REPORT

Preliminary Geotechnical and Hydrogeological Investigation Report

Proposed Residential Development at Old Church Road, Caledon East, Ontario

Submitted to:

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Distribution List

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Stylux Caledon Inc. (Stylux) to carry out a preliminary geotechnical and hydrogeological investigation for a proposed residential development to be constructed near Old Church Road and Russel Mason Ct., in Caledon East, Ontario (the Site), as shown in the Site and Borehole Location Plan, Figure 1. The terms of reference for the geotechnical/hydrogeological consulting services are included in Golder's Proposal No. P18111428, dated February 28, 2019.

The purpose of the investigation is to obtain information on the general subsurface soil and shallow groundwater conditions at the site by means of a limited number of boreholes and geotechnical laboratory tests. Based on our interpretation of the factual information collected as a part of the preliminary geotechnical investigation carried out at this site, a general description of the subsurface conditions across the site is presented herein. The interpreted subsurface conditions and available project details were used to develop preliminary engineering recommendations on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

This report provides the results of the preliminary geotechnical/hydrogeological investigation and should be read in conjunction with the *"Important Information and Limitations of This Report"* (attached). The reader's attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report. The factual data, interpretations and 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. If the project is modified in concept, location or elevation, or if the project is not initiated within eighteen months of the date of the report, Golder should be given an opportunity to confirm that the recommendations in this report are still valid.

2.0 SITE AND PROPOSED DEVELOPMENT DESCRIPTION

The Site consists of a gross area of approximately 1.61 ha and is located at 6098, 6126, 6142 Old Church Rd., 2 Russel Mason Ct. and 1 Marilyn Street, Caledon East, Ontario as shown in Figure 1. We understand that future site development may also include properties 3 and 5 Marilyn Street; however, at the time of this investigation access to these properties was not made available to Golder.

The property is bounded by Old Church Road to the south, Marylin Street to the west and residential houses to the north and east. At the time of our investigation, the property contained five residential houses, assumed to have basements, various storage sheds, asphalt/gravel driveways, open field areas from previously demolished houses, landscape areas with grasses, shrubs and trees.

Based on our understanding, the Site is to be redeveloped into a higher density residential use. Based on a Concept Plan by KLM Planning Partners Project No. P-2967 dated November 27, 2018 provided by Stylu, the purposed development will include 19 single detached residential homes with basements and the supporting roadway and underground infrastructure.

3.0 INVESTIGATION PROCEDURE

The field work for this preliminary geotechnical/hydrogeological investigation was carried out between March 25 and April17, 2019, during which time five boreholes (designated as Boreholes BH19-1 to BH19-5) were advanced at the Site to depths between about 4.42 m and 9.75 m below existing ground surface at the approximate locations shown in the Site and Borehole Location Plan, Figure 1, attached. The borehole locations were determined in the field using a GPS instrument based on UTM coordinates.

Prior to initiating the field work, Golder contacted Ontario One Call; which in turn notified public utility companies to locate and clear existing underground services. As boreholes were located on private property, Golder also retained a private utility locating contractor to scan the borehole locations for buried services prior to drilling.

The boreholes were advanced using a truck-mounted drill rig supplied and operated by a specialist drilling contractor, subcontracted to Golder. Standard Penetration Testing (SPT) and sampling were carried out at regular intervals of depth in the boreholes using conventional 35 mm internal diameter split-spoon sampling equipment advanced using an automatic hammer in accordance with ASTM D1586-08a.

The shallow groundwater conditions were recorded in the open boreholes during and immediately following the drilling operations. Three of the boreholes advanced at the site were equipped with 50 mm diameter monitoring wells to permit further monitoring of the groundwater levels, and to carry out in-situ hydrogeological testing. The well installation details and water level readings are presented on the Record of Borehole sheets.

The field work for this investigation was directed by members of our engineering staff who also logged the boreholes, directed the sampling and cared for the samples obtained. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder’s Mississauga geotechnical laboratory for further examination and laboratory testing. Classification testing, consisting of water content determinations and grain size distribution were carried out on selected soil samples. The results of the geotechnical laboratory tests are included in Appendix B and also on the Record of Borehole sheets, in Appendix A.

4.0 SUBSURFACE CONDITIONS

The subsurface soil and shallow groundwater conditions encountered in the boreholes, as well as the results of the field and laboratory testing, are shown in detail on the Record of Boreholes sheets, in Appendix A. Method of Soil Classification and Symbols and Terms Used on the Records of Boreholes are provided to assist in the interpretation of the Record of Boreholes. It should be noted that the boundaries between the strata have been inferred from drilling observations and non-continuous samples. They generally represent a transition from one soil type to another and should not be inferred to represent an exact plane of geological change. Further, conditions will vary between and beyond the boreholes. The following is a summarized account of the subsurface conditions encountered in the boreholes advanced during this investigation, followed by more detailed descriptions of the major soil strata and shallow groundwater conditions.

In general, the subsurface conditions encountered at the boreholes advanced at the site consist of a surficial layer of topsoil/gravel/silty sand fill layer underlain by non-cohesive silty sand/sandy silt/gravelly sand.

Details of the observations of the groundwater during and upon completion of drilling are summarized on the Record of Boreholes sheets. Shallow groundwater was encountered at depths ranging from 6.1 m below existing ground surface to a depth of up to about 7.2 m below existing ground surface upon completion of drilling. Shallow groundwater levels measured in the three, 50 mm diameter monitoring wells installed at the site were recorded at depths of about 6.3 m to 7.3 m below the existing ground surface on April 3 and 17, 2019.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.1 Topsoil

Topsoil materials were encountered in Borehole BH19-4 with an approximate thickness of about 150 mm.

Materials identified as topsoil in this report were classified based on visual and textural evidence as no other testing for organic content or other nutrients was carried out. As such, the ability for these materials to support vegetation has not been assessed.

4.2 (SM) Surficial Fill Materials - Silty Sand

A non-cohesive silty sand fill was encountered in Boreholes BH19-1 and BH19-3 at existing ground surface and BH19-4 below the topsoil layer. The existing surficial fill material consists of various amounts of gravel, some to mixed organics, plastic fines, and rootlets and is generally brown to dark brown in colour. The thickness of the fill material ranges from about 0.3 m to 0.6 m.

The SPT ‘N’ values measured in this fill layer ranged from about blows 19 to 36 blows per 0.3 m of penetration, indicating a compact to dense compactness.

The natural water content measured on the silty sand fill layer ranged from about 14 per cent to 27 per cent.

4.3 (SM/ML) Silty Sand to Sandy Silt

A non-cohesive silty sand to sandy silt deposit was encountered at ground surface at BH19-2 and BH19-5 and underneath the fill/topsoil in Boreholes BH19-1, BH19-3 and BH19-4 and ranged from about 4.4 m to 9.8 m in thickness. BH19-1, BH19-2, BH19-3 and BH19-4 terminated within this deposit. The silty sand to sandy silt deposit consists of various amount of gravel with periodic silt seams and is light brown to grey in color. Cobbles and/or boulders are inferred to be present in borehole BH19-1 by auger grinding at a depth of about 5.8 m below ground surface.

The SPT ‘N’ values of this non-cohesive deposit ranged from about 3 blows to 53 blows per 0.3 m of penetration, indicating a compactness ranging from very loose to very dense, but generally was found to be compact throughout the deposit.

The natural water content measured within this deposit ranged from about 3 per cent to about 19 per cent, but generally was found to be less then 10 per cent.

The results of grain size distribution test carried out on four samples of the silty sand are shown on Figure B1-A.

The results of grain size distribution test carried out on three samples of the sandy silt to sand and silt are shown on Figure B2.

4.4 Gravelly Sand

A non-cohesive gravelly sand was encountered underneath the non-cohesive silty sand at borehole BH19-5 at a depth of about 7.0 m below existing ground surface. Borehole BH19-5 was terminated within this deposit.

The SPT ‘N’ value of this deposit was measured to be 9 blows per 0.3 m of penetration, indicating a loose, compactness.

The result of a grain size distribution test carried out on a sample of this deposit is presented in Figure B3.

The water content of selected samples ranged from about 8.2 per cent.

4.5 Groundwater Levels

Groundwater observations were carried out in the open boreholes during and upon completion of drilling. Subsequent water level measurements in the monitoring wells installed at Boreholes BH19-1, BH19-2 and BH19-4 were also carried out. The shallow groundwater levels measured in the monitoring wells on selected dates are summarized as follows:

Table 1: Groundwater Level Measurements

Borehole No.	Measurements Upon Completion of Drilling		Measurements in Monitoring Wells			
	Approximate Groundwater Depth (mbgs)	Date	Approximate Groundwater Depth (mbgs)	Date	Approximate Groundwater Depth (mbgs)	Date
BH19-1	7.2	March 26, 2019	7.2	April 3, 2019	7.3	April 17, 2019
BH19-2	6.1	March 25, 2019	6.6	April 3, 2019	6.8	April 17, 2019
BH19-3	Dry (>4.4)	March 25, 2019	No monitoring well installed			
BH19-4	7.0	March 25, 2019	7.3	April 3, 2019	7.3	April 17, 2019
BH19-5	Dry (>8.2)	March 26, 2019	No monitoring well installed			

Note:
mbgs = meters below ground surface.

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year.

4.6 Hydraulic Testing

To estimate the hydraulic conductivity of the soils adjacent to the screened interval in the monitoring wells installed in Boreholes BH19-1, BH19-2 and BH19-4, single-well response tests were carried out by Golder on April 3, 2019. The tests were carried out by rapidly purging a known volume of water with a dedicated Waterra tube and footvalve and monitoring the subsequent water level recovery.

The Bouwer-Rice (1976) method was applied to rising head test data using the unconfined solution. The data was analyzed using the AQTESOLV for Windows version 4.50 Professional software. The estimated hydraulic conductivity values obtained from the rising head tests are summarized in the table below. A summary of the single-well response test data and the AQTESOLV printouts are provided in Appendix C.

Monitoring Well ID	Screened Interval Depth (mbgs)	Groundwater Condition	Screened Stratigraphy	Est. Hydraulic Conductivity (m/s)
BH19-1	6.1 to 9.1	Unconfined	SILTY SAND	1×10^{-6}
BH19-2	4.6 to 7.6	Unconfined	SILTY SAND / sandy SILT	5×10^{-6}
BH19-4	6.1 to 9.1	Unconfined	SILTY SAND / SAND and SILT	1×10^{-6}

Notes:
mbgs = meters below ground surface
m/s = metres per second

The estimated hydraulic conductivity values are considered reasonable for the units tested.

5.0 GEOTECHNICAL ENGINEERING DISCUSSION

This section of the report provides preliminary geotechnical engineering recommendations on the geotechnical aspects of the proposed development based on our interpretation of the limited borehole information and on our understanding of the project scope and requirements. The information in this portion of the report is provided for the guidance of the design engineers and professionals. Where comments are made on construction, they are provided only in order to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking any work at the site should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, equipment capabilities, costs, sequencing and the like.

This report addresses only the geotechnical (physical) aspects of the subsurface conditions at this site. The geo-environmental (chemical) aspects, including the consequences of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources, are outside of the terms of reference for this report.

Based on the results of this preliminary investigation, the subsurface soil conditions encountered at the site are considered to generally be suitable for the proposed residential development which is understood to comprise of residential houses with one underground basement level, underground services and paved driveways, roads or laneway. However, at the time of preparation of this report, proposed design grades (i.e., finished floor, pavement subgrade and utility invert levels) were not available for the proposed development. The following engineering recommendations regarding the geotechnical design aspects of the project including underground services, pavements and building foundations should be considered as preliminary only and should be reviewed when the final design grades and utility invert levels have been finalized to confirm that they are still applicable.

5.1 Site Preparation

5.1.1 Subgrade Preparation

Based on the existing site topography, it is assumed that only minor cut and/or fill site grading operations of less than 1.0 m will be required to establish subgrade levels and permit the construction of the proposed residential development. However, in the area of the existing residential dwellings, fills of up to 2.5 m may be required once the former underground structures/basement are removed during the redevelopment.

Any filling carried out at the site in conjunction with regrading (with the exception of future green spaces) should be carried out as engineered fill. Recommendations for the placement of engineered fill are outlined in Section 5.1.2 of this report, titled “Engineered Fill Requirements”.

In general, the existing site vegetation, surficial topsoil/organics, surficial asphalt/concrete or the sandy silt fill with organics and other near-surface soils containing significant amounts of organic matter or construction debris are not considered to be suitable for the subgrade support of engineered fill, building foundations, floor slabs, or other settlement sensitive structures. These materials should be completely stripped prior to placing any engineered fill or construction of foundations or interior or exterior slab-on-grade(s), following appropriate environmental procedures. Furthermore, excessively wet soils should be dried before reuse as engineered fill.

The thicknesses of the concrete slabs within the footprint of the existing buildings and the condition of any fill underneath the slab or around the existing residential houses, was not assessed during this investigation. Therefore, when the granular fill and the underlying subgrade material is encountered underneath the existing structures or concrete slabs during construction activities, the acceptance of such fill as suitable to reuse on the site should be assessed by a qualified geotechnical engineer.

Former structures (existing buildings, sewers, etc.) located on site, will have to be removed or decommissioned. Remedial actions, such as removal of existing foundations or re-compaction of backfill will be required, as directed by the geotechnical engineer and the recommendations contained in the report.

Following the stripping of the surficial topsoil, fill and soils containing significant amounts of organics and/or soft/disturbed surficial soils, the exposed subgrade should be heavily proof-rolled with suitable equipment, in conjunction with inspection by qualified geotechnical personnel to confirm that the exposed soils are competent and have been adequately stripped of ponded water and all disturbed, loosened, softened, organic and other deleterious material. Remedial work (i.e., further subexcavation and replacement) should be carried out on poorly-performing areas identified during the proof-rolling activities, as directed by Golder.

5.1.2 Engineered Fill Requirements

As stated above, the anticipated site grading activities include both cutting and filling to meet the final design site grades.

Following the stripping of existing topsoil and removal of fill material containing organic materials, in general, the existing fill material and silty sand native material is considered to be acceptable for reuse as engineered fill. Based on the laboratory test results, the water content of the soils present at the site are considered to be generally near or below the optimum water content for compaction, and therefore will probably require some wetting prior to placement.

It should be noted that the fill and native material at the site are silty in nature, and as such are **susceptible** to wet/inclement weather and freezing temperature. Therefore, it is recommended that site grading activities not be carried out during late fall, winter, early spring seasons or any periods of inclement weather conditions. All oversized cobbles (i.e., greater than 150 mm in size) and boulders, if present, should be removed from excavated material that will be used as engineered fill.

If imported material is required for the engineered fill process, the material that is proposed for use as engineered fill should be approved by the geotechnical engineer at its source, prior to importing the material to the site. Suitable soils, free of topsoil, organic matter or other deleterious materials can be used as engineered fill provided that the water content of the soil at the time of placement does not vary by more than 2 percent above or below its optimum water content for compaction. Otherwise, the soils may require treatment (i.e., drying or wetting) prior to placement.

Following the inspection and approval of the subgrade as described previously in this report, engineered fill materials should be placed in maximum 300 mm-thick loose lifts and uniformly compacted to 98 percent of the Standard Proctor maximum dry density (SPMDD). Filling should continue until the design elevations are achieved.

Full-time monitoring and in-situ density testing should be carried out by Golder during placement of engineered fill.

The final surface of the engineered fill should be protected as necessary from construction traffic and should be sloped to provide positive drainage for surface water during the construction period. If the engineered fill materials will be left exposed (i.e. uncovered) during periods of freezing weather, additional soil cover should be placed above final subgrade to provide some level of frost protection. Prior to placing the granular subbase and/or base courses within pavement areas, the surface of the engineered fill/subgrade should be inspected by Golder.

For the silty sand to sandy silt material, normal post-construction settlement of the engineered fill materials should be anticipated with the majority of such settlement taking place within about 1 to 3 months following completion of filling operations. If, however, the specified degree of compaction is reduced and/or the engineered fill operations are completed during the winter months, post-construction settlements will increase beyond typical anticipated values, and settlements will be reflected at the ground surface.

5.2 Installation of Underground Services

5.2.1 Temporary Excavations

Details of underground servicing for the proposed development are unknown at the time of this investigation; as such, for the purpose of this report, the maximum depth of the underground services was assumed to be about 3 m below the existing ground surface. Once detailed design is complete, review of the underground services should be completed by this office for compliance with the recommendations contained herein.

The founding soils are anticipated to generally consist of the native silty sand deposits. These materials are considered to be suitable for supporting the underground services provided that the integrity of the base of the trench excavations is maintained during construction. Where softened or disturbed native soils or other deleterious materials are encountered at the base of excavations for settlement-sensitive services, these materials should be sub-excavated and replaced with compacted fills approved by the geotechnical engineer.

Based on the groundwater levels measured in monitoring wells during the investigation (April 2019), the groundwater levels at the Site range from approximately 6.6 m to 7.3 m below existing ground surface (mbgs). Assuming the underground services will be about 3 mbgs, the underground services will be above the local water table. As such, it is not anticipated that lowering of the water table will be required. Removal of direction precipitation into the excavations can probably be handled, as required, by pumping from properly constructed and filtered sumps located within the excavations. This finding should be reviewed upon finalization of the design invert elevations. It should be noted that water takings (dewatering) in excess of 50 m^3/day are regulated by the Ministry of Environment, Conservation and Parks (MECP). Certain takings of groundwater and storm water for construction site dewatering purposes with a combined total less than 400 m^3/day qualify for self-registration on the MECP's Environmental Activity and Sector Registry (EASR). A Category 3 Permit to Take Water (PTTW) is required where the proposed water taking is greater than 400 m^3/day.

Care should be taken to direct surface water away from any open excavations and all temporary excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects.

Excavations for the site servicing would generally extend through the fill material and silty sand native deposits. Conventional excavation equipment should be suitable to excavate through these materials.

For trench excavations (i.e., for servicing) extending predominantly through the fill material and silty sand native deposit it is anticipated that conventional temporary open cuts may be developed with side slopes not steeper than 1 horizontal to 1 vertical; this is under the assumption that the existing non-cohesive sandy soils would be under dry conditions.

5.2.2 Pipe Bedding and Cover

The bedding for the sewers and watermains should be compatible with the size, type and class of pipe and the surrounding subsoil and the requirements of the Region of Peel and Town of Caledon. If granular bedding is deemed to be acceptable, then Ontario Provincial Standard Specifications (OPSS) Granular A should be used from at least 150 mm below invert to springline. Clear stone should not be used as bedding material. From springline to 300 mm above the obvert of the pipe, sand cover could be used. All bedding and cover material should be placed in 150 mm loose lifts and uniformly compacted to at least 100 percent of SPMDD. Where variable fill materials, softened or disturbed native soils or other deleterious materials are encountered at the base of excavations for settlement-sensitive services, these materials should be subexcavated and replaced with compacted fills approved by the geotechnical engineer.

5.2.3 Trench Backfill

The excavated materials from the site will consist predominantly of silty sand materials. Based on the measured water contents, in general, the fill materials are estimated to be above optimum water contents for compaction and will require drying prior to placement. The native materials encountered at the site are estimated to be near their optimum water contents for compaction, and therefore, will probably require only minor drying or wetting prior to placement.

Care should be taken to maintain the water content of the soils close to/at the optimum water content for compaction during the construction operations, as difficulties with compaction and/or backfill performance would be anticipated with fine-grained soils where the water content is significantly above the optimum for compaction purposes. Soils that contain significant quantities of organics or debris are also not suitable for use as trench backfill within settlement-sensitive areas. In addition, all boulders and cobbles greater than 150 mm in size should be removed from the trench backfill materials. If there is a shortage of suitable in-situ material, an approved imported material such as Ontario Provincial Standard Specifications Select Subgrade Material should be used for trench backfill. Again, as noted above, the trench backfill materials are silty in nature and are very susceptible to wetting/freezing temperatures. Backfilling trenches during cold or wet weather is not recommended.

Trench backfill should be placed in maximum 300 mm loose lift thickness and uniformly compacted to at least 98 percent of the SPMDD of the material. Soil that is frozen should not be used as backfill.

Normal post-construction settlement of the compacted trench backfill should be anticipated with the majority of such settlement taking place within about 6 months following the completion of trench backfilling operations. If the trench backfill operations are completed during the winter months, post-construction settlements may increase beyond typical anticipated values. These settlements will be reflected at the ground surface. If the asphalt binder course is laid shortly following the completion of the trench backfilling operations, any settlement that may be reflected by subsidence of the surface of the binder asphalt should be compensated for by placing an additional thickness of binder asphalt or by padding. If possible, the surface course asphalt should not be placed over the binder course asphalt for about 12 months. Where scheduling requires that the surface course be placed over the binder course asphalt before this period, trench backfill settlement would be reflected by subsidence and possible cracking of the finished pavement surface in these areas which, depending upon the extent and magnitude, may require local repairs.

5.3 Exterior Flatwork

5.3.1 Subgrade Inspection

In case exterior concrete flatwork such as sidewalks may be required, the subgrade for the exterior concrete flatwork should be inspected during construction to determine if it is capable of supporting the proposed loads and to identify soft zones and areas of unsuitable subgrade soil. Therefore, during construction, the prepared subgrade should be proofrolled in conjunction with an inspection by Golder. Remedial work should be carried out on any softened, disturbed, wet or poorly performing zones as directed by Golder. Any low areas may then be brought up to within at least 300 mm of the underside of the walkway slab, as required, using OPSS Granular B, Type I material or other approved non-frost susceptible materials, placed in maximum 150 mm loose lifts and uniformly compacted to at least 98 percent of the standard Proctor maximum dry density (SPMDD).

The final lift of granular fill beneath concrete flatwork should consist of a minimum thickness of 300 mm of OPSS Granular A material, uniformly compacted to at least 100 percent of SPMDD. Any filling operations should be inspected and tested by Golder.

5.3.2 Frost Susceptibility

The native soils encountered in the boreholes are classified as highly frost susceptible.

Frost heave is caused by formation of ice and/or ice lenses within the soil which increase the overall soil volume which in turn can generate considerable pressure in the process. For ice lenses to form, three conditions must be present:

- The soil temperature must be below freezing;
- The soil must be frost susceptible; and
- Sufficient water must be present at or near the freezing temperature.

For any proposed concrete flatwork that will not be within a heated area, the subsurface soils will be exposed to freezing temperatures during winter. The flatworks will generally be cleared of snow and ice and therefore the frost penetration during the winter may potentially extend to the full local maximum frost penetration of 1.2 m below the ground surface. Any grass covered areas adjacent to the flatwork or construction joints will allow water to infiltrate and pool within the granular base and serve as a source of water for ice lensing.

To minimize the effects of frost heave, frost susceptible soils within 1.2 m of the final grade may be removed and replaced with non-frost susceptible soils (generally free draining sandy soils with minimal (<10% fines). Alternatively, consideration could be given to thermal insulation which would reduce or eliminate the requirement for subgrade soil removal and replacement with non-frost susceptible material.

In order to reduce frost heaving, we recommend that the following measures be considered in the design, in addition to the replacement of the subgrade soils with non-frost susceptible material:

- Install solid subdrains associated with strategically located grated surface drains and underslab perforated and filtered subdrains. The subdrains should outlet to the nearby catchbasins. Golder would be pleased to recommend locations and design details for the drains, if requested;
- Sealing all of the concrete joints with flexible caulking to help prevent storm water runoff from infiltrating into the granular base;
- Perimeter subdrains should be installed along walkways/sidewalks to help prevent water originating from precipitation and plant/grass watering efforts from entering the granular base prior to freeze-up; and,
- Subgrade beneath the flat work should be graded towards the subdrains to promote effective drainage.

5.4 Residential Building Foundations

As noted in Section 5.1, the existing site vegetation, surficial topsoil/organics, and other near-surface soils containing significant amounts of organic matter or construction debris are not considered to be suitable for the subgrade support of engineered fill, building foundations, floor slabs, or other settlement sensitive structures. These materials should be completely stripped prior to placing any engineered fill or construction of foundations or interior or exterior slab-on-grade.

Based on the subsurface conditions encountered in the boreholes, strip and spread footings that may be used, provided that the footings are founded on the native silty sand deposit or on engineered fill placed in accordance with the recommendation outlined in Section 5.1.2, and maintained a minimum depth of embedment below finished adjacent ground surface and top of interior slab of 0.5 m for strip and spread footings.

For such strip and spread footings, a factored geotechnical resistance at Ultimate Limit States (ULS) of 200 kPa and a geotechnical reaction at Serviceability Limit States (SLS) of 125 kPa may be assumed for design purposes, provided that the footings have a minimum width of 0.5 m and a maximum width of 1.0 m.

All foundation excavations at the site should be carried out in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. The founding materials are susceptible to disturbance by construction activity especially during wet weather and care should be taken to preserve the integrity of the materials as bearing strata. Prior to pouring concrete for the footings, the foundation excavations should be inspected by the geotechnical engineer to confirm that the footings are founded within an undisturbed and competent bearing stratum that has been cleaned of ponded water and all disturbed, softened, loosened, organic and other deleterious material. It is essential that footings founded on engineered fill be inspected by the geotechnical engineer prior to pouring concrete.

In general, for any houses placed wholly or in part on engineered fill, it is recommended that the foundations be provided with nominal reinforcement, consisting of reinforcing steel at the top and bottom of the foundation walls, or alternatively placed in the footing and top of the foundation walls. However, once the final thicknesses and extent of engineered fill are known, the need for and design of any reinforcement can be determined on a lot-by-lot basis by the builder’s structural engineer, in consultation with the geotechnical engineer.

The perimeter house basement walls should be backfilled with a free draining, non-frost susceptible granular material carefully placed and compacted in lifts and should be designed using a lateral earth pressure coefficient of 0.5 and a unit weight of backfill of 21 kN/m^3. Alternatively, where site excavated material comprised of silty sand is to be reused for all backfill, an approved geocomposite drainage system should be used directly against the wall. The final lift of backfill should be sloped away from the house. Properly filtered perimeter drains at foundation level leading to a permanent outlet, such as a continuously pumped sump should be provided.

If stepped footings are constructed at different founding levels, the difference in elevation between individual footings should not be greater than one half the clear distances between the footings. In addition, the lower footings should be constructed first so that if it is necessary to construct the lower footings at a greater depth than anticipated, the elevations of the upper footings can be adjusted accordingly. Stepped strip footings, if required, should be constructed in accordance with the Ontario Building Code (2012), Section 9.15.3.9.

The maximum total and differential settlements are expected to be less than 25 mm and 20 mm; respectively, for footings designed, constructed and inspected as outlined above.

All exterior footings and footings in unheated areas should be provided with at least 1.2 m of soil cover after final grading in order to minimize the potential for damage due to frost action. In addition, the bearing soil and fresh concrete should be protected from freezing during cold weather construction.

Where spread footings are constructed at different elevations, the difference in elevation between the individual footings should not be greater than one half the clear distance 650 mm between the footings. In addition, the lower footings should be constructed first so that if it is necessary to construct the lower footings at a greater depth than anticipated, the elevation of the upper footings can be adjusted accordingly. Stepped strip footings should be constructed in accordance with the Ontario Building Code (2012), Section 9.15.3.9.

Resistance to lateral forces/sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The unfactored coefficient of friction, tan δ, for the interface between the cast-in-place concrete footing and the properly-prepared subgrade can be assumed to be 0.35.

5.5 Slab-on-Grade Floor

The underground basement level floor slab can be designed as a concrete slab-on-grade. The floor slab may be placed on native undisturbed subgrade approved by Golder. We have assumed that the underground parking level will be heated and that the slab-on-grade will not be subjected to cold temperatures and frost. The exposed native subgrade should be proofrolled at the time of the slab construction in conjunction with an inspection carried out by Golder. Remedial work should be carried out on any softened, disturbed, wet or poorly performing zones as directed by the geotechnical engineer. Any low areas may then be brought up to within at least 150 mm of the underside of the floor slab, as required, using OPSS.MUNI Granular ‘B’, Type I material or other approved material, placed in maximum 200 mm loose lifts and uniformly compacted to at least 98 percent of the material’s Standard Proctor Maximum Dry Density (SPMDD) using suitable compaction equipment.

The final lift of granular fill beneath the floor slab should consist of a minimum thickness of 300 millimetres of OPSS Granular A, uniformly compacted to at least 100 percent of SPMDD. Special care should be taken to ensure adequate compaction around columns and adjacent to foundation walls. This should provide a modulus of subgrade reaction, for a 1 foot square plate placed directly on the subgrade material, kv_1, of approximately 18 MPa/m. Any filling operations should be monitored and tested by Golder.

As noted above the native material is considered to be frost susceptible. As such, where the backfill against the exterior walls is to support <u>settlement/frost heave sensitive structures</u>, such as concrete slabs, pavements or walkways, a free draining OPSS Granular A or B material, uniformly compacted to at least 98 percent of standard Proctor maximum dry density should be used to backfill the foundation walls. In addition, a perforated perimeter drain connected to a permanent storm sewer outlet should also be utilized in the design. As an alternative, the use of a ridged foam insulation board may be used; however, the design length, thickness and depth would have to be reviewed, prior to installation.

5.6 Seismic Site Classification

Seismic hazard is defined in the 2012 Ontario Building Code (OBC) by uniform hazard spectra (UHS) at spectral coordinates of 0.2 second, 0.5 second, 1.0 second and 2.0 seconds and a probability of exceedance of 2% in 50 years. The OBC method uses a site classification system defined by the average soil/bedrock properties (e.g. shear wave velocity, Standard Penetration Test (SPT) resistance, undrained soil shear strength, etc.) in the 30 m below the foundation level. There are six site classes from A to F, decreasing in ground stiffness from A, hard rock, to E, soft soil; with site class F used to denote problematic soils (e.g. sites underlain by thick peat deposits and/or liquefiable soils). The site class is then used to obtain acceleration and velocity-based site coefficients F_a and F_v; respectively, used to modify the UHS to account for the effects of site-specific soil conditions in design.

Based on the results of the preliminary geotechnical investigation and assuming soils below the maximum depth investigated exhibit similar properties / strengths, a **Site Class D** is estimated for planning purposes. The Site Class will need to be verified, and adjusted as necessary, during detail design.

5.7 Permanent Below-Grade Walls

The design of the foundation walls for the basement level and the perimeter wall for the basement ramp should take into account the horizontal soil loads, hydrostatic pressure as well as surcharge loads that may occur during or after construction. The permanent below-grade wall is considered to be a rigid structure and should be designed to resist at-rest lateral earth pressures calculated as follows:

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

where

p = lateral earth pressure acting depth z, kilopascals

$K = K_o$ = at rest earth pressure coefficient, use 0.5 for the foundation wall

$K = K_a$ = active pressure coefficient, use 0.33 for the retaining wall

γ = unit weight of retained soil/backfill, a value of 20 kN/cubic meter may be assumed

h = depth to point of interest in soil, meters

q = equivalent value of surcharge on the ground surface, kilopascals where:

The above expression assumes that the perimeter drainage system prevents the build-up of any hydrostatic pressure behind the wall. Should the hydrostatic pressures build up behind the walls, they must be included in calculating the lateral earth pressures.

All foundation elements in unheated areas must be provided with at least 1.2 meter of earth cover for frost protection purposes. In addition, the bearing soil and fresh concrete should be protected from freezing during cold weather construction.

To avoid problems with frost adhesion and heaving, the foundation and retaining wall should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular B Type I. In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill immediately adjacent to the building and the more frost susceptible native materials which exist beyond the wall backfill. To reduce the severity of this differential heaving, the backfill adjacent to the wall should to be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.2 metres below finished exterior grade at a slope of 3 horizontal to 1 vertical, or flatter, away from the wall. The backfill materials should be placed evenly in lifts not exceeding 200 millimetres loose thickness. The layers should be compacted to at least 95 percent of the materials' SPMDD. Light compaction equipment should be used immediately adjacent to the wall; otherwise compaction stresses on the wall may be greater than that imposed by the backfill material. The upper 0.3 metres of backfill should consist of clayey material to provide a relatively impermeable cap and the exterior grade should also be shaped to slope away from the building.

Based on the groundwater conditions encountered in the boreholes and the anticipated founding elevations of 4.2 m below existing grade, the foundations will be above the local groundwater table and we have assumed that the under slab drainage system will be not be required. The details of the drainage system should be reviewed/assessed after final grades and construction methods have been determined.

6.0 PAVEMENT DESIGN

This section of the report provides preliminary engineering information for the pavement structures throughout the project. Based on our understanding of the project and limited field investigation, preliminary pavement structure designs have been provided.

As traffic information was not available, minimum pavement structure requirements have been provided. It is anticipated that the access road will be used by passenger vehicles with periodic heavier load from service vehicles.

The minimum pavement structures provided in Table 2, below, are recommended for this site:

Table 2: Pavement Design

Material		Minimum Thickness of Pavement Components (mm)	
		Light-Duty Traffic Areas (Laneway)	Heavy-Duty Traffic Areas (18 m wide Road)
Asphaltic Concrete (OPSS 1150)	HL 3 Surface Course	40	40
	HL 8 Binder Course	65	80
Granular Materials (OPSS 1010)	Granular A Base	150	150
	Granular B Type II Subbase	400	450
Total Pavement Thickness		655	720
Prepared and Approved Subgrade			

As part of the subgrade preparation, proposed access roads/parking lots should be stripped of topsoil and other obviously unsuitable fill or organic materials. Fill required to raise the grades to design elevations should conform to the engineered fill requirements outlined previously in the report. Soft or spongy trench backfill areas should be sub-excavated and properly replaced with suitable approved backfill compacted to 98 percent SPMDD. Prior to placing pavement subbase and/or base materials, the exposed soil subgrade should be heavily proof-rolled in conjunction with an inspection by Golder. The granular subbase and base materials should be uniformly compacted to 100 percent of their SPMDD. The asphalt materials should be compacted to a minimum of 92.0 percent of their Marshall Maximum Relative Density according to OPSS 310, as measured in the field using a nuclear density gauge.

Where new pavement abuts existing pavement (e.g. at the development limits), proper longitudinal lap joints should be constructed to key the new asphalt into the existing asphalt surface. The existing asphalt edges should be provided with a proper sawcut edge prior to keying in the new asphalt. It should be ensured that any undermining or broken edges resulting from the construction activities are removed by the sawcut.

It should be noted that in some cases, even though the compaction requirements have been met, the subgrade strength may not be adequate to support heavy construction loading especially during spring time, wet weather or where backfill materials wet of optimum have been placed. In this regard, the design granular subbase thickness may not be sufficient as a construction haul road and additional granular subbase (in the order of 300 mm) may be required. In any event, the subgrade should be proof-rolled and inspected by geotechnical personnel prior to placing the granular subbase and additional granular placed, as required, consistent with the prevailing weather conditions and anticipated use by construction traffic.

7.0 SUBGRADE DRAINAGE

To preserve the integrity of the completed paved areas, a permanent drainage system is recommended. It is anticipated that the drainage would consist of a system of catchbasins draining to storm sewers. In this regard, the subgrade should be carefully proofrolled to a smooth surface and sloped towards the catchbasins to prevent ponding or entrapment of water in the subbase, which would lead to deterioration of the pavement (i.e., alligator cracks, potholes, etc.).

The drainage system should consist of a 100 mm to 150 mm-diameter geotextile wrapped perforated pipe, placed inside a trench and surrounded by clear stone or filter sand. If clear stone is used, the trench should be lined with a suitable geotextile prior to placing the clear stone. At the top of the trench, the geotextile should overlap a minimum of 300 mm. The geotextile should conform to OPSS 1860, Class 1 and be non-woven with a F.O.S. in the range of 75 to 150 micron. The drain invert should be at least 0.5 m below the bottom of the granular subbase.

At internal catchbasin locations, consideration should be given to properly grade and provide continuous subdrains from the internal catchbasins to the perimeter edges of the access road or storm sewer system. If this is not feasible, short (5 m to 6 m long) perforated stubdrains should be provided at the internal catchbasin locations. In addition, consideration should be given to providing continuous subdrains along the sides of the access to promote drainage of the granular materials.

8.0 INSPECTION AND TESTING

During construction, full-time observation should be carried out during engineered fill and site servicing backfill placement, and sufficient foundation inspections, subgrade inspections and in-situ materials testing should be carried out to confirm that the conditions exposed are consistent with those encountered in the boreholes and to monitor conformance to the pertinent project specifications.

9.0 CLOSING

We trust that this preliminary report provides enough preliminary geotechnical engineering information to proceed with the detailed design of the proposed development. Once a more detailed geotechnical report is completed, the geotechnical aspects of the final design drawings and specifications should be reviewed by this office prior to tendering and construction, to confirm that the intent of this report has been met.

If you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact this office.

Signature Page

Yours truly,

Golder Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
M. W. KELLY
100125796
Feb. 25 2020
PROVINCE OF ONTARIO

Matthew Kelly, P.Eng.
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Jeff Tolton, C.E.T.
Associate, Senior Geotechnical Technologist

EM/JET/JJG/JP/MWK/sm/sv/mlk

https://golderassociates.sharepoint.com/sites/35081g/deliverables/reports/preliminary geotech hydro investigation/final/18111428 rep 2019'05'25 preliminary geotechhydro investigation - old church road.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

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The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Ground Water Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

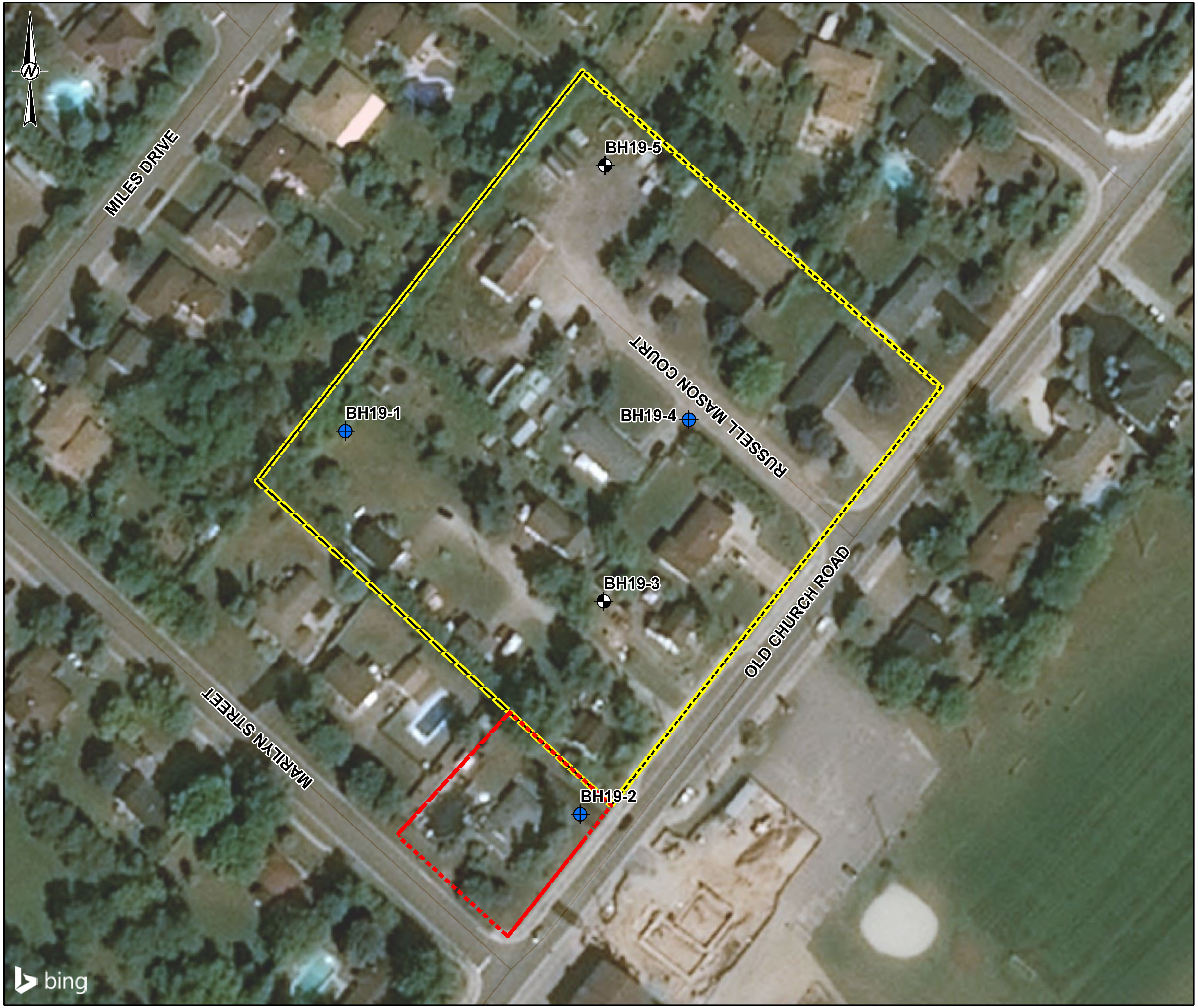
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client’s expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder’s report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder’s report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder’s report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder’s report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder’s responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



LEGEND

⊕ APPROXIMATE BOREHOLE LOCATION

⊕ APPROXIMATE BOREHOLE AND MONITORING WELL LOCATION

🟨 CURRENT PROPOSED SITE DEVELOPMENT BOUNDARY

🟥 FUTURE PROPOSED DEVELOPMENT SITE BOUNDARY


KEY MAP

SCALE 1:400,000

0 20 40

1:1,000 METRES

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

BASE DATA: MNR LIO (2018)

IMAGERY: © 2019 MICROSOFT CORPORATION © 2019 DIGITALGLOBE ©CNES (2019) DISTRIBUTION AIRBUS DS

SOURCES: ESRI, HERE, GARMIN, USGS, INTERMAP, INCREMENT P, NRCAN, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), ESRI KOREA, ESRI (THAILAND), NGCC, © OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY

PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 1983 UTM ZONE 17N

CLIENT

STYLUX CALEDON INC.

PROJECT

PROPOSED DEVELOPMENT AT MARILYN STREET AND
OLD CHURCH ROAD, CALEDON EAST, ONTARIO

TITLE

SITE AND BOREHOLE LOCATION PLAN

CONSULTANT

GOLDER

YYYY-MM-DD	2019-05-06
DESIGNED	LMM
PREPARED	LMM
REVIEWED	EM
APPROVED	JET

PROJECT NO.	CONTROL	REV.	FIGURE
18111428	-	-	1

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IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

25mm

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APPENDIX A

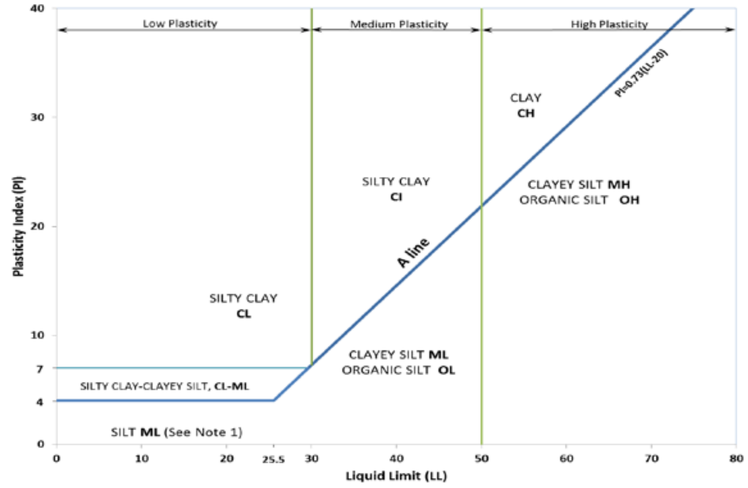
Record of Boreholes BH19-1 to BH19-5

METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators					Organic Content	USCS Group Symbol	Primary Name
				Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)			
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% **(see Note 2)**	CL	SILTY CLAY
			Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat							75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.

Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.

For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.

For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.

A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.
2. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson’s ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l – w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w – w_p) / I_p$
I_C	consistency index = $(w_l – w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} – e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

---

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

PROJECT: 18111428

LOCATION: N: 4858421.837 E: 591165.555
(See Figure 1)

RECORD OF BOREHOLE: BH19-1

BORING DATE: March 26, 2019

SHEET 1 OF 2

DATUM: Local

BORING METHOD: CME 55 Track Mount, 108 mm O.D. Soild Stem Power Auger

DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE						
FILL - (SM) SILTY SAND, some organics, rootlets; dark brown; non-cohesive, dry, compact	0.00	1A	SS			Cuttings
(SM) SILTY SAND, trace gravel; brown to light grey; non-cohesive, dry to moist, very loose to compact	0.46	1B	SS	19		
		2	SS	3		
		3	SS	9		
		4A	SS	13		
		4B	SS	13		
						Bentonite
		5	SS	27		
		6	SS	28		
		7	SS	25		
- Inferred cobbles/boulder from auger grindings below depth of 5.8 m						Sand
		8	SS	25	MH	
						17/04/2019
- Sandy silt at a depth of 7.6 m						Screen
		9	SS	25	MH Non-Plastic	
- Becoming gravelly silty sand, very dense at a depth of 9.1 m						
		10	SS	51		
END OF BOREHOLE.	9.75					

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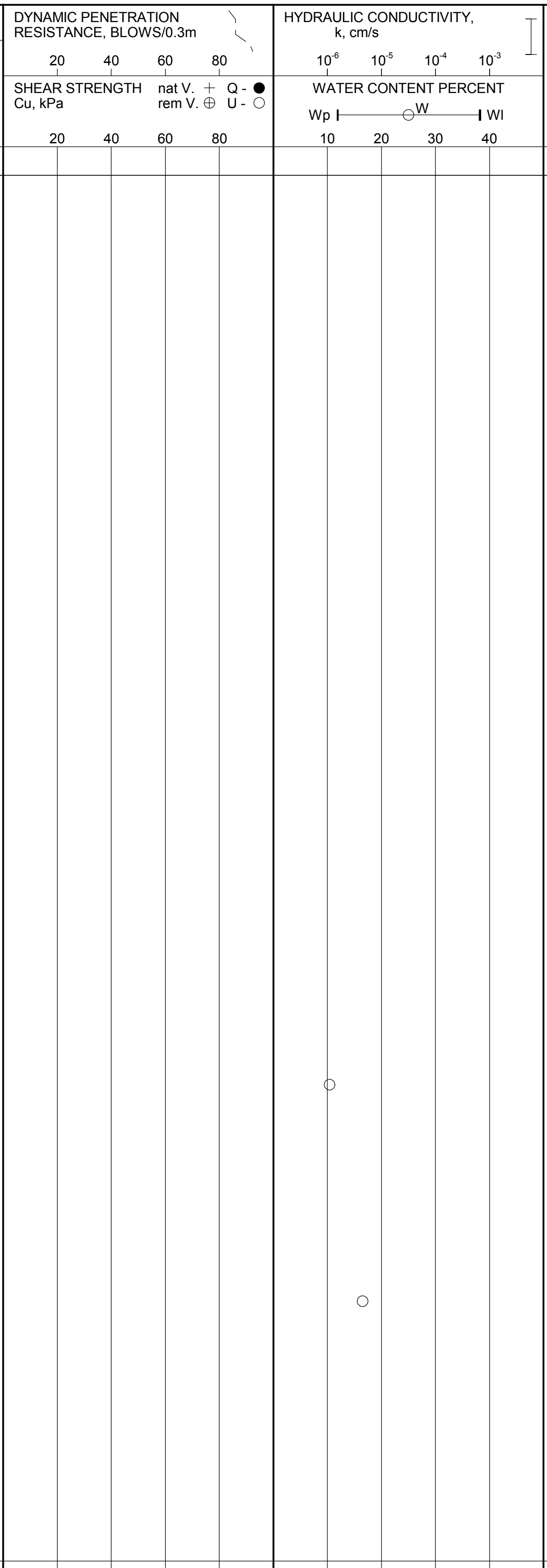
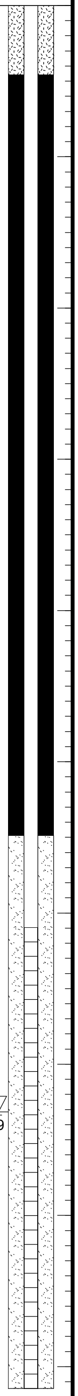




DEPTH SCALE 1 : 50

LOGGED: JD

CHECKED: EM

PROJECT: 18111428
LOCATION: N: 4858421.837 E: 591165.555
(See Figure 1)

RECORD OF BOREHOLE: BH19-1

BORING DATE: March 26, 2019

SHEET 2 OF 2
DATUM: Local

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10		Notes: 1. Water level measured at a depth of 7.24 mbgs upon completion of drilling. 2. Water level measured as follows: Date: Depth April 3, 2019 7.17 mbgs April 17, 2019 7.32 mbgs									
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JD
CHECKED: EM

GTA-BHS 001 G:_CLIENTS\STYLUX\OLD CHURCH ROAD\12_GINT\18111428-OLD CHURCH ROAD BH LOGS.GPJ GAL-MIS.GDT 22/4/19

PROJECT: 18111428

LOCATION: N: 4858335.704 E: 591219.213
(See Figure 1)

RECORD OF BOREHOLE: BH19-2

BORING DATE: March 25, 2019

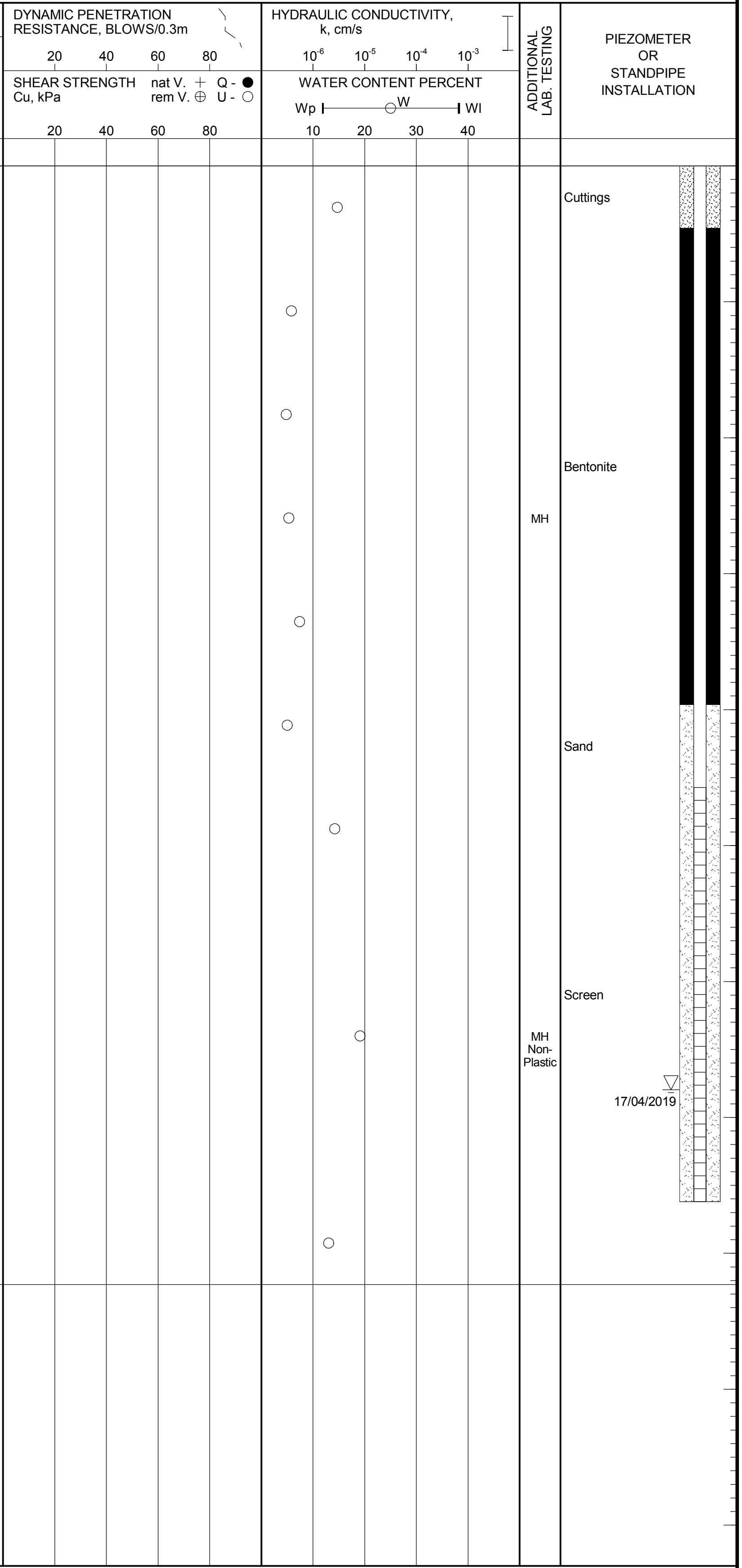
SHEET 1 OF 1

DATUM: Local

BORING METHOD: CME 55 Track Mount
108 mm O.D. Soild Stem Power Auger

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	
(SM) SILTY SAND, some gravel; brown to light brown; non-cohesive, dry to moist, compact - Silt seams at a depth of 4.9 m	0.00
(SM/ML) sandy SILT, trace gravel; light brown to light grey; non-cohesive, moist to wet, compact	5.74
END OF BOREHOLE.	8.23

SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	16	
2	SS	8	
3	SS	18	
4	SS	19	MH
5	SS	17	
6	SS	23	
7	SS	19	
8	SS	21	MH Non- Plastic
9	SS	24	



Notes:

1. Water level measured at a depth of 6.10 mbgs upon completion of drilling.
2. Water level measured as follows:

Date:	Depth
April 3, 2019	6.63 mbgs
April 17, 2019	6.80 mbgs

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JD
CHECKED: EM

PROJECT: 18111428

LOCATION: N: 5413962.809 E: 583241.219
(See Figure 1)

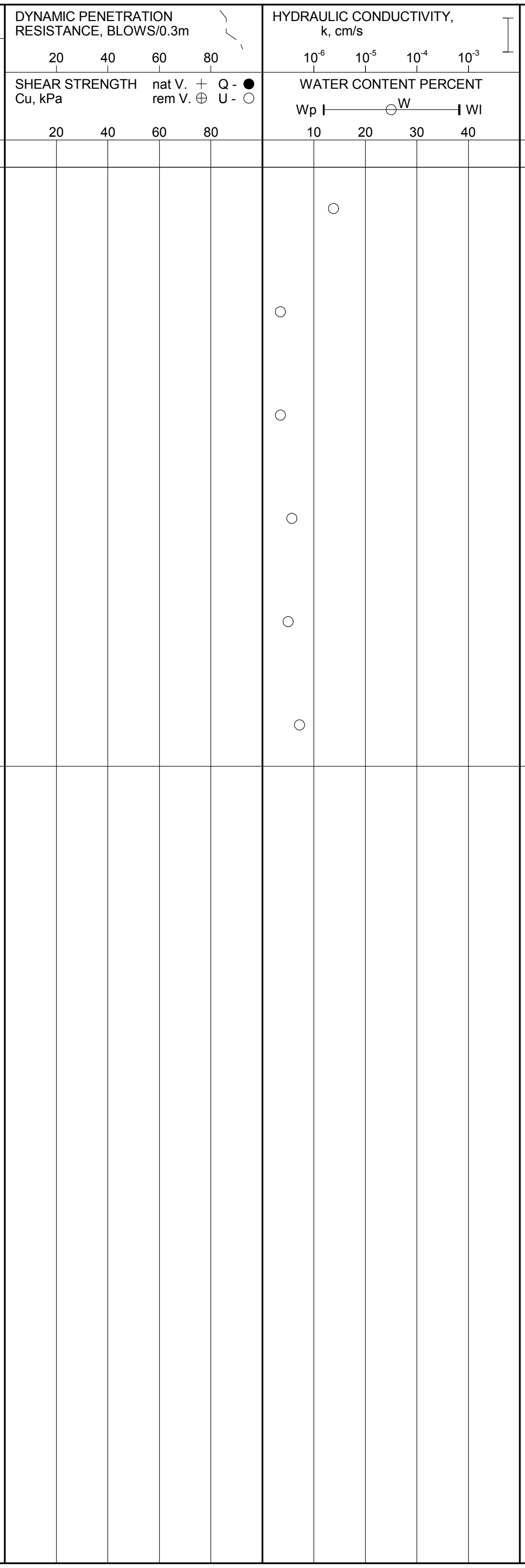
RECORD OF BOREHOLE: BH19-3

BORING DATE: March 25, 2019

SHEET 1 OF 1

DATUM: Local

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
		GROUND SURFACE					
0	CME 55 Track Mount 108 mm O.D. Soild Stem Power Auger	FILL - (SM) gravelly SILTY SAND, trace organics; brown to dark brown; non-cohesive, moist, dense	▨	0.00	1	SS	34
		(SM) SILTY SAND, trace gravel; light brown to light grey; non-cohesive, dry to moist, compact		0.61	2	SS	15
					3	SS	26
					4	SS	20
					5	SS	21
					6	SS	20
		END OF BOREHOLE. Notes: 1. Borehole dry upon completion of drilling.		4.42			



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JD
CHECKED: EM

PROJECT: 18111428
LOCATION: N: 4858430.942 E: 591254.228
(See Figure 1)

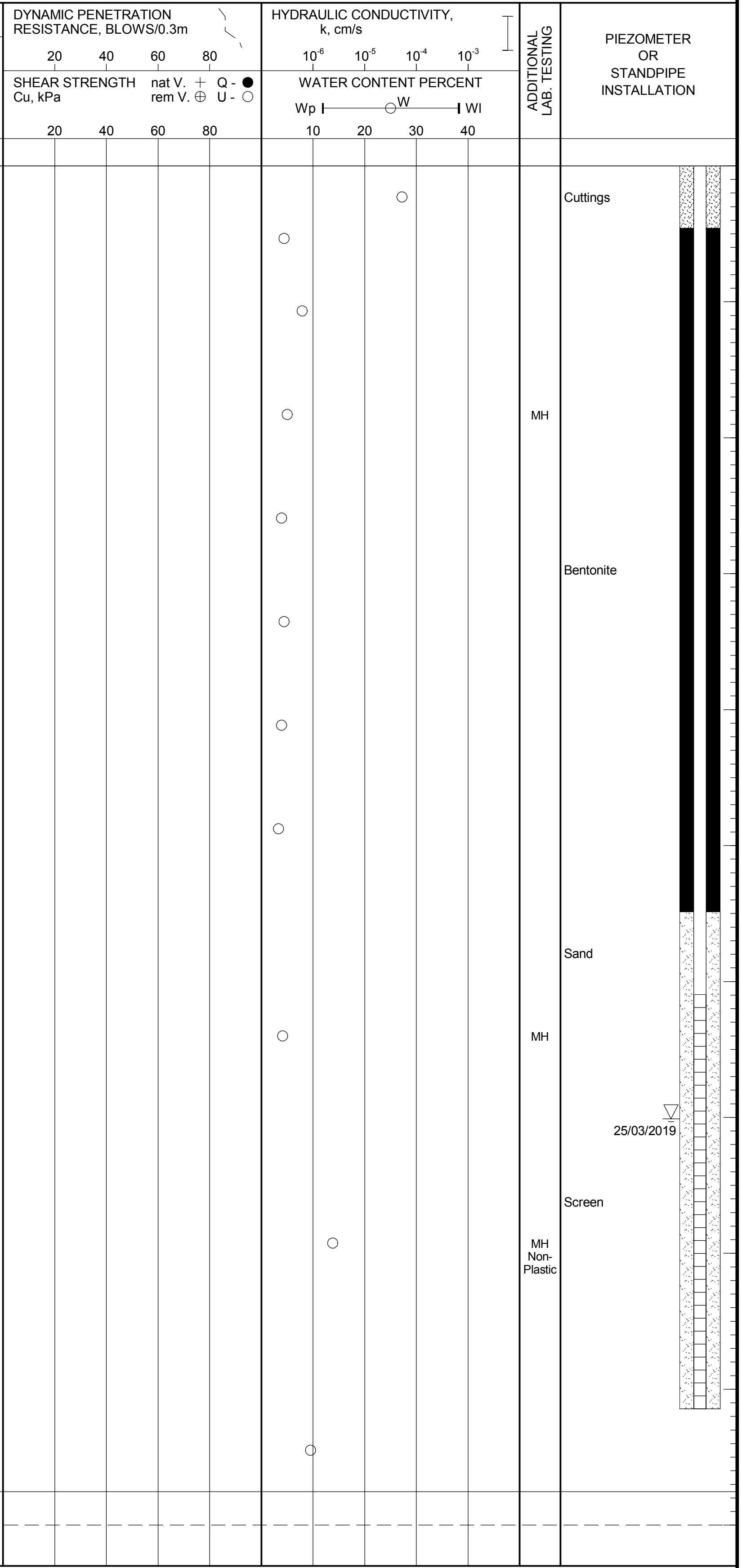
RECORD OF BOREHOLE: BH19-4

BORING DATE: March 25, 2019

SHEET 1 OF 2
DATUM: Local

BORING METHOD: CME 55 Track Mount; 108 mm O.D. Soild Stem Power Auger

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
	GROUND SURFACE					
0	TOPSOIL (150 mm)	0.00				
	FILL - (SM/ML) sandy SILT, some plastic fines, some gravel; dark brown; non-cohesive, dry, compact	0.15	1A	SS		
	(SM) SILTY SAND, trace to some gravel; light brown to grey; non-cohesive, dry to moist, compact	0.46	1B	SS	36	
1			2	SS	13	
2			3	SS	18	MH
			4	SS	20	
3			5	SS	20	
4			6	SS	26	
5	- Becoming very dense to dense at a depth of 4.8 m		7	SS	53	
6			8	SS	41	MH
7	(SM/ML) SAND and SILT, trace gravel; grey to light brown; non-cohesive, wet, compact	7.32				
8			9	SS	18	MH Non-Plastic
9			10	SS	27	
	END OF BOREHOLE.	9.75				
10						



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DEPTH SCALE
1 : 50

GOLDER

LOGGED: JD
CHECKED: EM

PROJECT: 18111428
LOCATION: N: 4858430.942 E: 591254.228
(See Figure 1)

RECORD OF BOREHOLE: BH19-4

BORING DATE: March 25, 2019

SHEET 2 OF 2
DATUM: Local

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10		Notes: 1. Water level measured at a depth of 7.01 mbgs upon completion of drilling. Date: Depth April 2, 2019 7.26 mbgs April 17, 2019 7.27 mbgs									
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JD
CHECKED: EM

PROJECT: 18111428

RECORD OF BOREHOLE: BH19-5

SHEET 1 OF 1

LOCATION: N: 4858493.678 E: 591227.892
(See Figure 1)

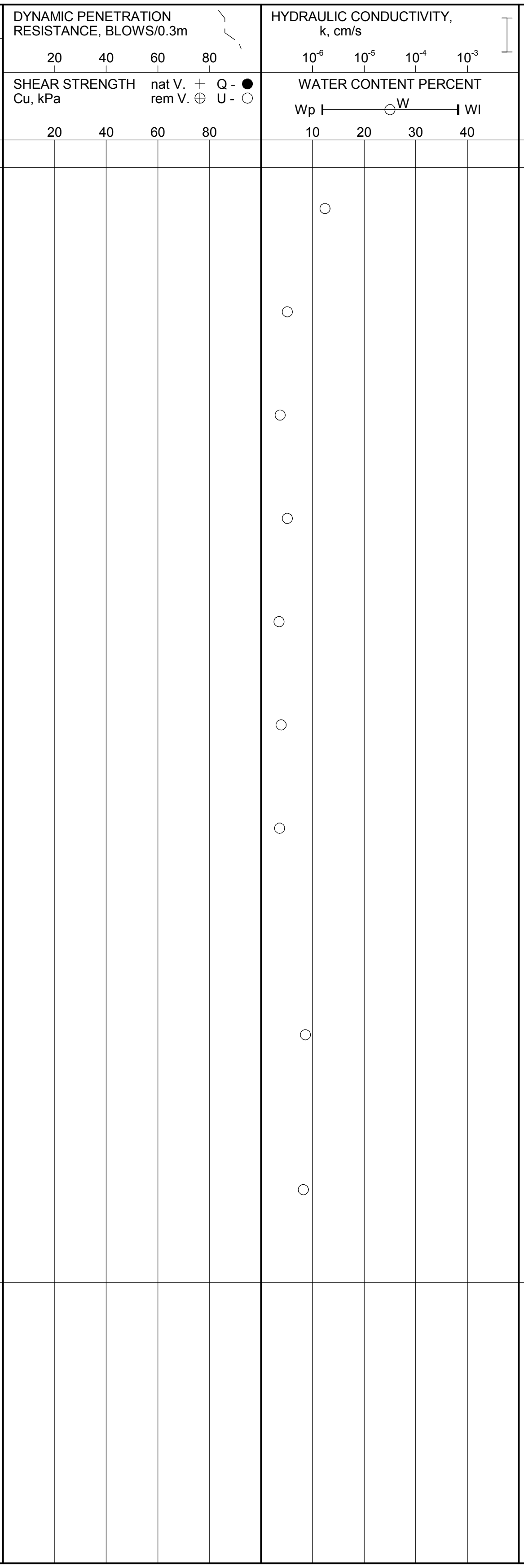
BORING DATE: March 26, 2019

DATUM: Local

BORING METHOD: CME 55 Track Mount
108 mm O.D. Soild Stem Power Auger

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)
	GROUND SURFACE	
0	(SM) SILTY SAND, some to trace gravel; brown to light brown; non-cohesive, dry to moist, compact	0.00
7	(SM-SW) gravelly SAND, some silt; grey-light brown; non-cohesive, wet, loose	7.01
	END OF BOREHOLE. Notes: 1. Borehole dry upon completion of drilling.	8.23

SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	11	
2	SS	5	
3	SS	21	
4	SS	21	
5	SS	32	
6	SS	27	
7	SS	20	
8	SS	13	
9	SS	9	MH



PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JD
CHECKED: EM

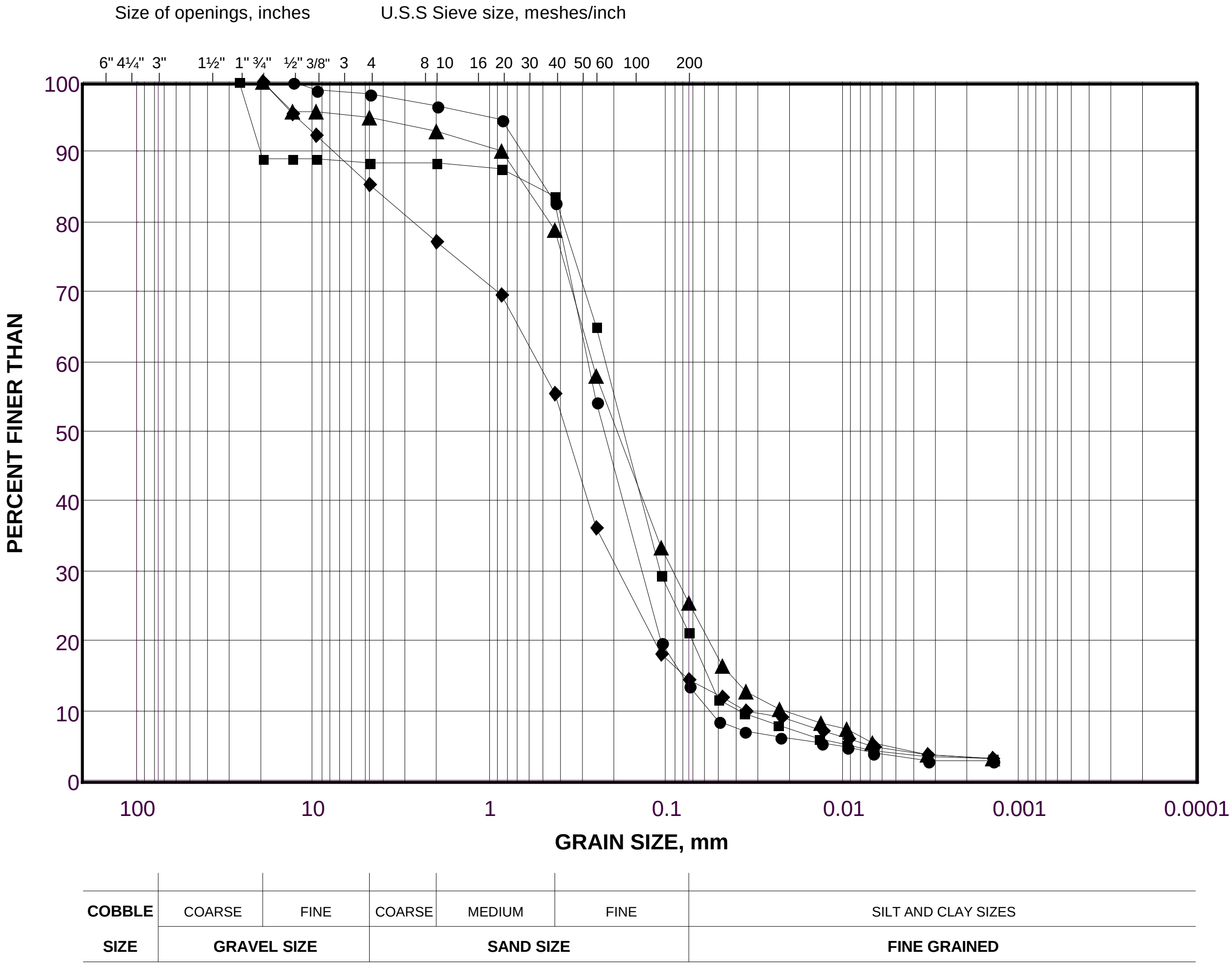
APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

(SM) SILTY SAND

FIGURE B1



LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	BH19-04	3	1.52 - 2.13
■	BH19-02	4	2.29 - 2.90
◆	BH19-04	8	6.10 - 6.71
▲	BH19-01	8	6.10 - 6.71

Project Number: 18111428 (1000)

Checked By:

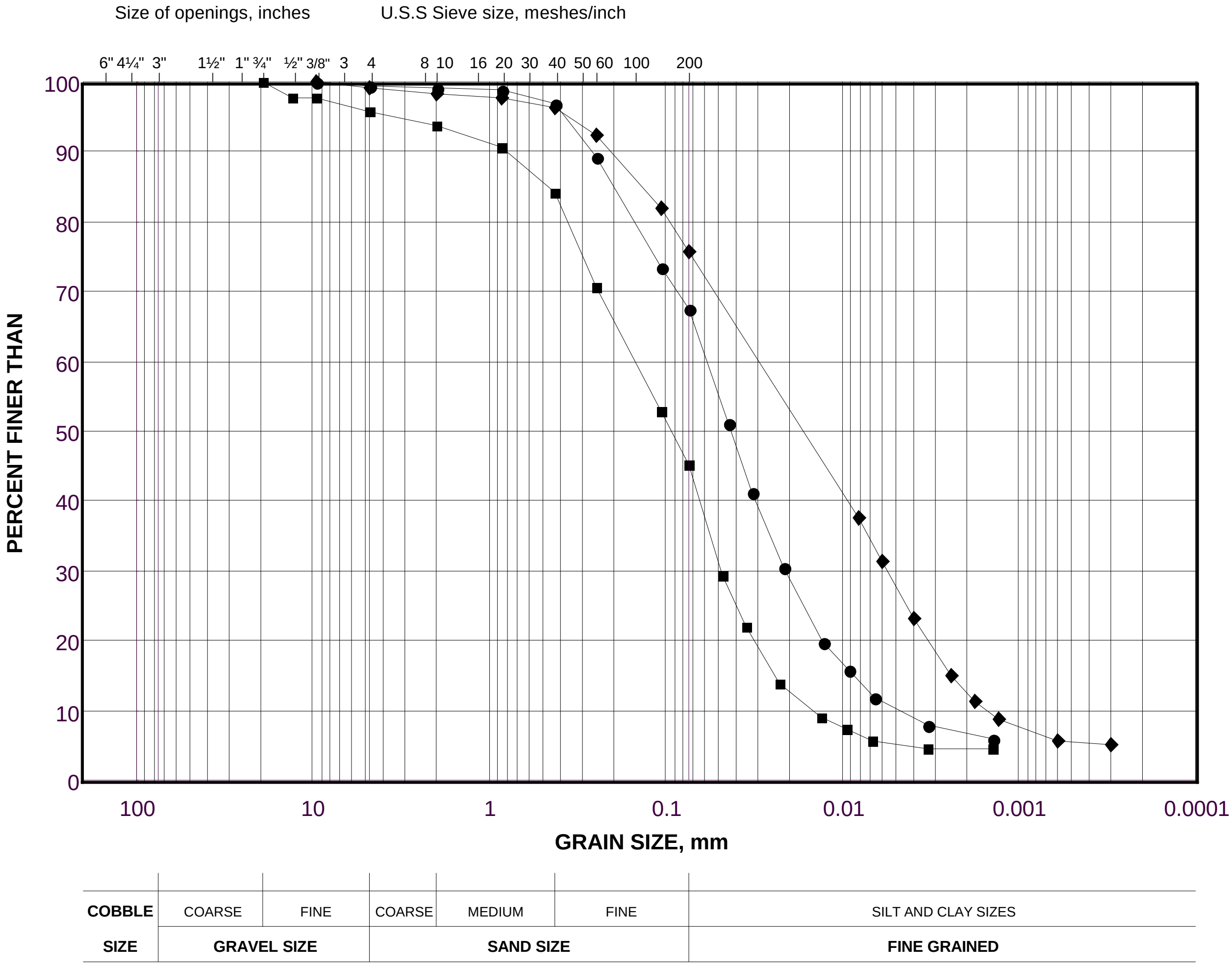
Golder Associates

Date: 24-Apr-19

GRAIN SIZE DISTRIBUTION

(SM/ML) sandy SILT to SAND & SILT

FIGURE B2



LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	BH19-2	8	6.10 - 6.71
■	BH19-4	9	7.62 - 8.23
◆	BH19-1	9	7.62 - 8.23

Project Number: 18111428 (1000)

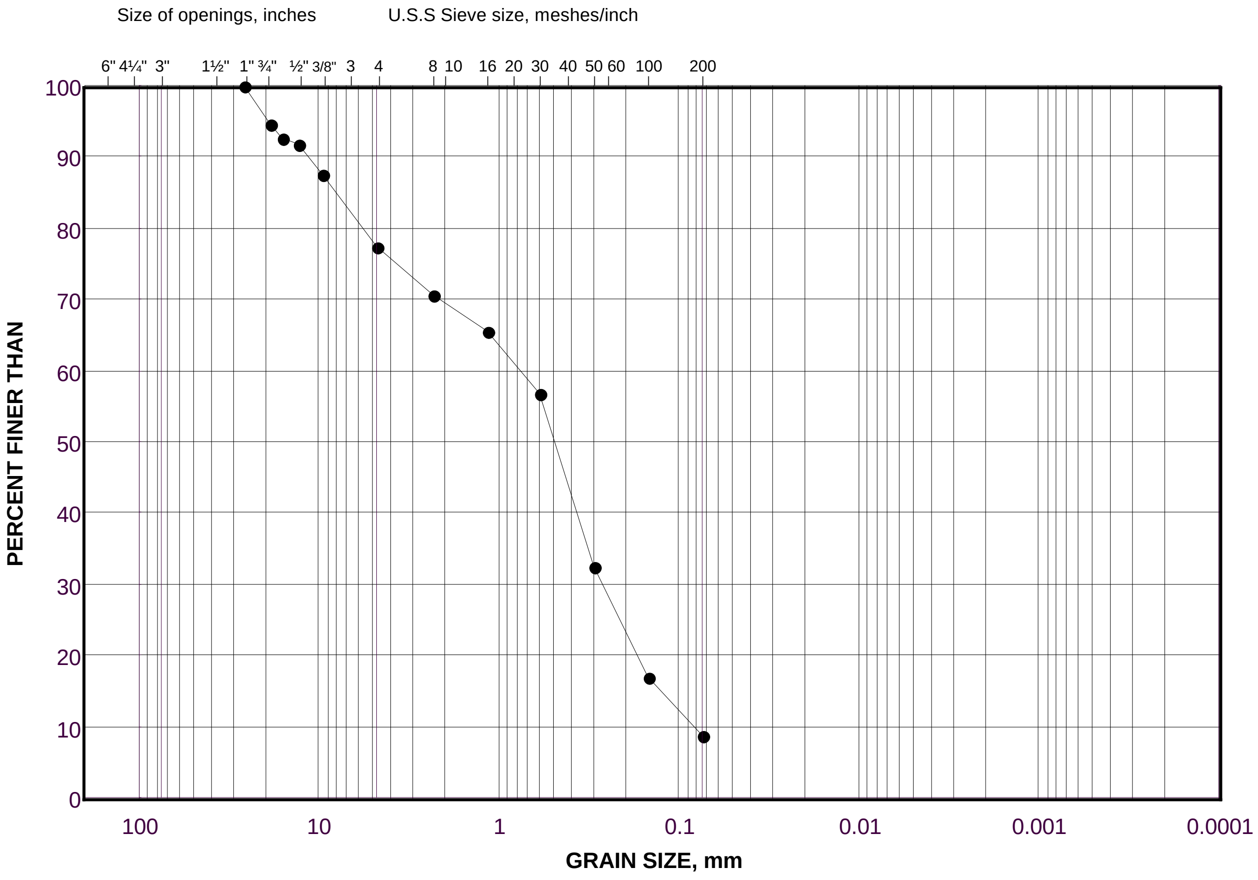
Checked By:

Golder Associates

Date: 24-Apr-19

GRAIN SIZE DISTRIBUTION
(SM-SW) gravelly SAND

FIGURE B3



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	**GRAVEL SIZE**		**SAND SIZE**			**FINE GRAINED**

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	BH19-5	9	7.62 - 8.23

Project Number: 18111428 (1000)

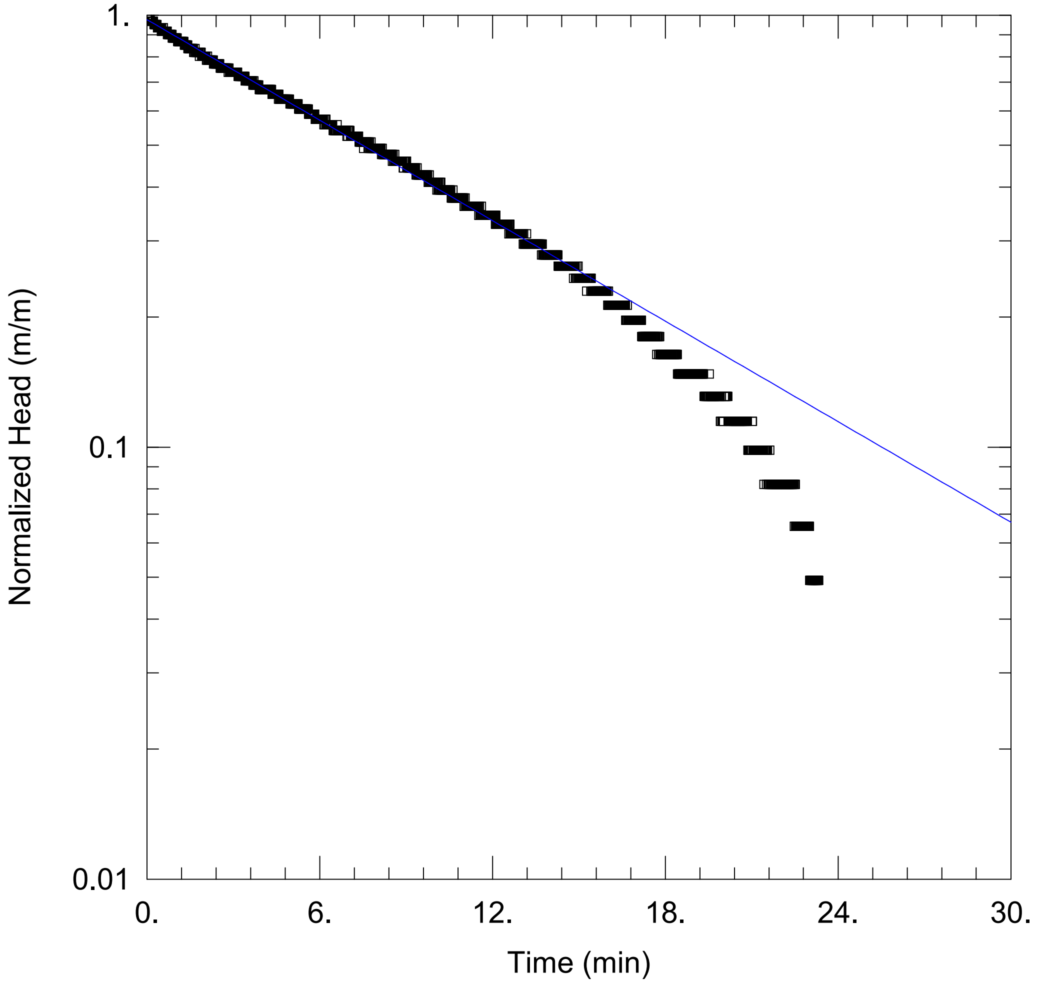
Checked By:

Golder Associates

Date: 24-Apr-19

APPENDIX C

Hydraulic Testing



WELL TEST ANALYSIS

Data Set: C:\Users\JGopaul\Desktop\Old Church Road\BH19-1_JJG.aqt
Date: 05/15/19 Time: 16:14:30

PROJECT INFORMATION

Company: Golder Associates Ltd.
Project: 18111428
Location: BH19-1
Test Well: BH19-1
Test Date: 02/04/19

AQUIFER DATA

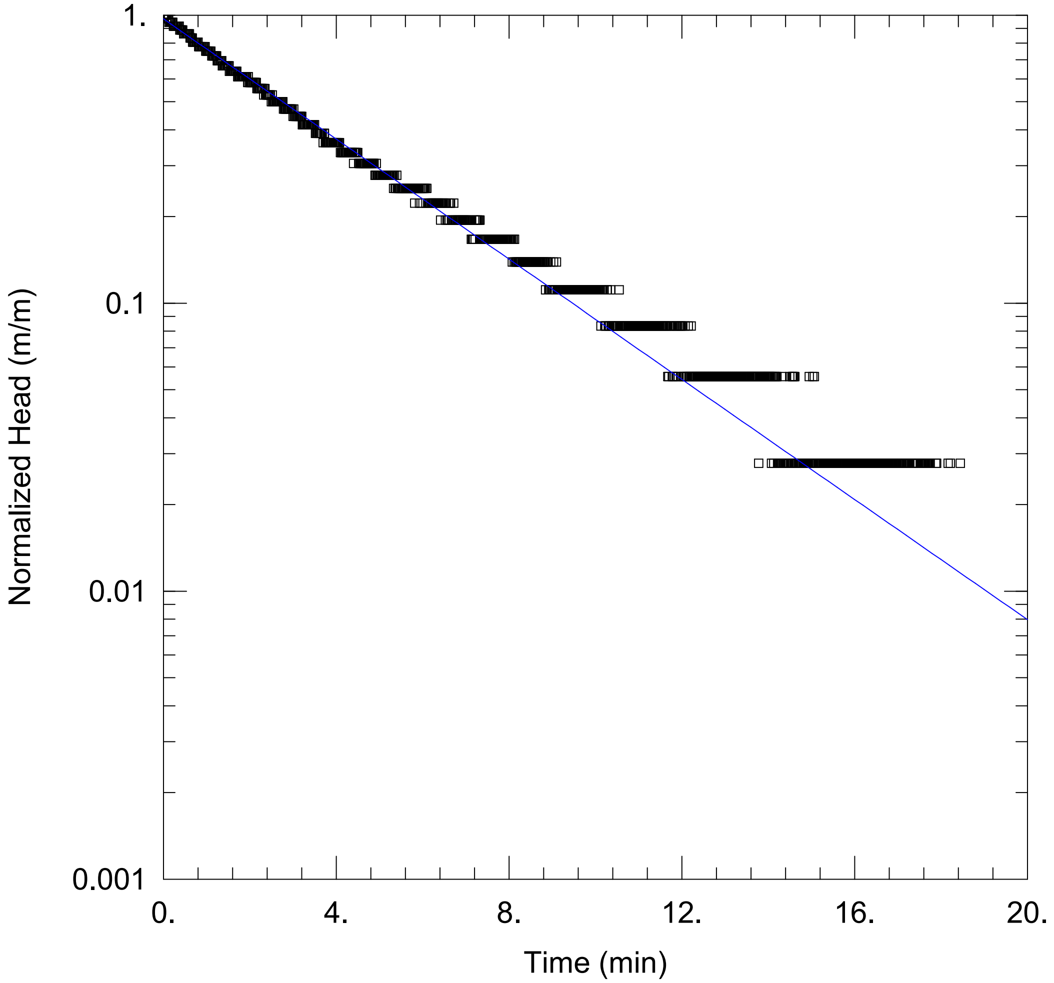
Saturated Thickness: 2.58 m
Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (BH19-1)

Initial Displacement: 0.61 m
Static Water Column Height: 2.58 m
Total Well Penetration Depth: 3.65 m
Screen Length: 3.65 m
Casing Radius: 0.036 m
Well Radius: 0.054 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice
K = 1.165E-6 m/sec
y0 = 0.5917 m



WELL TEST ANALYSIS

Data Set: C:\Users\JGopaul\Desktop\Old Church Road\BH19-2_JJG.aqt
Date: 05/15/19 Time: 16:47:48

PROJECT INFORMATION

Company: Golder Associates Ltd.
Project: 18111428
Location: BH19-2
Test Well: BH19-2
Test Date: 02/04/19

AQUIFER DATA

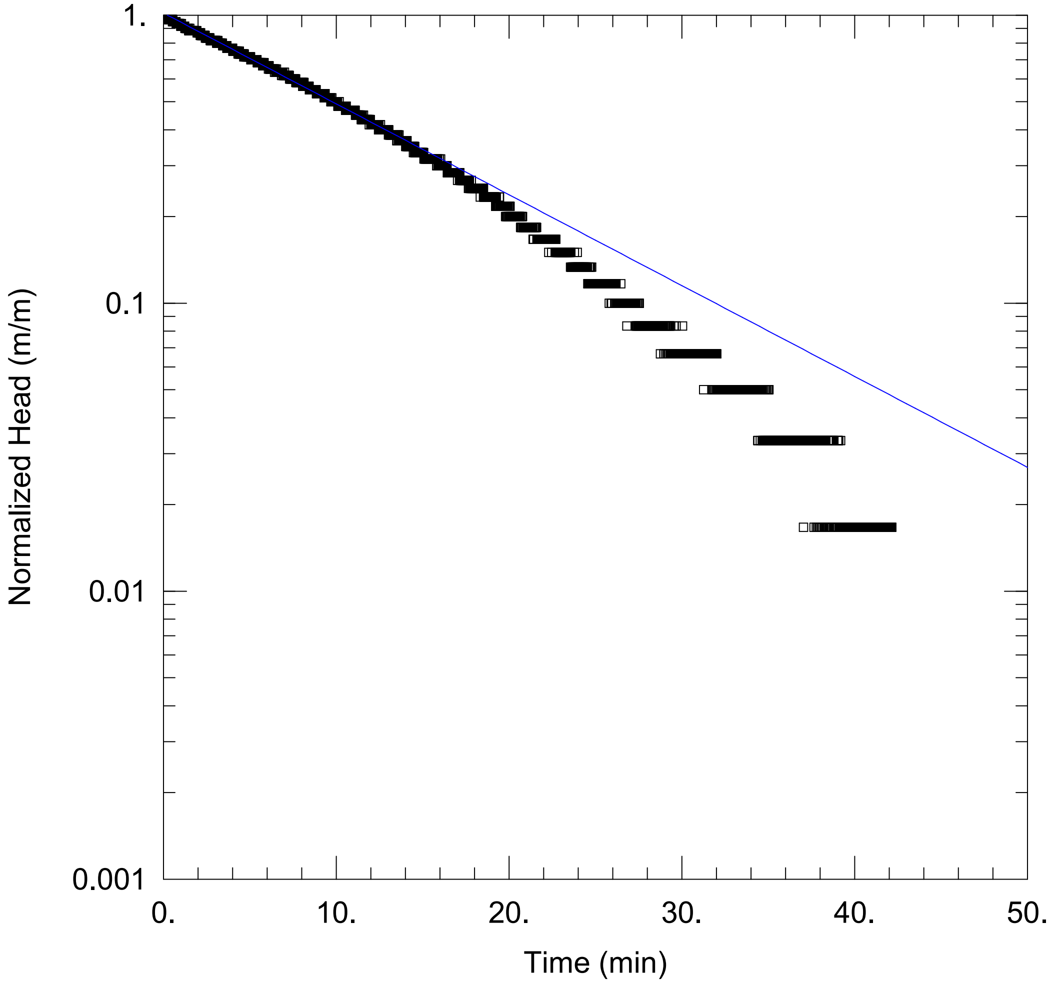
Saturated Thickness: 1.6 m
Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (BH19-2)

Initial Displacement: 0.36 m
Static Water Column Height: 1.6 m
Total Well Penetration Depth: 3.65 m
Screen Length: 3.65 m
Casing Radius: 0.036 m
Well Radius: 0.054 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice
K = 5.004E-6 m/sec
y0 = 0.3454 m



WELL TEST ANALYSIS

Data Set: C:\Users\JGopaul\Desktop\Old Church Road\BH19-4_JJG.aqt
Date: 05/15/19 Time: 17:13:50

PROJECT INFORMATION

Company: Golder Associates Ltd.
Project: 18111428
Location: BH19-4
Test Well: BH19-4
Test Date: 02/04/19

AQUIFER DATA

Saturated Thickness: 2.49 m
Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (BH19-4)

Initial Displacement: 0.6 m
Total Well Penetration Depth: 3.6 m
Casing Radius: 0.036 m
Static Water Column Height: 2.49 m
Screen Length: 3.6 m
Well Radius: 0.054 m
Gravel Pack Porosity: 0.

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice

K = 1.034E-6 m/sec
y0 = 0.6213 m

GOLDER

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