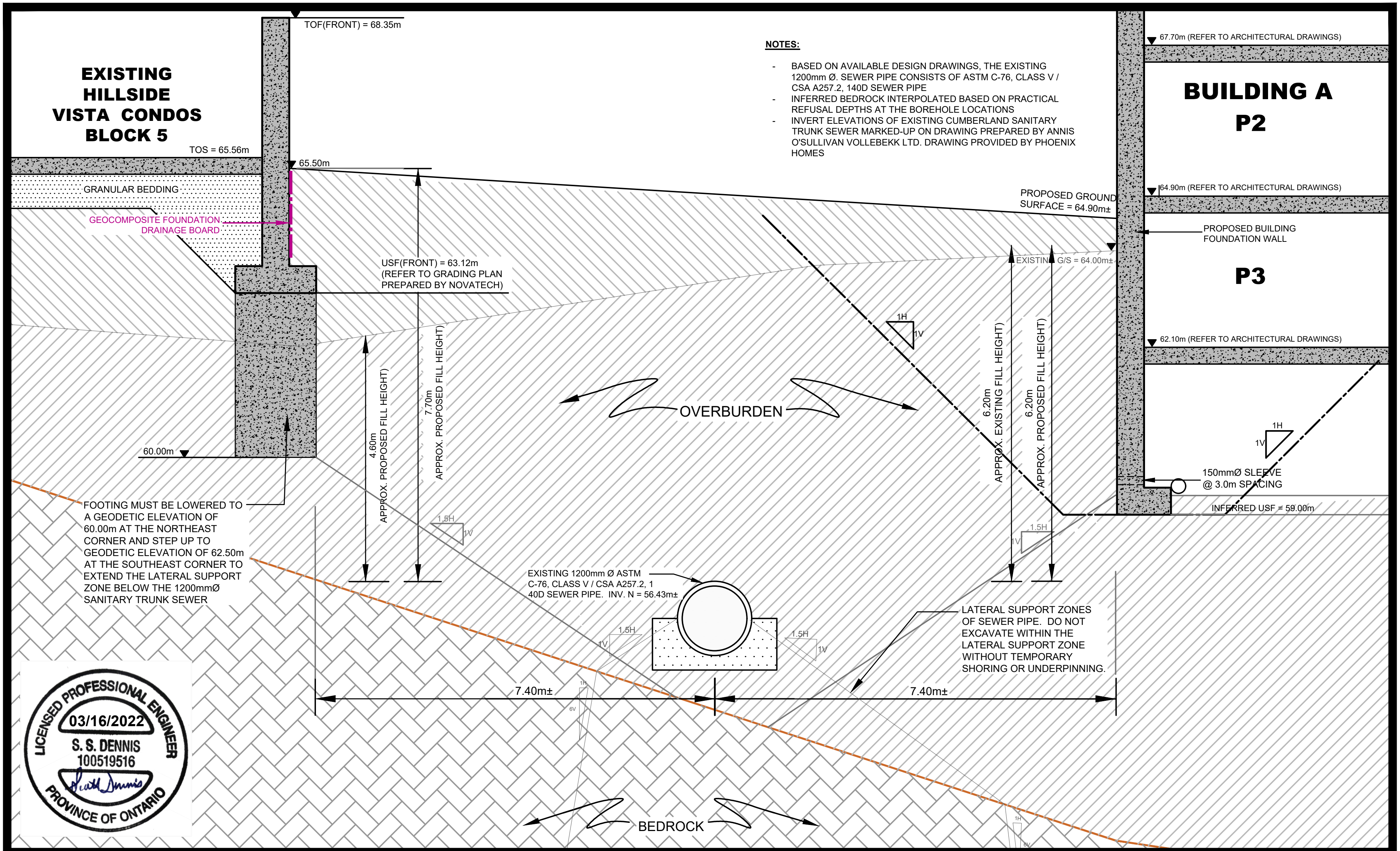

NOTES:

- BASED ON AVAILABLE DESIGN DRAWINGS, THE EXISTING 1200mm Ø. SEWER PIPE CONSISTS OF ASTM C-76, CLASS V / CSA A257.2, 140D SEWER PIPE
- INFERRED BEDROCK INTERPOLATED BASED ON PRACTICAL REFUSAL DEPTHS AT THE BOREHOLE LOCATIONS
- INVERT ELEVATIONS OF EXISTING CUMBERLAND SANITARY TRUNK SEWER MARKED-UP ON DRAWING PREPARED BY ANNIS O'SULLIVAN VOLLEBEKK LTD. DRAWING PROVIDED BY PHOENIX HOMES

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GEOTECHNICAL INVESTIGATION - HILLSIDE DEV.
PROPOSED MULTI-STOREY BUILDING - 3277 ST.JOSEPH BOULVARD
OTTAWA, ONTARIO

Title:

CROSS SECTION G-G

Scale: 1:100
Date: 03/2021
Drawn by: RCG
Report No.: PG5625
Checked by: RG
Drawing No.: FIG. 7
Approved by: SD
Revision No.:

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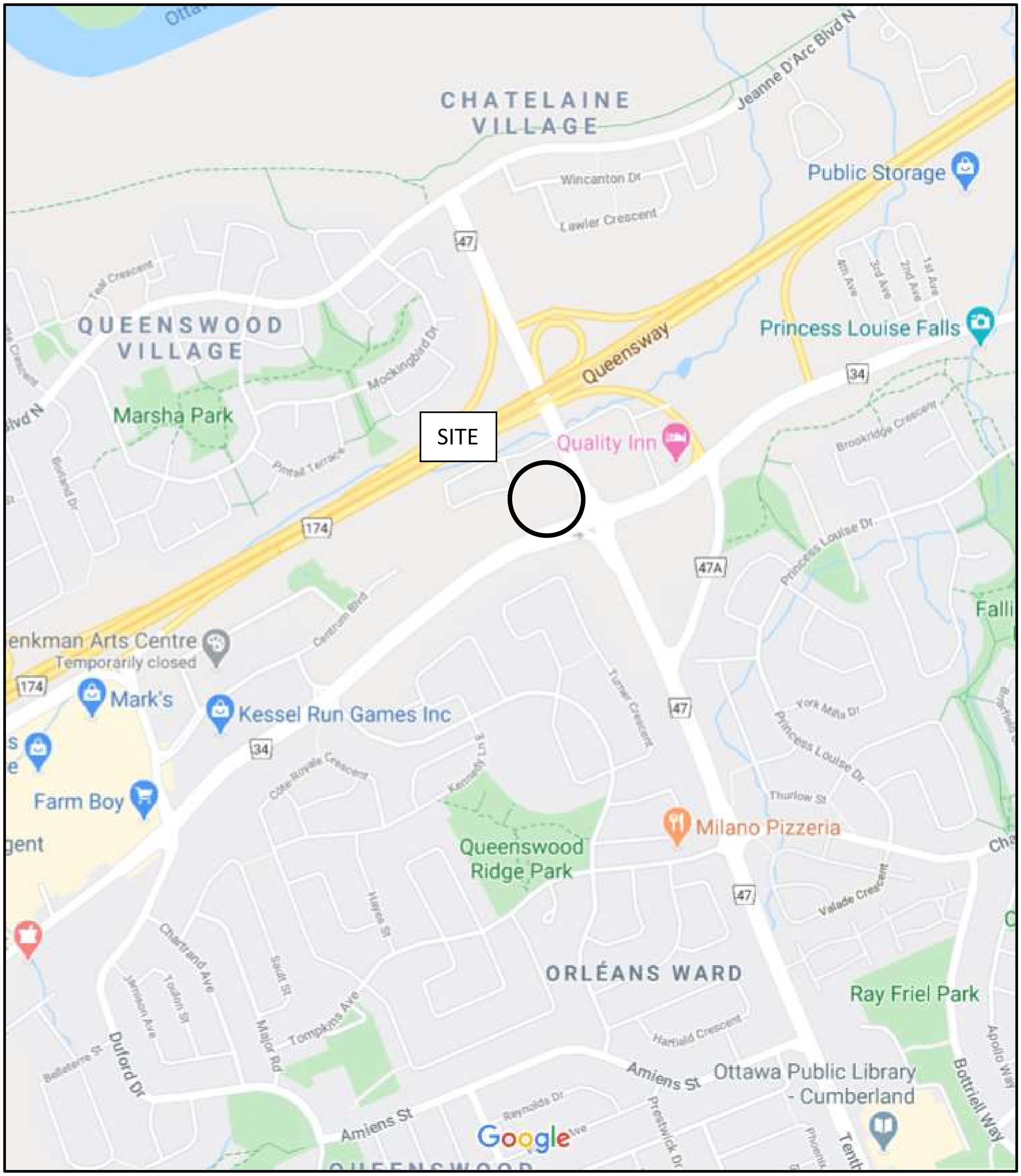
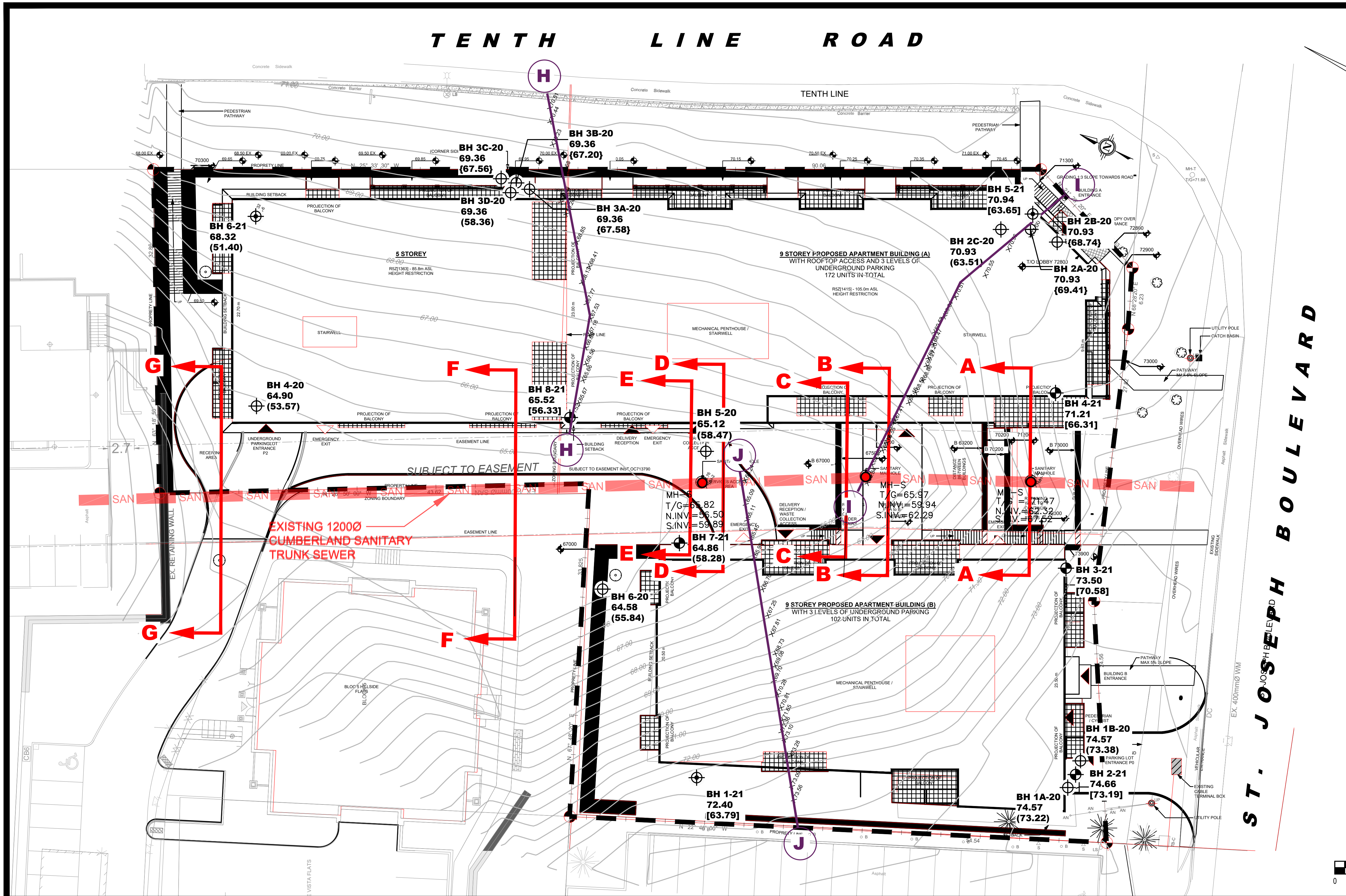


FIGURE 8

KEY PLAN



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
⌖	BOREHOLE LOCATION, PREVIOUS INVESTIGATION (PG5349, 2020)
74.57	GROUND SURFACE ELEVATION (m)
(73.22)	PRACTICAL DCPT / AUGERING REFUSAL ELEVATION (m)
{72.33}	PRACTICAL AUGER REFUSAL ON INFERRED BOULDER ELEVATION (m)
[73.19]	PRACTICAL AUGER REFUSAL ON CONFIRMED BEDROCK ELEVATION (m)
↑_↑	BUILDING CROSS SECTION
Ⓙ—Ⓙ	SLOPE STABILITY CROSS SECTION

ELEVATIONS AT TEST HOLES ARE REFERENCED TO A GEODETIC DATUM

BASEPLAN PROVIDED BY ROSSMAN ARCHITECTURE

INVERT ELEVATIONS OF THE EXISTING CUMBERLAND SANITARY TRUNK SEWER PROVIDED BY PHOENIX HOMES

0 5 10 15 20 25m

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NO.	REVISIONS	DATE	INITIAL
3	CONCEPTUAL PLAN UPDATED	31/03/2022	RG
2	CONCEPTUAL PLAN UPDATED	23/02/2022	RG
1	2021 BOREHOLES ADDED, CONCEPTUAL PLAN UPDATED, SLOPE STABILITY INFORMATION ADDED	08/03/2021	RG

DCR PHOENIX C/O LANDRIC HOMES

GEOTECHNICAL INVESTIGATION - HILLSIDE DEVELOPMENT
PROPOSED MULTI-STOREY BUILDING - 3277 ST JOSEPH BLVD.

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400
Date: 03/2021
Drawn by: RCG
Report No.: PG5625-1
Checked by: RG
Dwg. No.: PG5625-1
Approved by: SD
Revision No.: 1

11x17

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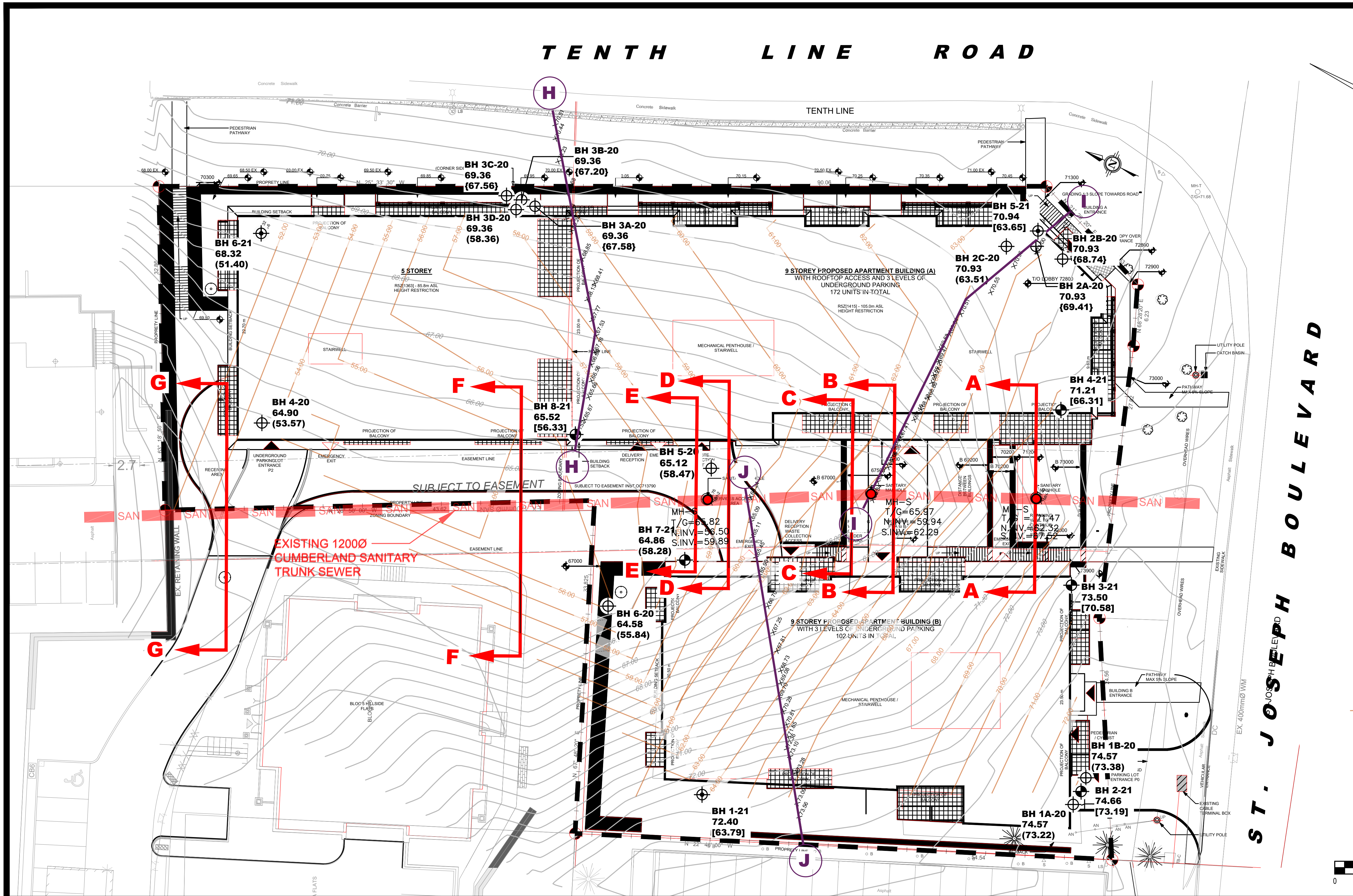

LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
⌖	BOREHOLE LOCATION, PREVIOUS INVESTIGATION (PG5349, 2020)
74.57	GROUND SURFACE ELEVATION (m)
(73.22)	PRACTICAL DCPT / AUGERING REFUSAL ELEVATION (m)
{72.33}	PRACTICAL AUGER REFUSAL ON INFERRED BOULDER ELEVATION (m)
[73.19]	PRACTICAL AUGER REFUSAL ON CONFIRMED BEDROCK ELEVATION (m)
↑—↑	BUILDING CROSS SECTION
H—H	SLOPE STABILITY CROSS SECTION
70.00	BEDROCK CONTOUR (m)

ELEVATIONS AT TEST HOLES ARE REFERENCED TO A GEODETIC DATUM

BASEPLAN PROVIDED BY ROSSMAN ARCHITECTURE

INVERT ELEVATIONS OF THE EXISTING CUMBERLAND SANITARY TRUNK SEWER PROVIDED BY PHOENIX HOMES

0 5 10 15 20 25m

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NO.	REVISIONS	DATE	INITIAL
2	CONCEPTUAL PLAN UPDATED	31/03/2022	RG
1	CONCEPTUAL PLAN UPDATED	23/02/2022	RG

DCR PHOENIX C/O LANDRIC HOMES

GEOTECHNICAL INVESTIGATION - HILLSIDE DEVELOPMENT
PROPOSED MULTI-STOREY BUILDING - 3277 ST.JOSEPH BOULVARD
OTTAWA, ONTARIO

Title:

BEDROCK CONTOUR PLAN

Scale: 1:400
Drawn by: RCG
Checked by: RG
Approved by: SD
Date: 03/2021
Report No.: PG5625-1
Dwg. No.: PG5625-2
Revision No.: 2

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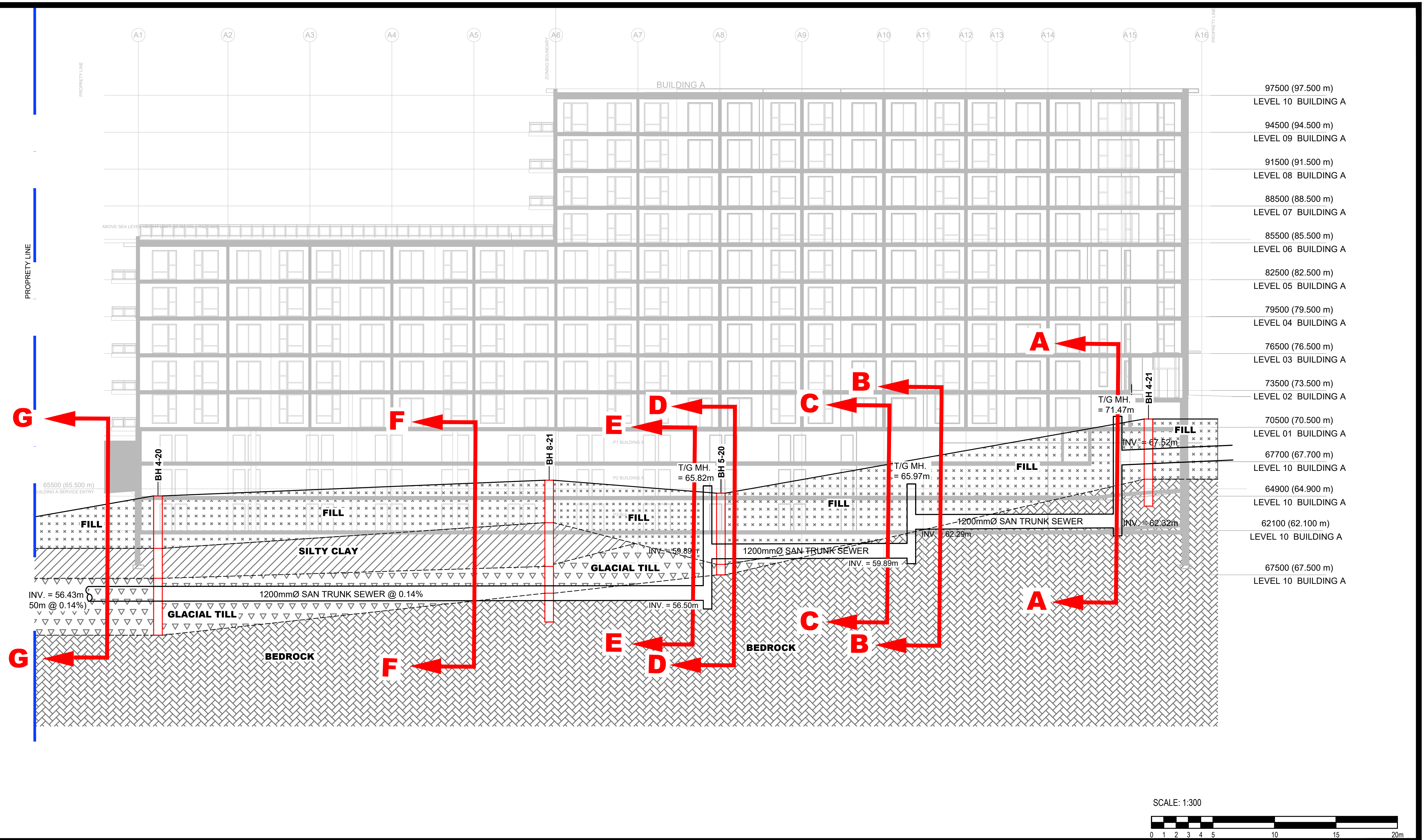
11x17



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NO.	REVISIONS	DATE	INITIAL
0			

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GEOTECHNICAL INVESTIGATION - HILLSIDE DEV.
PROPOSED MULTI-STOREY BUILDING - 3277 ST.JOSEPH BOULVARD
OTTAWA, ONTARIO

Title:
BUILDING PROFILE VIEW - BUILDING A

Scale: 1:300
Date: 03/2021
Drawn by: RCG
Report No.: PG5625
Checked by: MS
Dwg. No.: PG5625-3A
Approved by: SD
Revision No.:

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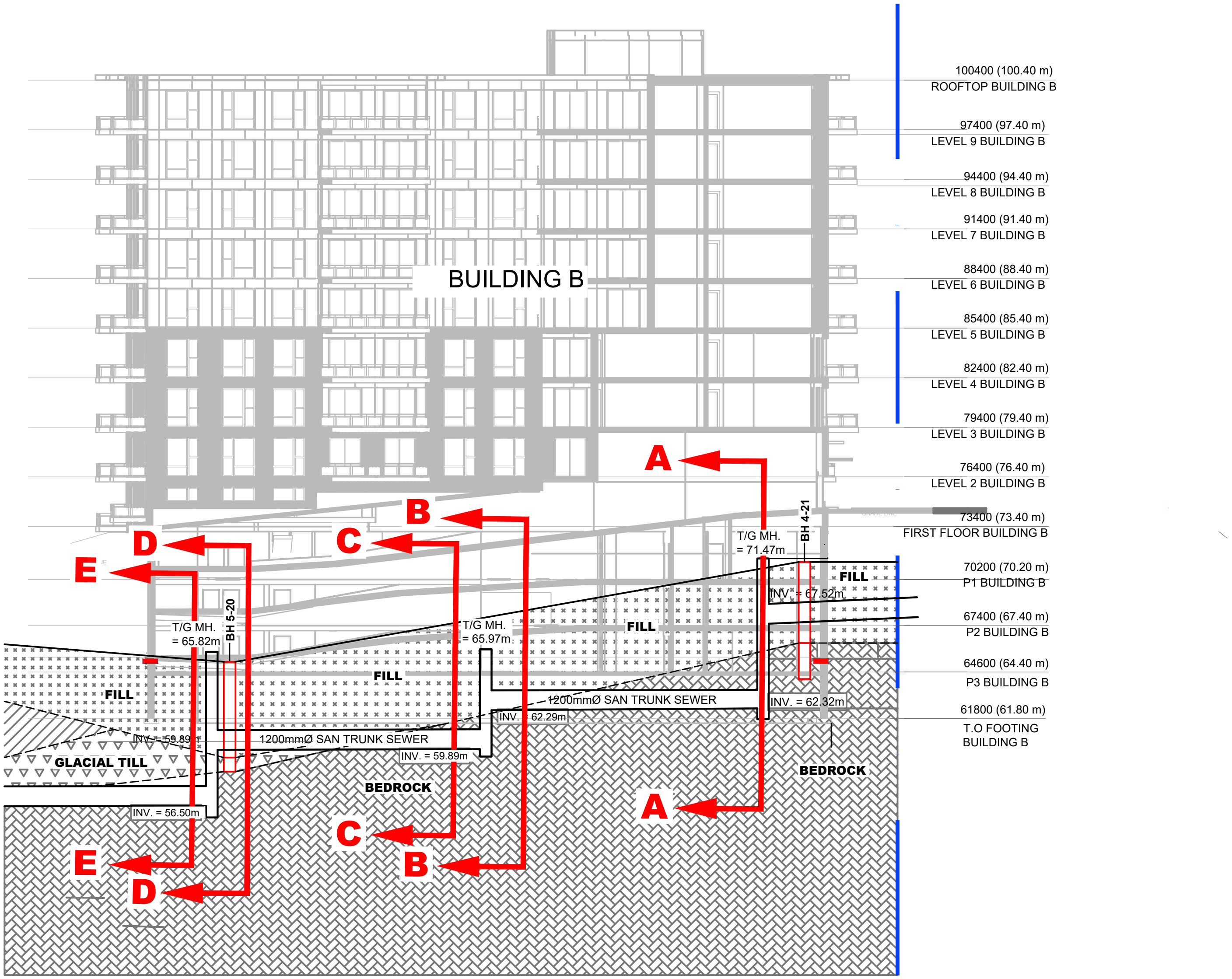
11x17



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GEOTECHNICAL INVESTIGATION - HILLSIDE DEV.
PROPOSED MULTI-STOREY BUILDING - 3277 ST.JOSEPH BOULVARD
OTTAWA, ONTARIO

Title:

BUILDING PROFILE VIEW - BUILDING B

Scale: 1:400
Date: 03/2021
Drawn by: RCG
Report No.: PG5625
Checked by: RG
Drawing No.: PG5625-3B
Approved by: SD
Revision No.:

c:\users\robertg\downloads\pg5349-2(rev.5) (1).dwg