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July 25, 2019
File: PG4777-LET.01R

Greely Family Farm Inc.
6598 Pebble Trail Way
Greely, Ontario
K4P 0B6

Attention: **Mr. Daniel Payer, P.Eng.**

Subject: **Updated Geotechnical Investigation**
Proposed Commercial Development
6075 Bank Street
Ottawa (Greely)

Dear Mr. Payer,

1.0 Introduction

Further to your request and authorization, Paterson Group Inc. (Paterson) completed a stand-alone geotechnical investigation report for the proposed commercial development to be located at the aforementioned site. This updated report version presents our findings and recommendations from a geotechnical perspective for the proposed project, as well as including revisions and clarifications arising from review comments by the Infrastructure Approvals Department of the City of Ottawa.

The subject parcel being developed is located at the southwest corner of a much larger parcel of land previously investigated by Paterson for the same client, in our Report No. PG3957-1R, dated March 27, 2017. The overall parcel is considered to be a future “campus” of commercial buildings. It is understood that commercial development is proposed at the subject site, to consist of a group of four (4) basementless slab-on-grade buildings.

2.0 Method of Investigation

Field Program

The boreholes described in this investigation were put down as part of the previous investigation, referenced above. The applicable boreholes have been renumbered from BH 26-16 to BH 31-16, inclusive, to BH 1 to BH 6, inclusive, for presentation in this report.

The field program for the subject boreholes was carried out on October 19, 2016, and included the putting down of six (6) boreholes to depths of between 3.0 and 6.9 m.

The borehole locations and the ground elevations were provided and determined by Ark Engineering. The ground elevations are referenced to geodetic datum.

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

Sampling and In Situ Testing

The sides of the upper part of each borehole was scraped and a disturbed soil sample was recovered from the auger flights to log the upper part of the profile. Subsequently, samples of the soil were recovered from the boreholes using a 50 mm diameter split-spoon sampler. The depths at which the auger and split spoon samples were recovered from the test holes are shown as, AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1. All soil samples were classified on site, using visual and tactile field soil description methods, placed in sealed plastic bags and were transported to our laboratory for further review and testing.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm. The Standard Penetration Tests were done in general accordance with ASTM D1586-11 - Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils. Note that the investigation fieldwork was done in 2016.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the attached Soil Profile and Test Data sheets for specific details of the soil profile encountered at the test hole locations.

The boreholes were instrumented with standpipes to allow for groundwater measurements to be made subsequent to the completion of the drilling program.

3.0 Observations

3.1 Surface Conditions

The subject site is fairly level, with ground elevations between 90.1 and 90.5 m. The site is approximately at grade with the surrounding properties and the Village Centre Place roadway to the north, and is 1±m lower than Bank Street, to the southwest. Based on the present ground elevations, the finished floor levels of the proposed buildings will represent a grade raise of between 1.0 and 1.4 m from current levels.

The ground surface has been covered with fill materials. As the fill material consists of native material excavated from the site, the transition between the fill material and existing native material is difficult to distinguish in the absence of a well defined topsoil layer. The existing fill deposits extend to depths of up to 1.5 m below the existing ground surface. Based on the interpreted native soil surface levels, the grade raise from original grade is 1.2 to 2.7 m.

3.2 Subsurface Profile

Fill Material

A layer of clean fill material was found over the greater part of the subject property and is assumed to have been placed on site from local earthmoving operations, including the excavation for a nearby existing SWM pond. The fill material consists of various site excavated material, generally sandy silt to silty sand, that may be difficult to differentiate from native materials. The fill layer has been interpreted to be up to 1.5 m in thickness.

Sandy Silt, Silty Sand and Sand-Gravel

The predominantly coarse grained soils within the subject development parcel consist of deposits of silty fine to coarse sand, sandy silt and sand-gravel. Based on geological mapping, it is our interpretation that the sandy silt, silty fine sand and sand-gravel are post glacial deposits.

A gradation curve of a sample of the sand-gravel is attached to this report. The gradation curve was determined based on washing and sieve analysis. The testing was performed in general accordance with ASTM C117 Test Method for Materials Finer Than 75-m (No. 200) Sieve in Mineral Aggregates by Washing and ASTM C136 - Test Method for Sieve Analysis of Fine and Coarse Aggregates.

The results of the SPTs indicate that the state of compaction of these coarse-grained soils is predominantly within the compact to very dense ranges.

Bedrock

Practical refusal to augering was encountered in BHs 1, 2, 5 and 6 at depths between 3.0 and 6.9 m. The other two boreholes terminated in dense granular material after practical split spoon sampler refusal. Refer to the attached Soil Profile and Test Data sheets for specific details of the soil profile encountered at each test hole location.

The depths of practical refusal to augering do not necessarily indicate the position of the bedrock and in most cases are expected to be within dense granular soil and/or on cobbles and/or boulders.

Based on digital geological mapping produced by Natural Resources Canada, sourced from the Geological Survey of Canada, the bedrock in this area consists of dolomite of the Oxford formation with an overburden drift thickness of 5 to 15 m depth.

3.3 Groundwater

The measured groundwater levels from the investigation are presented in Table 1, on the following page. The groundwater levels in all the boreholes were measured in the standpipes on October 26, 2016, after the installations had been given time to stabilize. A groundwater level was also recorded in BH 4 on June 10, 2019.

Table 1: Summary of Groundwater Level Readings

Borehole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 1	90.53	2.88	87.65	October 26, 2016
BH 2	90.05	2.96	87.09	October 26, 2016
BH 3	90.24	3.19	87.05	October 26, 2016
BH 4	90.51	2.61	87.90	October 26, 2016
	90.51	2.76	87.75	July 10, 2019
BH 5	90.27	2.91	87.36	October 26, 2016
BH 6	90.12	3.03	87.09	October 26, 2016

Notes: 1. The ground surface elevations at the test hole locations were provided by Ark Engineering and are referenced to geodetic datum.
2. Goundwater readings were measured in the standpipe tubing.

Where the groundwater was encountered, the recorded groundwater depths varied from 2.6 to 3.2 m and the elevations varied from to 87.1 to 87.9 m.

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

4.0 Discussion

4.1 Geotechnical Assessment

Based on the results of the geotechnical investigation, the subject site is suitable, from a geotechnical perspective, for the proposed commercial development. Existing fill materials may present issues with respect to the need for subexcavation and replacement, but the underlying soils are competent for building support. These and other considerations are further discussed in the following sections.

4.2 Site Grading and Preparation

Stripping Depth

Topsoil, and any fill containing deleterious or organic materials, should be stripped from under any buildings and other settlement sensitive structures, such as retaining walls, hard landscaping and pavements.

Fill Placement

Fill used for grading beneath building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A, Granular B Type II, Granular B Type I or select subgrade material. This material should be tested and approved prior to delivery to the site. Testing may consist of the suppliers own Quality Control testing, or samples can be submitted to the geotechnical consultant for testing. Initial acceptance testing can consist of gradation analyses and comparison to OPSS MUNI 1010 (Nov, 2013) gradation limits.

The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Engineered fill placed beneath the buildings should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD) value. Engineered fill placed below the subgrade level for pavements should be compacted to at least 95% of its SPMDD value. The materials comprising the pavement structures should be compacted to at least 100% of their SPMDD values.

The laboratory testing reference for the specified density is ASTM D698-12 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft^3 (600 kN-m/m^3)).

Site-excavated soil, along with non-specified existing fill, can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. The compaction or consolidation requirement for landscaped areas is strictly to avoid excessive settlement of the ground surface over the unspecified landscaping fill materials. As such, it is only necessary to break up clods of the soil fill materials, and provide light consolidation, so running over the fill as it is placed with tracked equipment is sufficient fill consolidation. It is anticipated that site-excavated material within the glacial till areas will include cobbles and large boulders. Boulders should be culled from the material before or during reuse to allow the resulting material to be placed in appropriate lift thicknesses.

If site-excavated soil materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. These materials should meet the requirements for Select Subgrade Material (SSM) under OPSS MUNI 1010 (Nov, 2013). Non-specified existing fill and site-excavated soils, are not suitable as backfill against foundation walls unless a drainage geocomposite, connected to a perimeter drainage system, is provided.

4.3 Foundation Design

Bearing Resistance for Shallow Foundations

The commercial structures proposed for the subject development can be supported by footing foundations. Footings should be founded either on undisturbed in situ soil bearing media or on engineered granular fill materials placed over undisturbed in situ soil subgrade media and compacted to a minimum of 98% of their SPMDD value.

The bearing resistance values for the various founding media are provided below and then an interpretation of the conditions at each of the presently proposed buildings on the subject property is provided later in this section to provide clarification regarding the application of the recommendations.

In localized areas where loose granular soils are present below, and in close proximity to, the footing level, such as at BH 1, where a loose silty sand layer was encountered, these materials should be densified by vibratory compaction (i.e. precompacted) and/or removed in whole or in part and replaced with compacted engineered native fill and/or engineered granular fill.

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

Strip or square footings, up to 2.5 m wide, placed on an undisturbed, compact to dense coarse grained soil bearing surface can be designed using a bearing resistance at SLS value of **120 kPa** and a factored bearing resistance at ULS value of **200 kPa**. A geotechnical resistance factor of 0.5 has been applied to the above noted bearing resistance at ULS value. These values should be confirmed by field review by geotechnical personnel at the time of construction. These bearing resistance values are applicable to the sandy silt, silty sand, sand and sand-gravel.

Note that the allowable soil pressure for working stress design can be taken to be equal to the bearing resistance at SLS value, as noted above.

Where the placing of engineered granular fill is required, to establish the bearing medium, the bearing resistance values can be taken to be equivalent to the bearing resistance values of the parent subgrade soil, as detailed above, provided OPSS Granular B Type II or Granular A materials, compacted to a minimum of 98% of their SPMDD values are used.

Settlement and Permissible Grade Raises

Footings designed using the above-noted bearing resistance at SLS values will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. These are the generally accepted tolerable settlement values for steel-frame commercial construction.

The subject site is not underlain by compressible soil deposits and, as such, there is essentially no practical grade raise restriction.

4.4 Building Specific Interpretation

General Comments

It is our understanding that the proposed buildings will consist of single storey basementless slab-on-grade configuration. The interior underside of footing (USF) levels will depend on the finished floor level (FFL) of the buildings, and the thickness of the footings, but can typically be located at least 1.0±m below the FFL. The USF for the perimeter (exterior) footings needs consideration of the exterior finished grade to ensure that adequate soil cover (1.5 m) for frost protection is provided. For retail buildings, the exterior finished grades are typically close to the FFL to facilitate access from outside without a step, so a perimeter USF of about 1.7 m below the FFL is generally adequate. The Ark Engineering Grading Plan, Drawing No. GP – Grading Plan, Revision 1, dated June 25, 2019, indicates that the USF values for the footings are located 2.0 m below the slab-on-grade (SOB) level for each building. This report section has been updated for the grading plan and the building numbering.

Building A

Building A has a FFL or SOB level of 91.55 m, USF of 89.55 m and finished exterior grades of 91.50 m.

Based on review of the applicable boreholes, interior and perimeter footings are expected to be founded directly on the native soil, at the specified USF, although engineered granular fill could be required in localized areas. It is recommended that the footings be designed using a bearing resistance at SLS value of **120 kPa** and a factored bearing resistance at ULS value of **200 kPa**.

Building B

Building B a FFL or SOB level of 91.45 m, USF of 89.45 m and finished exterior grades of 91.40 m.

Based on review of the applicable boreholes, interior and perimeter footings are expected to be founded on 0.3 to 0.7 m of engineered granular fill, at the specified USF. It is recommended that the footings be designed using a bearing resistance at SLS value of **120 kPa** and a factored bearing resistance at ULS value of **200 kPa**.

Building C

Building C has a FFL or SOB level of 91.35 m, USF of 89.35 m, and finished exterior grades of 91.30 m.

Based on review of the applicable boreholes, interior and perimeter footings are expected to be founded on 0.2 to 0.6 m of engineered granular fill, at the specified USF. It is recommended that the footings be designed using a bearing resistance at SLS value of **120 kPa** and a factored bearing resistance at ULS value of **200 kPa**.

Building D

Building D has a FFL or SOB level of 91.25 m, USF of 89.25 m, and finished exterior grades of 91.20 m.

Based on review of the applicable boreholes, interior and perimeter footings are expected to be founded on 0.5 to 0.6 m engineered granular fill, at the specified USF, although portions could be founded directly on the native soil. It is recommended that the footings be designed using a bearing resistance at SLS (serviceability limit states) value of **120 kPa** and a factored bearing resistance at ULS (ultimate limit states) value of **200 kPa**.

4.5 Design for Earthquakes

The proposed development can be taken to have a seismic site response Class C as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A) for foundations considered at this site. An analysis, using representative shear wave velocity values, and a depth to bedrock of 10 m below the footing level (deeper than any refusal depth) is provided in Figure 1, attached.

The soils underlying the site and below the groundwater level have Standard Penetration Test “N” values that are in excess, to well in excess, of those required to provide assurance of adequate stability concerning seismic liquefaction. The soils underlying the site are not susceptible to seismic liquefaction.

4.6 Slab on Grade Construction

With the removal of all topsoil, and loose and/or organic-rich existing fill, within the proposed building footprint, the native soil surface will be considered an acceptable subgrade surface on which to commence backfilling for floor slab-on-grade construction.

Provision should be provided for proof-rolling the soil subgrade with heavy vibratory compaction equipment prior to placing any fill. Any soft areas should be removed and backfilled with appropriate backfill material.

4.7 Pavement Structures

Proposed Pavement Structures

For design purposes, the pavement structures presented in the tables on the following page could be used for the design of car parking areas (Table 2) or access lanes, including fire lanes, (Table 3).

**Table 2: Recommended Pavement Structure
Car Parking Areas**

Thickness (mm)	Material Description
50	**Wear Course:** Superpave 12.5 Asphaltic Concrete (or HL-3)
150	**BASE:** OPSS Granular A Crushed Stone
300	**SUBBASE:** OPSS Granular B Type II
SUBGRADE:	Either in situ soils, engineered native fill (min. 95% SPMDD) or OPSS Granular B Type I or II material placed over in situ soil (min. 95% SPMDD).

**Table 3: Recommended Pavement Structure
Access Lanes - Truck Traffic**

Thickness (mm)	Material Description
40	**Wear Course:** Superpave 12.5 Asphaltic Concrete (or HL-3)
50	**Binder Course:** Superpave 19.0 Asphaltic Concrete (or HL8)
150	**BASE:** OPSS Granular A Crushed Stone
400	**SUBBASE:** OPSS Granular B Type II
SUBGRADE:	Either in situ soils, engineered native fill (min. 95% SPMDD) or OPSS Granular B Type I or II material placed over in situ soil (min. 95% SPMDD).

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material. Use of a geotextile between the subgrade and the pavement structure is not routinely required. Field review should be conducted during subgrade preparation to evaluate whether any supplementary measures are required, such as subexcavation and replacement of unsuitable materials, use of a geotextile or geogrid and/or proof-rolling of subgrades.

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

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. Asphaltic concrete mixes should be in conformance with OPSS MUNI 1151 (Nov 2006), for Ontario Traffic Category C. The asphaltic concrete should be compacted in conformance with OPSS 310 (Nov 2012), Table 9.

5.0 Design and Construction Precautions

5.1 Foundation Drainage and Backfill

It is recommended, but not mandatory, that a perimeter foundation drainage system be provided for each of the proposed structures. The system should consist of a 100 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 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.

Several of the granular soils encountered during the investigation have an elevated silt content, such as the sandy silt and silty sand. The on-site fill materials also consist predominantly of sandy silt materials. The high silt content of these materials makes them frost susceptible. The frost susceptible soils are not recommended for re-use as backfill against the foundations especially at areas where exterior slabs are close to the grade of the FFL/SOB. Where minor frost action would be tolerable, unselected site excavated soils can be used in conjunction with a composite drainage system (such as system Platon or Miradrain 6000 or G100N), connected to a foundation drainage system.

Alternatively, backfill against the exterior sides of the foundation walls can consist of free-draining non frost susceptible granular materials, such as selected native clean sand or sand-gravel, or OPSS Granular B Type I material. No geotextile separator is required between these materials and the native soils, although the perforated foundation drainage pipe should have a geotextile “sock”.

Please note that, in the case of basementless slab-on-grade structures, it is our understanding that the provision of a foundation drainage system is not a requirement of the Ontario Building Code (OBC 2012). However, the provision of foundation drainage generally ensures that groundwater will not be present adjacent to the foundations of the applicable structures.

As such, non-frost susceptible granular backfill materials preferably in conjunction with a foundation drainage system, should be used, especially at areas where exterior slabs are at grade with the FFL. Where minor frost action is tolerable, and a foundation drainage system is not provided, a “bond-break” should be provided on the exterior face of the foundation wall if frost susceptible backfill materials are to be used, in order to prevent adfreezing (adhesion frost heaving) from occurring.

5.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as those for isolated exterior piers and wing walls, if used, may require more soil cover or a combination of soil cover and insulation.

Manufactured insulation can be used to supplement soil cover. As this generally requires a specific engineered design, incorporating specific foundation details, it should be addressed, if required, at Building Permit stage.

5.3 Excavation Side Slopes

Unsupported Excavations

The side slopes of excavations in the soil overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the applicable structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavations for the commercial structures to be undertaken by open-cut methods (i.e. unsupported excavations).

Above the groundwater level, which is the expected condition for the building and services trench excavations, all the native soils are Type 2, according to the Occupational Health and Safety Act, Regulations for Construction Projects (OHSA) criteria, except for the loose silty sand in BH 1 that is Type 3. Below the groundwater level, all the native soils are Type 3, according to OHSA criteria. Most of the excavations, including those for services installation, are expected to terminate above the groundwater level.

Excavation side slopes above the groundwater level, and extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. This depth is expected to be sufficient for the building excavations and the installation of site services. Flatter excavation slopes, such as 2H:1V to 3H:1V will be required for the portions of the side slopes of excavations that extend below the groundwater level. These portions of the side slopes can also be stabilized by placing clear crushed stone over the saturated soil to allow for the groundwater to collected and pumped out, while retaining the soil. The latter method is described in more detail under the Basal Instability subsection of this report section, below.

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

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress. The frequency of monitoring can be established by the geotechnical consultant at the time when the need arises, based on the specific conditions that warrant the monitoring.

Trench Excavations and Support

The installation of the proposed services in soils can be carried out safely within the confines of a trench box or in an open cut. An open cut in overburden materials will require that all side slopes be cut back at appropriate inclinations, as noted above, to maintain stability. If a shoring system (i.e. trench box) is used to support the walls of the cut, the trench box design should, for safety purposes, allow for additional surcharge pressures associated with construction equipment and stockpiled fill materials above the cut, although stockpiling of materials above excavations is strongly discouraged.

The interpretation of the soil descriptions in the OHSA, Regulations for Construction Projects, for purposes such as trench box design, should be undertaken by experienced geotechnical personnel. The information provided in the attached Soil Profile and Test Data sheets can be consulted for this purpose, with due consideration that the information is only accurate at the applicable test hole locations, and in consideration that the contractor's equipment and methods, as well as the depth of the excavation, can have a significant effect on the actual earth pressures in force at the time of construction. Paterson has interpreted that the native soils above the groundwater level are Type 2 soils and those below the groundwater level are Type 3 soils.

Basal Instability of Trench Excavations

Where trench excavations will extend below the groundwater level in areas of sand, silty sand and sandy silt soils, basal instability could occur due to groundwater influx. This phenomenon is referred to as a "quick" condition, where the effective strength of the soil is reduced by upward flowing groundwater. This situation is exacerbated where the sand is overlain by a low permeability "confining" layer, such as silty clay and/or glacial till, that extend below the groundwater level. When the excavation penetrates the confining layer, the sand is under higher groundwater pressure and the ensuing seepage into the excavation leads to a quick condition. The soil profiles encountered at the boreholes within this development consist of fairly pervious materials, so no confining layer is present.

It is recommended that trench excavations in areas where basal instability is observed should be dewatered from within the excavation by pumping in a slow and controlled manner in order to give time for the groundwater to be lowered beyond the excavation limits. In order to reduce the loss of soil fines from the trench base and walls, non-woven geotextile can be placed against the soil and covered with fine clear crushed stone. The geotextile will retain the soil fines, be held in place by the clear stone, and the clear stone will allow the influx water to be collected and pumped.

In extreme conditions, the basal instability can be avoided by lowering the groundwater level in the offending sand, silty sand or sandy silt soil stratum by pumping from deep wells or well-points installed outside the proposed excavation so that the excavation is completed “in the dry”. Based on the expected levels of the services, trenches are not expected to extend into the groundwater in most cases.

Extensive dewatering should not be conducted without consideration of potential off-site effects if the work is being conducted in close proximity to existing structures that could be adversely affected by groundwater lowering. This is not expected to be a risk for this development and the adjacent buildings are located outside the expected influence zone of the excavations from the subject development.

5.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. Trench details should be as per the applicable cases shown in Detail Drawing Nos. W17, S6 and S7 (attached).

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD value. The bedding material should extend at least to the spring line of the pipe.

Where site conditions at the bedding subgrade are poor, it may be necessary to subexcavate and increase the thickness of the bedding, as indicated in Note 5 of both Detail Drawing Nos. W17 and S6. For this development, such a case would be the result of unexpected conditions encountered during construction and would, therefore, be an evaluation to be made in the field. This would be an issue that would be evaluated as part of the field review services during construction.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD value.

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

5.5 Groundwater Control

It is anticipated that groundwater infiltration into excavations below the groundwater level would be moderate to high depending upon depth of excavation. Pumping from open or cased sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The use of non-woven geotextile and clear stone may be necessary to control silt and prevent clogging of submersible pumps. 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.

For typical ground or surface water volumes, being pumped during the construction phase, of 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 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.

Paterson is of the opinion that required pumping rates will be less than 50,000 L/day and it will not be necessary to register with MOECP. This is based on the observed groundwater levels being below building excavations and below the servicing levels. Should higher pumping rates be required, the developer can register with MOECP for an EASR within a short period of time, as described above.

5.6 Winter Construction

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

The subsoil conditions at this site mostly consist of frost susceptible materials with a high silt content. 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 use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. Also, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure.

5.7 Landscaping Considerations

Tree Planting Restrictions

Based on our review of the subsurface conditions, the soils underlying the site are predominantly coarse-grained and not susceptible to shrinkage. No sensitive marine clay (SMC) soils were encountered within the subject development parcel. As such, there is no minimum regulated tree to structure distance related to potential soil shrinkage.

5.8 Corrosion Potential and Sulphate

The results of the analytical testing for corrosion of four (4) soil samples from the previous investigation of the entire campus property are attached. Soil samples selected for testing consisted of a silty sand (BH 7-SS3), a sandy silt (BH 12-SS4), a sandy silty (BH 18-SS4), and a silty sand with trace gravel (BH 24-SS4).

Paracel Laboratories (Paracel), of Ottawa, performed the laboratory analysis of the soil sample submitted for analytical testing. Paracel is a member of the Standards Council of Canada/Canadian Association for Environmental Analytical Laboratories (SCC/CAEAL). Paracel is accredited and certified by SCC/CAEAL for specific tests registered with the association.

The following testing guidelines were utilized for the submitted soil samples. The anions were analyzed using EPA 300.1, the pH was analyzed using EPA 150.1, the resistivity was analyzed using EPA 120.1, and the percent solids was determined using gravimetrics.

The results of analytical testing show that the sulphate content is less than 0.1% (1 mg/g). This result indicates that Type GU (general use) cement, as per CSA A23.1 Section 4.2.1.1.2, is appropriate for this site.

The chloride content is less than 400 mg/g and the pH of the sample is greater than 5. These results indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site. The resistivity values range from 2530 to 9940 ohm-cm and are indicative of a non-aggressive to aggressive corrosive environment for exposed ferrous metals at this site, with the lower resistivity values indicating more aggressive conditions.

The appropriate concrete exposure class is “N”, for soil contact based on chloride content, where freezing and thawing (F-1 or F-2 exposure class) is not an issue.

6.0 Recommendations

It is recommended that the following be carried out once the site development plans are finalized and during site development:

1. Geotechnical review of development grading plans. Note that the present grading for the development, as per Revision 1 of the Grading Plan, has been reviewed and conforms to geotechnical recommendations, as per Paterson File PG4777-LET.03, dated July 25, 2019.

2. Observation of all bearing surfaces prior to the placement of concrete.

3. Sampling and testing of the concrete and fill materials used.

4. Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

5. Observation of all subgrades prior to backfilling.

6. Field density tests to determine the level of compaction achieved.

7. 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 field review and materials testing program by the geotechnical consultant.

7.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities.

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) or entities other than Greely Family Farm Inc. or their agent(s) is not authorized without review by Paterson Group Inc. for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Tovell

Andrew J. Tovell, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Jul. 25-19
A. J. TOVELL
46812509
PROVINCE OF ONTARIO

Report Distribution:

☐ Greely Family Farm Inc. (3 copies)
☐ Paterson Group Inc. (1 copy)

Attachments:

☐ Soil Profile and Test Data Sheets (BH 1 to BH 6, inclusive)
☐ Symbols and Terms
☐ Sieve Analysis Chart
☐ Analytical Testing Results (2 pages)
☐ Figure 1: Seismic Site Class
☐ City of Ottawa Detail Drawings W17, S6 and S7
☐ Drawing PG4777-1, Rev. 2 - Test Hole Location Plan

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Village Centre Place
Ottawa, Ontario

DATUM Ground surface elevations provided by Ark Engineering.

REMARKS

BORINGS BY CME 55 Power Auger

DATE October 19, 2016

FILE NO. PG4777

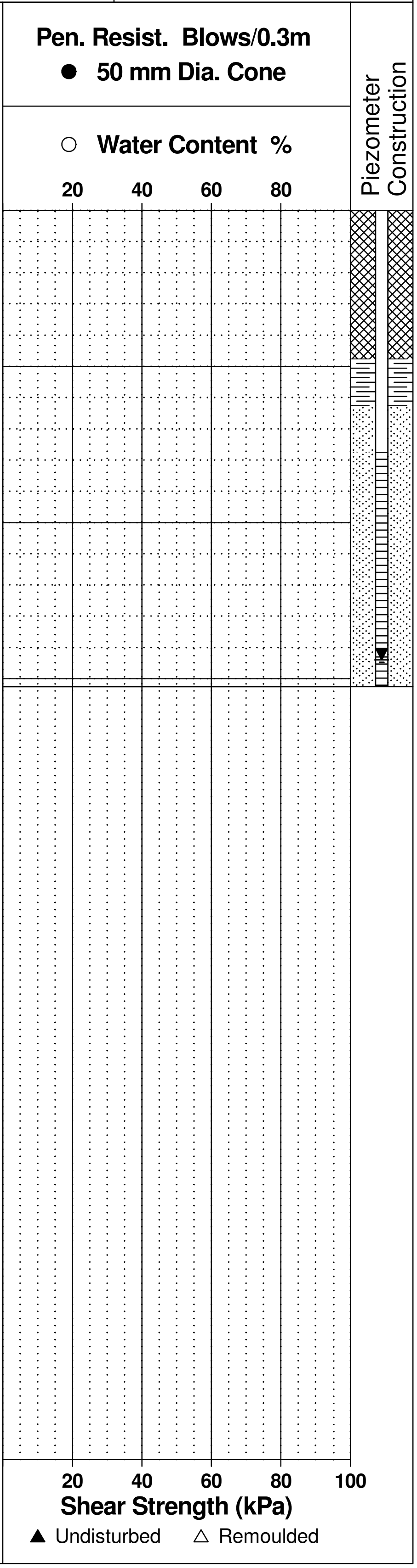
HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	90.53
FILL: Brown silty sand to sandy silt, trace rootlets	AU	1				
0.60						
Loose, brown **SILTY SAND**	SS	2	62	9	1	89.53
1.52						
Very dense, brown medium to coarse **SAND** with gravel, inferred cobbles and boulders	SS	3	50	50+	2	88.53
	SS	4	71	50+		
3.05					3	87.53

End of Borehole

Practical refusal to augering at 3.05m depth

(GWL @ 2.88m-October 26, 2016)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Village Centre Place
Ottawa, Ontario

DATUM Ground surface elevations provided by Ark Engineering.

FILE NO. PG4777

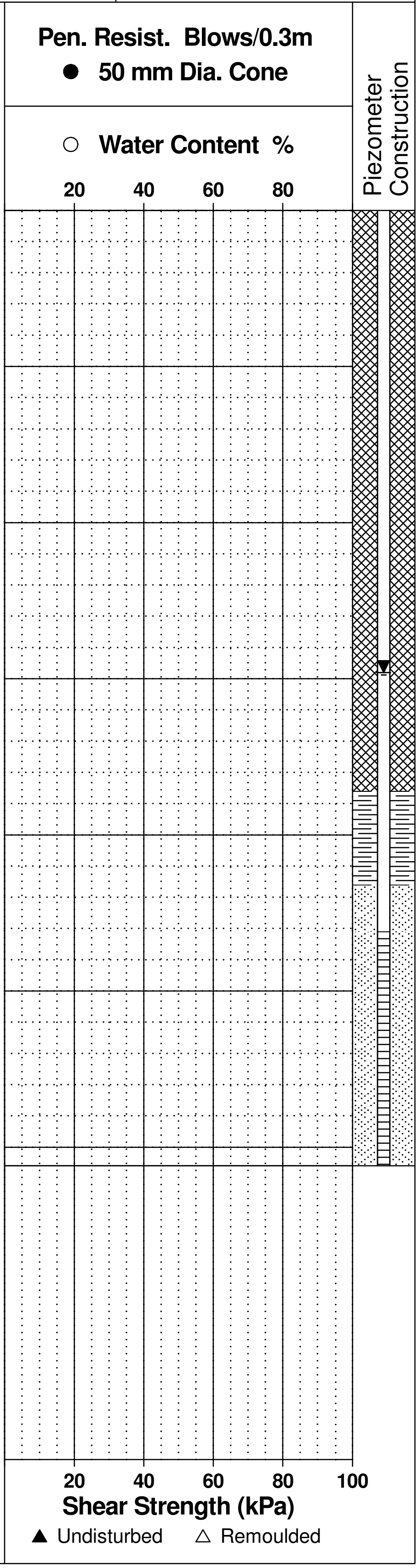
REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger

DATE October 19, 2016

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	90.05
FILL: Brown silty sand with rootlets — 0.60	AU	1				
FILL: Red-brown silty sand — 0.91						
Compact, brown **SANDY SILT to SILTY SAND** — 2.13	SS	2	58	19	1	89.05
	SS	3	62	21	2	88.05
Very dense, brown **SAND-GRAVEL,** trace silt, inferred cobbles and boulders — 4.27	SS	4	62	50+		
	SS	5	58	50+	3	87.05
	SS	6	33	35	4	86.05
Very dense to dense, brown fine to medium **SAND,** trace silt, inferred cobbles and boulders — 6.12	SS	7	46	50+	5	85.05
	SS	8	62	35	6	84.05
End of Borehole Practical refusal to augering at 6.12m depth (GWL @ 2.96m-October 26, 2016)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Village Centre Place
Ottawa, Ontario

DATUM Ground surface elevations provided by Ark Engineering.

REMARKS

BORINGS BY CME 55 Power Auger

DATE October 19, 2016

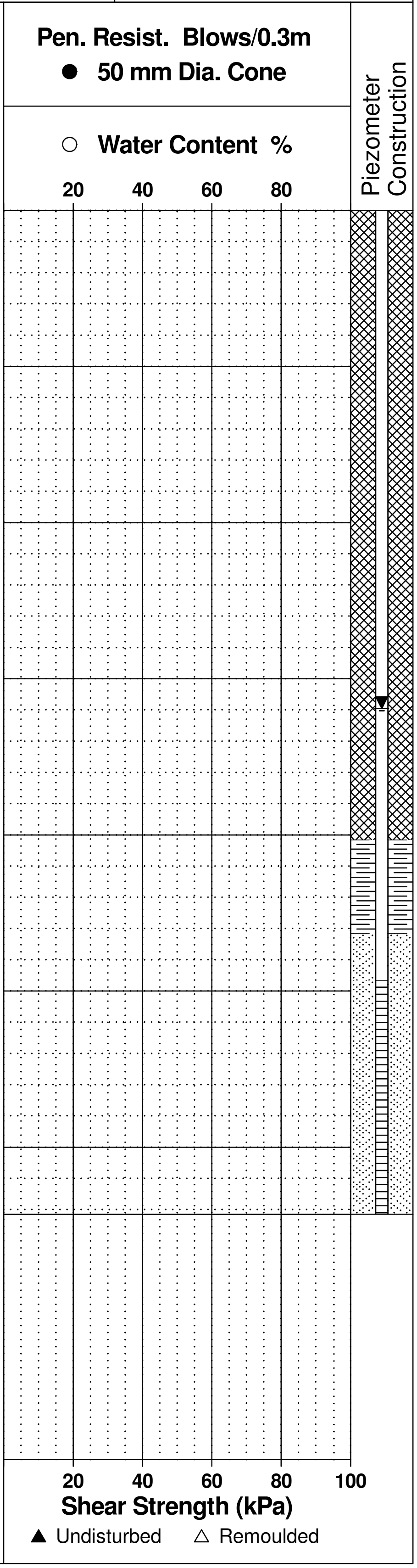
FILE NO. PG4777

HOLE NO. BH 3

SOIL DESCRIPTION	
GROUND SURFACE	
FILL; Grey sandy silt with rootlets	0.10
FILL: Grey sandy silt	0.76
FILL: Brown sandy silt, trace clay and gravel	1.52
Compact, brown **SILTY SAND**	2.13
Compact to dense, grey **SANDY SILT**	3.96
Dense to very dense, grey **SANDY-GRAVEL**	6.43
End of Borehole (GWL @ 3.19m-October 26, 2016)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	62	9
SS	3	67	20
SS	4	62	25
SS	5	58	45
SS	6	58	21
SS	7	33	50+
SS	8	38	35
SS	9	85	50+

DEPTH (m)	ELEV. (m)
0	90.24
1	89.24
2	88.24
3	87.24
4	86.24
5	85.24
6	84.24



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Village Centre Place
Ottawa, Ontario

DATUM Ground surface elevations provided by Ark Engineering.

REMARKS

BORINGS BY CME 55 Power Auger

DATE October 19, 2016

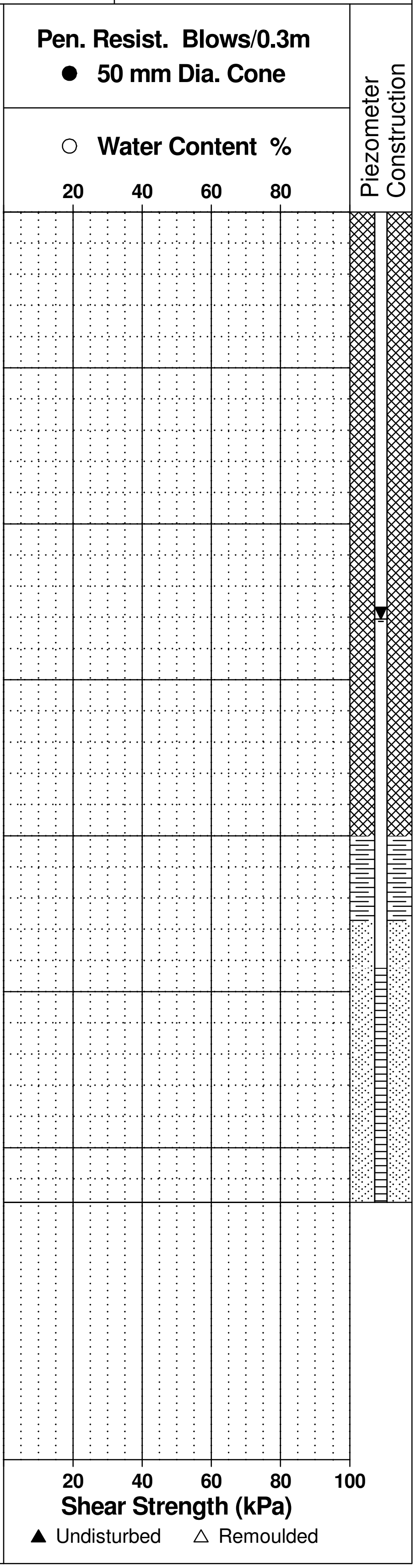
FILE NO. PG4777

HOLE NO. BH 4

SOIL DESCRIPTION	
GROUND SURFACE	
FILL: Brown silty clay, trace cobbles and rootlets	0.20
FILL: Brown sandy silt to silty sand, trace cobbles	0.76
Compact, brown **SANDY SILT to SILTY SAND**	1.37
Compact to dense, grey **SANDY SILT**	3.81
Compact to vey dense, grey **SAND-GRAVEL,** trace to some silt	6.35
End of Borehole (GWL @ 2.61m-October 26, 2016)	

SAMPLE			
TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	19
SS	3	62	28
SS	4	58	23
SS	5	50	43
SS	6	12	19
SS	7	42	28
SS	8	50	50+
SS	9	30	50+

DEPTH (m)	ELEV. (m)
0	90.51
1	89.51
2	88.51
3	87.51
4	86.51
5	85.51
6	84.51



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Village Centre Place
Ottawa, Ontario

DATUM Ground surface elevations provided by Ark Engineering.

FILE NO. PG4777

REMARKS

HOLE NO. BH 5

BORINGS BY CME 55 Power Auger

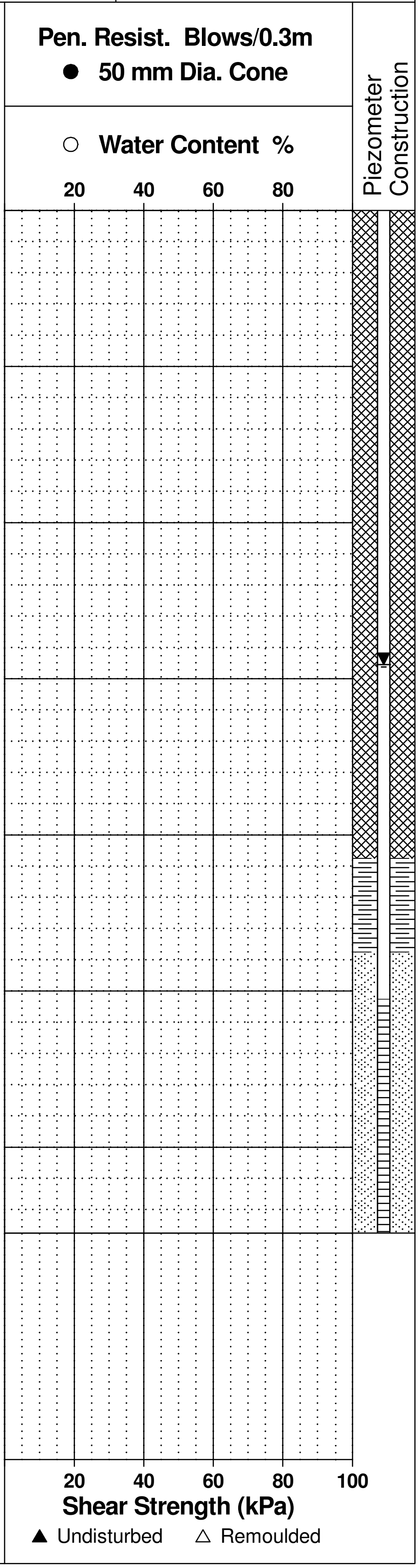
DATE October 19, 2016

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	90.27
FILL: Brown sandy silt, trace gravel and rootlets 0.13	AU	1				
FILL: Brown sandy silt, trace gravel	SS	2	62	21	1	89.27
1.52						
Compact to dense, grey **SANDY SILT**	SS	3	67	16		
					2	88.27
	SS	4	67	41		
					3	87.27
	SS	5	58	47		
3.66						
Very dense, grey **SAND-GRAVEL,** trace silt, cobbles and inferred boulders					4	86.27
	SS	6	42	50+		
4.42						
Dense, grey **SANDY SILT** with gravel, trace inferred cobbles and boulders	SS	7	42	50+		
5.18					5	85.27
Dense to very dense, grey **SAND-GRAVEL,** trace silt, inferred cobbles and boulders	SS	8	75	41		
					6	84.27
	SS	9	45	50+		
6.55						

End of Borehole

Practical refusal to augering at 6.55m depth

(GWL @ 2.91m-October 26, 2016)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Commercial Development - Village Centre Place
Ottawa, Ontario

DATUM Ground surface elevations provided by Ark Engineering.

REMARKS

BORINGS BY CME 55 Power Auger

DATE October 19, 2016

FILE NO. PG4777

HOLE NO. BH 6

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Brown **SANDY SILT,** trace rootlets	0.10
Compact, brown **SANDY SILT to SILTY SAND**	0.76
Compact to dense, grey **SANDY SILT**	4.40
Dense to compact, brown **SAND-GRAVEL,** trace silt, inferred cobbles and boulders	6.40
Dense, grey **SANDY SILT**	6.93

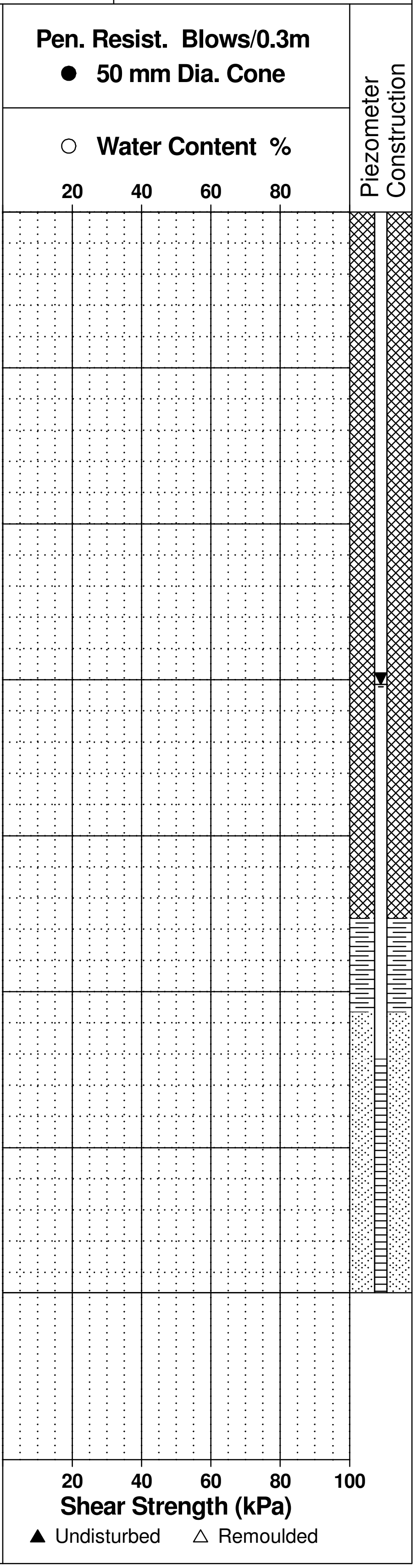
End of Borehole

Practical refusal to augering at 6.93m depth

(GWL @ 3.03m-October 26, 2016)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	11
SS	3	75	21
SS	4	62	50+
SS	5	62	42
SS	6	54	21
SS	7	42	30
SS	8	38	26
SS	9	58	41

DEPTH (m)	ELEV. (m)
0	90.12
1	89.12
2	88.12
3	87.12
4	86.12
5	85.12
6	84.12



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

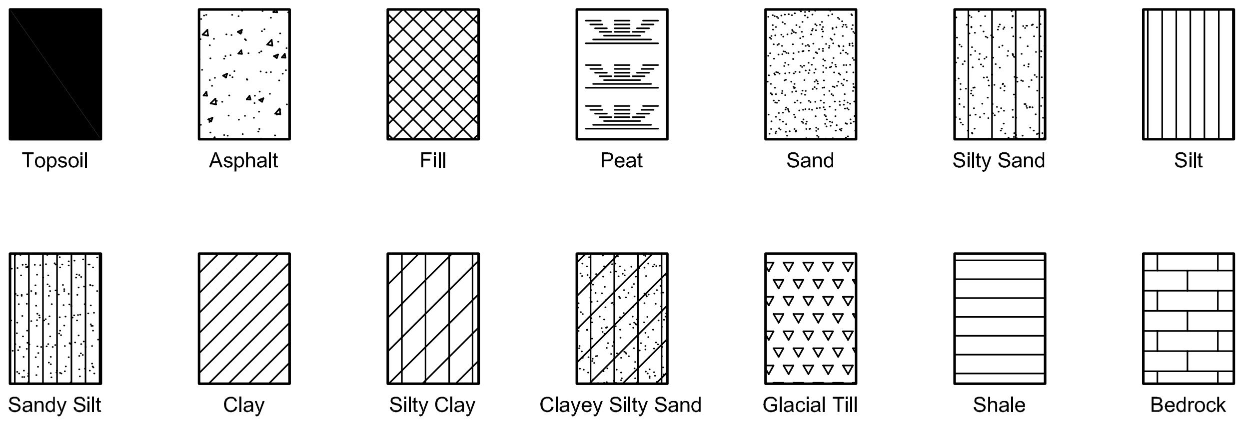
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

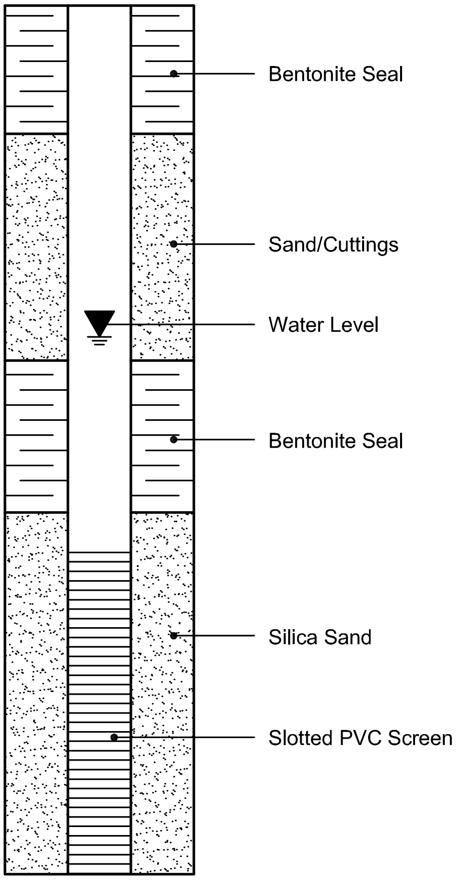
SYMBOLS AND TERMS (continued)

STRATA PLOT

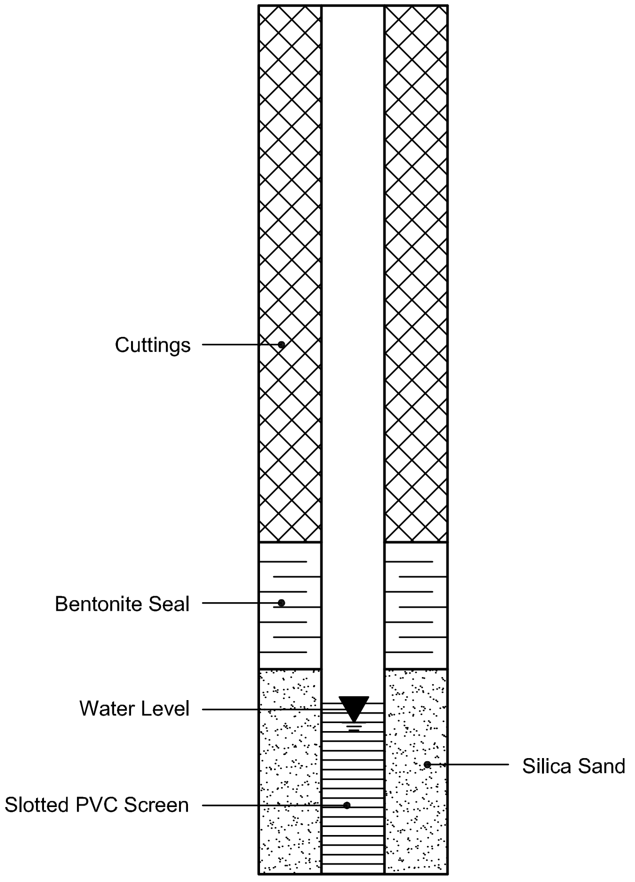


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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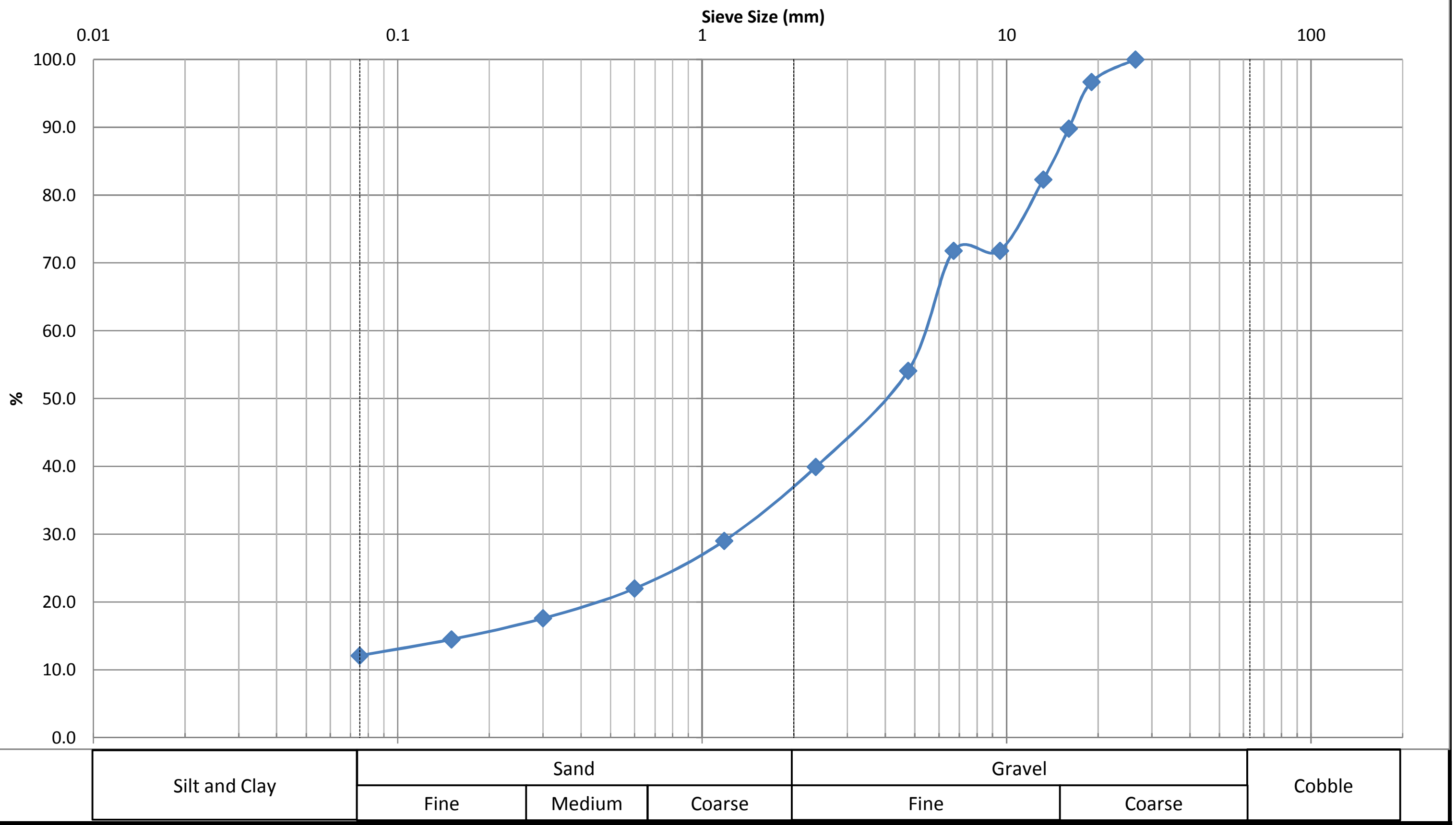
SIEVE ANALYSIS
ASTM C136

CLIENT:	Greely Family Farm Inc.	DESCRIPTION:	Sand - Gravel	FILE NO:	PG4777
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO:	89817
PROJECT:	#7616 to #7646 Village Centre Place	INTENDED USE:	-	DATE RECEIVED:	25-Oct-16
		PIT OR QUARRY:	-	DATE TESTED:	28-Oct-16
DATE SAMPLED:	13-Oct-16	SOURCE LOCATION:	BH 2 (BH27)	DATE REPORTED:	28-Oct-16
SAMPLED BY:	E. Ardley	SAMPLE LOCATION:	SS6	TESTED BY:	DB



Identification	**Soil Classification**			**MC(%)**	**LL**	**PL**	**PI**	**Cc**	**Cu**
								7.82	135.0
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
	27	5.4	1.3	0.04	45.9	42.0	12.1	

Comments	

Certificate of Analysis
Client: Paterson Group Consulting Engineers
Client PO: 20963

Report Date: 24-Oct-2016
Order Date: 17-Oct-2016
Project Description: PG3957

	Client ID:	BH12-SS4	-	-	-
	Sample Date:	14-Oct-16	-	-	-
	Sample ID:	1643068-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	85.2	-	-	-
General Inorganics					
SAR	0.01 N/A	1.94	-	-	-
Conductivity	5 uS/cm	256	-	-	-
pH	0.05 pH Units	8.12	-	-	-
Resistivity	0.10 Ohm.m	39.1	-	-	-
Anions					
Chloride	5 ug/g dry	110	-	-	-
Sulphate	5 ug/g dry	37	-	-	-

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 21106

Report Date: 25-Oct-2016

Order Date: 19-Oct-2016

Project Description: PG3957

Client ID:		BH15-SS4	BH7-SS3	BH18-SS4	BH24-SS4
Sample Date:		17-Oct-16	12-Oct-16	17-Oct-16	18-Oct-16
Sample ID:		1643286-01	1643286-02	1643286-03	1643286-04
	MDL/Units	Soil	Soil	Soil	Soil
Physical Characteristics					
% Solids	0.1 % by Wt.	80.8	88.0	80.6	87.5
General Inorganics					
SAR	0.01 N/A	0.46	-	-	-
Conductivity	5 uS/cm	346	-	-	-
pH	0.05 pH Units	-	7.54	7.82	7.92
Resistivity	0.10 Ohm.m	-	43.7	25.3	99.4
Anions					
Chloride	5 ug/g dry	-	10	102	10
Sulphate	5 ug/g dry	-	146	98	8

Figure 1: Seismic Site Class - OBC 2012

Project: Greely Family Farm Inc. - #7616 to #7646 Village Centre Place
File No: PG4777-LET.01 **Date:** 28-Dec-18

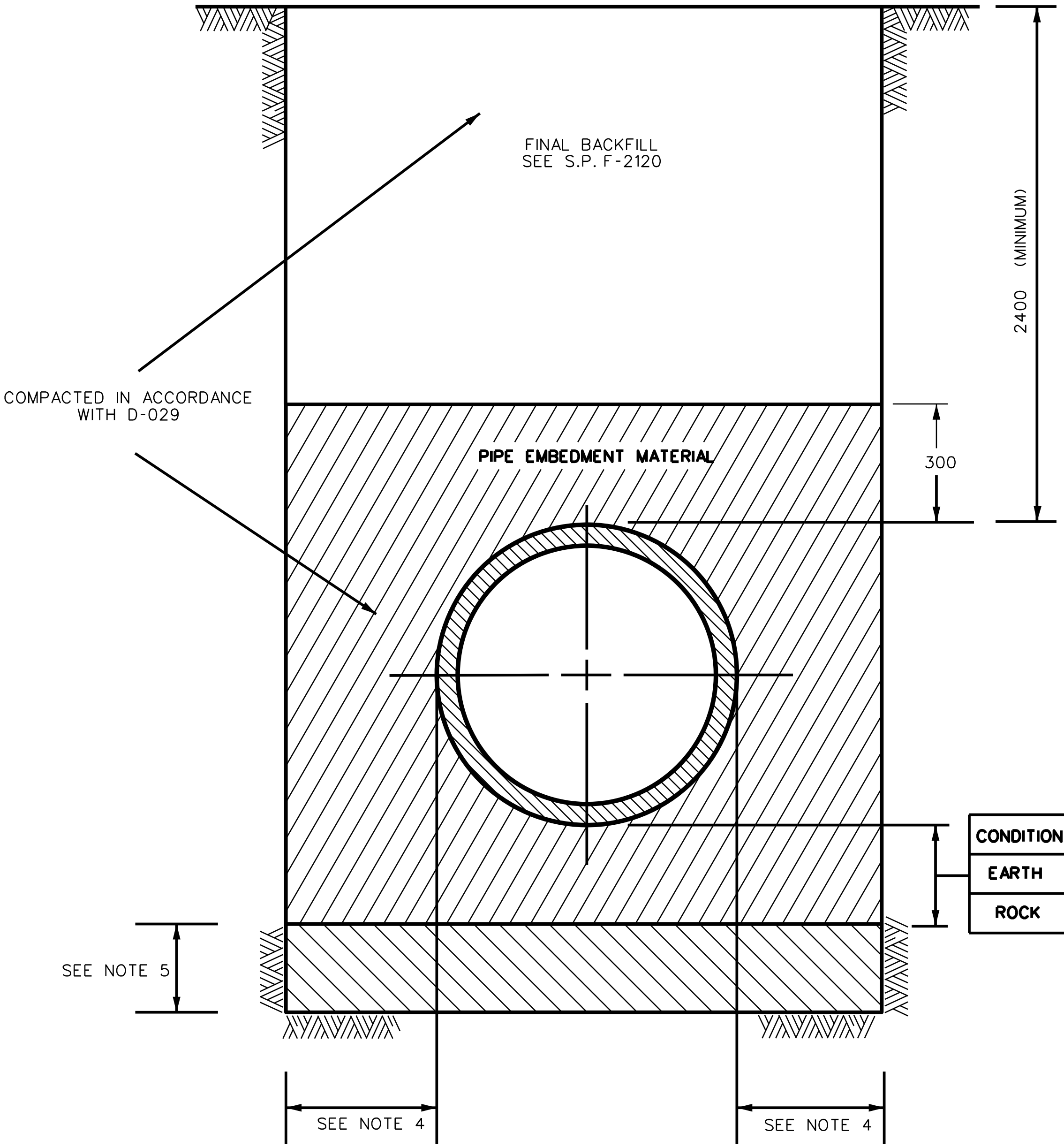
PGA 0.32 **Region:** Ottawa (Greely)

Layer Description	**Layer Properties**		**Cumulative**	**Thickness**
	Vs	**Thickness**	**Thickness**	**Vs**
silty clay crust	200	0.0	0.0	0.0000
grey silty clay (Vs by eqn)	125.0	0.0	0.0	0.0000
Vs = 125 + 1.1667*Z				
compact granular soil	220	1.5	1.5	0.0068
post-glacial clay	200	0.0	1.5	0.0000
dense granular soil	350	8.5	10.0	0.0243
weak or weathered bedrock	1500	1.0	11.0	0.0007
sound bedrock	2000	19.0	30.0	0.0095
Totals	**N/A**	**30.0**	**N/A**	**0.0413**

Average Shear Wave Velocity = **726.9**

Site Class for Seismic Response = **Class** **C**

Site Class	**Description**	**Vs Min.**	**Vs Max.**	**N60 Range**	**Cu Range**
A	Hard rock	1500	>1500	N/A	N/A
B	Rock	760	1500	N/A	N/A
C	Soft rk VD soil	360	760	N>50	Cu>100
D	Stiff soil	180	360	15<N<50	50<Cu<100
E	Soft soil	0	180	N<15	Cu<50



CONDITION	WATERMAIN	SERVICES
EARTH	150 Min.	150 Min.
ROCK	300 Min.	150 Min.

NOTES:

1. PIPE EMBEDMENT MATERIAL - GRANULAR 'A'.
2. FINAL BACKFILL - APPROVED NATIVE MATERIAL OR SELECT SUBGRADE IN ACCORDANCE WITH F-2120
3. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS SHOWN OTHERWISE.
4.

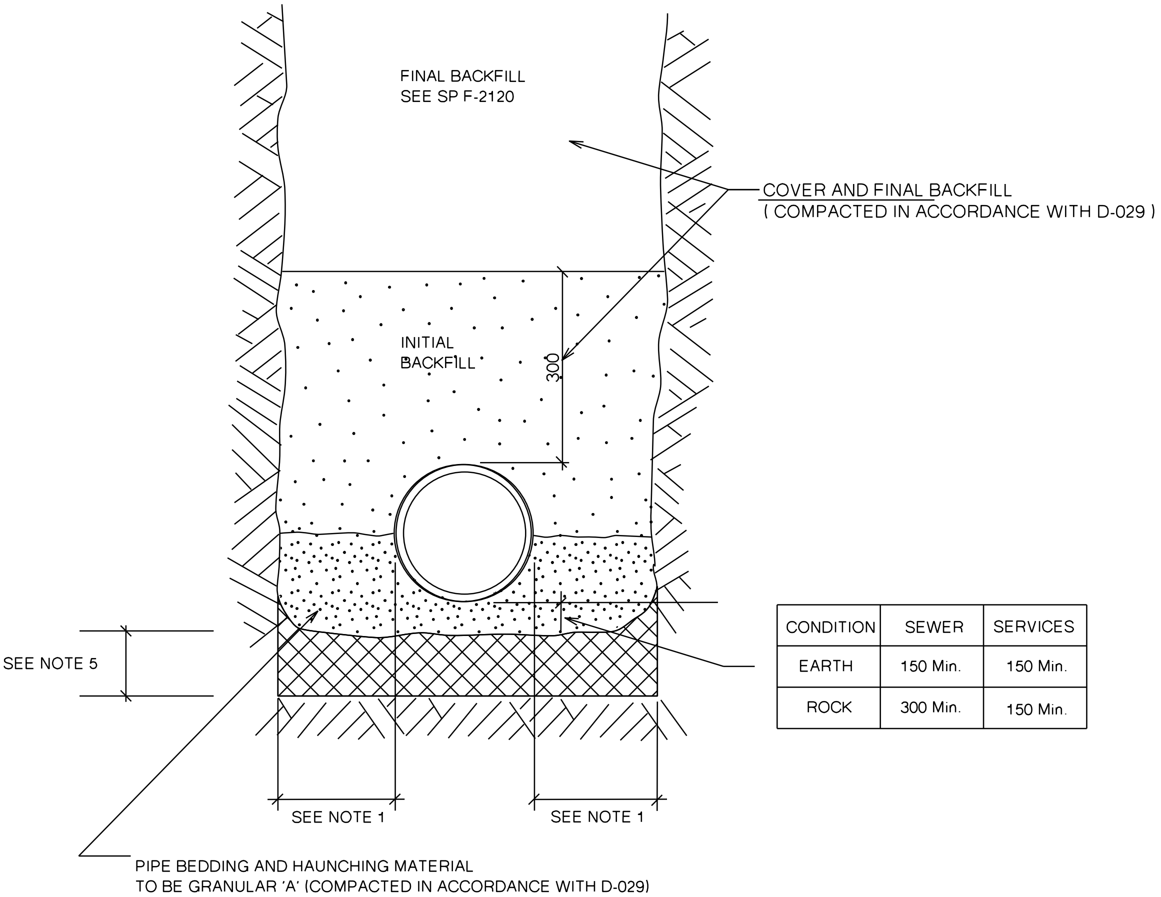
PIPE OUTSIDE DIA. (mm)	CLEARANCE (mm)
900 or LESS	450
OVER 900	500

5. WHEN APPROVED BY THE CONTRACT ADMINISTRATOR, POOR SOILS SHALL BE EXCAVATED TO CREATE A FOUNDATION THAT SHALL BE FILLED TO THE BEDDING WITH GRANULAR 'B'.

Ottawa

STANDARD TRENCH DETAIL

DATE: **MAY 2001**
REV. DATE: **MARCH 2016**
DWG. No.: W17



CONDITION	SEWER	SERVICES
EARTH	150 Min.	150 Min.
ROCK	300 Min.	150 Min.

NOTES:

1.

PIPE INSIDE DIAMETER (mm)	CLEARANCE (mm)
900 OR LESS	CONC 450 PVC 450
OVER 900	500

2. FINAL BACKFILL - APPROVED NATIVE MATERIAL OR SELECT SUBGRADE IN ACCORDANCE WITH F-2120.
3. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS SHOWN OTHERWISE.
4. INITIAL BACKFILL MATERIAL: CONCRETE PIPE - GRANULAR 'A' OR GRANULAR 'B'
   MATERIAL WITH 100% PASSING THE 37.5mm SIEVE
   - PVC PIPE - GRANULAR 'A'
5. WHEN APPROVED BY THE CONTRACT ADMINISTRATOR, POOR SOILS SHALL BE EXCAVATED TO CREATE A FOUNDATION THAT SHALL BE FILLED TO THE BOTTOM OF THE BEDDING WITH GRANULAR 'B'.

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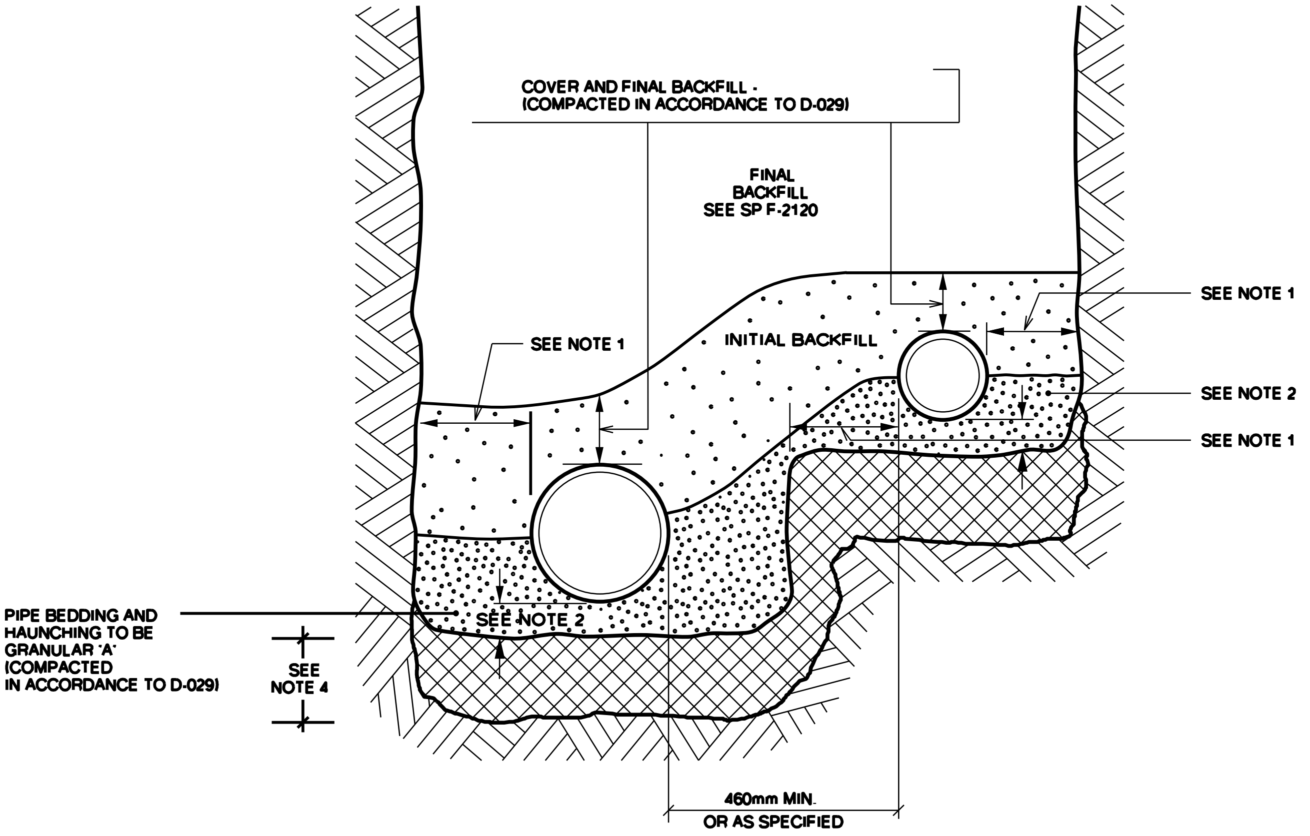
SINGLE TRENCH
(SEWER & SEWER SERVICES)

DATE: MAY 2001
REV. DATE: MARCH 2019
DWG. No.: S6



NOTES:

1.

PIPE INSIDE DIAMETER (mm)	CLEARANCE (mm)
150 TO 900	CONC 450 PVC 450
OVER 900	500

2.

CONDITION	SEWER	SERVICES
EARTH	150 Min.	150 Min.
ROCK	300 Min.	150 Min.

3. FINAL BACKFILL - APPROVED NATIVE MATERIAL OR SELECT SUBGRADE IN ACCORDANCE WITH F-2120.

4. WHEN APPROVED BY THE CONTRACT ADMINISTRATOR, POOR SOILS SHALL BE EXCAVATED TO CREATE A FOUNDATION THAT SHALL BE FILLED TO THE BOTTOM OF THE BEDDING WITH GRANULAR 'B'.

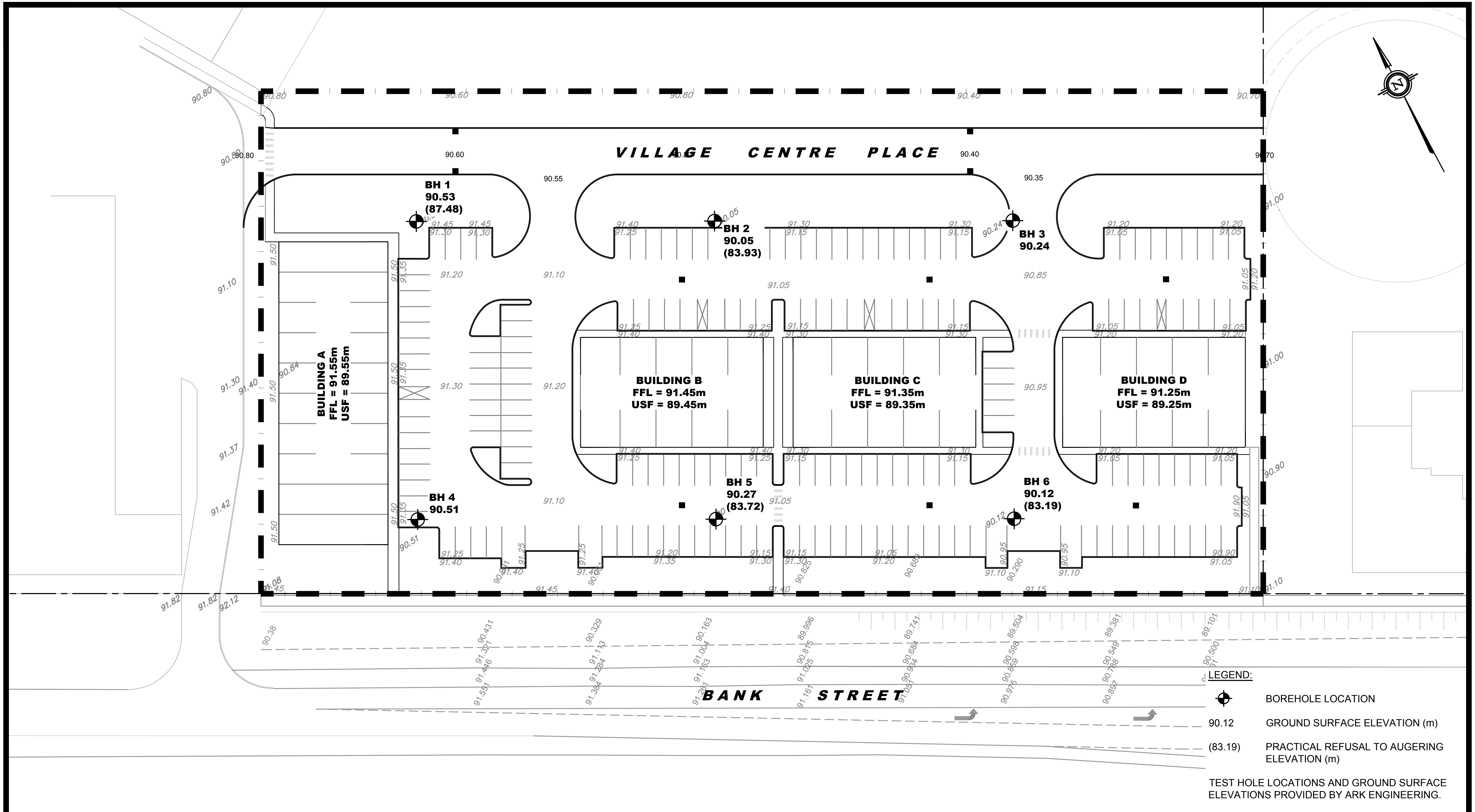
5. SEE S11.3 FOR TYPICAL SINGLE, SEMI-DETACHED AND TOWNHOUSE LOT SERVICING

6. INITIAL BACKFILL: CONCRETE PIPE - GRANULAR 'A' OR GRANULAR 'B'
WITH 100% PASSING THE 37.5mm SIEVE
PVC PIPE - GRANULAR 'A'

Ottawa

COMBINED TRENCH
(SEWERS & SEWER SERVICES)

DATE: MAY 2001
REV. DATE: MARCH 2019
DWG. No.: S7



LEGEND:

⌖	BOREHOLE LOCATION
90.12	GROUND SURFACE ELEVATION (m)
(83.19)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY ARK ENGINEERING.

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NO.	REVISIONS	DATE	INITIAL
2	RELABELED BUILDINGS	16/07/2019	AJT
1	UPDATED TO LATEST BASE PLAN	20/02/2019	AJT

GREELY FAMILY FARM INC.
GEOTECHNICAL INVESTIGATION
PROP. COMMERCIAL DEVELOPMENT - 6075 BANK STREET
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:600
Date: 12/2018
Drawn by: RCG
Report No.: PG4777-LET-01R
Checked by: AJT
Dwg. No.: PG4777-1
Approved by: AJT
Revision No.: 2

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