Geotechnical Investigation

Atlantic Development Group Ltd.
c/o GSP Group Inc.

Project Name:
Proposed Development
69 Ainslie Street South
Cambridge, Ontario

Project Number:
LON-00017843-GE

Prepared By:

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Date Submitted:
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Atlantic Development Group Ltd.

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1. Introduction and Background

1.1 Introduction

EXP Services Inc. (EXP) was retained by **Atlantic Development Group Ltd.** to carry out a geotechnical investigation and prepare a geotechnical report relating to a proposed development at 69 Ainslie Street South in Cambridge, Ontario, hereinafter referred to as the 'Site'.

It is our understanding that the project consists of proposed 20-storey and 15-storey residential towers with an attached 5 and 7-storey podium for commercial and parking use. The development will also include new site services, access road, at grade parking, and landscaped areas.

Based on an interpretation of the factual test hole data and a review of soil and groundwater information from test holes advanced at the site, EXP has provided geotechnical engineering guidelines to support the proposed Site development.

1.2 Terms of Reference

The geotechnical investigation was generally completed in accordance with the scope of work outlined in EXP's Proposal P19-412 (revised) dated November 14, 2019. Authorization to proceed with this investigation was received from Mr. Adeel Khan on behalf of **Atlantic Development Group Ltd.**

The purpose of the investigation was to examine the subsurface conditions at the Site by advancing a series of boreholes at the locations shown on the attached Borehole Location Plan (Drawing 1).

Based on an interpretation of the factual borehole data, and a review of soil and groundwater information from test holes excavated at the Site, EXP has provided engineering guidelines for the geotechnical design and construction of the proposed development. More specifically, this report provides comments on excavations, site preparation, foundations, slab-on-grade construction, bedding and backfill, earthquake design considerations, and pavement design.

This report is provided on the basis of the terms of reference presented above, and on the assumption that the design will be in accordance with applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning geotechnical aspects of the codes and standards, this office should be contacted to review the design.

The information in this report in no way reflects on the environmental aspects of the soil. Should specific information in this regard be needed, additional testing may be required.

Reference is made to Appendix C of this report, which contains further information necessary for the proper interpretation and use of this report.

2. Methodology

The fieldwork was conducted on March 25 and 26, 2020 and consisted of advancing five (5) boreholes at the approximate locations shown on Drawing 1. These test holes within the Site are designated as BH1 to BH5.

Prior to the drilling, buried service clearances were obtained by EXP for the test hole locations.

The boreholes were completed by a specialist drilling subcontractor under the full-time supervision of EXP geotechnical staff. The boreholes were advanced using a track-mounted drill rig equipped with continuous flight hollow stem augers, soil sampling and soil testing equipment. In each borehole, disturbed soil samples were recovered at depth intervals of 0.75 m and 1.5 m using conventional split spoon sampling equipment and Standard Penetration Test (SPT) methods or auger samples. At Boreholes 1 and 3, the bedrock was cored with NQ sized core bit and samples were retained for examination and selective testing.

During the drilling, the stratigraphy in the boreholes was examined and logged in the field by EXP geotechnical personnel.

Short-term groundwater levels within the open boreholes were observed. These observations pertaining to groundwater conditions at the test hole locations are recorded in the borehole logs in Appendix A. Following the drilling, the boreholes were backfilled with the excavated materials and bentonite, to satisfy the requirements of O.Reg. 903.

Representative samples of the various soil strata encountered at the test locations were taken to our laboratory in Cambridge for further examination by a Geotechnical Engineer and laboratory classification testing. Laboratory testing for this investigation consisted of routine moisture content determinations, of the soil overburden and uniaxial compressive strength testing on two representative samples of the bedrock.

Samples remaining after the classification testing will be stored for a period of three months following the issuance of report. After this time, they will be discarded unless prior arrangements have been made for longer storage.

The location of each test hole was established in the field in conjunction with a site plan provided by the client. Elevations of the boreholes were surveyed to a local, temporary benchmark. The benchmark used was the top of the fire hydrant located on the west side of Ainslie Street South, across from the southwest corner of the Site (Assumed Elevation 100.00 m).

3. Site and Subsurface Conditions

3.1 Site Description

The subject Site is currently developed with an unoccupied one-storey building in the central section, surrounded by an asphalt parking lot and some vegetated section along the east side. The Site is bound to the north by a regional bus terminal and to the east and west by Ainslie Street South and Wellington Street, respectively. The Site is about at the same grade as adjacent properties along Ainslie Street South and is partially sloped up, towards the east.

The following sections provide a summary of the soil conditions and groundwater conditions.

3.2 Soil Stratigraphy

The detailed stratigraphy encountered in each test pit is shown on the test pit logs found in Appendix A and summarized in the following paragraphs. It must be noted that the boundaries of the soil indicated on the test pits logs are inferred from non-continuous sampling and observations during excavation. These boundaries are intended to reflect transition zones for geotechnical design and should not be interpreted as exact planes of geological change.

Asphalt

Asphalt was encountered at surface at all borehole locations. The thickness of the asphalt ranged from about 75 to 100 mm.

Fill

At all borehole locations, fill was encountered beneath the asphalt, to depths from about 0.15 to 1.5 m below existing grade. In general, the fill was noted to consist of brown sand having some gravel. At some location, the sand fill was also noted to contain coal, cinders, brick fragments and traces of staining. At Borehole 3, a lower layer of sandy silt fill was noted beneath the sand fill. The compactness condition of the fill is very loose to dense, based on SPT N-values of 3 to 33. The *in situ* moisture content of the fill ranges from about 3 to 14 percent, indicating damp to moist conditions.

Sand

Beneath the fill at Boreholes 1, 3, and 4, a layer of sand was encountered to depths ranging from about 1.5 to 1.9 m below existing grade. In general, the sand was noted to be brown to greyish-brown, with some gravel and traces of cobbles. The compactness condition of the sand is compact to very dense and the *in situ* moisture content of the sand ranges from about 3 to 5 percent, indicating moist conditions.

Bedrock

The fill and native soil deposits are underlain by dolostone bedrock. At the borehole locations, the bedrock was encountered at depths from approximately 0.2 to 1.9 m below existing grade, corresponding to a range in relative Elevation of 98.1 to 100.0 m. Auger refusal and/or sampler refusal on assumed bedrock was met in all boreholes. With standard hollow stem augers, it is estimated that it was possible to penetrate a maximum refusal depth of about 0.8 m into the bedrock, at some locations. It is possible that auger refusal is on boulders above the bedrock. Nevertheless, bedrock was proven at two locations by rock coring techniques.

Bedrock at the site was confirmed to be dolostone of the Guelph Formation. In general, the bedrock is grey with brown zones, porous, and fine to medium crystalline. Traces of fossils and crystal-filled vugs, ranging in sizes were also noted throughout. Traces of fossils, mainly corals and shell, were also noted with some highly concentrated, fossiliferous zones. The dolostone is moderately weathered in the upper 2.0 m and is slightly weathered to unweathered below this depth. The joints are typically horizontal bedding joints with joint surfaces that are planar and rough. Some iron staining, as well clay and calcite infilling was noted to be associated with the joints. Joint spacing increases with depth to an average of 10 to 20 cm. Occasional vertical joints were observed but are discontinuous as seen in the cores. It is noted that vertical joints should be expected because of the inherent ‘blocky’ mass of the dolostone.

The Total Core Recovery (TCR) ranged from 85 to 100 percent. Rock Quality Designation (RQD) values of 0 to 15 percent were noted in the upper 1.0 m of the bedrock, indicative of “ very poor quality” and typical values of 35 to 90 were noted below this depth, indicative of “poor quality” to “good quality”. The individual rock fragments were noted to be very hard.

The results of uniaxial compressive testing on selected cores samples of the dolostone bedrock are shown in Table 1 and the approximate depth to bedrock at the borehole locations is shown in Table 2.

Table 1 – Summary of Bedrock Compressive Strength Testing

Borehole	Depth (m)	Unconfined Compressive Strength (MPa)
1	3.1 – 3.2	51.3
1	4.6 – 4.7	68.5
3	3.2 – 3.3	59.2

The unconfined compressive strength of the three samples above, indicative of a “medium strong” strength.

Table 2 – Approximate Bedrock Elevation at the Borehole Locations

Borehole ID	Ground Surface Elevation (m)	Depth to Bedrock (m)	Elevation of Bedrock (m)
BH1	99.9	1.6	98.3
BH2	100.2	0.2	100.0
BH3	100.0	1.9	98.1
BH4	99.9	1.5	98.4
BH5	100.0	0.9	99.1

The bedrock relief at the Site, as noted in the boreholes, is in the order of 2 m.

3.3 Groundwater Conditions

Short-term groundwater level observations are recorded on the attached borehole logs. The boreholes were open and dry upon completion. It is noted that insufficient time was available for the measurement of the depth to the stabilized groundwater table prior to backfilling the boreholes.

It is also noted that the depth to the groundwater table may vary in response to climatic or seasonal conditions, and as such, may differ at the time of construction, with higher levels in wet seasons.

4. Discussion and Recommendations

4.1 General

Based on available project drawings, the proposed development consists of the construction of a new multi-tower and podium structure that will have one underground level. Based on configuration the existing and expected proposed grades, excavation will be required to about 3 m below existing grade, approximate relative Elevation of 97.0 m. The proposed footprint of the building will encompass the majority of the Site.

The following sections of this report provide geotechnical comments and recommendations regarding site preparation, excavations and dewatering, foundations, slab-on-grade design, drainage, bedding and backfill, earthquake design considerations, and pavement design.

4.2 Site Preparation

Prior to placement of foundations and placement of imported subgrade fill, all existing fill or otherwise deleterious materials should be removed from the footprint of the proposed building and within parking/access road areas. This preparation will also include the existing construction debris, footings, services, and slabs of the existing building(s). It is expected that this process will mostly result in full removal of the fill/overburden, down to weathered bedrock. In landscaped areas and surface parking areas, it may be possible to leave the native sand in place, depending on design subgrade elevations, and subject to on-site review.

Where the exposed subgrade requires reconstruction to achieve the design elevations, structural fill should be used. Along the proposed parking areas and access roads, fill material used to raise grades should consist of imported granular material such as OPSS Granular ‘B’ or similar. The fill material should be inspected and approved by a Geotechnical Engineer and should be placed in maximum 300 mm (12 inch) thick loose lifts and uniformly compacted to 95/98 percent Standard Proctor Maximum Dry Density (SPMDD) within 3 percent of optimum moisture content in order to provide adequate stability for the new pavements. *In situ* compaction testing should be carried out during the fill placement to ensure that the specified compaction is being achieved.

Excess materials should be removed from the site and disposed of in accordance with Ministry of the Environment, Conservation and Parks (MECP) guidelines and requirements. Analytical sampling and testing may be required in accordance with O. Reg. 153 for transportation and off-site disposal of excavated material.

If imported fill material is used at the site, verification of the suitability of the fill may be required from an environmental standpoint. Conventional geotechnical testing will not determine the suitability of the material in this regard. Analytical testing and environmental site assessment may be required at the source. This will best be assessed prior to the selection of the material source. A quality assurance program should be implemented to ensure that the fill material will comply with the current (MECP) standards for placement and transportation.

The disposal of excavated materials must also conform to the MECP Guidelines and requirements. EXP can be of assistance if an assessment of the materials is required.

4.3 Excavation and Groundwater Control

General

Excavations for the proposed building and site services are expected to penetrate the fill and extend into the upper zone of the dolostone bedrock as deep as 2 to 3 metres. It is assumed that excavations will remain above the water table.

Excavation of the overburden should be undertaken with hydraulic equipment that is capable of removing cobbles, boulders, and large concrete debris within the fill material.

Based on relative grades, the service trenches, foundations, and elevator shafts will extend into the sound bedrock. Based on observations during the fieldwork, excavation of the weathered bedrock in the upper 0.3 to 0.8 m is expected to be relatively straight forward. However, excavation of the sound bedrock will require pneumatic rock hammers or blasting techniques. All loosened bedrock fragments should be removed from the excavation sidewalls as work proceeds. Groundwater seepage through discontinuities in the bedrock may loosen small fragments, and excavation sidewalls should be periodically monitored. If explosives are used to shatter the rock, the vibration should be closely monitored

Side slopes of temporary excavations must conform to Regulation 213/91 of the Occupational Health and Safety Act (OHSA) of Ontario. The fill and sand at the Site are classified as Type 3 soils and the bedrock is classified as Type 2 soil. It is expected that all excavation work at the site will extend through Type 3 soils. Therefore, in accordance with OHSA, excavation side slopes must be cut back at a maximum inclination of about 1H:1V from the base of the excavation. Where space allows and where Type 3 soils are removed, excavations in the bedrock can be 1.2 m vertical from the base and then sloped 1H:1V. Should groundwater egress loosen the side slopes, flatter slopes may be required.

Construction Dewatering

Based on the borehole information, no major groundwater dewatering problems are envisioned during construction of the buildings, parking lots, or the installation of the underground services. Some seepage of groundwater from within the fill and possibly from the upper zone of the bedrock should be anticipated during construction. Where minor groundwater infiltration is encountered it can most likely be accommodated using conventional sump pumping techniques; provided that the sump pits are lined with a suitable geotextile filter fabric and pump inlet is set in a clear stone, which must fill the sump pit completely. Use of a filtered system will result in migration of sandy soil particles that will loosen the soil deposits.

The collected water should be discharged a sufficient distance away from the excavated area to prevent the discharge water from returning to the excavation. Sediment control measures should be provided at the discharge point of the dewatering system. Caution should also be taken to avoid any adverse impacts to the environment.

Although not anticipated for foundation excavations to conventional depths, it is important to mention that for any projects requiring positive groundwater control with a removal rate of 50,000 litres to less than 400,000 litres per day, an Environmental Activity and Sector Registry (EASR) or Permit to Take Water (PTTW) will be required. PTTW applications are required for removal rates more than 400,000 L per day and will need to be approved by the MECP per Sections 34 and 98 of the Ontario Water Resources Act R.S.O. 1990 and the Water Taking and Transfer Regulation O. Reg. 387/04. It is noted that a standard geotechnical investigation will not determine all the groundwater parameters which may be required to support the application. Accordingly, a detailed hydrogeological assessment from a quantitative point of view may be required to estimate the quantity of water to be removed. EXP can assist if the need arises.

4.4 Foundation Construction

Conventional Strip and Spread Footings

The fill and sand encountered at the borehole locations are not considered suitable to safely support the new structures. It is recommended that footings for the buildings be founded on the dolostone bedrock, which is located at depths ranging from approximately 0.2 to 1.9 m below existing grade, relative Elevation 98.1 to 100.0 m. Strip and spread footings or drilled piers founded below any excessively weathered zone of the bedrock can be designed using a factored bearing resistance of 2000 kPa at ULS. The bearing resistance at SLS is not expected to govern design of footings on dolostone bedrock which is non-yielding base.

The bearing value and founding depths are subject to verification through close field evaluation. The depth of the weathered bedrock had been identified as variable across the site and difficult to establish accurately from the borehole information. Therefore, the actual founding elevations would have to be determined by field inspection during the installation of footings.

Socketed Caisson Foundations

In light of the height of the proposed building and consequently the high foundation loads, founding the footings on the strong dolostone bedrock can be considered.

Drilled caissons socketed into strong dolostone bedrock about 3.0 m bgs can be used if high bearing resistance is required. The socketed caisson will develop resistance through a combination of sidewall shear and end-bearing in the rock socket. A minimum centre-to-centre spacing of three caisson diameters should be maintained between caissons.

The axial geotechnical resistance can be computed using the following factored values of end-bearing resistance and sidewall shear:

Factored Bearing Resistance at ULS:	4000 kPa

The bearing resistance at SLS is not expected to govern design of footings on strong dolostone bedrock which is non-yielding base.

The recommended resistance values assume that the socket will extend at least 3.0 m bgs into the strong bedrock, and the socket sidewall will not be softened or smeared by the drilling method. Substantially higher values of sidewall shear resistance could be used if the socket walls are artificially roughened or grooved and the design is based on sidewall shear alone.

Use of a steel liner might be required to support the sidewalls in the upper fill and sand layers.

It should be noted that the recommended bearing capacity has been calculated by EXP from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, if more specific information becomes available with respect to conditions between boreholes when foundation construction is underway. The interpretation between the boreholes and the recommendations of this report must therefore be checked through field inspections provided by EXP to validate the information for use during the construction stage.

4.5 Slab-on-Grade and Permanent Drainage

The floor slabs for the proposed structure may be constructed using conventional concrete poured slab techniques, following removal of all existing fill, debris, or otherwise deleterious soils and preparation of the subgrade as outlined in the previous Section 4.2.

It is recommended that an impermeable soil seal such as clay, asphalt or concrete be provided on the surface to minimize water infiltration from the exterior of the building. See Drawing 2 for Drainage and Backfill recommendations for slab-on-grade construction.

A moisture barrier, consisting of a 200 mm (8 in.) thick, compacted layer of 19 mm (3/4 in.) clear stone, should then be placed between the prepared granular sub-base and the floor slab. The installation and requirement of a vapour barrier under a concrete slab should conform to the flooring manufacturer's and designer's requirements. Moisture emission testing will be required to determine the concrete condition prior to any flooring installation on concrete slabs. To minimize the potential for excess moisture in the floor slab at the time of the flooring installation, a concrete mixture with a low water-to-cement ratio (i.e., 0.45 to 0.55) should be used. Chemical additives may be required at the time of placement to make the concrete workable and should be used in place of additional water at the point of placement. Ongoing liaison from this office will be required.

For slab-on-grade design, the modulus of subgrade reaction (k) can be taken as 100 MPa/m for the compacted stone layer over the compacted granular subbase.

The water-to-cement ratio and slump of concrete used in the floor slabs should be strictly controlled to minimize shrinkage of the slabs. Adequate joints should be provided in the floor slab to further control cracking. During placement of concrete at the construction site, testing should be performed on the concrete.

Around the perimeter of the buildings, the ground surface should be sloped on a positive grade away from the structure to promote surface water run-off and reduce groundwater infiltration adjacent to the foundations.

Perimeter drainage is required to remove any water adjacent to the exterior foundation walls. In order to prevent the build-up of water adjacent to the basement walls, it would be prudent to incorporate an exterior vertical drainage system attached to the backside of the basement walls, outletted through the wall and connected to a frost-free outlet inside the building. The exterior vertical drainage should consist of Terradrain 600 or equivalent covering the entire basement wall in order to reduce the risk of water penetration.

Assuming that the retained soil is free-draining and the backfill is drained to the base of the wall, the earth pressure acting on basement walls at any depth (h) may be computed using the following expression:

$$P = K (\gamma h + q)$$

where: P = pressure at any depth, in kPa;

K = coefficient of earth pressure considered appropriate for subsurface walls, 0.35 to 0.40;

γ = unit weight of the retained soil, use 20.0 kN/m^3 for approved granular material;

h = depth, in metres; and

q = surcharge loads, if any, in kPa.

4.6 Foundation Backfill

The existing fill at the Site is not considered suitable for re-use as foundation or subgrade backfill. Imported granular material such as OPSS Granular 'B' should be used where required.

The backfill must be brought up evenly on both sides of walls not designed to resist lateral earth pressures and the backfill materials should be compacted to 98 percent SPMDD.

During construction, the fill surface around the perimeter of structures should be sloped in such a way that the surface runoff water does not accumulate around the structure.

4.7 Site Servicing

The subgrade beneath the water and sewer pipes which will service the Site is expected to consist of dolostone bedrock. No bearing problems are anticipated for flexible or rigid pipes founded on the bedrock.

Water and sewer lines installed outside of heated areas should be provided with a minimum 1.2 m (4 ft) of soil cover or equivalent insulation for frost protection. Excavations for the construction of underground services should be conducted as recommended in Section 4.3 of this report.

For services constructed on the dolostone bedrock, the bedding should conform to OPS Standards. Bedding aggregate should be placed around the pipe to at least 300 mm (12 inch) above the pipe and be compacted to a minimum 95 percent SPMDD.

Backfill above the bedding aggregate can consist of the excavated (inorganic) soils, compacted in maximum 300 mm thick lifts to a minimum of 95 % SPMDD. For compaction of service trenches within settlement-sensitive areas, the backfill within the top metre should be compacted to a minimum of 98 % SPMDD. A program of *in situ* density testing should be set up to ensure that satisfactory levels of compaction are achieved. The use of any imported material is subject to review and approval by the geotechnical consultant.

As indicated previously, the disposal of any excavated materials off site should conform to current MECP guidelines.

4.8 Earthquake Design Considerations

The recommendations for the geotechnical aspects to determine the earthquake loading for design using the OBC 2012 are presented below.

The subsoil and groundwater information at this Site have been examined in relation to Section 4.1.8.4 of the OBC 2012. The subsoils at the Site generally consist of thin topsoil and fill overlying dolostone bedrock. It is anticipated that the proposed structures will be founded on the dolostone bedrock.

Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2012 indicated that to determine the site classification, the average properties in the top 30 m (below the lowest basement level) are to be used. The boreholes for this investigation were advanced to a maximum depth of about 5.8 m below existing grade. Therefore, the Site Classification recommendation would be based on the available information as well as our interpretation of conditions below the boreholes based on our knowledge of the soil conditions in the area.

Based on the presence of shallow bedrock and the known local geological conditions, the Site Class for the proposed development is “B” as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012.

4.9 Site Pavement Design

Areas to be paved should be stripped of all fill as noted in Section 4.2. All fill required to raise the subgrade to design levels must conform to requirements outlined previously. Provided the preceding recommendations are followed, the pavement thickness design requirements given in the following table are recommended for the anticipated site use and the anticipated subgrade conditions.

Table 3 – Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Light-Duty Pavement Structure (Cars Only)	Heavy-Duty Pavement Structure (Cars and Trucks)
Asphaltic Concrete	92% MRD[1] or 97% BRD[1]	40 mm HL-3 50 mm HL-8	50 mm HL-3 60 mm HL-8
Granular 'A' (Base)	100% SPMDD[1]	150 mm	150 mm
Granular 'B' (Base)	100% SPMDD[1]	200 mm	3000 mm

*Notes: 1) SPMDD denotes Standard Proctor Maximum Dry Density, MRD denotes Maximum Relative Density, BRD denotes Bulk Relative Density.
2) The subgrade must be compacted to 98% SPMDD.
3) The above recommendations are minimum requirements.

The recommended pavement structures provided in the above table are based on the properties of the expected subgrade material. Consequently, the recommended pavement structures should be considered for preliminary design purposes only. Other granular configurations may also be possible provided the granular base equivalency (GBE) thickness is maintained. These recommendations on thickness design are not intended to support heavy and concentrated construction traffic, particularly where only a portion of the pavement section is installed.

If construction is undertaken under adverse weather conditions (i.e., wet or freezing conditions) subgrade preparation and granular sub-base requirements should be reviewed by the Geotechnical Engineer.

If only a portion of the pavement will be in place during construction, the granular subbase may have to be thickened, and/or the subgrade improved with a geotextile separator or geogrid stabilizing layer. This is best determined in the field during the site servicing stage of construction, prior to road construction.

Samples of both the Granular 'A' and Granular 'B' aggregates should be checked for conformance to OPSS 1010 and City of Cambridge Standards prior to use on Site, and during construction. The Granular 'B' subbase and the Granular 'A' base courses must be compacted to 100 percent SPMDD.

The asphaltic concrete paving materials should conform to the requirements of OPSS 1150. The asphalt should be placed in accordance with OPSS 310 and compacted to at least 97 percent of the Marshall mix design bulk relative density or 92% of maximum relative density. A tack coat should be applied between the surface and binder asphalt courses.

Good drainage provisions will optimize pavement performance. The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum grade of two percent) to provide effective surface drainage toward catch basins. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas. In low areas, sub-drains should be installed to intercept excess subsurface moisture and prevent subgrade softening, as shown on Drawing 3. This is particularly important in heavier traffic areas at the site entrances. The locations and extent of sub-drainage required within the paved areas should be reviewed by this office in conjunction with the proposed grading.

A program of *in situ* density testing must be carried out to verify that satisfactory levels of compaction are being achieved.

To minimize the effects of differential settlements of service trench fill, it is recommended that wherever practical, placement of binder asphalt be delayed for approximately six months after the granular sub-base is put down. The surface course asphalt should be delayed for a further one year. Prior to the surface asphalt being placed, it is recommended that a pavement evaluation be carried out on the base asphalt to identify repair areas or areas requiring remedial works prior to surface asphalt being placed.

4.10 Inspection and Testing Requirements

An effective Inspection and Testing program is an essential part of construction monitoring. For this project, the Inspection and Testing Program typically includes the following items:

- Inspection and Materials testing during site servicing works, including soil sampling, laboratory testing (moisture contents and Standard Proctor density test on the pipe bedding and trench backfill) and in situ density testing
- Footing Base Examinations to confirm suitability to support the design bearing pressures; and, visual examination of concrete reinforcing steel placement in structural elements
- Inspection and testing for underfloor subgrade and granular placement
- Materials testing for concrete foundations, floor slab, walls, columns
- Inspection and Materials testing during paved area construction, including subgrade examination of the paved area subgrade soils following site servicing, laboratory testing (grain size analyses and Standard Proctor density tests on the Granular A and B material placed on site roadways), and in situ density testing
- Inspection and Materials testing for base and surface asphalt, including laboratory testing on asphalt samples

EXP would be pleased to prepare an inspection and testing work program prior to construction, incorporating the above items.

5. General Comments

The information presented in this report is based on a limited investigation designed to provide information to support an assessment of the current geotechnical conditions within the subject property. The conclusions and recommendations presented in this report reflect site conditions existing at the time of the investigation. Consequently, during the future development of the property, conditions not observed during this investigation may become apparent. Should this occur, EXP Services Inc. should be contacted to assess the situation, and the need for additional testing and reporting. EXP has qualified personnel to provide assistance in regards to any future geotechnical and environmental issues related to this property.

Our undertaking at EXP, therefore, is to perform our work within limits prescribed by our clients, with the usual thoroughness and competence of the engineering profession.

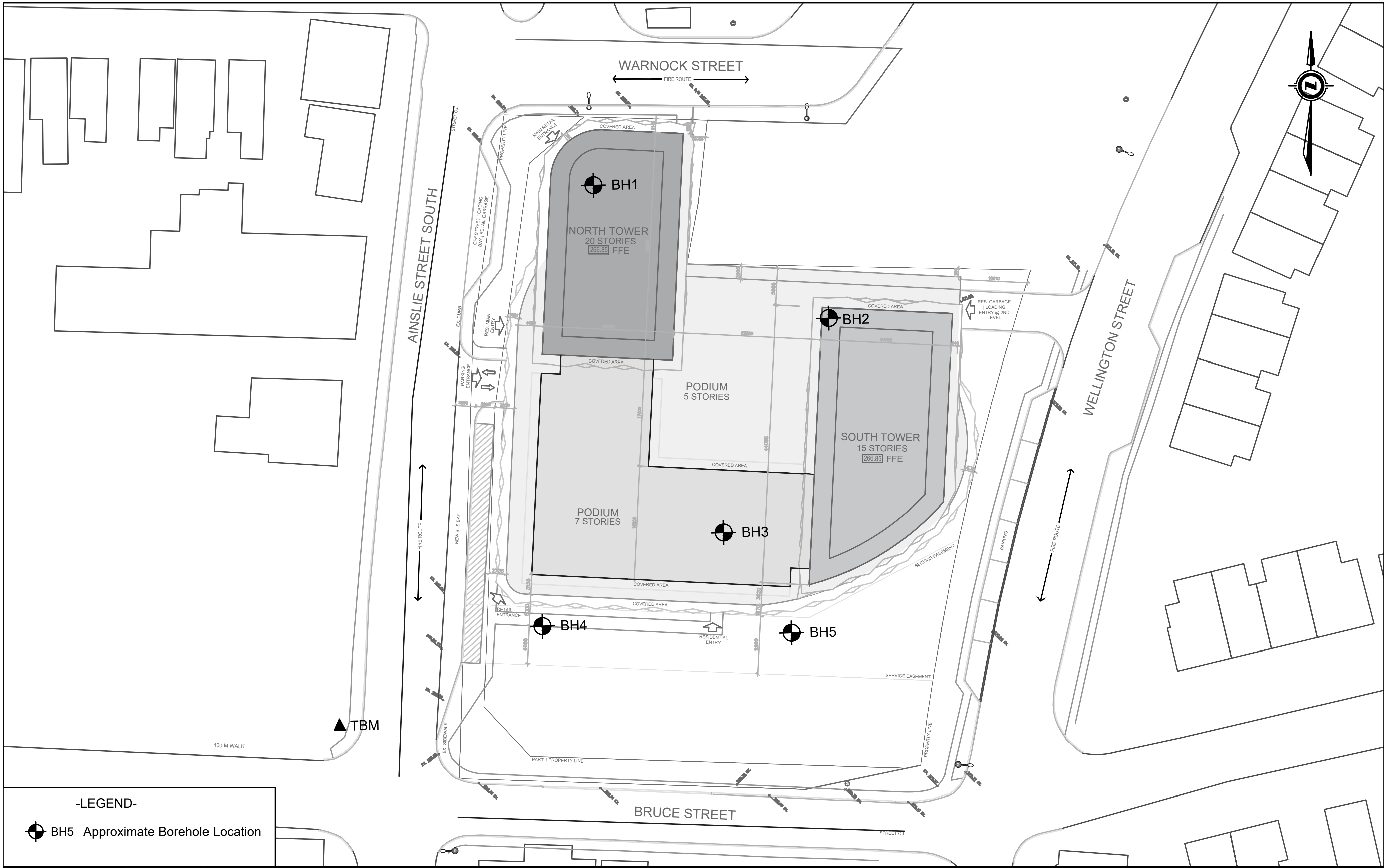
The comments given in this report are intended only for the guidance of design engineers. The number of test holes required to determine the localized underground conditions between test holes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

EXP Services Inc. should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not afforded the privilege of making this review, EXP Services Inc. will assume no responsibility for interpretation of the recommendations in this report.

This report was prepared for the exclusive use of **Atlantic Development Group Ltd.** and may not be reproduced in whole or in part, without the prior written consent of EXP, or used or relied upon in whole or in part by other parties for any purposes whatsoever. Any use which a third party makes of this report, or any part thereof, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. EXP Services Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

Drawings



-LEGEND-

● BH5 Approximate Borehole Location

-NOTES-

1. The boundaries and soil types have been established only at test hole locations. Between test holes they are assumed and may be subject to considerable error.
2. Soil samples will be retained in storage for 3 months and then destroyed unless client advises that an extended time period is required.
3. Topsoil quantities should not be established from the information provided at the test hole locations.
4. The site plan was reproduced from a drawing provided by the client and should be read in conjunction with EXP Geotechnical Report LON-00017843-GE.
5. Boreholes were surveyed to the top of spindle on the fire hydrant at the northwest corner of the intersection of Bruce Street and Ainslie Street South (Assumed Elevation: 100.0m)

Geotechnical Investigation

Proposed Development

69 Ainslie Street South,
Cambridge, Ontario

CLIENT
Atlantic Development Group Ltd.

TITLE
Borehole Location Plan

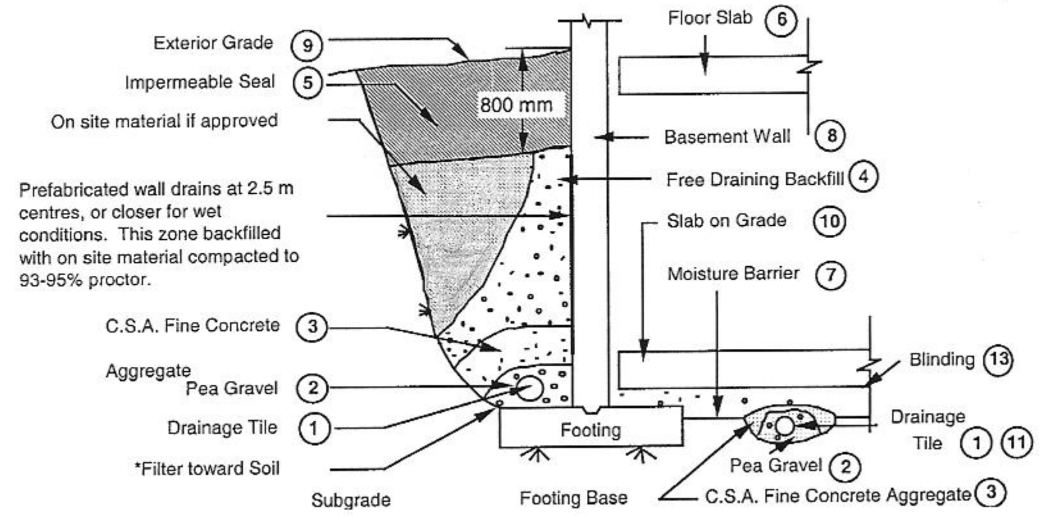
Prepared By: M.B.

Reviewed By: G.F.

EXP Services Inc.
15701 Robin's Hill Road, London, ON, N5V 0A5

DATE	SCALE	PROJECT NO.	DWG.
JULY 2020	NTS	LON-00017843-GE	1

BACKFILL AND BASEMENT DRAINAGE DETAIL
(NOT TO SCALE)

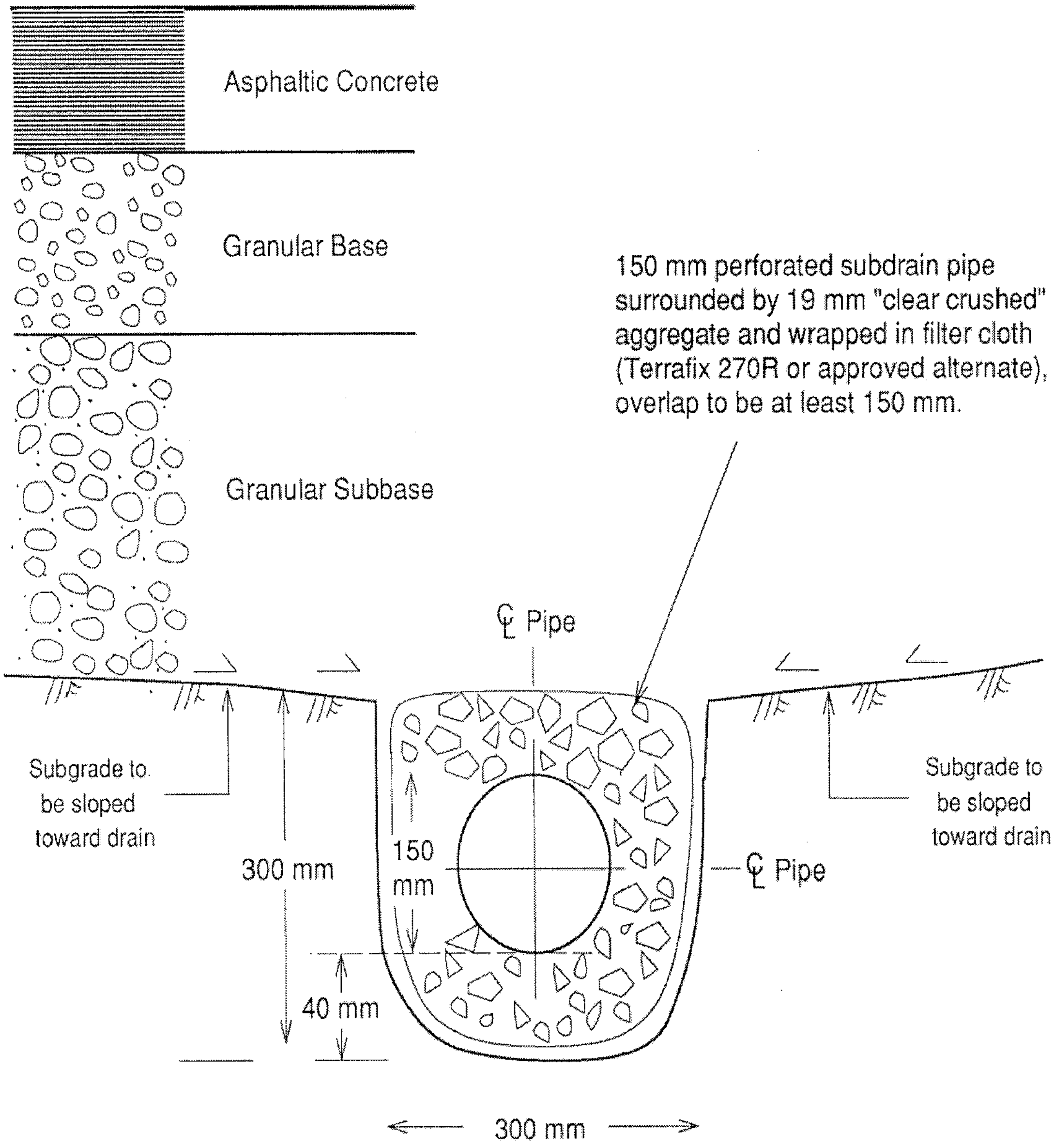


NOTES:

1. Drainage tile to consist of 100 mm (4 in.) diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet. Invert to be minimum of 150 mm (6 in.) below underside of floor slab.
2. Pea gravel 150 mm (6 in.) top and sides of drain. If drain is not on footing, place 100 mm (4 in.) of pea gravel below drain. 20 mm (3/4 in.) clear stone may be used provided if it is covered by an approved porous geotextile fabric membrane (Terrafix 270R or equivalent).
3. C.S.A. fine concrete aggregate to act as filter material. Minimum 300 mm (12 in.) top and side of drain. This may be replaced by an approved porous geotextile membrane (Terrafix 270R or equivalent).
4. Free-draining backfill - OPSS Granular ‘B’ or equivalent compacted to 93 to 95 (maximum) percent Standard Proctor density. Do not compact closer than 1.8 m (6 ft) from wall with heavy equipment. Use hand-controlled light compaction equipment within 1.8 m (6 ft) of wall.
5. Impermeable backfill seal of compacted clay, clayey silt or equivalent. If original soil is free-draining, seal may be omitted.
6. Do not backfill until wall is supported by basement and floor slabs or adequate bracing.
7. Moisture barrier to consist of compacted 20 mm (3/4 in.) clear stone or equivalent free-draining material. Layer to be 200 mm (8 in.) minimum thickness.
8. Basement walls to be damp-proofed.
9. Exterior grade to slope away from wall.
10. Slab-on-grade should not be structurally connected to wall or footing.
11. Underfloor drain invert to be at least 300 mm (12 in.) below underside of floor slab. Drainage tile placed in parallel rows 6 to 8 m (20 to 25 ft.) centres one way. Place drain on 100 mm (4 in.) of pea gravel with 150 mm (6 in.) of pea gravel top and sides. CSA fine concrete aggregate to be provided as filter material or an approved porous geotextile membrane (as in 2 above) may be used.
12. Do not connect the underfloor drains to perimeter drains.
13. If the 20 mm (3/4 in.) clear stone requires surface binding, use 6 mm (1/4 in.) clear stone chips.

Note:
- a) Underfloor drainage can be deleted where not required (see report).
- b) Free-draining backfill, item 4 may be replaced by wall drains, as indicated, if more economical.

PAVEMENT SUBDRAIN DETAIL



NOTES:

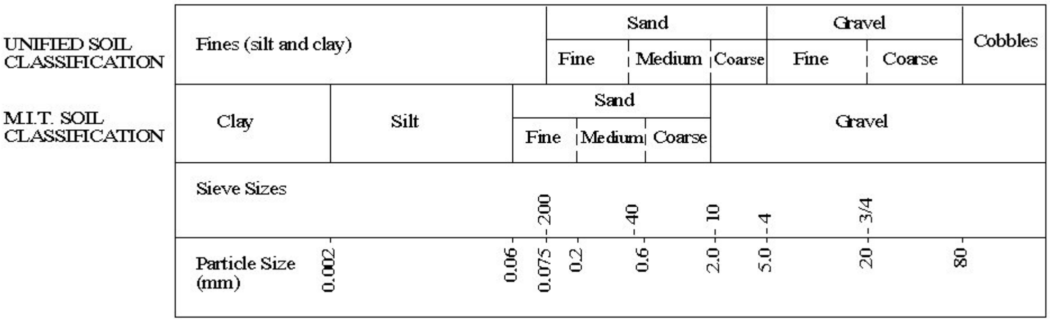
1. All dimensions in millimetres.
2. All subdrains to be set on at least 1% grade draining to a positive outlet.
3. Subgrade soil conditions should be verified onsite, during subgrade preparation works, following site servicing installations.

Scale: N.T.S.

Appendix A – Borehole Logs

NOTES ON SAMPLE DESCRIPTIONS

1. All descriptions included in this report follow the 'modified' Massachusetts Institute of Technology (M.I.T.) soil classification system. The laboratory grain-size analysis also follows this classification system. Others may designate the Unified Classification System as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain size analysis has been carried out, all samples are classified visually and the accuracy of the visual examination is not sufficient to differentiate between the classification systems or exact grain sizing. The M.I.T. system has been modified and the EXP classification includes a designation for cobbles above the 75 mm size and boulders above the 200 mm size.



2. **Fill**: Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description therefore, may not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces or subsurface basements, floors, tanks, even though none of these obstructions may have been encountered in the borehole. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. The fill at this site has been monitored for the presence of methane gas and the results are recorded on the borehole logs. The monitoring process neither indicates the volume of gas that can be potentially generated or pinpoints the source of the gas. These readings are to advise of a potential or existing problem (if they exist) and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic waste that renders the material unacceptable for deposition in any but designated land fill sites; unless specifically stated, the fill on the site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common, but not detectable using conventional geotechnical procedures.

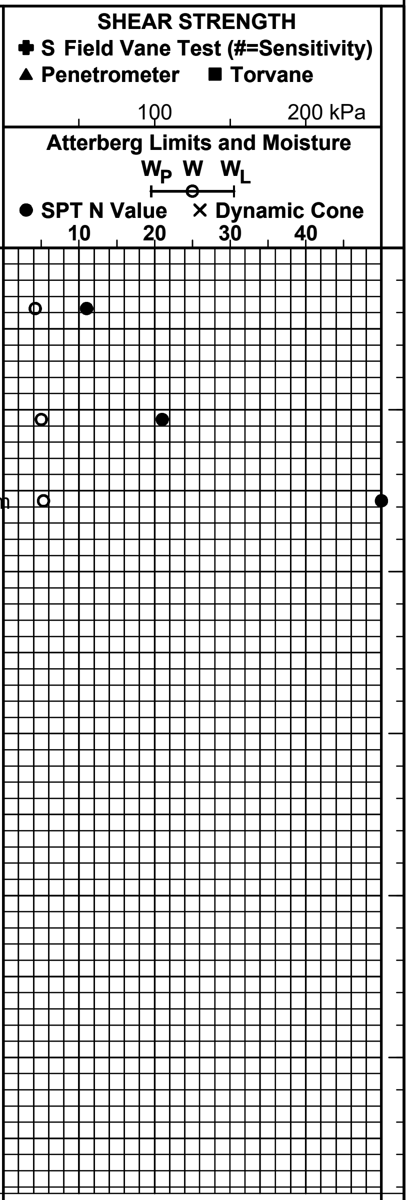
3. **Glacial Till**: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process, the till must be considered heterogeneous in composition and as such, may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (75 to 200 mm in diameter) or boulders (greater than 200 mm diameter) and therefore, contractors may encounter them during excavation, even if they are not indicated on the borehole logs. It should be appreciated that normal sampling equipment can not differentiate the size or type of obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited area; therefore, caution is essential when dealing with sensitive excavations or dewatering programs in till material.

BOREHOLE LOG

BH1
Sheet 1 of 1

PROJECT **Geotechnical Investigation, 69 Ainslie Street South, Cambridge, ON** PROJECT NO. **LON-00017843-GE**
CLIENT **Atlantic Development Group Ltd.** DATUM **Local**
DRILL TYPE/METHOD **H.S. Augers/Diamond Core** DATES: Boring **Mar 26, 2020** Water Level **Mar 26, 2020**

DEPTH (m)	ELEVATION (m) 99.9	STRATA DESCRIPTION	TYPE	NUMBER	RECOVERY (mm) or (%)	N VALUE (blows) or RQD (%)	PENETROMETER (kPa)
0	99.82	**ASPHALT:**~75 mm thick					
		FILL: Sand, brown, some gravel, with layer of black coal and cinders, moist, compact	▨	S1	355	11	
	99.09	**SAND:** Brown, some gravel, trace silt, moist, compact	▨	S2	460	21	
	98.24	- becoming greyish-brown, some gravel, trace cobbles,	▨	S3	75	50 /125 mm	
		DOLOSTONELight grey with some dark brown banding, fine to medium crystalline and fossiliferous, trace crystal-filled vugs, bedding joints are rough planar with spacing at 50 to 100 mm (and with discontinuous vertical joints) above 3.0 m and 50 to 150 mm below 3.0 m, fragments are hard	◫	S4	680	15	
			◫	S5	250	0	
			◫	S6	1400	35	
			◫	S7	1530	40	
6	94.04	**End of Borehole at 5.84 m depth.**					



NOTES

1) Borehole interpretation requires assistance by EXP before use by others. Borehole Logs must be read in conjunction with EXP Report LON-00017843-GE. For definition of terms used on logs, see sheets prior to logs.

2) Upon completion of augering and prior to coring, no groundwater noted to 1.8 m.

SAMPLE LEGEND
☒ AS Auger Sample ▨ SS Split Spoon ■ ST Shelby Tube
◫ Rock Core (eg. BQ, NQ, etc.) ◫ VN Vane Sample

OTHER TESTS

G Specific Gravity	C Consolidation
H Hydrometer	CD Consolidated Drained Triaxial
S Sieve Analysis	CU Consolidated Undrained Triaxial
γ Unit Weight	UU Unconsolidated Undrained Triaxial
P Field Permeability	UC Unconfined Compression
K Lab Permeability	DS Direct Shear

WATER LEVELS
▽ Apparent ▼ Measured ▲ Artesian (see Notes)

BOREHOLE LOG

BH2
Sheet 1 of 1

PROJECT **Geotechnical Investigation, 69 Ainslie Street South, Cambridge, ON**
PROJECT NO. **LON-00017843-GE**
CLIENT **Atlantic Development Group Ltd.**
DATUM **Local**
DRILL TYPE/METHOD **H.S. Augers/Diamond Core**
DATES: Boring **Mar 26, 2020**
Water Level **Mar 26, 2020**

DEPTH (m)	ELEVATION (m) 100.2	STRATA DESCRIPTION	STRATA PLOT	WELL LOG	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm) or (%)	SAMPLES N VALUE (blows) or RQD (%)	PENETROMETER (kPa)	SHEAR STRENGTH ✚ S Field Vane Test (#=Sensitivity) ▲ Penetrometer ■ Torvane 100 200 kPa; Atterberg Limits and Moisture W_P W W_L; ● SPT N Value × Dynamic Cone 10 20 30 40
0	100.09	**ASPHALT:**~75 mm thick								
	100.01	**FIL L:** Sand and gravel, moist, compact			▨	S1	75	50	75 mm	●
		End of Borehole at 0.15 m depth with auger refusal on assumed bedrock.								
1										
2										
3										
4										
5										
6										
7										

NOTES

1) Borehole interpretation requires assistance by EXP before use by others. Borehole Logs must be read in conjunction with EXP Report LON-00017843-GE. For definition of terms used on logs, see sheets prior to logs.

2) Upon completion, borehole open to 0.15 m, no groundwater observed.

SAMPLE LEGEND
☒ AS Auger Sample ▨ SS Split Spoon ■ ST Shelby Tube
◫ Rock Core (eg. BQ, NQ, etc.) ◫ VN Vane Sample

OTHER TESTS

G Specific Gravity	C Consolidation
H Hydrometer	CD Consolidated Drained Triaxial
S Sieve Analysis	CU Consolidated Undrained Triaxial
γ Unit Weight	UU Unconsolidated Undrained Triaxial
P Field Permeability	UC Unconfined Compression
K Lab Permeability	DS Direct Shear

WATER LEVELS
▽ Apparent ▼ Measured ▲ Artesian (see Notes)

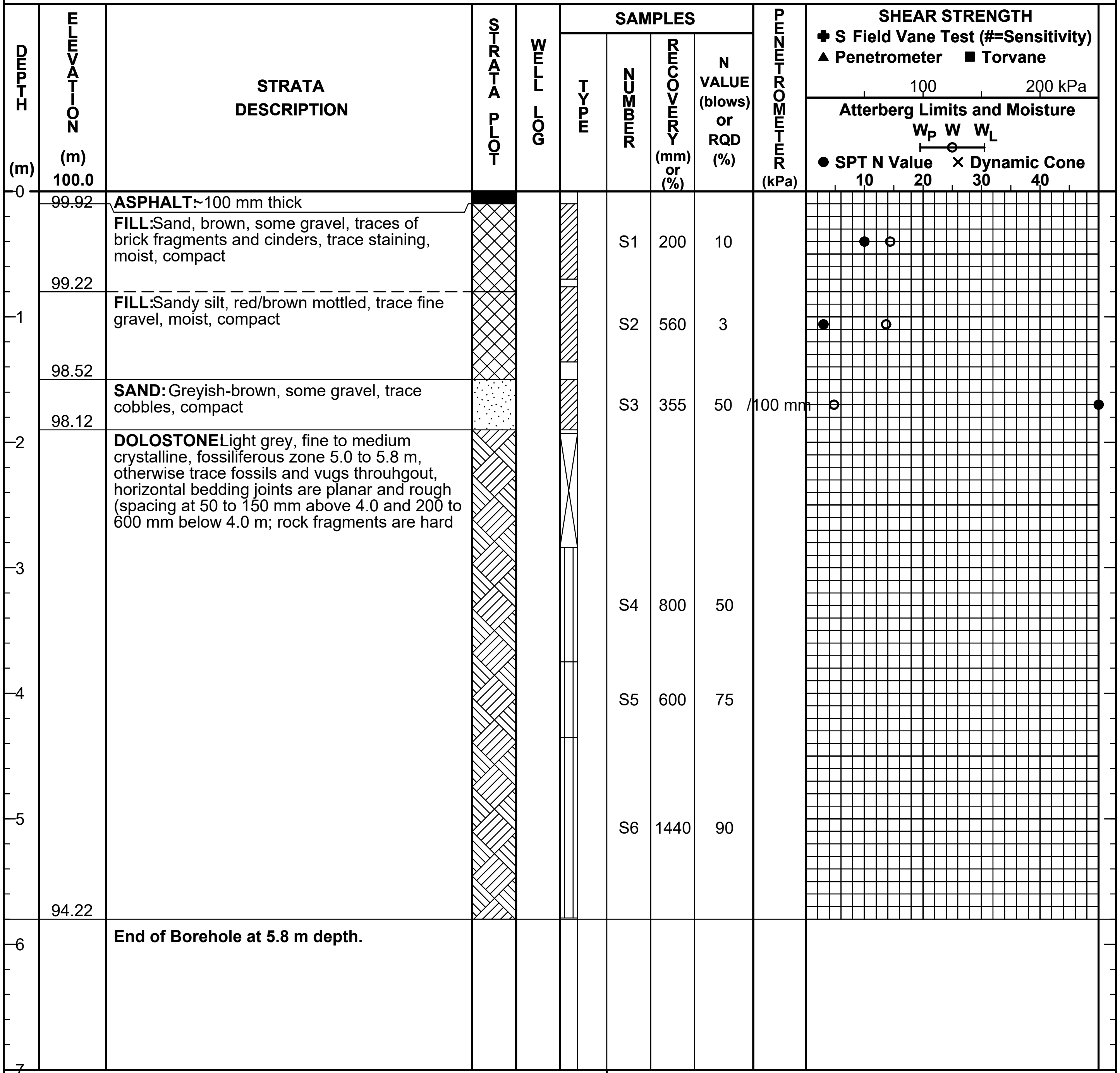
BOREHOLE LOG

BH3
Sheet 1 of 1

PROJECT **Geotechnical Investigation, 69 Ainslie Street South, Cambridge, ON** PROJECT NO. **LON-00017843-GE**

CLIENT **Atlantic Development Group Ltd.** DATUM **Local**

DRILL TYPE/METHOD **H.S. Augers/Diamond Core** DATES: Boring **Mar 25, 2020** Water Level **Mar 25, 2020**



DEPTH (m)	ELEVATION (m) 100.0	STRATA DESCRIPTION	SAMPLES NUMBER	RECOVERY (mm) or (%)	N VALUE (blows) or RQD (%)	PENETROMETER (kPa)
0	99.92	**ASPHALT:** ~100 mm thick				
	99.22	**FILL:** Sand, brown, some gravel, traces of brick fragments and cinders, trace staining, moist, compact	S1	200	10	
1	98.52	**FILL:** Sandy silt, red/brown mottled, trace fine gravel, moist, compact	S2	560	3	
	98.12	**SAND:** Greyish-brown, some gravel, trace cobbles, compact	S3	355	50 /100 mm	
2	94.22	**DOLOSTONE** Light grey, fine to medium crystalline, fossiliferous zone 5.0 to 5.8 m, otherwise trace fossils and vugs throuhgout, horizontal bedding joints are planar and rough (spacing at 50 to 150 mm above 4.0 and 200 to 600 mm below 4.0 m; rock fragments are hard	S4 S5 S6	800 600 1440	50 75 90	
6		**End of Borehole at 5.8 m depth.**				

NOTES

1) Borehole interpretation requires assistance by EXP before use by others. Borehole Logs must be read in conjunction with EXP Report LON-00017843-GE. For definition of terms used on logs, see sheets prior to logs.

2) Upon completion of augering and prior to coring, no groundwater observed to 2.7 m.

SAMPLE LEGEND
☒ AS Auger Sample ▨ SS Split Spoon ■ ST Shelby Tube
◫ Rock Core (eg. BQ, NQ, etc.) ◫ VN Vane Sample

OTHER TESTS

G Specific Gravity	C Consolidation
H Hydrometer	CD Consolidated Drained Triaxial
S Sieve Analysis	CU Consolidated Undrained Triaxial
γ Unit Weight	UU Unconsolidated Undrained Triaxial
P Field Permeability	UC Unconfined Compression
K Lab Permeability	DS Direct Shear

WATER LEVELS
▽ Apparent ▼ Measured ▲ Artesian (see Notes)

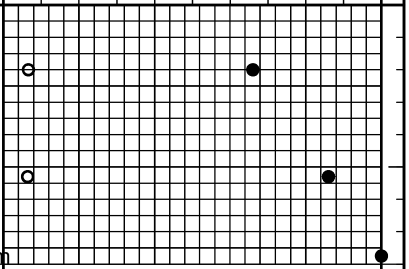
BH4
Sheet 1 of 1

BOREHOLE LOG

PROJECT **Geotechnical Investigation, 69 Ainslie Street South, Cambridge, ON** PROJECT NO. **LON-00017843-GE**
CLIENT **Atlantic Development Group Ltd.** DATUM **Local**
DRILL TYPE/METHOD **H.S. Augers/Diamond Core** DATES: Boring **Mar 25, 2020** Water Level **Mar 25, 2020**

DEPTH (m)	ELEVATION (m) 99.9	STRATA DESCRIPTION	SAMPLES TYPE	NUMBER	RECOVERY (mm) or (%)	N VALUE (blows) or RQD (%)	PENETROMETER (kPa)
0	99.75	**ASPHALT:**~100 mm thick					
		FILL: Sand, brown, some gravel, moist, dense	▨	S1	460	33	
	99.05	**SAND:** Sand, some gravel, trace cobbles, moist, dense	▨	S2	600	43	
	98.35 98.25	**DOLOSTONE:**Assumed bedrock, fragments, damp	▨	S3	50	50 /100 mm	
		End of Borehole at 1.6 m depth.					

SHEAR STRENGTH
✚ S Field Vane Test (#=Sensitivity)
▲ Penetrometer ■ Torvane
100 200 kPa
Atterberg Limits and Moisture
W_P W W_L
● SPT N Value × Dynamic Cone
10 20 30 40



NOTES

1) Borehole interpretation requires assistance by EXP before use by others. Borehole Logs must be read in conjunction with EXP Report LON-00017843-GE. For definition of terms used on logs, see sheets prior to logs.

2) Upon completion, borehole open to 1.6 m, no groundwater observed.

SAMPLE LEGEND
☒ AS Auger Sample ▨ SS Split Spoon ■ ST Shelby Tube
◫ Rock Core (eg. BQ, NQ, etc.) ◫ VN Vane Sample

OTHER TESTS

G Specific Gravity	C Consolidation
H Hydrometer	CD Consolidated Drained Triaxial
S Sieve Analysis	CU Consolidated Undrained Triaxial
γ Unit Weight	UU Unconsolidated Undrained Triaxial
P Field Permeability	UC Unconfined Compression
K Lab Permeability	DS Direct Shear

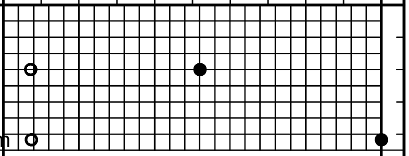
WATER LEVELS
▽ Apparent ▼ Measured ▲ Artesian (see Notes)

BOREHOLE LOG

BH5
Sheet 1 of 1

PROJECT **Geotechnical Investigation, 69 Ainslie Street South, Cambridge, ON** PROJECT NO. **LON-00017843-GE**
CLIENT **Atlantic Development Group Ltd.** DATUM **Local**
DRILL TYPE/METHOD **H.S. Augers/Diamond Core** DATES: Boring **Mar 25, 2020** Water Level **Mar 25, 2020**

DEPTH (m)	ELEVATION (m) 100.0	STRATA DESCRIPTION	STRATA PLOT	WELL LOG	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm) or (%)	SAMPLES N VALUE (blows) or RQD (%)	PENETROMETER (kPa)	SHEAR STRENGTH ✚ S Field Vane Test (#=Sensitivity) ▲ Penetrometer ■ Torvane 100 200 kPa Atterberg Limits and Moisture W_P W W_L ● SPT N Value × Dynamic Cone 10 20 30 40
0	99.86	**ASPHALT:**~100 mm thick								
		FILL: Sand, brown, some gravel, trace cobbles, moist, compact			▨	S1	450	26		
	99.09				▨	S1	50	50 /125 mm		
1	99.05	**DOLOSTONE:**Grey fragments, some sand, damp **End of Borehole at 0.91 m depth on sampler refusal on assumed bedrock.**								
2										
3										
4										
5										
6										
7										



NOTES

1) Borehole interpretation requires assistance by EXP before use by others. Borehole Logs must be read in conjunction with EXP Report LON-00017843-GE. For definition of terms used on logs, see sheets prior to logs.

2) Upon completion, borehole open to 0.9 m, no groundwater observed.

SAMPLE LEGEND
☒ AS Auger Sample ▨ SS Split Spoon ■ ST Shelby Tube
◫ Rock Core (eg. BQ, NQ, etc.) ◫ VN Vane Sample

OTHER TESTS

G Specific Gravity	C Consolidation
H Hydrometer	CD Consolidated Drained Triaxial
S Sieve Analysis	CU Consolidated Undrained Triaxial
γ Unit Weight	UU Unconsolidated Undrained Triaxial
P Field Permeability	UC Unconfined Compression
K Lab Permeability	DS Direct Shear

WATER LEVELS
▽ Apparent ▼ Measured ▲ Artesian (see Notes)

Appendix B – Limitations and Use of Report

LIMITATIONS AND USE OF REPORT

BASIS OF REPORT

This report (“Report”) is based on site conditions known or inferred by the geotechnical investigation undertaken as of the date of the Report. Should changes occur which potentially impact the geotechnical condition of the site, or if construction is implemented more than one year following the date of the Report, the recommendations of EXP may require re-evaluation.

The Report is provided solely for the guidance of design engineers and on the assumption that the design will be in accordance with applicable codes and standards. Any changes in the design features which potentially impact the geotechnical analyses or issues concerning the geotechnical aspects of applicable codes and standards will necessitate a review of the design by EXP. Additional field work and reporting may also be required.

Where applicable, recommended field services are the minimum necessary to ascertain that construction is being carried out in general conformity with building code guidelines, generally accepted practices and EXP’s recommendations. Any reduction in the level of services recommended will result in EXP providing qualified opinions regarding the adequacy of the work. EXP can assist design professionals or contractors retained by the Client to review applicable plans, drawings, and specifications as they relate to the Report or to conduct field reviews during construction.

Contractors contemplating work on the site are responsible for conducting an independent investigation and interpretation of the borehole results contained in the Report. The number of boreholes necessary to determine the localized underground conditions as they impact construction costs, techniques, sequencing, equipment and scheduling may be greater than those carried out for the purpose of the Report.

Classification and identification of soils, rocks, geological units, contaminant materials, building envelopment assessments, and engineering estimates are based on investigations performed in accordance with the standard of care set out below and require the exercise of judgment. As a result, even comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations or building envelope descriptions involve an inherent risk that some conditions will not be detected. All documents or records summarizing investigations are based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated. Some conditions are subject to change over time. The Report presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, these should be disclosed to EXP to allow for additional or special investigations to be undertaken not otherwise within the scope of investigation conducted for the purpose of the Report.

RELIANCE ON INFORMATION PROVIDED

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to EXP by the Client and others. The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose as communicated by the Client. EXP has relied in good faith upon such representations, information and instructions and accepts no responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of any misstatements, omissions, misrepresentation or fraudulent acts of persons providing information. Unless specifically stated otherwise, the applicability and reliability of the findings, recommendations, suggestions or opinions expressed in the Report are only valid to the extent that there has been no material alteration to or variation from any of the information provided to EXP.

STANDARD OF CARE

The Report has been prepared in a manner consistent with the degree of care and skill exercised by engineering consultants currently practicing under similar circumstances and locale. No other warranty, expressed or implied, is made. Unless specifically stated otherwise, the Report does not contain environmental consulting advice.

COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment form part of the Report. This material includes, but is not limited to, the terms of reference given to EXP by its client (“Client”), communications between EXP and the Client, other reports, proposals or documents prepared by EXP for the Client in connection with the site described in the Report. In order to properly understand the suggestions, recommendations and opinions expressed in the Report, reference must be made to the Report in its entirety. EXP is not responsible for use by any party of portions of the Report.

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The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. No other party may use or rely upon the Report in whole or in part without the written consent of EXP. Any use of the Report, or any portion of the Report, by a third party are the sole responsibility of such third party. EXP is not responsible for damages suffered by any third party resulting from unauthorized use of the Report.

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