Geotechnical Investigation Report

Proposed Norwood Park Subdivision - Phase 4, Albine Street, Norwood, Ontario

DPH Developments Inc.

8 October 2021

The Power of Commitment

Contents

Table Index

Figure Index

Appendices

1. Introduction

GHD Limited (GHD) has been retained by DPH Developments Inc. (the Client), being represented by Engage Engineering Ltd. (EEL) to conduct a geotechnical investigation for the design and construction of phase 4 of the proposed Norwood Park subdivision along Albine Street and west of Keeler Court (phase 3), in Norwood, Ontario (the Site). The Site location map is presented as **Figure 1** in the attachment section of this report.

It is GHD's expectation that the proposed development will consist of typical 1- and 2-storey residences, with or without basements, with associated asphalt-paved roadways, and in-ground servicing. A preliminary conceptual plan provided by the Client illustrated the site location and proposed road network and lot layout. Further details of the proposed development, such as site grading plans and servicing plans, were not available at the time of preparation of this report.

The purpose of the geotechnical investigation was to assess the subsurface soil and groundwater conditions within the proposed development area, and to provide geotechnical engineering recommendations relevant to earthwork construction, reuse of existing soils as backfill material, service installation, infiltration rates for potential Low Impact Development (LID) features, storm water management (SWM) pond design, foundations, slabs, and pavement structure for roadway construction. The geotechnical investigation was completed in general accordance with our proposal PG-5227 dated July 6, 2021.

The factual data, interpretations and preliminary recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or site location. This report should be read in conjunction with the Statement of Limitations appended to this report. The reader's attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report.

2. GHD's Scope of Work

GHD's scope of work was carried out from July to October 2021 and involved the following tasks:

- Pre-Planning activities:
  - Preparation of a Site-specific health and safety plan (HASP).
  - Completion of underground utility locate clearances (public and private).
- Field activities:
  - Advancement of four (4) boreholes identified as MW1-21, MW2-21, MW3-21 and BH4-21 as shown on **Figure 2** to depths ranging between 2.7 to 4.0 metres below ground surface (mbgs). Monitoring wells were installed in three (3) of the borehole locations (MW1-21, MW2-21, and MW3-21) to facilitate groundwater monitoring and testing.
  - Advancement of eleven (11) test pits identified as TP1-21 through TP11-21 on **Figure 2** to depths ranging between 1.0 to 3.7 mbgs.
  - Standard Penetration Test (SPT) and associated split spoon soil sampling in accordance with ASTM D1586.
  - Hydraulic testing (i.e. single well response slug testing) was completed at the three (3) monitoring wells and permeameter testing was completed at three test pit locations, to evaluate hydraulic conductivity for potential dewatering and infiltration rates for potential Low Impact Development (LID) design.
- Completion of geotechnical laboratory testing on the selected soil samples.
- Preparation of geotechnical investigations report (factual data, analysis and recommendations).

3. Methodology

3.1 Safety Planning

Upon project initiation, a Site-specific Health and Safety Plan (HASP) was prepared for implementation during the field investigation program. The HASP presents the visually observed Site conditions to identify potential physical hazards to field personnel. Required personal protective equipment was also listed in the HASP. It is mandatory for all GHD personnel involved in the field program, to read the HASP and have a copy of the HASP available at the Site during the investigative work. Health and Safety requirements in the HASP were implemented during the field investigation program.

In addition to the abovementioned safety measures, GHD's safety protocol related to COVID-19 issues was implemented and all preventive measures were reinforced. GHD and sub-contractor maintained the required social distancing throughout the field investigations.

3.2 Utility Clearance

GHD completed a pre-drilling Site visit to review the Site conditions and access restrictions. Based on the limits of approach, the test holes were positioned appropriately to avoid potential obstructions. The test holes were placed in the field based on the preliminary conceptual plan.

Prior to initiating the subsurface investigation activities, all applicable utility companies (gas, hydro, network cables, water, waste water, etc.) were contacted, to demarcate the location of their respective underground utilities to ensure that service lines would not be damaged during the investigative works.

GHD also retained a specialist private services locator (Utility Marx) to locate any underground private utilities that could potentially be present at the Site. The proposed boreholes and test pits were positioned at appropriate locations to avoid existing service lines.

3.3 Field Investigation

The test hole program associated with the geotechnical investigation was conducted on August 18th and August 27th, 2021. The subsurface investigation consisted of advancing a total of four (4) exploratory boreholes and eleven (11) test pits. Monitoring wells were installed in three (3) of borehole locations. The test hole locations are presented on **Figure 2**.

GHD's safety protocol related to COVID-19 issues was implemented and all preventive measures were applied for the bringing of samples into the lab. The sample bags were decontaminated before carrying out the sample review and laboratory testing.

3.3.1 Test Hole Advancement and Sample Collection

The boreholes were located as shown in **Figure 2**. The drilling work was carried out by a track mounted drilling rig, supplied, and operated by Landshark Drilling under the full-time supervision of a GHD experienced technical representative. The boreholes were advanced to depths ranging between approximately 2.7 to 4.0 mbgs.

The boreholes were advanced using continuous solid stem augers and soil samples were collected using a 50 millimetre (mm) outside diameter split spoon sampler in general accordance with the specifications of the Standard Penetration Test Method (ASTM D1586). The relative density or consistency of the subsurface soil layers were measured using the Standard Penetration Test (SPT) method, by counting the number of blows ('N') required to drive a conventional split barrel soil sampler 300 mm depth.

Monitoring wells were installed in three (3) borehole locations (MW1-21, MW2-21 and MW3-21). The monitoring wells consist of a 50 mm diameter polyvinyl chloride (PVC) slotted well screen and completed to the ground surface using a riser pipe. A silica sand pack was placed in the annular space between the PVC screen and riser pipe to approximately 0.3 m above the top of the screen. A bentonite seal was installed in the remaining borehole annulus above the sand pack.

The test pits were located as shown in **Figure 2**. The test pitting was carried out by a track mounted excavator, supplied, and operated by Drain Brothers Excavating Ltd under the full-time supervision of a GHD experienced technical representative. The test pits were advanced to bedrock at depths ranging between approximately 1.1 to 3.7 mbgs.

Groundwater level observations and measurements were made in the test holes as drilling and test pits proceeded and upon completion of the test holes. The observed conditions and measured groundwater levels are provided in the logs of the drilled boreholes and excavated test pits. The boreholes in which a monitoring well was not installed were backfilled upon completion and sealed in accordance with Ontario Regulation 903.

The GHD technical representative logged the soil samples and bedrock encountered in the test holes and examined the samples as they were obtained. The recovered samples were transferred to the GHD Peterborough laboratory, where they were reviewed by a senior geotechnical engineer. The detailed results of the examination are recorded on the test hole logs presented in Appendix A.

3.4 Test Hole Locations and Ground Surface Elevations

UTM coordinates and ground surface elevations for each test hole location was surveyed using a Leica RX1250X GPS system connected to the Real-Time Kinematic (RTK) network. The location of each test hole is referenced to UTM (Zone 17). The following table presents a summary of investigated depths, surface elevations, and UTM coordinates for the test hole locations:

Table 3.1 Summary of Test Holes

Borehole and Test Pit ID	Location – UTM Coordinates System		Test Hole Depth (mbgs)	Ground Elevation (m)
	Northing	Easting		
MW1-21	4919307	261628	3.1	211.9
MW2-21	4919275	261660	3.7	210.3
MW3-21	4919153	261503	4.0	211.7
BH4-21	4919103	261664	2.7	208.3
TP1-21	4919431	261740	1.7	212.3
TP2-21	4919371	261770	1.8	212.2
TP3-21	4919287	261795	3.2	212.9
TP4-21	4919218	261816	3.7	211.9
TP5-21	4919166	261690	1.6	209.7
TP6-21	4919126	261566	1.4	209.9
TP7-21	4919161	261501	2.0	212.0
TP8-21	4919211	261535	1.8	212.2
TP9-21	4949243	261669	1.6	210.1
TP10-21	4919299	261520	1.7	212.2
TP11-21	4919320	261644	1.1	212.2

It should be noted that the provided coordinates and elevations are approximate, and should not be used for construction purposes. The locations of the test holes are shown on the Test Hole Location Plan presented as **Figure 2**. Details of the subsurface conditions encountered are discussed in Section 5 of this report and are presented on the individual logs attached to this report in Appendix A.

3.5 Geotechnical Laboratory Testing

Geotechnical laboratory testing was completed in accordance with the latest editions of the ASTM standards. Geotechnical laboratory testing consisted of moisture content tests on recovered soil samples, as well as grain size distribution analysis (hydrometers) on five (5) selected soil samples.

The collected soil samples were classified / described in general accordance with the ASTM D2487 - Standard Practice for Classification of Soils for engineering purposes (Unified Soil Classification System-USCS).

The results of the moisture content and grain size distribution analysis are recorded at their corresponding depths on the individual test hole logs provided in Appendix A. The gradation curves are provided in Appendix B.

3.6 Hydraulic Conductivity and Infiltration Testing

In-situ constant head permeameter testing was conducted at test pits TP3-21 (2 tests), TP7-21 and TP11-21 at depths from about 0.3 to 1.4 mbgs. Infiltration testing was completed using an ETC Pask (constant head well) permeameter. The infiltration testing results are provided in Appendix C.

In-situ hydraulic response testing was completed in the monitoring wells installed in boreholes MW1-21, MW2-21 and MW3-21. The testing consisted of falling head testing and was completed by introducing a known quantity of potable water in the piezometers, and then measuring the water levels using a data logger programmed to record readings at three (3) second intervals. The data was analyzed using AQTESOLV and the Bouwer-Rice solution for each test. The results of the hydraulic response testing are presented graphically in Appendix C.

4. Site Location and Description

The proposed development area is located along Albine Street to the west of Keeler Court with the bulk of the property on the north side of Albine Street, and a smaller portion on the south side of Albine Street, in Norwood, Ontario. The Site currently consists of vacant agricultural fields. The ground surface within the proposed development area generally comprised of recently harvested hay crops (cut grassed areas) delineated by older fence lines. Power transmission lines border the property at the northern extent. The Site topography is consists of rolling terrain with elevations generally dropping the south.

5. Regional Geology and Subsurface Conditions

5.1 Regional Geology

According to the Quaternary Geology of Ontario Map 2556 ("Quaternary Geology of Ontario-Southern Sheet", prepared by the Ministry of Northern Development and Mines (MNDM), published in 1997), the quaternary deposits in the area of the subject Site consist of till deposits comprised of predominantly sandy silt to silt matrix, commonly rich in clasts, often high in total matrix carbonate content.

According to the Paleozoic Geology of Southern Ontario map, the bedrock in the area consists of limestone and shale of the Verulam Formation, Simcoe Group of the Middle Ordovician era. The bedrock in this area is expected to be encountered near the surface.

5.2 Subsurface Conditions

Subsurface conditions at the test hole locations were generally found to be consistent with the regional geology. Details of the subsurface conditions encountered in the four (4) boreholes and eleven (11) test pits advanced at Site during the GHD investigation are summarized in the following sections of the report. Detailed stratigraphy is shown on the detailed test hole logs presented in Appendix A. It should be noted that the subsurface conditions are only confirmed at the test hole locations and may vary between and beyond the test hole locations. The boundaries between the various strata, as shown on the test hole logs are based on non-continuous sampling and drilling resistance noted and observed at the time of drilling. These boundaries represent an inferred transition between the various strata, rather than precise planes of geological change.

5.2.1 Topsoil

A layer of topsoil was encountered at the surface in the boreholes and test pits. This topsoil layer ranged from 125 to 325 mm in thickness. This soil was observed to be in a damp, loose state, with a silty, highly organic content. As such, it is expected to be devoid of any structural engineering properties.

5.2.2 Sandy Silt / Silty Sand

The surficial layer of topsoil was underlain by native soils generally consisting of sandy silt or silty sand extending to depths ranging from 0.6 mbgs to 3.4 mbgs in the investigated test hole locations. These soils were generally brown in colour and were observed to contain increasing amounts of gravel, cobbles and boulders with depth. SPT N values obtained from within the sandy silt and silty sand layer varied from 4 blows/300mm to over 76 blows/150mm, indicating a loose to very dense in-situ state of relative density.

Samples of this material were visually described to be in a generally moist condition. Measured moisture contents ranged from 1 percent to 16 percent by weight.

5.2.3 Bedrock

Each test hole encountered practical refusal to further advancement due inferred/confirmed bedrock. Practical refusal occurred at depths ranging from 1.7 to 4.0 mbgs.

The depth at which practical refusal was encountered was interpreted by GHD as being the depth of competent bedrock for the purpose of logging the test holes. It is noted that bedrock typically exhibits a certain degree of weathering and fracturing in its upper zone. This weathering effect can increase significantly in shale/limestone bedrock. A number of the test pits penetrated partly into the bedrock, (i.e., through this upper zone of more fractured /

weathered bedrock) before encountering refusal on bedrock interpreted to be of more sound composition. This layer of weathered bedrock appears in the test holes with the exception of test pit TP2-21. Refer to Appendix A for additional details.

The estimated depths of the top of bedrock surface are provided on the test hole logs (Appendix A) and are summarized in Table 5.1.

Table 5.1 Depth to Bedrock

Borehole ID	Depth / Elevation to Inferred / Confirmed Weathered Bedrock (mbgs / m)	Depth / Elevation to Inferred / Confirmed Sound Bedrock (mbgs / m)
MW1-21	1.8 / 210.1	3.1 / 208.8
MW2-21	2.3 / 208.0	3.7 / 206.7
MW3-21	3.4 / 208.4	4.0 / 207.8
BH4-21	2.1 / 206.2	2.7 / 205.6
TP1-21	0.6 / 211.7	1.7 / 210.6
TP2-21	Not Observed	1.8 / 210.4
TP3-21	1.5 / 211.4	3.2 / 209.7
TP4-21	3.4 / 208.5	3.7 / 208.2
TP5-21	0.8 / 208.8	1.6 / 208.0
TP6-21	0.8 / 209.2	1.4 / 208.5
TP7-21	0.9 / 211.1	2.0 / 210.0
TP8-21	0.8 / 211.5	1.8 / 210.4
TP9-21	1.2 / 208.9	1.6 / 208.5
TP10-21	0.9 / 211.3	1.7 / 210.5
TP11-21	0.9 / 211.3	1.1 / 211.1

Note: Bedrock inferred within the boreholes; Bedrock confirmed in the test pits.

5.2.4 Groundwater

Groundwater seepage was not observed in any of the test holes during the drilling and excavation operations. Monitoring wells were installed by GHD in three (3) of the borehole locations (MW1-21, MW2-21 and MW3-21). All three monitoring wells were measured to be dry on August 27, 2021. Test hole information is available in Appendix A.

It is noted that groundwater may be subject to seasonal fluctuations and could rise and decline in response to major weather events.

5.2.5 Geotechnical Laboratory Test Results

A total of five soil samples were collected from native soils at select depths and tested for grain size distribution analysis. The laboratory test results are summarized in the following table and detailed test results are presented in Appendix B.

Table 5.2 Summary of Laboratory Results

Location	Depth (mbgs)	Grain Size Distribution				Observed Soil Unit
		%Gravel	%Sand	%Silt	%Clay	
MW1-21	1.5 – 1.8	35	40	19	6	SM – Silty Sand, Gravelly
MW3-21	2.3 – 2.6	30	45	19	6	SM – Silty Sand, Gravelly
BH4-21	0.8 – 1.1	0	35	57	8	ML – Sandy Silt
TP1-21	0.8 – 0.9	52	30	15	3	Sample of Weathered Bedrock
TP4-21	0.6 – 0.9	0	7	90	3	ML – Silt
Soil description based on Unified Soil Classification System (ASTM D 2487)						

5.2.6 Hydraulic Conductivity and Infiltration Testing Results

In-situ constant head permeameter testing was conducted at test pits TP3-21 (2 tests), TP7-21 and TP11-21 at depths from about 0.3 to 1.4 mbgs. The values obtained correlate to field saturated hydraulic conductivity (K_{fs}) values that range between 10^{-3} cm/sec and 10^{-4} cm/sec for the native soils within the zone tested and correlates to estimated infiltration rates ranging from about 50 to 75 mm/hour based upon Supplementary Guidelines to the Ontario Building Code 2012. The infiltration testing results are provided in Appendix C.

In-situ hydraulic response testing was completed in the monitoring wells installed in boreholes MW1-21, MW2-21 and MW3-21. The testing consisted of falling head testing since the monitoring wells were dry at the outset and was completed by introducing a known quantity of potable water in the monitoring wells and then measuring the water levels using a data logger programmed to record readings at three (3) second intervals. The data was analyzed using AQTESOLV and the Bouwer-Rice solution for each test. The testing indicated hydraulic conductivity values ranging from 10^{-4} cm/sec to 10^{-5} cm/sec within the screened interval of the tested native soils and weathered bedrock units. The results of the hydraulic response testing are presented graphically in Appendix C.

6. Discussion and Recommendations

A preliminary conceptual plan provided by the Client illustrated the proposed site location, road network and lot layout. Proposed invert grades for utility piping (storm, sanitary, water) were not available to GHD as of writing this report. It is expected that the residential buildings will be up to 2-storeys in height, possibly with basements where feasible (based on final grading, bedrock depths, etc).

Based upon the above comments and on the test hole information, and assuming them to be representative of the subsoil conditions across the Site, the following comments and recommendations are offered.

6.1 Site Preparation, Grading and Backfill

The test holes generally encountered a surficial layer of topsoil over compact to very dense native sandy silt and silty sand native soils, containing increasing amounts of gravel, cobbles and boulders with depth, underlain by bedrock.

Any topsoil, vegetation, disturbed earth, fill, organic and organic-bearing material should be removed from the footprint of the proposed building area and from within pavement areas prior to site grading activities. Care will be required during excavation to separate materials containing significant amounts of topsoil/organics or rootlets from the clean excavated material.

Prior to Site grading activity, the subgrade soils exposed after the removal of topsoil and disturbed native soils within the proposed buildings and unsuitable materials within proposed pavement areas should be visually inspected, compacted if required, and proof rolled using large axially loaded equipment. Any loose, organic, or unacceptable areas should be subexcavated and removed as directed by the Engineer and replaced with suitable fill materials compacted to a minimum of 98 percent Standard Proctor Maximum Dry Density (SPMDD). Clean earth fill used to raise grades in the proposed buildings and pavement areas should be placed in thin layers (200 mm thick or less) and compacted by a heavy appropriate roller to 100 percent SPMDD.

The native soils encountered at the Site are generally suitable for reuse as backfill to raise site grades (where required), or as backfill against foundation walls or as trench backfill during installation of buried services, provided they are free of organic material, and are within the optimum moisture content. Control of moisture content during placement and compaction will be essential for maintaining adequate compaction.

Installation of engineered fill, where required, must be continuously monitored on a full-time basis by qualified geotechnical personnel.

6.2 Service Installation

The proposed utility invert depths were not available to GHD at the time of writing this report. The material encountered during this investigation at the anticipated service invert elevations (2 to 3 mbgs) typically consists of native silty sand or weathered bedrock and bedrock. As such, a normal compacted Class “B” bedding is recommended for all underground services, where moisture conditions inside the trench will allow for placement and compaction of bedding material. Class “B” bedding is Granular “A”, or 19 mm crusher run limestone, as per Ontario Provincial Standard Specifications (OPSS). The minimum recommended bedding thickness for the underground services is 150 mm. If any bedding subgrade consists of unsuitable or otherwise incompetent soils, either subexcavate to competent soils, and/or thicken the bedding material to 300 mm. All bedding, surround, and cover materials should be compacted to at least 98 percent of its SPMDD.

It is recommended that covering of the underground services be accomplished using Granular “A”, sand, or other suitable material as allowed by the Municipality’s standards, to a minimum of 300 mm above the pipe. Compaction of this material should attain a minimum of 100 percent SPMDD. It is expected that some of the excavated soils may be suitable for reuse as trench backfill, conditional upon suitable moisture content (within 2 % of optimum), final review and approval by an experienced geotechnical engineer at the time of construction, and regular monitoring and inspection of such reuse throughout construction. Compaction of any native soil in service trenches is recommended to be a minimum of 98 % of its SPMDD.

In order to minimize possibility of differential settlement due to variable subgrade material, where bedding subgrade transitions from soil to bedrock, the bedding thickness can be increased up to 300 mm on the bedrock, and taper at 10H:1V back to the minimum 150 mm thickness as the pipe alignment advances away from the soil subgrade and further into the bedrock subgrade. Alternatively (for watermains) the Client may consider the use of restrained joints at locations where this transition occurs.

6.3 Road Construction

Based on the results of this investigation, we would recommend the following procedures be implemented to prepare the proposed asphalt paved access way and parking areas for its construction:

1. Remove all topsoil, organics, organic-bearing materials and other deleterious materials from the planned pavement areas to a minimum depth to allow for the new pavement structure at which point an assessment of the exposed soils by a member of GHD will deem whether further removal and/or placement of suitable geotextile material or other treatment is required.

2. Inspect and proof roll the subgrade for the purpose of detecting possible zones of overly wet or soft subgrade. Any deleterious areas thus delineated should be replaced with approved granular material compacted to a minimum of 98 percent of its SPMDD. Approved excavated soils can be reused as road subgrade backfill provided the soil is workable and at a moisture content that will permit adequate compaction. A final review and approval to reuse any soils must be made during construction.
3. Contour the subgrade surface to prevent ponding of water during the construction and to promote rapid drainage of the sub-base and base course materials.
4. To maximize drainage potential, 150 mm diameter perforated pipe subdrains should be installed below any curb lines. The pipe should be encased in filter fabric and surrounded by clear stone aggregate. It is recommended that the subdrains discharge to a suitable, frost-free outlet.
5. Construct transitions between varying depths of granular base materials at a rate of 1:10 minimum.

The subgrade materials in the proposed pavement areas will consist of native silty sand till or fill soils. The frost susceptibility of these soils is assessed as being generally moderate to high. In this regard, the following minimum flexible pavement structures are recommended for the construction of the new access and parking areas.

Table 6.1 Minimum Pavement Structure for Local Residential Roads

Profile	Material	Thickness (mm)	In Conformance with OPSS Form
Asphalt Surface	H.L.3	40	1150
Asphalt Base	H.L.8	50	
Granular Base	Granular “A”	150	1010
Granular Subbase	Granular “B”	300	

The following steps are recommended for optimum construction of paved areas:

1. The Granular “A” and “B” courses should be compacted to a minimum 100 percent of their respective SPMDD’s.
2. All asphaltic concrete courses should be placed, spread and compacted conforming to OPSS 310 or equivalent. All asphaltic concrete should be compacted to a minimum 92.0 percent of their respective laboratory Maximum Relative Densities (MRD’s).
3. Adequate drainage should be provided to ensure satisfactory pavement performance.

It is recommended that all fill material be placed in uniform lifts not exceeding 200 mm in thickness before compaction. It is suggested that all granular material used as fill should have an in-situ moisture content within 2 percent of their optimum moisture content. All granular materials should be compacted to 100 percent SPMDD. Granular materials should consist of Granular “A” and “B” conforming to the requirements of OPSS 1010 or equivalent.

The performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved as much as practically possible. It is noted that the above recommended pavement structures are for the end use of the project. The most severe loading conditions on pavement areas and the subgrade may occur during construction. As such, during construction of the project, the recommended granular depths may not be sufficient to support loadings encountered. Consequently, special provisions such as restricted lanes, half-loads during paving, etc. may be required, especially if construction is carried out during unfavourable weather.

6.4 Foundations

In general, it is recommended that structural loading for one- to two-storey buildings be supported on spread and continuous strip footings for column and load bearing walls, respectively. The footings should be founded on the compact native soils, bedrock, or on engineered fill placed directly on the compact native soils or bedrock. The following table summarizes the depths at which suitably competent soils and bedrock were first encountered in each test hole:

Table 6.2 Depth to Competent Bearing Soil and Bedrock

Borehole ID	Depth / Elevation to competent native soils (mbgs / m)	Depth / Elevation to Practical Refusal (Inferred / Confirmed Sound Bedrock , mbgs / m)
MW1-21	0.8 / 211.1	4.0 / 207.8
MW2-21	0.8 / 209.5	2.7 / 205.6
MW3-21	0.3 / 212.1	1.7 / 210.6
BH4-21	1.2 / 207.1	1.8 / 210.4
TP1-21	0.4 / 211.9	3.2 / 209.7
TP2-21	0.6 / 211.6	3.7 / 208.2
TP3-21	1.2 / 211.6	1.6 / 208.0
TP4-21	1.5 / 210.3	1.4 / 208.5
TP5-21	0.8 / 208.8	2.0 / 210.0
TP6-21	0.5 / 209.5	1.8 / 210.4
TP7-21	0.3 / 211.7	1.6 / 208.5
TP8-21	0.3 / 211.8	1.7 / 210.5
TP9-21	0.8 / 209.3	1.1 / 211.1
TP10-21	0.6 / 211.6	4.0 / 207.8
TP11-21	0.8 / 211.4	2.7 / 205.6

For design purposes, and based on one- to two-storey residential houses, it is generally recommended that footings constructed on the compact native soils, bedrock or engineered fill be proportioned using the following bearing capacities:

Table 6.3 Preliminary Bearing Pressure for Foundation Design

<table>
<tr><th rowspan="3">Parameter</th><th colspan="5">Bearing Pressure</th></tr>
<tr><th rowspan="2">Sound Bedrock</th><th rowspan="2">Compact Undisturbed Native Soils</th><th colspan="3">Engineering Fill</th></tr>
<tr><th>Rock-based Fill (2)</th><th>Granular Fill (3)</th><th>Earth Borrow Fill (3)</th></tr>
<tr><td>Factored Bearing Capacity at ULS (1)</td><td rowspan="2">1 MPa</td><td>180 kPa</td><td>255 kPa</td><td>205 kPa</td><td>155 kPa</td></tr>
<tr><td>Bearing Capacity at SLS</td><td>120 kPa</td><td>150 kPa</td><td>120 kPa</td><td>90 kPa</td></tr>
<tr><td colspan="6">Notes:
(1) Resistance factor Φ =0.5 applied to the ULS bearing pressure for design purposes.
(2) At least 1m of Rock-based fill. Quality of material is to be approved prior to use as engineered fill.
(3) At least 0.3m of Granular or Earth Borrow fill. Quality of material is to be approved prior to use as engineered fill.</td></tr>
</table>

Any engineered fill upon which footings are placed must be a minimum thickness corresponding to the notes that accompany the above table. Rock-based fill must be completely encapsulated with suitable filter fabric to minimize any migration of fine-grained particles from surrounding soils into the voids within the rock fill. Footings (and foundation walls) placed on engineered fill must be suitably reinforced; as a minimum, and where not already specified in the design drawings, this reinforcing should use 2 continuous runs of 15M rebar throughout the footings, and 2 runs of 15M rebar throughout near the top and bottom of the foundation walls.

The following is recommended for the construction of any engineered fill for the foundations:

1. Remove any and all existing vegetation, topsoil, fill, organics, and organic-bearing soils to the competent, undisturbed native soil from within the area of the proposed engineered fill.
2. The area of the engineered fill should extend horizontally 1m beyond the outside edge of the building foundations and then extend downward at a 1:1 slope to the competent native soil.
3. The base of the engineered fill area must be approved by a member of GHD prior to placement of any fill, to ensure that all unsuitable materials have been removed, that the materials encountered are similar to those observed, and that the subgrade is suitable for the engineered fill.
4. All engineered fill material is to be approved by GHD at the time of construction.
5. Place approved engineered fill, in maximum 200 mm lifts, compacted to 100 percent of its SPMDD. Any fill material placed under sufficiently wet conditions should consist of an approved, rock-based fill, with the inclusion of appropriate geotextile fabric around the rock-based fill should the rock fill contain enough voids to warrant.
6. Full time testing and inspection of the engineered fill will be required, to ensure compliance with material and compaction specifications.

Under no circumstances should the foundations be placed above organic materials, loose, frozen subgrade, construction debris, or within ponded water. Prior to forming, all foundation excavations must be inspected and approved by a member of GHD's geotechnical group. This will ensure that the foundation bearing material has been prepared properly at the foundation subgrade level and that the soils exposed are similar to those encountered during this investigation.

Should basement or otherwise subgrade areas be incorporated into any of the buildings' designs, it is recommended that for drainage purposes, perimeter drains be installed about the structure. The subdrains would serve to drain seepage water that infiltrates the backfill. The drains should consist of a perforated pipe, at least 150 mm in diameter, surrounded by clear, crushed stone and suitable filter protection. The drain should discharge to a positive sump or other permanent frost free outlet. It is also strongly recommended that the building's foundation walls be sealed and waterproofed.

For foundations constructed in accordance with the foregoing manner, total and differential settlements are estimated to be less than 25 mm.

6.5 Depth of Frost Penetration

It is recommended that all exterior foundations or footings in unheated areas have a minimum soil cover of at least 1.4 m in according to OPSD 3090.101 (2010), or equivalent insulation. Footings for heated structures, such as perimeter foundation for the proposed building structure, must be provided with a minimum of 1.2 m of earth cover or equivalent insulation.

During winter construction exposed surfaces to support foundations must be protected against freezing by means of loose straw and tarpaulins, heating, etc.

6.6 Seismic Site Classification

The latest Ontario Building Code (OBC) requires the assignment of a Seismic Site Class for calculations of earthquake design forces and the structural design based on a two percent probability of exceedance in 50 years. According to the latest OBC, the Seismic Site Class is a function of soil profile, and is based on the average properties of the subsoil strata to a depth of 30 m below the ground surface. The OBC provides the following three methods to obtain the average properties for the top 30 m of the subsoil strata:

– Average shear wave velocity.
– Average Standard Penetration Test (SPT) values (uncorrected for overburden).
– Average undrained shear strength.

For design purposes, based on the criteria listed in Table 4.1.8.4.A. of the OBC and the results obtained from standard penetration resistance of the underlying subsurface conditions, estimated undrained shear strength and our knowledge of the regional geology, a Seismic Site Class ‘C’ can be used for the design of the proposed buildings.

6.7 Slab-On-Grade Construction

Floors may generally be constructed as normal slabs-on-grade, on granular or 19 mm clear stone over native, inorganic subsoils. The floor slab should be formed over a base course consisting of at least 150 mm of Granular “A” backfill as per OPSS or (19 mm clear stone beneath basement areas) compacted to a minimum of 100 percent of its SPMDD. All grade increases or infilling below the granular “A” or clearstone should be constructed in accordance with the engineered fill steps provided in Section 6.5 of this report. If the groundwater table is intersected by any basement excavations, the floor slabs should incorporate under slab drains, and a vapour barrier should be installed beneath the slab to prevent migration of moisture vapour. All fill placed as engineered fill must be inspected, approved and compaction verified by personnel from GHD.

6.8 Basement and Retaining Walls

It is recommended that free draining backfill to basement and retaining walls be provided. Walls located above the groundwater table may be designed for lateral earth pressures using the following equation:

p = k (w h + q), where:

- p = the lateral earth pressure in kPa acting on the subsurface wall at depth h;
- ka= the coefficient of active earth pressure;

  (= 0.3 for walls restrained from the bottom only);

  (= 0.5 for walls restrained at the top and bottom*);
- kp= the coefficient of passive earth pressure, (= 3.0);
- w= the granular or native soil bulk density in kN/m^3;

  (= 21.0 kN/m^3 for well compacted, OPSS-approved Granular "B" or native soils);
- h = the depth (in metres) below the exterior grade at which the earth pressure is being calculated; and
- q = the equivalent value of any surcharge (in kN/m3) acting on the ground surface adjacent to the walls.

(*) This value is recommended for rigid walls retaining compacted backfill.

The recommended value for the coefficient for sliding friction between the soil and the concrete is 0.4. Also, any additional surcharge loading that will influence the wall must be taken into account in its design.

6.9 Storm Water Management Pond

It is GHD's understanding that a SWM pond is proposed in the southwest corner of the site and is to be located within the area of test holes TP7-21 and MW3-21 as shown on the Test Hole Location Plan. The proposed base elevation of the SWM pond is not known at the time of writing this report, however it is expected that the bottom of the SWP will consist of gravelly silty sand native soils or bedrock. The hydraulic conductivity of the sandy gravel is expected to be on the order of about 10^{-3} to 10^{-4} cm/sec and the hydraulic conductivity of the bedrock is expected to range from 10^{-1} to 10^{-7} cm/sec depending on the presence of fractures and voids within the bedrock. (Note that even greater hydraulic conductivities may exist within the bedrock where larger fractures, voids, or other groundwater conduits exist and are intersected by excavations).

Based on the soils observed, and the assumed base elevations, it appears that construction of the SWM pond in this area is feasible. In general, excavation of the soils and/or bedrock for the SWM pond are expected to be straightforward, provided that appropriate measures are taken during construction to minimize any overland or near-surficial flow of water into the area. Groundwater and surficial water inflow into the open SWM pond excavation may be encountered depending on the time of the year in which construction is conducted, however this is expected to be controlled by pumping from within the excavation, along with further measures if required including up-gradient cutoff trenching with appropriate drainage outletting.

It is recommended that the SWM pond subgrade surfaces be proof rolled, and a representative of GHD approve the subgrade prior to construction of the berms. Construction of the berms may utilize excavated soils, such as the sandy silt or gravelly silty sand native soils. Such operations should place soils in lifts no thicker than 150 mm prior to compaction, and compacted to at least 95 percent SPMDD. The native, undisturbed soils or the fractured / weathered bedrock are not expected to have a sufficiently low permeability where they could substitute for a liner, as such the bottom of the SWM pond must be lined with a more suitable (i.e less hydraulically conductive) material.

Liner material should have a hydraulic conductivity of no greater than 10^{-6} cm/sec. The thickness of the liner typically depends on the hydraulic conductivity of the material used. Liners constructed using materials with the maximum allowable hydraulic conductivity (10^{-6} cm/sec) should be a minimum of 450 mm thick, and may be required to be as thick as 600 mm if containing high amounts of gravel and/or cobbles. Liners constructed with less permeable materials, such as clay soils containing hydraulic conductivity of 10^{-8} cm/sec or lower are typically required to be a minimum of 300 mm in thickness. The liner should be placed in lifts no thicker than 150 mm prior to compaction, and compacted to 100 percent SPMDD. It is recommended that compaction of any liner material be carried out under dry weather conditions and at an in-situ moisture content within 2 percent of the material's optimum moisture content. The surface of each liner lift should be scarified prior to placement of the overlying lift. Failure to achieve this will result in a poor performance and a liner with a higher relative hydraulic conductivity. It is noted that materials encountered during this investigation are typically expected to possess hydraulic conductivities greater than 10^{-6} cm/sec (with elevated sand and gravel content), and are therefore generally expected to be unsuitable for use as the SWM pond liner. Further geotechnical assessment of the exposed SWM pond subgrade and any soils being considered for reuse as a liner material should be made during construction.

For the purpose of the proposed SWP, the soils observed should be stable from slip circle failure if sloped at 3 horizontal to 1 vertical (3H:1V) or flatter in the long term both above and below the water table. Between the stable water level and the expected high water level, it is recommended that the slopes be lessened to 4H:1V (or flatter) to guard against erosion by wavelet action. The native material will require vegetative root mass (or otherwise suitable erosion protection) to minimize erosional forces on exposed slopes.

Slopes and berms of the SWP should be constructed so as to reduce or eliminate the effects of surficial erosion. Features to do so may include slope vegetation, installation of erosion or gabion mats, rip rap, and/or other acceptable stabilizing features.

6.10 Excavation and Temporary Shoring

The Occupational Health and Safety Act (OHSA) regulations require that if workmen must enter an excavation deeper than 1.2 m, the excavation must be suitably sloped and/or braced in accordance with the OHSA requirements. OHSA specifies maximum slope of the excavations for four broad soil types as summarized in the following table:

Soil Type	Base of Slope	Maximum Slope Inclination
1	Within 1.2 metres of bottom	1 horizontal to 1 vertical
2	Within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
3	From bottom of excavation	1 horizontal to 1 vertical
4	From bottom of excavation	3 horizontal to 1 vertical

The earth fill and native soils underlying the Site are considered Type 3 soils above groundwater level, and Type 4 if affected by surface water or groundwater seepage. If the above recommended excavation side slopes cannot be maintained due to lack of space or any other reason, the excavation side slopes must be supported by an engineered shoring system. The shoring system should be designed in accordance with Canadian Engineering Foundation Manual (4th Edition) and the OHSA Regulations for Construction Projects.

Depending on the depth of foundations and underground services, construction excavation operations will likely encounter and extend into bedrock. It is recommended that a unit price allowance for bedrock removal be included in the construction contract due to the variable bedrock elevations that are expected during the proposed construction, should any bedrock require excavating. Excavation of any highly fractured / weathered bedrock may be possible using a large hydraulic backhoe. The use of hydraulic breaking techniques and/or blasting (combined with precondition surveys of surrounding properties and vibration monitoring during construction) may be required.

6.11 Temporary Dewatering Requirements

Groundwater seepage was not observed in any of the test holes during the drilling and excavation operations. The monitoring wells installed in boreholes MW1-21, MW2-21 and MW3-21 were measured to be dry on August 27, 2021. In the long-term, seasonal fluctuations of groundwater may occur. Perched groundwater could accumulate within the fill or at the interface between the bedrock and overburden soils after heavy precipitation and/or during spring thaw; however, is expected to be seasonal and temporary based upon the conditions observed during our geotechnical investigation. A permanent groundwater table was not encountered during this drilling program.

Based on the conditions observed during the drilling, and the anticipated excavation depths for the proposed development, groundwater seepage is not expected but could occur at some locations depending on the time of the year. Any groundwater or surficial water infiltration into open excavations is expected to be controlled by pumping from a sump to an acceptable outlet. Should any excavations extend into the bedrock, groundwater-bearing zones may be encountered within any bedding planes and/or fractures and/or other such conduits within the bedrock. Based on hydraulic response testing conducted on the installed monitoring wells, any seepage encountered within the native soils and weathered bedrock may be expected to have a hydraulic conductivity value ranging from 10^{-4} cm/sec to 10^{-5} cm/sec. It should be noted that hydraulic conductivities can vary over a vertical and horizontal extent, and may be outside the stated range if pockets or zones of sand, gravelly soils or more fractured bedrock is intersected.

If short-term pumping of groundwater at volumes greater than 50,000 L/day and less than 400,000 L/day is required during the construction stage, a permit through the Environmental Activity and Sector Registry (EASR) must be obtained. The EASR streamlines the process and water pumping may begin once the EASR registration is completed, the fee paid and supporting document prepared. If water taking in excess of 400,000 litres/day is required, a Permit to Take Water (PTTW) must be obtained in advance. PTTW applications may take up to 90 working days for the Ministry of the Environment, Conservation and Parks (MECP) to review and approve. The actual rate of groundwater taking performed during construction will be a function of the final design, time of year, and the contractor’s schedule, equipment, and techniques.

It is recommended that prior to commencing the construction, consideration be given to the excavation of a series of trial excavations to determine more accurately the soil behaviour and whether or not any significant dewatering works will be required.

6.12 Infiltration Rates for LID Design

In-situ constant head permeameter testing was conducted at test pits TP3-21 (2 tests), TP7-21 and TP11-21 at depths from about 0.3 to 1.4 mbgs. The values obtained correlate to field saturated hydraulic conductivity (K_{fs}) values that range between 10^{-3} cm/sec and 10^{-4} cm/sec for the native soils within the zone tested and correlates to estimated infiltration rates ranging from about 50 to 75 mm/hour based upon Supplementary Guidelines to the Ontario Building Code 2012. The infiltration testing results are provided in Appendix C.

It is noted that slight variations in the soil stratigraphy may cause variations in the permeability of the soil in both vertical and horizontal orientations.

A safety correction factor from Appendix C of the Low Impact Strom Water Management Planning and Design Guide must be applied to the measured infiltration rates once the base elevation for the LID feature is determined.

LIDs can be applied to any soil type; however, it is recommended that more permeable zones are targeted and that sub-grade infiltration locations be kept away from private lands. LIDs require maintenance and long-term care. If possible, naturally occurring infiltration strategies such as roof water discharged via downspouts to sodded lawns with adequate topsoil depths and minimum flow path distances are recommended. As indicated above, the LID features will be designed by others.

6.13 General Recommendations

6.13.1 Wells

The monitoring wells installed as part of this investigation were recorded and reported to the MECP, are still present and active as of writing this report, and are the property of the site's Owner. Any decommissioning of wells on-site must be performed by an appropriately- licensed and experienced well contractor, in compliance with O.Reg. 903.

6.13.2 Test Pit During Tendering

It is strongly recommended that test pits be excavated at representative locations of this Site during the construction tendering phase, with mandatory attendance of interested contractors. This will allow them to make their own assessments of any groundwater, soil and bedrock conditions at the Site and how these will affect their proposed construction methods, techniques and schedules.

6.13.3 Subsoil Sensitivity

The native subsoils are susceptible to strength loss or deformation if saturated or disturbed by construction traffic. Therefore, where the subgrade consists of approved soil, care must be taken to protect the exposed subgrade from excess moisture and from construction traffic.

6.13.4 Winter Construction

The subsoils encountered across the site are frost-susceptible and freezing conditions could cause problems to the structures. As preventive measures, the following recommendations are presented:

1. During winter construction, exposed surfaces intended to support foundations must be protected against freezing by means of loose straw and tarpaulins, heating, etc.

2. Care must be exercised so that any sidewalks and/or asphalt pavements do not interfere with the opening of doors during the winter when the soils are subject to frost heave. This problem may be minimized by any one of several means, such as keeping the doors well above outside grade, installing structural slabs at the doors, and by using well-graded backfill and positive drainage, etc.
3. Because of the frost heave potential of the soils during winter, it is recommended that the trenches for exterior underground services be excavated with shallow transition slopes in order to minimize the abrupt change in density between the granular backfill, which is relatively non-frost susceptible, and the more frost-susceptible native soils.

6.13.5 Construction Monitoring

The foundation installations and Engineered Fill placement must be closely monitored and inspected by qualified personnel to ensure consistency with the design bearing. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical design function and is required by Section 4.2.2.2 of the Ontario Building Code 2012.

Qualified Geotechnical personnel should inspect and test all stages of the proposed development. Specifically, they should ensure that the materials and conditions comply with this geotechnical assessment report. In addition, qualified geotechnical personnel should provide material testing services prior to and during backfilling and grade raising operation. Should soil conditions be encountered that vary from those described in this report, our office should be informed immediately such that the proper measures are undertaken.

7. Limitations of the Investigation

This report is intended solely for DPH Developments Inc. and their designers and is prohibited for use by others without GHD’s prior written consent. This report is considered GHD’s professional work product and shall remain the sole property of GHD. Any unauthorized reuse, redistribution of or reliance on the report shall be at the Client and recipient’s sole risk, without liability to GHD. No portion of this report may be used as a separate entity; it is to be read in its entirety and shall include all supporting drawings and appendices.

The recommendations made in this report are in accordance with our present understanding of the project, the current site use, ground surface elevation and conditions, and are based on the work scope approved by the Client and described in the report. The services were performed in a manner consistent with that level of care and skill ordinarily exercised by members of geotechnical engineering professions currently practicing under similar conditions in the same locality. No other representations, and no warranties or representations of any kind, either expressed or implied, are made. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties.

All details of design and construction are rarely known at the time of completion of a geotechnical study. The recommendations and comments made in the study report are based on our subsurface investigation and resulting understanding of the project, as defined at the time of the study. We should be retained to review our recommendations when the drawings and specifications are complete. Without this review, GHD will not be liable for any misunderstanding of our recommendations or their application and adaptation into the final design.

By issuing this report, GHD is the geotechnical engineer of record. It is recommended that GHD be retained during construction of all foundations and during earthwork operations to confirm the conditions of the subsoil are actually similar to those observed during our study. The intent of this requirement is to verify that conditions encountered during construction are consistent with the findings in the report and that inherent knowledge developed as part of our study is correctly carried forward to the construction phases.

It is important to emphasize that a soil investigation is, in fact, a random sampling of a site and the comments included in this report are based on the results obtained at the test locations only. The subsurface conditions confirmed at the test locations may vary at other locations. The subsurface conditions can also be significantly modified by the construction activities on site (e.g., excavation, dewatering and drainage, blasting, pile driving, etc.). These conditions can also be modified by exposure of soils or bedrock to humidity, dry periods or frost. Soil and groundwater conditions between and beyond the test locations may differ both horizontally and vertically from those encountered at the test locations and conditions may become apparent during construction which could not be detected or anticipated at the time of our investigation. Should any conditions at the site be encountered which differ from those found at the test locations, we request that we be notified immediately in order to permit a reassessment of our recommendations. If changed conditions are identified during construction, no matter how minor, the recommendations in this report shall be considered invalid until sufficient review and written assessment of said conditions by GHD is completed.

All of Which is Respectfully Submitted,

GHD

LICENSED PROFESSIONAL ENGINEER
Oct 8/2021
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100209777
PROVINCE OF ONTARIO

Leandro Ramos, P.Eng.

PROFESSIONAL GEOSCIENTIST
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Figures

Data Disclaimer

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Paper Size ANSI A

0 70 140 210 280

Metres

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 18N

N

GHD

DPH DEVELOPMENTS
158 ALBINE STREET, NORWOOD, ON
PT LOTS 18 & 19, CON 8, GEO. TOWNSHIP OF ASPHODEL
TOWNSHIP OF ASPHODEL-NORWOOD
COUNTY OF PETERBOROUGH

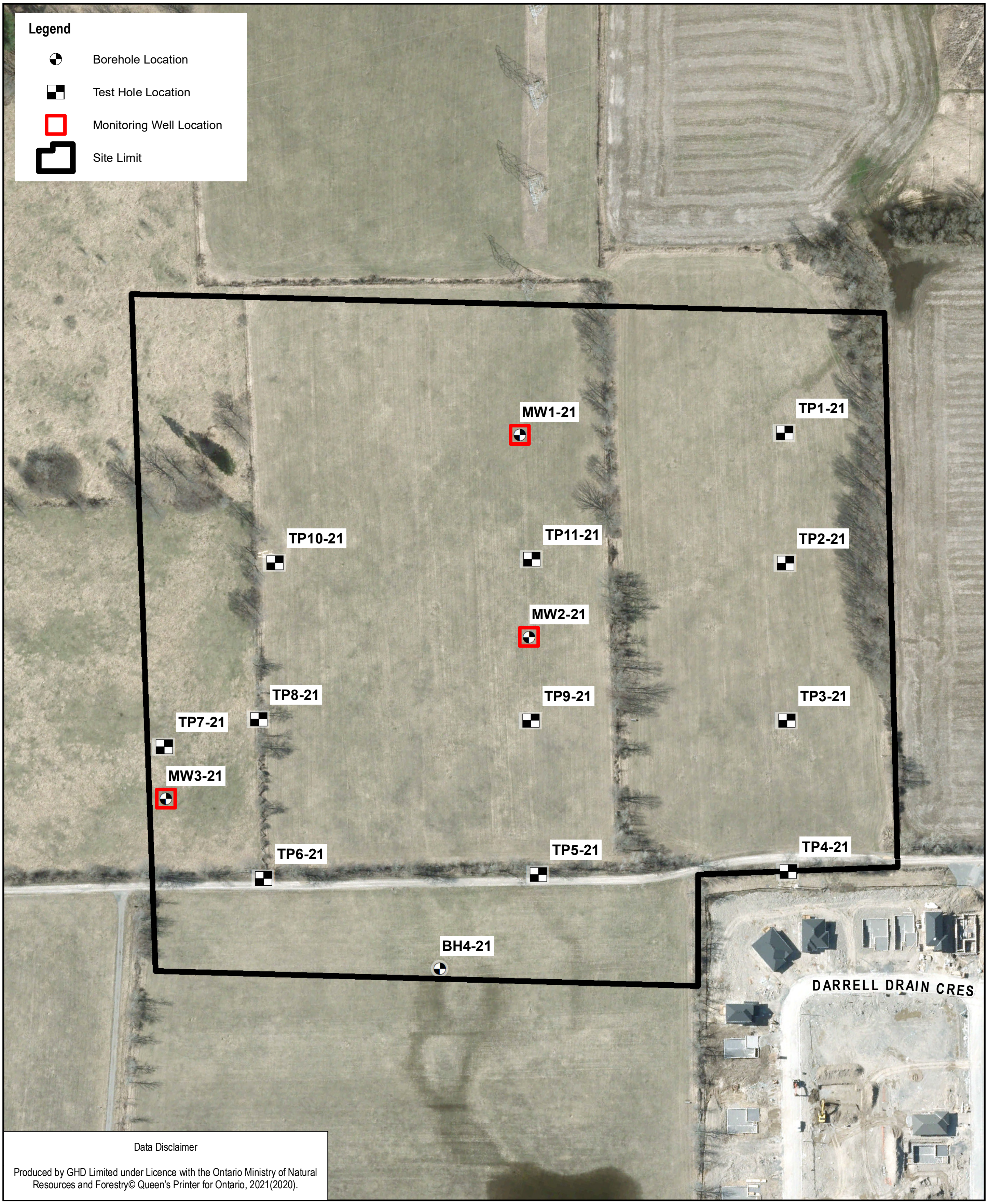
GEOTECHNICAL INVESTGATION
SITE LOCATION PLAN

Project No.	**11231077**
Revision No.	
Date	**Sep 2021**

FIGURE 1

Q:\GIS\PROJECTS\11231000s\11231077\Layouts\202109_RPT001\11231077_202109_RPT001_GIS001 - Site Location Plan.mxd
Print date: 24 Sep 2021 - 08:43

Data source: © County of Peterborough, 2018.



Paper Size ANSI A

0 10 20 30 40

Metres

Map Projection: Transverse Mercator
Horizontal Datum: North American 1983
Grid: NAD 1983 UTM Zone 18N

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GHD

DPH DEVELOPMENTS
158 ALBINE STREET, NORWOOD, ON
PT LOTS 18 & 19, CON 8, GEO. TOWNSHIP OF ASPHODEL
TOWNSHIP OF ASPHODEL-NORWOOD
COUNTY OF PETERBOROUGH

GEOTECHNICAL INVESTGATION
TEST HOLE PLAN

Project No.	**11231077**
Revision No.	
Date	**Sep 2021**

FIGURE 2

Q:\GIS\PROJECTS\11231000s\11231077\Layouts\202109_RPT001\11231077_202109_RPT001_GIS002 - Test Hole Plan.mxd
Print date: 07 Oct 2021 - 09:53

Data source: © County of Peterborough, 2018.

Appendices

Appendix A

Stratigraphy Logs

REFERENCE No.: 11231077

ENCLOSURE No.: 1

GHD

BOREHOLE No.: MW1-21

ELEVATION: 211.90 m

BOREHOLE REPORT

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern

CHECKED BY: L Ramos

DATE (START): 18 August 2021

DATE (FINISH): 18 August 2021

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- WATER LEVEL

NORTHING: 4919307

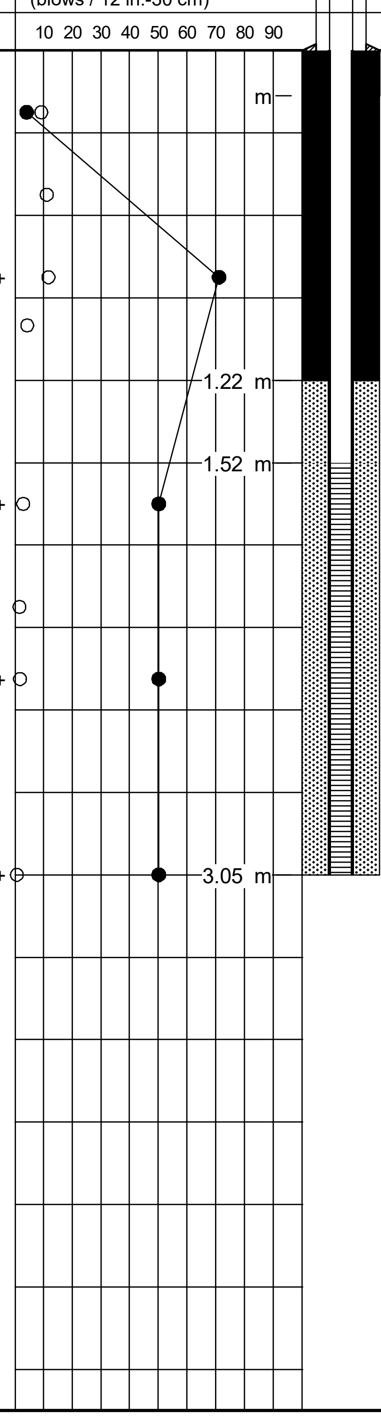
EASTING: 261628

Elevation (m) BGS	DESCRIPTION OF SOIL AND BEDROCK	Type and Number	Recovery/ TCR(%)	Moisture Content (%)	Blows per 15cm/ RQD(%)	'N' Value/ SCR(%)
211.90	GROUND SURFACE					
	TOPSOIL (300 mm)	SS1a	75	9	2-2-2-2	4
211.59	**NATIVE:** ML - SANDY SILT, gravelly, trace clay, loose, light brown, moist	SS1b		11	--	--
211.44	SM - SILTY SAND, gravelly, trace clay, cobbles, very dense, brown, moist	SS2a	100	12	6-21-50/5cm	71+
		SS2b		4	--	--
	gravel: 35%, sand 40%, silt: 19%, clay: 6%	SS3	83	3	40-50/15cm	50+
210.07	**WEATHERED BEDROCK**	AS4		2	--	--
		SS5	0	2	50/7.5cm	50+
208.85		SS6	0	1	50/0cm	50+

END OF BOREHOLE

NOTES:
- Borehole open and dry upon completion
- Borehole terminated at 3.1 m bgs due to auger refusal (solid bedrock inferred)
- 51 mm Monitoring well installed to 3.1 m bgs, 1.5 m screen
- Water level dry August 27, 2021
- bgs denotes 'below ground surface'

Shear test (Cu) △ Field □ Lab
Sensitivity (S)
○ Water content (%)
Atterberg limits (%) W_p W_l
● "N" Value (blows / 12 in.-30 cm)



Depth: Feet 0–16; Metres 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5

REFERENCE No.: 11231077 ENCLOSURE No.: 2

BOREHOLE REPORT

BOREHOLE No.: **MW2-21**
ELEVATION: 210.34 m

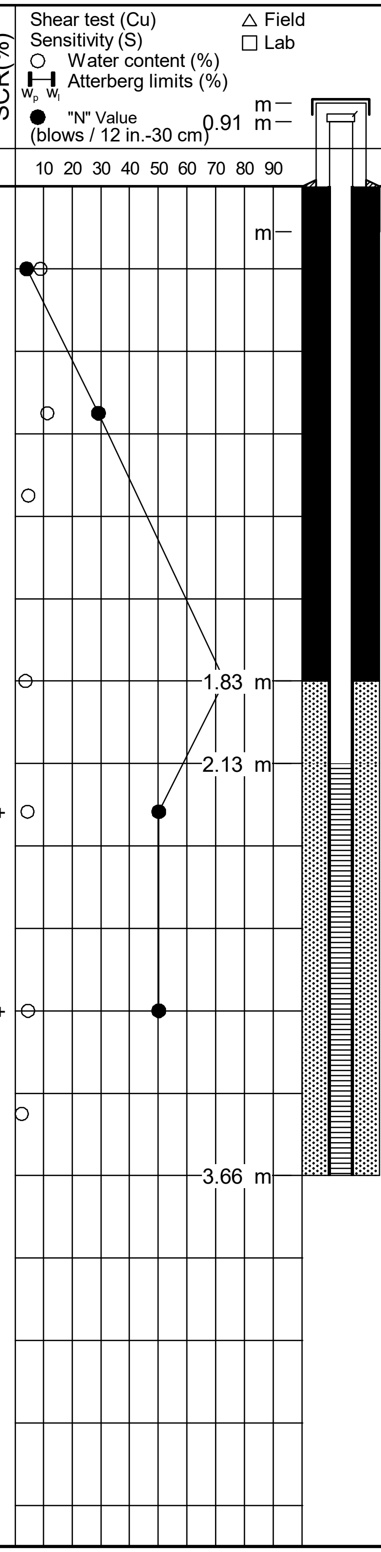
CLIENT: DPH Developments Inc
PROJECT: Geotechnical Investigation, Norwood Park Phase 4
LOCATION: Albine Street, Norwood
DESCRIBED BY: J McEachern CHECKED BY: L Ramos
DATE (START): 18 August 2021 DATE (FINISH): 18 August 2021

LEGEND
- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- ▼ - WATER LEVEL

NORTHING: 4919275 EASTING: 261657

Depth (Feet)	Depth (Metres)	Elevation (m) BGS	Description of Soil and Bedrock
0	0	210.34	GROUND SURFACE
0	0	210.34	**TOPSOIL** (300 mm)
1	0.3	210.04	**NATIVE:** ML - SANDY SILT, gravelly, with clay, loose, light brown, moist
3	0.9	209.43	SM - SILTY SAND, gravelly, trace clay, cobbles, very dense, brown, moist
7.5	2.3	208.00	**WEATHERED BEDROCK**
12	3.7	206.68	**END OF BOREHOLE**

State	Type and Number	Recovery/TCR(%)	Moisture Content (%)	Blows per 15cm/RQD(%)	'N' Value/SCR(%)
SS	SS1	46	9	4-2-2-4	4
SS	SS2a	75	11	9-14-15-16	29
SS	SS2b		5	--	--
SS	SS3	83	4	14-31-42-35	73
SS	SS4	100	4	50/5cm	50+
	SS5	0	5	50/0cm	50+
ST	AS6		2	--	--



NOTES:
- Borehole open and dry upon completion
- Borehole terminated at 3.7 m bgs due to auger refusal (solid bedrock inferred)
- 51 mm Monitoring well installed to 3.7 m bgs, 1.5 m screen
- Water level dry August 27, 2021
- bgs denotes 'below ground surface'

REFERENCE No.: 11231077 ENCLOSURE No.: 3

GHD

BOREHOLE No.: MW3-21

ELEVATION: 211.74 m

BOREHOLE REPORT

Page: 1 of 1

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern CHECKED BY: L Ramos

DATE (START): 18 August 2021 DATE (FINISH): 18 August 2021

LEGEND

- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- WATER LEVEL

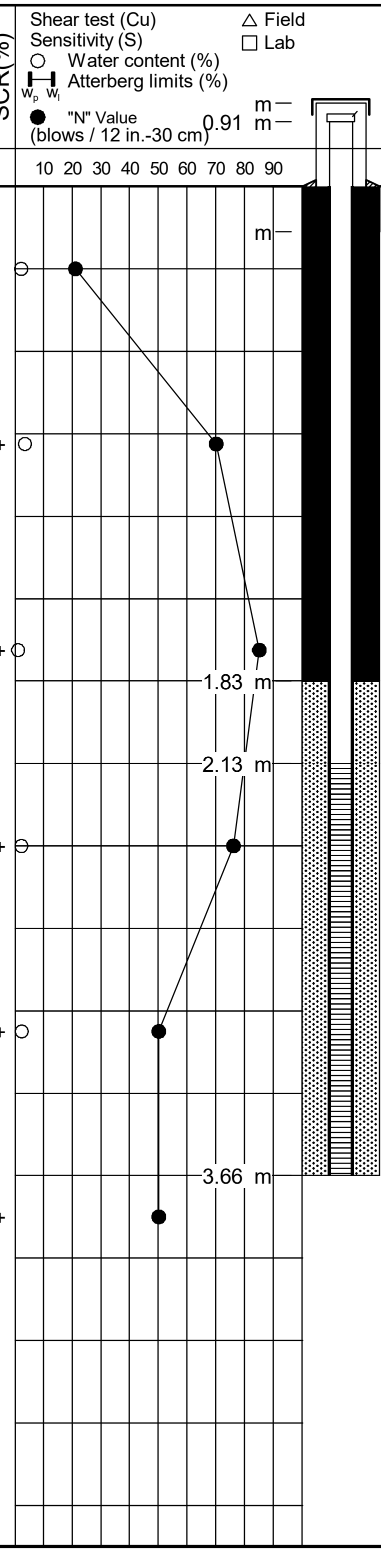
NORTHING: 4919153 EASTING: 261503

Depth (Feet)	Depth (Metres)	Elevation (m) BGS	DESCRIPTION OF SOIL AND BEDROCK	Type and Number	Recovery/ TCR(%)	Moisture Content (%)	Blows per 15cm/ RQD(%)	'N' Value/ SCR(%)
0		211.74	GROUND SURFACE					
			TOPSOIL (150 mm)					
		211.59	**NATIVE:** SM - SILTY SAND, gravelly, trace clay, cobbles, very dense, brown, moist					
1				SS1	50	2	10 10 11 12	21
3	1.0			SS2	93	3	12-20-50/7.5cm	70+
5.5				SS3	100	1	21-35-50/7.5cm	85+
8	2.5		gravel: 30%, sand 44%, silt: 19%, clay: 6%	SS4	100	2	14-26-50/0cm	76+
10	3.0			SS5	100	2	50/15cm	50+
11		208.39	**WEATHERED BEDROCK**					
12.5				SS6	0	--	50/0cm	50+
13	4.0	207.78	**END OF BOREHOLE**					

END OF BOREHOLE

NOTES:

- Borehole open and dry upon completion
- Borehole terminated at 4.0 m beg due to auger refusal (solid bedrock inferred)
- 51 mm Monitoring well installed to 3.7 m beg, 1.5 m screen
- Water level dry August 27, 2021
- bgs denotes 'below ground surface'



REFERENCE No.: 11231077 ENCLOSURE No.: 4

BOREHOLE REPORT

BOREHOLE No.: **BH4-21**

ELEVATION: 208.30 m

Page: 1 of 1

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern CHECKED BY: L Ramos

DATE (START): 18 August 2021 DATE (FINISH): 18 August 2021

LEGEND

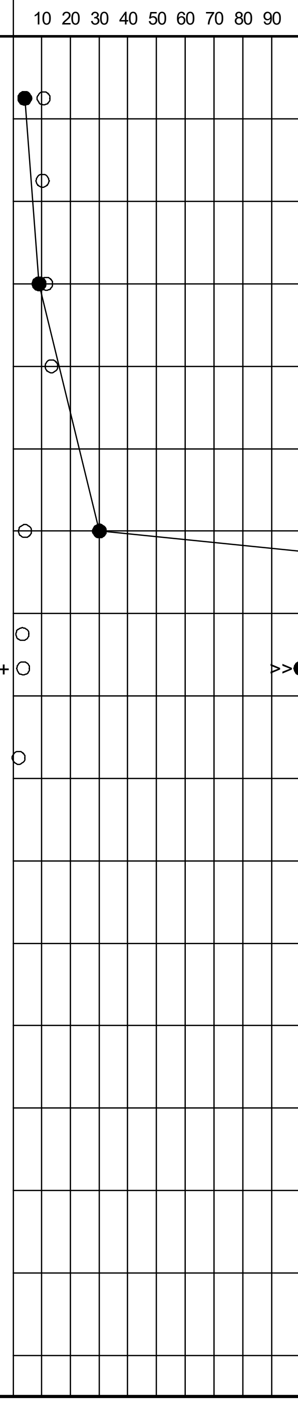
- SS - SPLIT SPOON
- ST - SHELBY TUBE
- AU - AUGER PROBE
- WATER LEVEL

NORTHING: 4919102 EASTING: 261664

Depth (Metres)	Elevation (m) BGS	DESCRIPTION OF SOIL AND BEDROCK	Type and Number	Recovery/ TCR(%)	Moisture Content (%)	Blows per 15cm/ RQD(%)	'N' Value/ SCR(%)
0.0	208.30	GROUND SURFACE					
0.0		**TOPSOIL** (150 mm)					
0.15	208.15	**NATIVE:** ML - SANDY SILT, trace clay, loose, light brown, moist	SS1	80	11	1-2-2-2	4
		reddish brown	SS1b		10	--	--
		gravel: 0%, sand 35%, silt: 57%, clay: 8%	SS2a	75	12	2-4-5-11	9
		brown	SS2b		13	--	--
1.22	207.08	SM - SILTY SAND, gravelly, trace clay, cobbles, very dense, brown, moist	SS3	50	4	15-14-16-14	30
2.13	206.17	**WEATHERED BEDROCK**	AS4		3	--	--
			SS5	50	4	50/10cm	500+
			AS6		2	--	--
2.74	205.56	**END OF BOREHOLE**					

NOTES:
- Borehole caving to 2.4 m bgs and dry upon completion
- Borehole terminated at 2.7 m beg due to auger refusal (solid bedrock inferred)
- bgs denotes 'below ground surface'

Shear test (Cu) △ Field □ Lab
Sensitivity (S)
○ Water content (%)
Atterberg limits (%) W_p W_l
● "N" Value (blows / 12 in.-30 cm)



REFERENCE No.: 11231077 ENCLOSURE No.: 5

GHD	TEST PIT No.: **TP1-21** ELEVATION: 212.33 m	# TEST PIT REPORT

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern DATE: 27 August 2021

CHECKED BY: L Ramos DATE: 27 August 2021

LEGEND

GSE - GRAB SAMPLE (environmental)
GS - GRAB SAMPLE (geotechnical)
Cu - SHEAR TEST
CHEM - CHEMICAL ANALYSIS
OVC - ORGANIC VAPOR CONCENTRATION
INF - INFILTRATION
▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS 212.33	Symbol	STRATIGRAPHY	Sample Type & Number	OVC ppm	Tests Type	▼ / INF
				TOPSOIL (250 mm)	GS1			
1		212.08		**NATIVE:** SM - SILTY SAND, reddish brown, moist	GS2			
	0.5	211.95		ML - SANDY SILT, light brown, moist				
2		211.72		**WEATHERED BEDROCK** gravel: 52%, sand 30%, silt: 15%, clay: 3%	GS3			
3	1.0							
4								
5	1.5				GS4			
6		210.60		**END OF TEST PIT** **NOTES:** - No noticeable groundwater seepage or accumulation at completion of test pit - Test pit terminated at 1.7 m bgs due to solid bedrock refusal - bgs denotes 'below ground surface'				
7	2.0							
8	2.5							
9								
10	3.0							
11								
12	3.5							
13	4.0							

REFERENCE No.: 11231077 ENCLOSURE No.: 6

	TEST PIT No.: TP2-21 ELEVATION: 212.21 m	TEST PIT REPORT

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern DATE: 27 August 2021

CHECKED BY: L Ramos DATE: 27 August 2021

LEGEND

GSE - GRAB SAMPLE (environmental)
GS - GRAB SAMPLE (geotechnical)
Cu - SHEAR TEST
CHEM - CHEMICAL ANALYSIS
OVC - ORGANIC VAPOR CONCENTRATION
INF - INFILTRATION
▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS 212.21	Symbol	STRATIGRAPHY	Sample Type & Number	OVC ppm	Tests Type	▼ / INF
				TOPSOIL (250 mm)	GS1			
1	0.5	211.96		**NATIVE:** ML - SANDY SILT, light brown, moist				
2		211.60		SM - SILTY SAND, gravelly, trace clay, cobbles, brown, moist Cobbles and boulders	GS2 GS3			
3	1.0							
4								
5	1.5							
6		210.38		**END OF TEST PIT** **NOTES:** - No noticeable groundwater seepage or accumulation at completion of test pit - Test pit terminated at 1.8 m bgs due to solid bedrock refusal - bgs denotes 'below ground surface'				
7	2.0							
8	2.5							
9								
10	3.0							
11								
12	3.5							
13	4.0							

REFERENCE No.: 11231077 ENCLOSURE No.: 7

GHD

TEST PIT No.: TP3-21

ELEVATION: 212.87 m

TEST PIT REPORT

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern DATE: 27 August 2021

CHECKED BY: L Ramos DATE: 27 August 2021

LEGEND

GSE - GRAB SAMPLE (environmental)
GS - GRAB SAMPLE (geotechnical)
Cu - SHEAR TEST
CHEM - CHEMICAL ANALYSIS
OVC - ORGANIC VAPOR CONCENTRATION
INF - INFILTRATION
▼ - WATER LEVEL

Depth (Feet)	Depth (Metres)	Elevation (m) BGS 212.87	Symbol	STRATIGRAPHY	Sample Type & Number	OVC ppm	Tests Type	▼ / INF
				TOPSOIL (250 mm)	GS1			
1		212.62		**NATIVE:** SM - SILTY SAND, reddish brown, moist	GS2			
	0.5	212.47		ML - SANDY SILT, light brown, moist				
2								
3	1.0							
4		211.65		SM - SILTY SAND, gravelly, trace clay, cobbles, brown, moist	GS3			
5	1.5	211.35		**WEATHERED BEDROCK**				
6								
7	2.0							
8	2.5							
9								
10	3.0				GS4			
		209.67		**END OF TEST PIT** **NOTES:** - No noticeable groundwater seepage or accumulation at completion of test pit - Test pit terminated at 3.2 m bgs due to solid bedrock refusal - bgs denotes 'below ground surface'				
11	3.5							
12								
13	4.0							

REFERENCE No.: 11231077 ENCLOSURE No.: 8

GHD	TEST PIT No.: **TP4-21** ELEVATION: 211.87 m	# TEST PIT REPORT

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern DATE: 27 August 2021

CHECKED BY: L Ramos DATE: 27 August 2021

LEGEND

GSE - GRAB SAMPLE (environmental)
GS - GRAB SAMPLE (geotechnical)
Cu - SHEAR TEST
CHEM - CHEMICAL ANALYSIS
OVC - ORGANIC VAPOR CONCENTRATION
INF - INFILTRATION
▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS 211.87	Symbol	STRATIGRAPHY	Sample Type & Number	OVC ppm	Tests Type	▼ / INF
				TOPSOIL (300 mm)	GS1			
1		211.56		**NATIVE:** SM - SILTY SAND, reddish brown, moist	GS2			
2	0.5	211.41		ML - SANDY SILT, light brown, moist gravel: 0%, sand 7%, silt: 90%, clay: 3%	GS3			
5	1.5	210.34		SM - SILTY SAND, gravelly, trace clay, cobbles, brown, moist cobbles and boulders				
11		208.51		**WEATHERED BEDROCK**	GS4			
12		208.21		**END OF TEST PIT** **NOTES:** - No noticeable groundwater seepage or accumulation at completion of test pit - Test pit terminated at 3.7 m bgs due to solid bedrock refusal - bgs denotes 'below ground surface'				

Depth scale: Feet 1–13; Metres 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0

REFERENCE No.: 11231077 ENCLOSURE No.: 9

GHD	TEST PIT No.: TP5-21 ELEVATION: 209.65 m	# TEST PIT REPORT

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern DATE: 27 August 2021

CHECKED BY: L Ramos DATE: 27 August 2021

LEGEND

GSE - GRAB SAMPLE (environmental)
GS - GRAB SAMPLE (geotechnical)
Cu - SHEAR TEST
CHEM - CHEMICAL ANALYSIS
OVC - ORGANIC VAPOR CONCENTRATION
INF - INFILTRATION
▼ - WATER LEVEL

Depth (Feet)	Depth (Metres)	Elevation (m) BGS 209.65	Symbol	STRATIGRAPHY	Sample Type & Number	OVC ppm	Tests Type	▼ / INF
				TOPSOIL (325 mm)				
1		209.33		**NATIVE:** SM - SILTY SAND, reddish brown, moist	GS1			
2	0.5	209.10		ML - SANDY SILT, light brown, moist	GS2			
3		208.84		**WEATHERED BEDROCK**	GS3			
4	1.0							
5	1.5	208.03			GS4			
6				**END OF TEST PIT** **NOTES:** - No noticeable groundwater seepage or accumulation at completion of test pit - Test pit terminated at 1.6 m bgs due to solid bedrock refusal - bgs denotes 'below ground surface'				
7	2.0							
8	2.5							
9								
10	3.0							
11								
12	3.5							
13	4.0							

REFERENCE No.: 11231077 ENCLOSURE No.: 10

GHD	**TEST PIT No.: TP6-21** **ELEVATION:** 209.94 m	**TEST PIT REPORT**

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern DATE: 27 August 2021

CHECKED BY: L Ramos DATE: 27 August 2021

LEGEND

GSE - GRAB SAMPLE (environmental)
GS - GRAB SAMPLE (geotechnical)
Cu - SHEAR TEST
CHEM - CHEMICAL ANALYSIS
OVC - ORGANIC VAPOR CONCENTRATION
INF - INFILTRATION
▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS 209.94	Symbol	STRATIGRAPHY	Sample Type & Number	OVC ppm	Tests Type	▼ / INF
				TOPSOIL (200 mm)				
1		209.73		**NATIVE:** ML - SANDY SILT, light brown, moist	GS1			
2	0.5	209.53		SM-SILTY SAND, gravelly, trace clay, cobbles, brown, moist	GS2			
3, 4	1.0	209.18		**WEATHERED BEDROCK**	GS3			
5, 6, 7, 8, 9, 10, 11, 12, 13	1.5, 2.0, 2.5, 3.0, 3.5, 4.0	208.51		**END OF TEST PIT** **NOTES:** - No noticeable groundwater seepage or accumulation at completion of test pit - Test pit terminated at 1.4 m bgs due to solid bedrock refusal - bgs denotes 'below ground surface'				

REFERENCE No.: 11231077 ENCLOSURE No.: 11

GHD	TEST PIT No.: TP7-21 ELEVATION: 212.00 m	TEST PIT REPORT

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern DATE: 27 August 2021

CHECKED BY: L Ramos DATE: 27 August 2021

LEGEND

GSE - GRAB SAMPLE (environmental)
GS - GRAB SAMPLE (geotechnical)
Cu - SHEAR TEST
CHEM - CHEMICAL ANALYSIS
OVC - ORGANIC VAPOR CONCENTRATION
INF - INFILTRATION
▼ - WATER LEVEL

Depth Feet	Depth Metres	Elevation (m) BGS 212.00	Symbol	STRATIGRAPHY	Sample Type & Number	OVC ppm	Tests Type	▼ / INF
				TOPSOIL (125 mm)				
		211.87		**NATIVE:** SM - SILTY SAND, gravelly, trace clay, cobbles, brown, moist				
1					GS1			
2	0.5							
3		211.09		**WEATHERED BEDROCK**				
4	1.0							
5	1.5				GS2			
6								
7	2.0	210.02		**END OF TEST PIT** **NOTES:** - No noticeable groundwater seepage or accumulation at completion of test pit - Test pit terminated at 2.0 m bgs due to solid bedrock refusal - bgs denotes 'below ground surface'				
8	2.5							
9								
10	3.0							
11								
12	3.5							
13	4.0							

REFERENCE No.: 11231077 | ENCLOSURE No.: 12

GHD	TEST PIT No.: **TP8-21** ELEVATION: 212.15 m	**TEST PIT REPORT**

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern DATE: 27 August 2021

CHECKED BY: L Ramos DATE: 27 August 2021

LEGEND

GSE - GRAB SAMPLE (environmental)
GS - GRAB SAMPLE (geotechnical)
Cu - SHEAR TEST
CHEM - CHEMICAL ANALYSIS
OVC - ORGANIC VAPOR CONCENTRATION
INF - INFILTRATION
▼ - WATER LEVEL

Depth (Feet)	Depth (Metres)	Elevation (m) BGS 212.15	Symbol	STRATIGRAPHY	Sample Type & Number	OVC ppm	Tests Type	▼ / INF
				TOPSOIL (175 mm)				
		211.98		**NATIVE:** SM - SILTY SAND, reddish brown, moist	GS1			
1		211.85		SM - SILTY SAND, gravelly, trace clay, cobbles, brown, moist	GS2			
	0.5							
2		211.54		**WEATHERED BEDROCK**				
3	1.0							
4								
5	1.5				GS3			
6		210.38		**END OF TEST PIT** **NOTES:** - No noticeable groundwater seepage or accumulation at completion of test pit - Test pit terminated at 1.8 m bgs due to solid bedrock refusal - bgs denotes 'below ground surface'				
7	2.0							
8	2.5							
9								
10	3.0							
11								
12	3.5							
13	4.0							

REFERENCE No.: 11231077 ENCLOSURE No.: 13

GHD	**TEST PIT No.:** **TP9-21** **ELEVATION:** 210.08 m	**TEST PIT REPORT**

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern DATE: 27 August 2021

CHECKED BY: L Ramos DATE: 27 August 2021

LEGEND

GSE - GRAB SAMPLE (environmental)
GS - GRAB SAMPLE (geotechnical)
Cu - SHEAR TEST
CHEM - CHEMICAL ANALYSIS
OVC - ORGANIC VAPOR CONCENTRATION
INF - INFILTRATION
▼ - WATER LEVEL

Depth (Feet)	Depth (Metres)	Elevation (m) BGS 210.08	Symbol	STRATIGRAPHY	Sample Type & Number	OVC ppm	Tests Type	▼ / INF
				TOPSOIL (300 mm)				
1		209.77		**NATIVE:** SM - SILTY SAND, reddish brown, moist	GS1			
2	0.5	209.57		ML - SANDY SILT, light brown, moist cobbles and boulders	GS2			
3	1.0	209.27		SM - SILTY SAND, gravelly, trace clay, cobbles, brown, moist	GS3			
4		208.86		**WEATHERED BEDROCK**				
5	1.5				GS4			
		208.45		**END OF TEST PIT** **NOTES:** - No noticeable groundwater seepage or accumulation at completion of test pit - Test pit terminated at 1.6 m bgs due to solid bedrock refusal - bgs denotes 'below ground surface'				
6								
7	2.0							
8	2.5							
9								
10	3.0							
11								
12	3.5							
13	4.0							

REFERENCE No.: 11231077

ENCLOSURE No.: 14

GHD	TEST PIT No.: **TP10-21** ELEVATION: 212.18 m	**TEST PIT REPORT**

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern DATE: 27 August 2021

CHECKED BY: L Ramos DATE: 27 August 2021

LEGEND

GSE - GRAB SAMPLE (environmental)
GS - GRAB SAMPLE (geotechnical)
Cu - SHEAR TEST
CHEM - CHEMICAL ANALYSIS
OVC - ORGANIC VAPOR CONCENTRATION
INF - INFILTRATION
▼ - WATER LEVEL

Depth (Feet)	Depth (Metres)	Elevation (m) BGS 212.18	Symbol	STRATIGRAPHY	Sample Type & Number	OVC ppm	Tests Type	▼ / INF
				TOPSOIL (250 mm)				
1		211.85		**NATIVE:** ML - SANDY SILT, light brown, moist	GS1			
2	0.5	211.63		SM - SILTY SAND, gravelly, trace clay, cobbles, brown, moist	GS2			
3	1.0	211.27		**WEATHERED BEDROCK**				
4					GS3			
5	1.5	210.51		**END OF TEST PIT** **NOTES:** - No noticeable groundwater seepage or accumulation at completion of test pit - Test pit terminated at 1.7 m bgs due to solid bedrock refusal - bgs denotes 'below ground surface'				
6								
7	2.0							
8	2.5							
9								
10	3.0							
11								
12	3.5							
13	4.0							

REFERENCE No.: 11231077

ENCLOSURE No.: 15

GHD	TEST PIT No.: TP11-21 ELEVATION: 212.21 m	TEST PIT REPORT

CLIENT: DPH Developments Inc

PROJECT: Geotechnical Investigation, Norwood Park Phase 4

LOCATION: Albine Street, Norwood

DESCRIBED BY: J McEachern DATE: 27 August 2021

CHECKED BY: L Ramos DATE: 27 August 2021

LEGEND

GSE - GRAB SAMPLE (environmental)
GS - GRAB SAMPLE (geotechnical)
Cu - SHEAR TEST
CHEM - CHEMICAL ANALYSIS
OVC - ORGANIC VAPOR CONCENTRATION
INF - INFILTRATION
▼ - WATER LEVEL

Depth (Feet)	Depth (Metres)	Elevation (m) BGS 212.21	Symbol	STRATIGRAPHY	Sample Type & Number	OVC ppm	Tests Type	▼ / INF
				TOPSOIL (250 mm)				
1		211.90		**NATIVE:** ML - SANDY SILT, light brown, moist				
2	0.5				GS1			
		211.45		SM - SILTY SAND, gravelly, trace clay, cobbles, brown, moist	GS2			
3		211.29		**WEATHERED BEDROCK**	GS3			
	1.0	211.14		**END OF TEST PIT** **NOTES:** - No noticeable groundwater seepage or accumulation at completion of test pit - Test pit terminated at 1.1 m bgs due to solid bedrock refusal - bgs denotes 'below ground surface'				
4								
5	1.5							
6								
7	2.0							
8	2.5							
9								
10	3.0							
11								
12	3.5							
13	4.0							

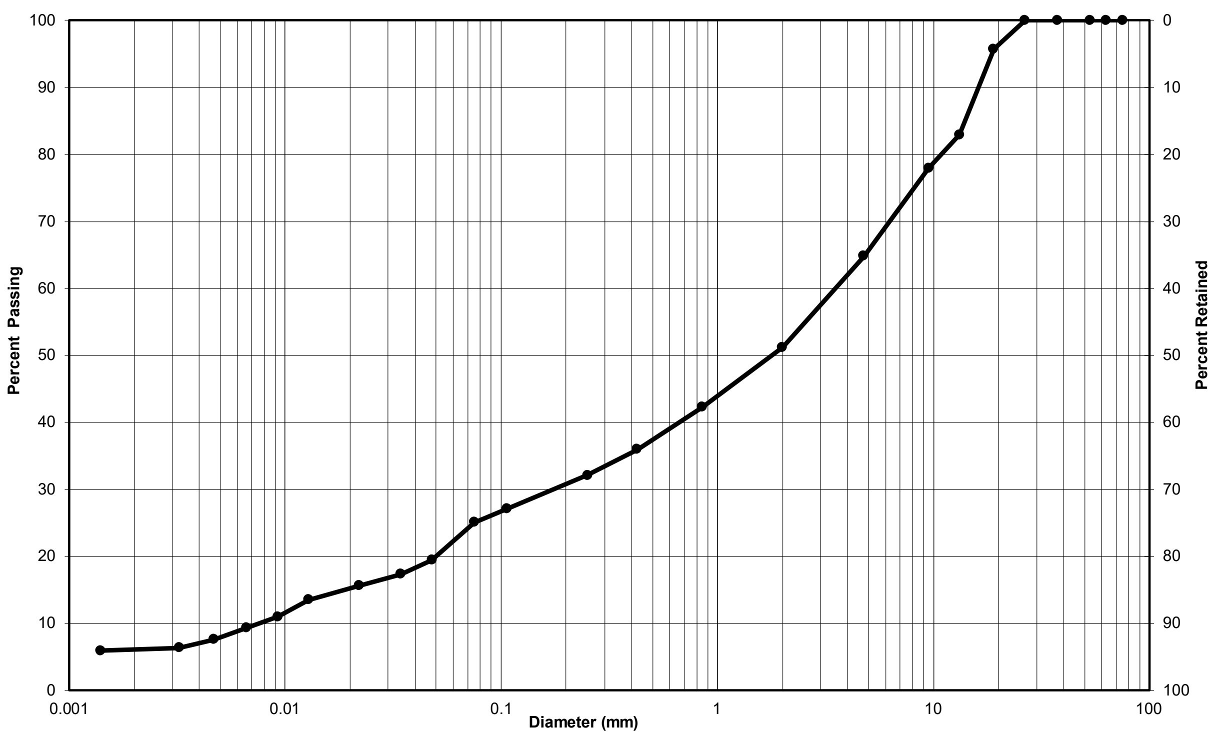
Appendix B

Geotechnical Laboratory Test Results

Particle-Size Analysis of Soils (Geotechnical) (USCS) (ASTM D422)

Client:	DPH Developments	**Lab No.:**	SS-21-70
Project/Site:	Norwood Subdivision	**Project No.:**	11231077

Borehole no.:	MW1-21	Sample no.:	SS3
Depth:	1.5 to 1.8m	Enclosure:	



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

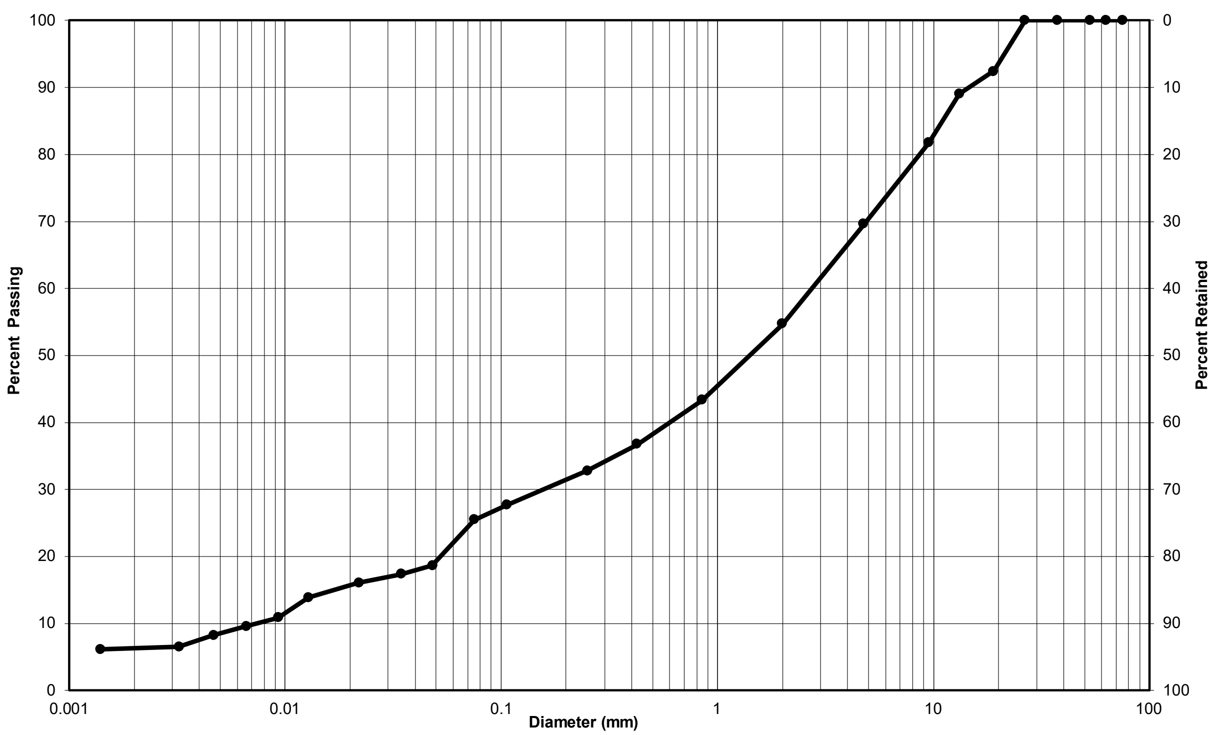
Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Gravelly, silty sand	35	40	25
Silt-size particles (%):	19		
Clay-size particles (%) (<0.002mm):	6		

Remarks:

Performed by:	Jade Gorman	**Date:**	August 27, 2021
Verified by:	Joe Sullivan	**Date:**	September 7, 2021

Particle-Size Analysis of Soils (Geotechnical) (USCS) (ASTM D422)

Client:	DPH Developments	**Lab No.:**	SS-21-70
Project/Site:	Norwood Subdivision	**Project No.:**	11231077
Borehole no.:	MW3-21	Sample no.:	SS4
Depth:	2.3 to 2.6m	Enclosure:	



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Gravelly, silty sand	30	44	25
Silt-size particles (%):	19		
Clay-size particles (%) (<0.002mm):	6		

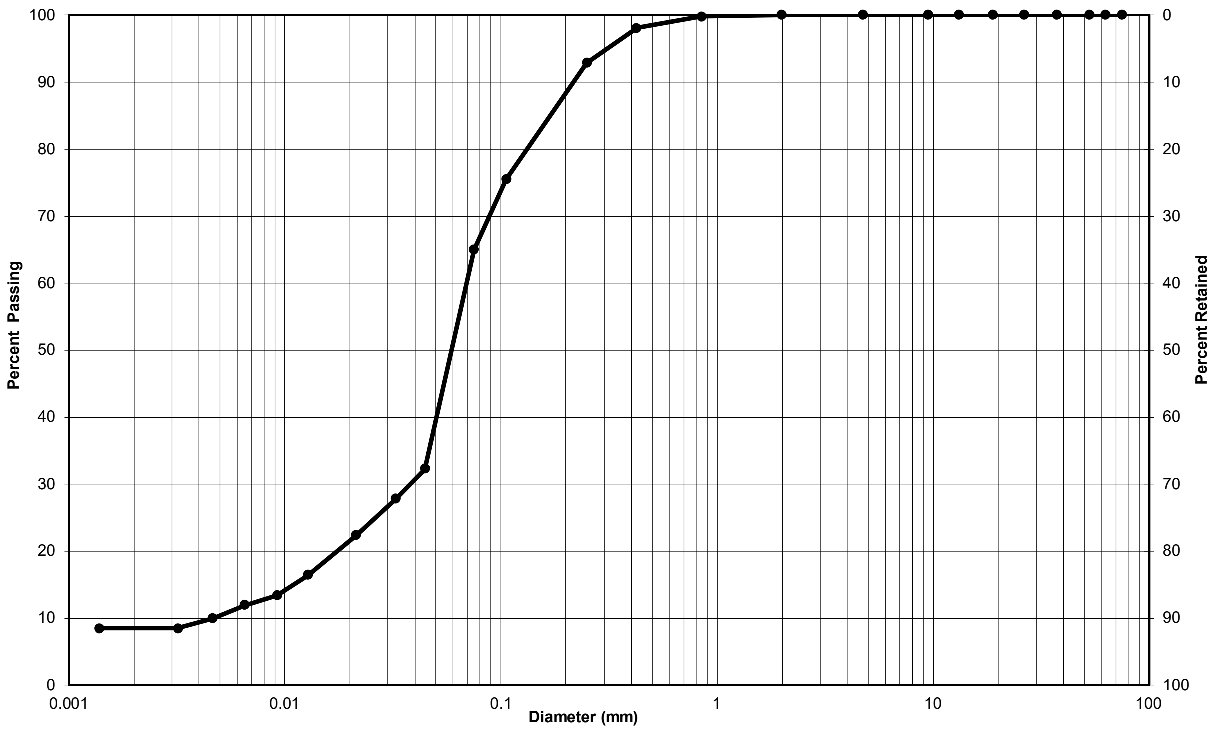
Remarks:

Performed by:	Jade Gorman	**Date:**	August 27, 2021
Verified by:	Joe Sullivan	**Date:**	September 7, 2021

Particle-Size Analysis of Soils (Geotechnical) (USCS) (ASTM D422)

Client:	DPH Developments	**Lab No.:**	SS-21-70
Project/Site:	Norwood Subdivision	**Project No.:**	11231077

Borehole no.:	BH4-21	Sample no.:	SS2A
Depth:	0.8 to 1.1m	Enclosure:	



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Sandy silt	0	35	65
Silt-size particles (%):	57		
Clay-size particles (%) (<0.002mm):	8		

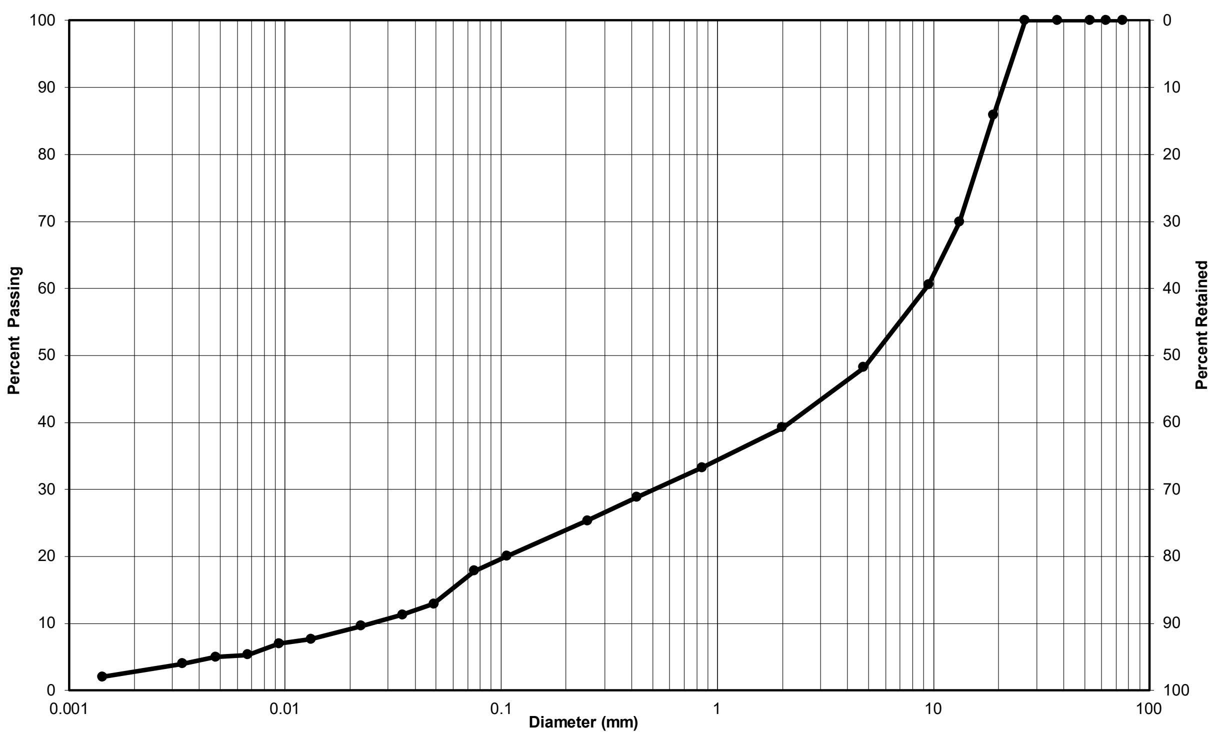
Remarks:

Performed by:	Jade Gorman	**Date:**	August 27, 2021
Verified by:	Joe Sullivan	**Date:**	September 7, 2021

Particle-Size Analysis of Soils (Geotechnical) (USCS) (ASTM D422)

Client:	DPH Developments	**Lab No.:**	SS-21-74
Project/Site:	Norwood Park Subdivision - Phase 4	**Project No.:**	11231077

Borehole no.:	TP1-21	Sample no.:	GS3
Depth:	0.8 to 0.9m	Enclosure:	



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Silty gravel with sand (GM)	52	30	18
Silt-size particles (%):	15		
Clay-size particles (%) (<0.002mm):	3		

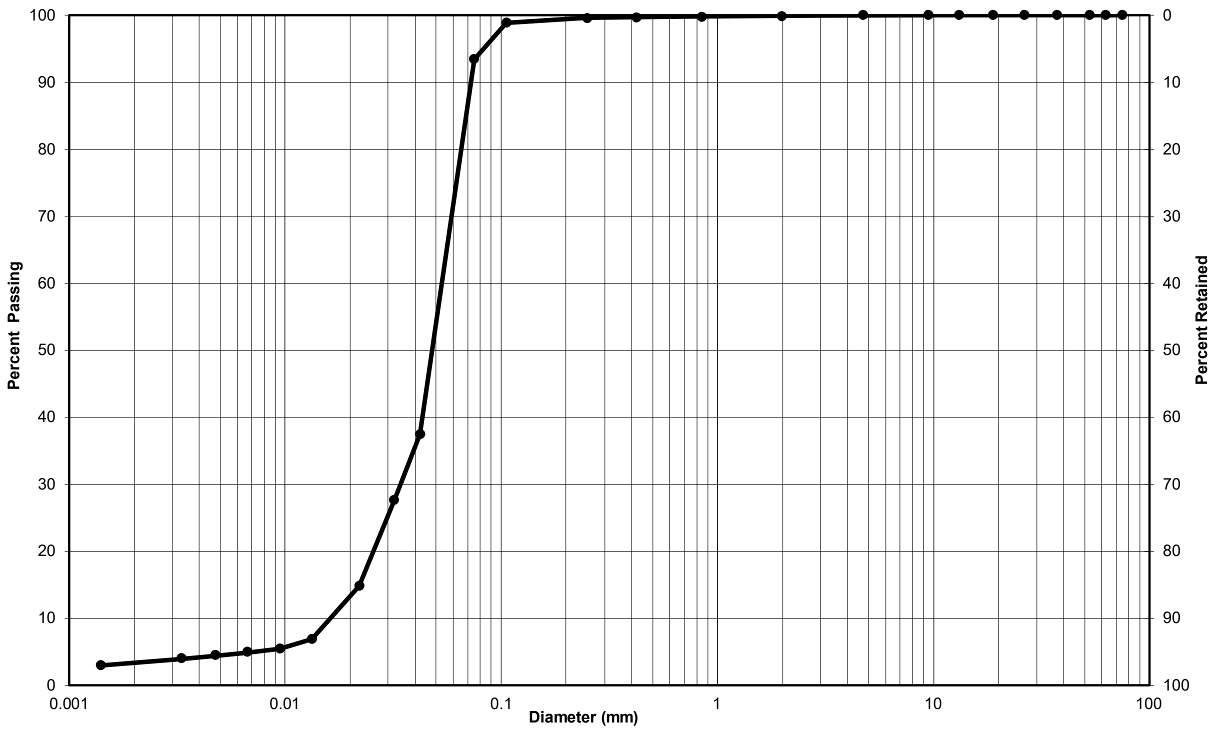
Remarks:

Performed by:	Jade Gorman	**Date:**	September 3, 2021
Verified by:	Joe Sullivan	**Date:**	September 13, 2021

Particle-Size Analysis of Soils (Geotechnical) (USCS) (ASTM D422)

Client:	DPH Developments	**Lab No.:**	SS-21-74
Project/Site:	Norwood Park Subdivision - Phase 4	**Project No.:**	11231077

Borehole no.:	TP4-21	Sample no.:	GS3
Depth:	0.6 to 0.9m	Enclosure:	



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

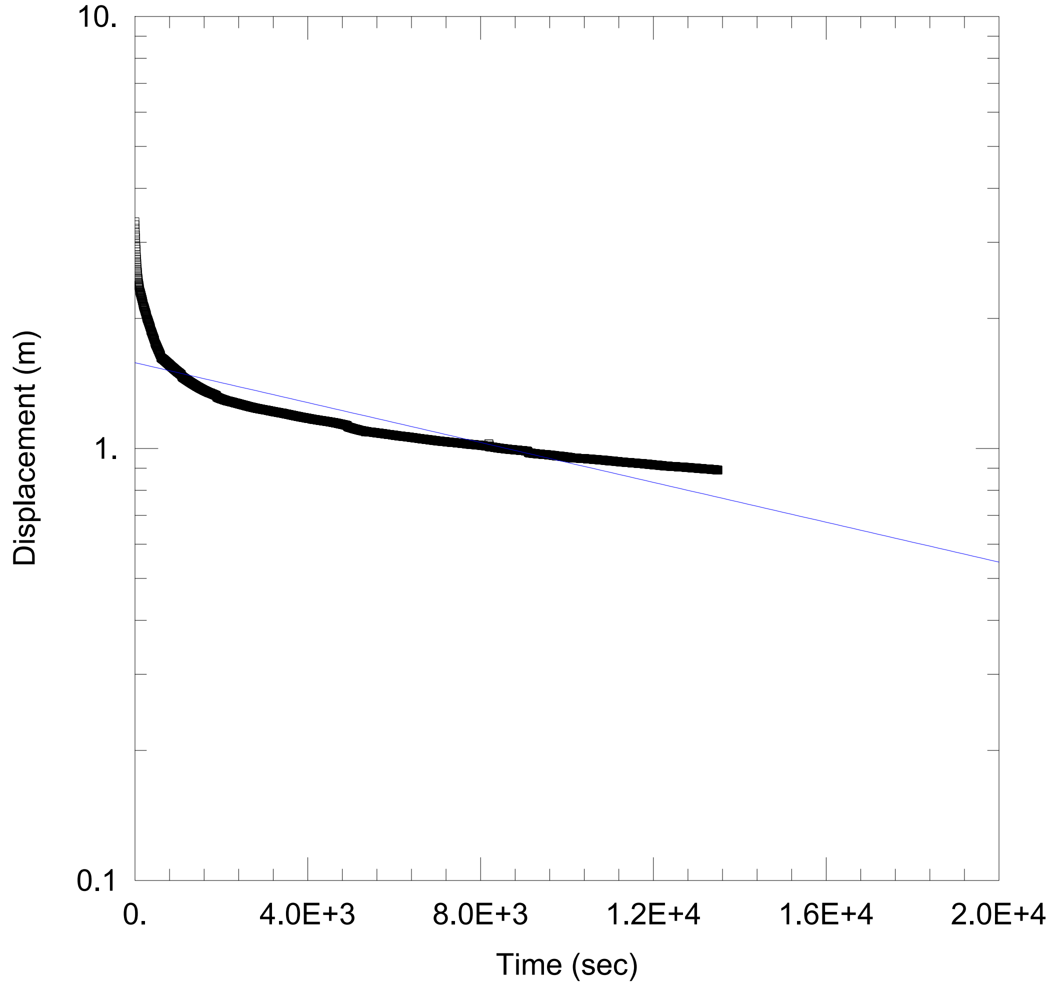
Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Silt (ML)	0	7	93
Silt-size particles (%):	90		
Clay-size particles (%) (<0.002mm):	3		

Remarks:

Performed by:	Jade Gorman	**Date:**	September 3, 2021
Verified by:	Joe Sullivan	**Date:**	September 13, 2021

Appendix C

Hydraulic Conductivity and Infiltration Testing Results



MW1-21 - FALLING HEAD TEST

Data Set: G:\...\11231077 - Falling Head Test MW1-21.aqt
Date: 10/05/21 Time: 09:28:34

PROJECT INFORMATION

Company: GHD
Client: DPH Developments Inc.
Project: 11231077
Location: Norwood Park Subdivision Ph.4
Test Well: MW1-21
Test Date: August 27, 2021

AQUIFER DATA

Saturated Thickness: 0.1 m Anisotropy Ratio (Kz/Kr): 1.

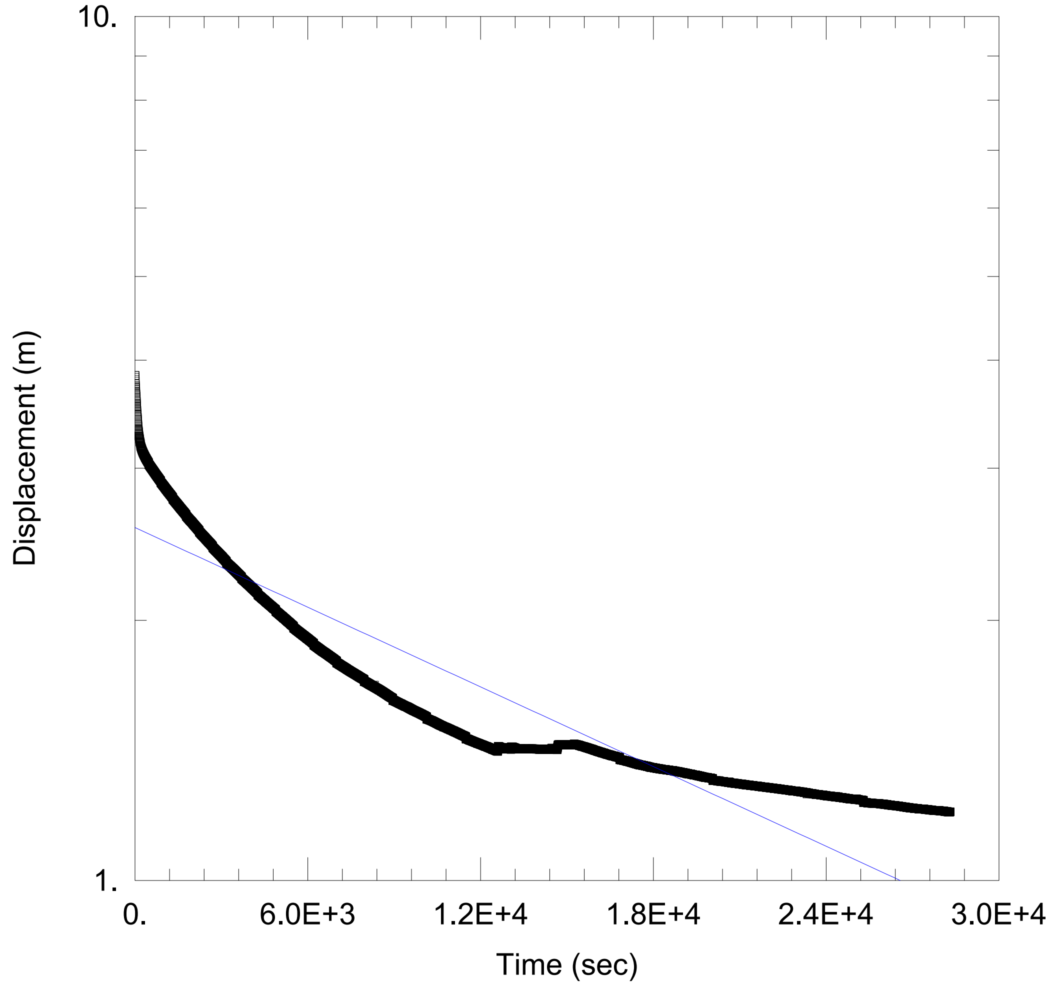
WELL DATA (MW1-21)

Initial Displacement: 3.345 m Static Water Column Height: 0.1 m
Total Well Penetration Depth: 1.6 m Screen Length: 1.5 m
Casing Radius: 0.025 m Well Radius: 0.025 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 3.661E-5 cm/sec y0 = 1.579 m



MW2-21 - FALLING HEAD TEST

Data Set: G:\...\11231077 - Falling Head Test MW2-21.aqt
Date: 10/05/21 Time: 09:31:47

PROJECT INFORMATION

Company: GHD
Client: DPH Developments Inc.
Project: 11231077
Location: Norwood Park Subdivision Ph.4
Test Well: MW2-21
Test Date: August 27, 2021

AQUIFER DATA

Saturated Thickness: 0.1 m
Anisotropy Ratio (Kz/Kr): 1.

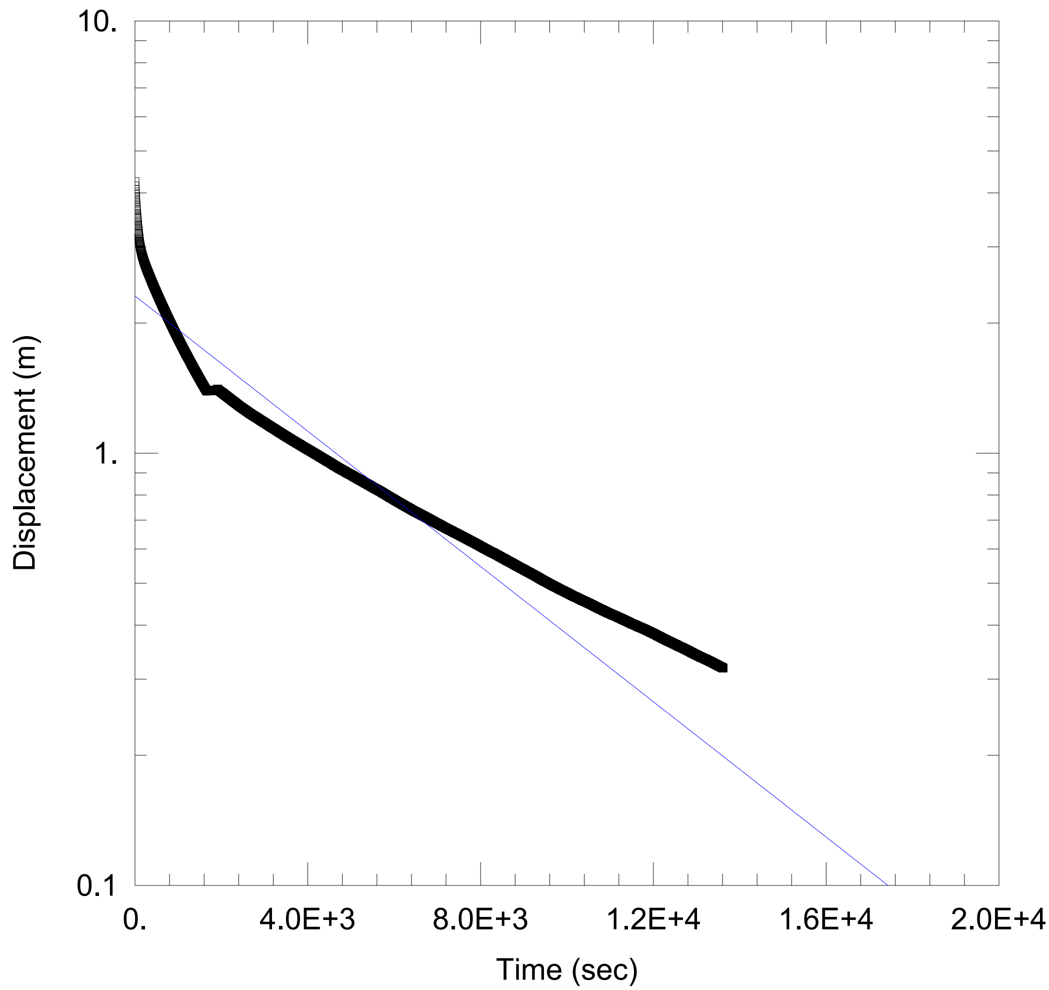
WELL DATA (MW2-21)

Initial Displacement: 3.842 m
Static Water Column Height: 0.1 m
Total Well Penetration Depth: 1.6 m
Screen Length: 1.5 m
Casing Radius: 0.025 m
Well Radius: 0.025 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice

K = 2.439E-5 cm/sec
y0 = 2.561 m



MW3-21 - FALLING HEAD TEST

Data Set: G:\...\11231077 - Falling Head Test MW3-21.aqt
Date: 10/05/21 Time: 09:34:44

PROJECT INFORMATION

Company: GHD
Client: DPH Developments Inc.
Project: 11231077
Location: Norwood Park Subdivision Ph.4
Test Well: MW3-21
Test Date: August 27, 2021

AQUIFER DATA

Saturated Thickness: 0.1 m
Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (MW3-21)

Initial Displacement: 4.249 m
Static Water Column Height: 0.1 m
Total Well Penetration Depth: 1.6 m
Screen Length: 1.5 m
Casing Radius: 0.025 m
Well Radius: 0.025 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice

K = 0.0001241 cm/sec
y0 = 2.311 m

Appendix C.2: Infiltration Testing (in-situu)

Project No. 11231077
Date: August 27, 2021
Equipment: ETC Pask Permeameter

Location:	TP-3		TP-3		TP-7		TP-11	
Depth of hole:	0.3 m		1.4 m		0.6 m		0.6 m	
	Test 1		Test 1		Test 1		Test 1	
	Elapsed Time (minutes)	Permeameter Level (cm)	Elapsed Time (minutes)	Permeameter Level (cm)	Elapsed Time (minutes)	Permeameter Level (cm)	Elapsed Time (minutes)	Permeameter Level (cm)
	0.0	40.0	0.0	40.0	0.0	45.5	0.0	43.5
	0.5	37.0	0.5	36.9	0.5	44.8	0.5	41.3
	1.0	34.3	1.0	35.9	1.5	42.5	1.0	40.4
	1.5	31.9	1.5	35.0	2.0	40.5	1.5	39.5
	2.0	29.5	2.0	34.5	2.5	36.5	2	38.5
	2.5	27.5	2.5	33.5	3.0	32.5	2.5	37.8
	3.0	25.5	3.0	32.9	3.5	24.5	3.0	36.9
	3.5	23.5	3.5	--	4.0	20.5	3.5	36.3
	4.0	21.5	4.0	31.6	4.5	16.5	4.0	35.5
	4.5	19.5	4.5	31.3	5.0	12.5	4.5	34.8
	5.0	17.5	5.0	30.5	5.5	8.5	5.0	34.0
	5.5	16.0	5.5	30.0	6.0	4.5	5.5	33.3
	6.0	14.0	6.0	29.6	6.5	0.5	6.0	32.5
	6.5	12.0	7.0	28.6			6.5	31.8
	7.0	10.5	8.0	27.7			7.0	31.2
	7.5	9.0	9.0	27.0			7.5	30.4
	8	7.5	10	26.3			8.0	29.8
	8.5	6.0	11.0	25.6			8.5	29.2
	9.0	4.5	12.0	24.8			9.0	28.4
	9.5	3.0	13.0	24.1			10.0	27.2
	10.0	1.5	14.0	23.4			11.0	25.6
			15.0	22.7			12.0	24.4
							14.0	22.0
							16.0	19.6
							18.0	17.2
Quasi Steady Flow Rate ® (cm/min)		3		0.7		8		1.2
Field-saturated Hydraulic Conductivity (Ksf) (m/sec)		1.60E-05		3.70E-06		4.30E-05		6.40E-06