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GEOTECHNICAL REPORT

Proposed 28 Lot Residential Development
893 & 911 Lockhart Road
Innisfil, Ontario

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1 Introduction

Central Earth Engineering Inc. (CEE) was retained by Soheil and Mohamed Fayaz to complete a geotechnical investigation and report for the proposed 28 lot residential development to be located at 893 and 911 Lockhart Road, in Innisfil, Ontario. A site location plan is provided as Figure 1. The portion of the property to be developed with the residential dwellings is rectangular in shape and measures approximately 240 metres long (east to west) by 80 metres wide (north to south). The majority of the property is generally vacant and vegetated with trees and grass, with the exception of a single-family residential dwelling located at 893 Lockhart Road. The site gently slopes from near Elev. 245 metres at the southwestern limit of the site to near Elev. 239 metres at the northeastern limit of the site. An aerial image of the site with existing topography is shown on Figure 2A.

CEE was provided with the following drawing in preparation of this report: *"Concept Plan 9a,"* Project No. FAY-19035, dated August 15, 2019, by Jones Consulting Group Ltd. It is understood that the site has a total area of 2.22 hectares and the proposed development will consist of demolishing any existing structures and constructing a new 28-lot residential subdivision. It is unknown if the proposed dwellings will have basement levels. A 20-metre-wide roadway will run through the site and will connect to Lockhart Road in two locations near the east and west ends of the site. A stormwater management area with an area of 0.15 hectares will be constructed near the northeastern entrance to the site. Proposed site grades were not provided to CEE but there are not expected to be any significant grade changes to accommodate the development. The proposed preliminary site conditions are shown on Figure 2B.

This area of Innisfil is not municipally serviced. Based on our correspondence, it is understood that this development will not proceed until such time that the property will have municipal water and sanitary services, such that no private wells or septic systems will be required. This report has been prepared reflecting this assumption.

The purpose of the geotechnical investigation was to assess the subsurface soil conditions at the site by advancing four (4) exploratory boreholes, each with a monitoring well installation, to provide geotechnical engineering recommendations in support of the proposed subdivision. This report summarizes the borehole findings, provides design recommendations for foundations, slabs on grade, site grading, earth pressures, and pavements, and provides considerations for constructability such as soil excavation, compaction, and temporary groundwater control. CEE has also been retained to complete a hydrogeological study for the site under a separate cover.

2 Procedures and Methodology

Prior to the commencement of drilling activities, the locations of underground utilities including natural gas, electrical, telephone, water, etc. were marked out by public and private utility locating companies. The fieldwork for the drilling program was carried out on October 24th, 2019. A total of four boreholes (Boreholes 1 to 4) were advanced on site by Drilltech Drilling using a track-mounted drill rig. To advance the boreholes, continuous flight solid stem augers and standard soil sampling equipment was utilized. All samples were collected as per ASTM D1586 to assess the strength characteristics of the substrate.

Borehole 4 was advanced adjacent to Borehole 1 to facilitate the installation of a shallower monitoring well, and as such, was augered straight to a depth of 4.6 metres without recovering soil samples.

The boreholes were advanced to depths of 4.6 to 9.6 metres below existing grade. The horizontal locations were laid out in the field by CEE prior to the drilling operations. Ground surface elevations of the boreholes were measured using survey equipment in reference to a geodetic benchmark (iron bars located at the eastern and western ends of the site) with known geodetic elevations. GPS measurements measured with a handheld GPS unit and referenced to the NAD 83 geodetic datum.

The CEE field staff examined and classified characteristics of the soils encountered in the boreholes, including the presence of fill materials, made groundwater observations during and upon completion of the drilling, recorded observations of borehole construction, and processed the recovered samples. Soil sampling was conducted at regular intervals for the full depth of the borehole. The boreholes were backfilled upon completion. All recovered soil samples were logged in the field, carefully packaged and transported to the laboratory for more detailed examination and classification. In the laboratory, the samples were classified as to their visual and textural characteristics and geotechnical laboratory testing was carried out with the results included in Appendix B. Four (4) monitoring wells were installed (one per borehole) to facilitate long-term ground water monitoring.

3 Subsurface Conditions

3.1 General Overview

The detailed soil profiles encountered in the boreholes are indicated on the attached borehole logs in Appendix A and a subsurface profile is included as Figure 3. The borehole locations are shown on Figure 2A (aerial image) and 2B (proposed site) and the geotechnical laboratory results are included in Appendix B. It should be noted that the conditions indicated on the borehole logs and subsurface profile are for specific locations only and can vary between and beyond the borehole locations. It should be noted that the soil boundaries indicated on the borehole logs and subsurface profile are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones and should not be interpreted as exact planes of geological change.

In addition, the descriptions provided in the borehole logs are inferred from a variety of factors, including: visual observations of the soil samples retrieved, laboratory testing, measurements prior to and after drilling, and the drilling process itself (speed of drilling, shaking/grinding of the augers, etc.). The passage of time also may result in changes in conditions interpreted to exist at locations where sampling was conducted.

Borehole 4 was advanced within 2 metres of Borehole 1 to facilitate the installation of a shallower monitoring well to determine perched water conditions in this area of the site. This borehole was augered straight to a depth of 4.6 metres without recovering soil samples. Due to the proximity of Borehole 4 to Borehole 1, the stratigraphic conditions are inferred to be consistent between these two boreholes.

3.2 Stratigraphy

Boreholes 1 to 3 encountered a layer of topsoil at the ground surface that ranged from 150 to 200 mm thick.

Boreholes 1 to 3 encountered a deposit of sand with some silt to being silty underlying the topsoil. The reddish-brown to brown and moist sand extended to depths of 0.8 to 1.0 metres below grade (Elev. 242.6 to 238.4 metres).

The Standard Penetration Test (SPT) results (“N” Values) measured in the sand ranged from 3 to 8 blows per 300 mm of penetration, indicating a very loose to loose relative density.

All three boreholes encountered a deposit of glacial till with a cohesive matrix consisting of clayey and sandy silt with trace gravel underlying the upper sand deposit. The clayey and sandy silt glacial till was encountered at depths of 0.8 to 1.0 metres below grade (Elev. 242.6 to 238.4 metres) and extended to depths of 3.1 to 6.1 metres below grade (Elev. 238.7 to 236.2 metres). The glacial till was brown to greyish-brown and moist. The SPT “N” Values measured in the glacial till ranged from 7 to 30 blows per 300 mm of penetration. Pocket penetrometer testing was also carried out on the recovered (but disturbed) glacial till samples to obtain a general sense of the undrained shear strength of the cohesive soil. The pocket penetrometer results indicate the undrained shear strength ranges from approximately 50 to greater than 225 kPa (typically greater than 225 kPa), indicating a firm to hard consistency.

The glacial till was interbedded by deposit of cohesionless soils with Boreholes 1 and 2. It is expected that these interbedded deposits are discontinuous and are of limited extent across the site. The cohesionless deposits were as follows:

- Borehole 1 encountered a silty sand with trace gravel from a depth of 3.1 metres (Elev. 240.2 metres) to a depth of 4.6 metres below grade (Elev. 238.7 metres). The silty sand was brown and wet, and the SPT “N” Value was 23 blows per 300 mm, indicating a compact relative density.
- Borehole 2 encountered a silt with some sand and trace clay from a depth of 1.0 metre (Elev. 242.3 metres) to 2.5 metres below grade (Elev. 240.8 metres). The silt was brown and moist, and the SPT “N” Values were 12 and 10 blows per 300 mm of penetration, indicating a compact relative density.

The glacial till deposit encountered in Boreholes 1 to 3 was underlain by a cohesive deposit of clayey silt with some sand. The grey and moist clayey silt was encountered at depths of 3.1 to 6.1 metres below grade (Elev. 238.7 to 236.2 metres). The clayey silt extended beyond the vertical depth of investigation in Boreholes 1 and 2 at 9.6 metres below grade (Elev. 233.7 to 233.6 metres) and extended to a depth of 9.1 metres (Elev. 230.1 metres) in Borehole 3. The SPT “N” Values measured in the clayey silt ranged from 21 to 90 blows per 300 mm of penetration. Pocket penetrometer testing was also carried out on the recovered (but disturbed) soil samples to obtain a general sense of the undrained shear strength of the cohesive soil. The pocket penetrometer results indicate the undrained shear strength ranges from approximately 175 to greater than 225 kPa (typically greater than 225 kPa), indicating a very stiff to hard consistency.

A deposit of fine to medium sand with trace silt was encountered in Borehole 3 underlying the clayey silt deposit. The fine to medium sand was encountered at a depth of 9.1 metres (Elev. 230.1 metres) and extended beyond the vertical depth of investigation at 9.6 metres below grade (Elev. 229.6 metres). The sand was greyish-brown and wet, and the SPT “N” value measured in the sand was 28 blows per 300 mm of penetration, indicating a compact relative density.

3.3 Ground Water

Unstabilized ground water level measurements and cave measurements were taken upon completion of drilling of each borehole. These measurements provide a rough estimate of the possible excavation and temporary ground water control constructability considerations that may arise. The unstabilized water level after completion of drilling ranged from being 2.3 metres below grade to being dry. Monitoring wells were installed in Boreholes 1 to 4 to facilitate measurements of stabilized and long-term ground water levels. The results shown on the borehole logs in Appendices A and are summarized in the table below.

Monitoring Well	**Well Screen Location**		**Strata Screened**	**Depth / Elevation (m) of Groundwater Table**		
	Depth (m)	**Elevation (m)**		**Oct. 30th, 2019**	**Nov. 15th, 2019**	**Dec. 2nd, 2019**
1	6.9 to 8.4	236.3 to 234.8	Clayey Silt	3.4 / 239.8*	2.2 / 241.0	3.3 / 239.9*
2	7.7 to 9.2	235.6 to 234.1		7.5 / 235.8*	3.3 / 240.0	7.5 / 235.8*
3	7.4 to 8.9	231.8 to 230.3	Clayey Silt (and likely underlying Fine to Medium Sand)	6.7 / 232.5	6.4 / 232.8	6.4 / 232.8
4	3.1 to 4.6	240.3 to 238.8	Silty Sand	2.1 / 241.3	1.8 / 241.6	1.8 / 241.5

***Note 1:** Due to the very low hydraulic conductivity of the clayey silt deposit, it is expected that the water level readings taken on October 30 (approx. 1 week after drilling) and December 2 (approx. 2 weeks after conducting rising head test) may not be reflective of the stabilized groundwater elevation.

Based on the results of the water levels and moisture contents of the recovered soil samples, it is expected that the prevailing groundwater table is located on the order of about 2 to 3 metres below grade across the site. It is expected that the groundwater flow gradient is from the higher elevation in the southwest of the site to the lower elevation in the northeast of the site.

The water level within Borehole 3 was noted to be significantly deeper, on the order of 6 metres below grade. This monitoring well was likely partially screened in the underlying wet sand deposit which may have a different piezometric head than the surficial clayey silt and upper silty sand deposits.

The cohesionless soils at the site are relatively permeable and will allow for the free flow of water when wet, whereas the clayey silt glacial till and clayey silt deposits have a significantly lower permeability, precluding free flow of water.

CEE is carrying out monthly groundwater level measurements for one year along with a hydrogeological study for the site, provided under a separate cover. Additional details pertaining to groundwater at the site are provided in the hydrogeological study and the seasonally high ground water levels will be determined through the monthly monitoring and summarized in a separate letter report.

4 Engineering Design Parameters & Analysis

It is understood that the site has a total area of 2.22 hectares and the proposed development will consist of demolishing any existing structures and constructing a new 28-lot residential subdivision. It is unknown if the proposed dwellings will have basement levels. A 20-metre-wide roadway will run through the site and will connect to Lockhart Road in two locations near the east and west ends of the site. A stormwater management area with an area of 0.15 hectares will be constructed near the northeastern entrance to the site. Proposed site grades were not provided to CEE but there are not expected to be any significant grade changes to accommodate the development. The proposed preliminary site conditions are shown on Figure 2B.

This area of Innisfil is not municipally serviced. Based on our correspondence, it is understood that this development will not proceed until such time that the property will have municipal water and sanitary services, such that no private wells or septic systems will be required. This report has been prepared reflecting this assumption.

Reference should be made to the Ontario Building Code which stipulate the geotechnical design and construction requirements for the type of residential structures being proposed at this site.

4.1 Foundation Design

The topsoil layer and upper loose to very loose deposit of sand are not suitable for the support of new building foundations. The undisturbed and stiff to hard clayey silt glacial till encountered in Boreholes 1 to 3 at depths of 0.8 to 1.0 metres below existing grade (Elev. 242.6 to 238.4 metres) and the compact silt encountered in Borehole 2 at a depth of 1.0 metre below grade (Elev. 242.3 metres) are suitable for the support of new foundations.

Foundations at this site may be constructed as conventional spread and strip footing foundations that extend down to bear on the undisturbed native soils as described above. Foundations set on undisturbed native soil at or below depths of 0.8 to 1.0 metres below existing grade may be designed using a geotechnical reaction at SLS of 75 kPa, for an estimated settlement of 25 mm or less. The maximum factored geotechnical resistance at ULS is 115 kPa.

It is important to note that these bearing capacities are applicable for foundations set onto suitable undisturbed native soils, which were encountered at depths of 0.8 to 1.0 metres below existing grades. If the grade is raised prior to foundation construction, the foundations must be extended through any new grade raise in addition to the required depth to reach the competent bearing level.

All footings exposed to ambient air temperature throughout the year must be provided with a minimum of 1.2 metres of earth cover or equivalent insulation for frost protection. The minimum strip and spread footing widths to be used shall be dictated as per the Ontario Building Code, regardless of loading considerations. Footings stepped from one level to another must be at a slope not exceeding 7 vertical to 10 horizontal. This concept should also be applied to excavations for new foundations in relation to existing footings or underground services unless rigid shoring is provided.

The foundation design parameters provided above are predicated on the assumption that the foundation subgrade surface is undisturbed, and that all deleterious, softened, disturbed, organic, and caved material is removed. The foundation excavation must be done in such a way that groundwater is controlled to prevent any disturbance to the foundation base. Temporary groundwater control is discussed in Section 5.2.

The foundation subgrade must be reviewed prior to concrete placement to ensure the above foundation design parameters are applicable, and to provide remedial recommendations if necessary. If the foundation excavation will be open for a prolonged period of time, the foundation subgrade should be protected with a skim coat of lean mix concrete (after inspection by the geotechnical engineer), to ensure that no deterioration will occur due to weather effects.

4.2 Earth Pressures

Underground levels, basements, retaining walls, cantilevered shoring walls and shoring walls with a single level of earth anchors all must be designed to resist unbalanced lateral earth pressures imparted from the weight of adjacent soils. Lateral earth pressures are calculated using the following equation:

$$\boldsymbol{P = K[\gamma h + q]}$$

where,
P = the horizontal pressure at depth, **h** (m)
K = the earth pressure coefficient (dimensionless)
h = depth below surface in metres
γ = the bulk unit weight of soil, (kN/m^3)
q = surcharge loading (kPa)

The above equation assumes that a drainage system is present which prevents the build up of any hydrostatic pressure behind the structure subjected to the unbalanced lateral earth pressures. If this is not the case, the equation must be revised to also incorporate the submerged unit weight of the soil multiplied by the earth pressure coefficient, in addition to the water pressure itself.

The values for use in the design of structures subjected to unbalanced lateral earth pressures at this site are as follows:

Soil Type	**γ - Bulk Unit Weight (kN/m^3)**	**φ - Friction Angle (degrees)**	**Earth Pressure Coefficient (dimensionless)**		
			K_a - Active	**K_o – At-Rest**	**K_p - Passive**
Granular 'B' (OPSS 1010)	21.0	32	0.31	0.47	3.25
Earth Fill	19.0	30	0.33	0.50	3.00
Loose Sand					
Clayey Silt Glacial Till	21.0	32	0.31	0.47	3.25
Compact Silt to Silty Sand	19.5	33	0.29	0.46	3.39

The calculation of the earth pressure coefficients is based on Rankine theory, which provides a conservative estimate as no friction between the soil and the structure is accounted for. The earth pressure coefficients provided above are only applicable for flat ground surfaces beyond the structure and must be increased for sloping ground surfaces.

The earth pressure coefficients referenced within the above table are a function of the friction angle of the adjacent soil, and both the degree and direction of movement of the structure subjected to unbalanced lateral earth pressures. For structures that are restrained at the top (such as basement walls), the at-rest earth pressure coefficient will apply. For structures that allow for 0.1 to 1% of movement away from the soil, the full active earth pressure coefficient will apply. For structures that allow for 1 to 10% of movement into the soil, the full passive earth pressure coefficient will apply. The percentage movement is based on the height of the structure.

Other types of structures such as shoring walls with multiple rows of tiebacks and soil nail walls are subject to different loading conditions and must be analyzed separately.

4.3 Slab on Grade Design

The topsoil and any soft, wet, organic, or disturbed native soils are not suitable for the support of a slab on grade. Undisturbed native soils are suitable for the support of a lightly supported unreinforced concrete slab on-grade. The subgrade for the slab on grade must be assessed by the geotechnical engineer, prior to the placement of an aggregate base. If the subgrade will consist of the upper loose to very loose sand, the sand must be surface compacted to 98% Standard Proctor Maximum Dry Density (SPMDD), and a subgrade consisting of the glacial till should be cut neat and inspected. If any soft or weak subgrade areas are identified, or if there are areas containing excessive amounts of deleterious/organic material, they must be locally sub-excavated and backfilled with approved clean earth fill or imported granular material and compacted to a minimum of 98% SPMDD.

The modulus of subgrade reaction appropriate for design of a slab-on-grade on the upper loose sand (provided it is surface compacted to 98% SPMDD) is 20,000 kPa/m. The modulus of subgrade reaction appropriate for design of a slab on grade on the undisturbed glacial till is 30,000 kPa/m.

All building floor slabs must be provided with a capillary moisture barrier and drainage layer. This is made by placing the concrete slab on a minimum 200 mm layer of 19 mm clear stone (OPSS.MUNI 1004) compacted by vibration to a dense state. The upper 50 mm of clear stone can be replaced with 19 mm crusher run limestone for a working surface. The clear stone and a cohesionless subgrade must be separated by a geotextile such as Terrafix 270R (or approved equivalent) to prevent the migration of fines into the clear stone layer which could result in loss of support for the slab. If the subgrade consists entirely of cohesive glacial till, the geotextile is not required.

4.4 Basement Drainage

For new structures that will be slab on grade with no basement levels, perimeter and under-slab drainage at the foundation level is not required, provided that the underside of concrete slab is at least 200 mm above the prevailing grade of the site and the surrounding surfaces slope away from the building at a gradient of at least 2% to promote surface water run-off and to reduce groundwater infiltration adjacent to foundations. To minimize infiltration of surface water, the upper 150 mm of backfill could comprise relatively impervious compacted soil material (such as the clayey silt glacial till or clayey silt deposits from the site).

Where basements are constructed, all basement foundation walls must be provided with damp-proofing provisions in conformance to the Ontario Building Code. Backfill along the foundation wall must consist of Granular ‘B’ Type 1 (OPSS 1010) for a minimum lateral distance of 600 mm out from the foundation wall. Alternatively, if a filtered cellular drainage media is provided adjacent to the foundation wall, the backfill may consist of common earth fill.

For buildings with basements, a perimeter drainage system must be installed that will remove any water that infiltrates into the building backfill, to ensure that any water does not infiltrate into the basement. The perimeter drains must consist of minimum 100 mm diameter perforated pipes wrapped in filter socks, sufficiently covered on all sides by 19 mm clear stone. Perimeter drains should be directed to the sump underneath the basement floor in solid pipes so as not to surcharge the underfloor drainage layer with water. All sump pumps should be on emergency power for redundancy in case of a power outage. A typical basement drainage detail is included in Appendix C.

If the basement level is set near or within the prevailing groundwater level, it is possible that perimeter drainage issues may occur in the future (e.g. sump pump failure, blockage of drainage pipes, etc.), which would lead to potential foundation cracking and basement flooding. These issues are typically more prevalent in soils that have a high permeability (i.e. sands) where higher groundwater flow and discharge occurs. Basements can be set below the groundwater table provided these risks are fully acknowledged and all obligations set by the governing bodies in the jurisdiction are met which stipulate minimum clearance distances between basement slab elevation and seasonal high groundwater table. It is noted that the Town of Innisfil typically requires a separation of 0.5 metres from the seasonally high groundwater table and any proposed basement slab, regardless of soil type.

CEE is carrying out monthly groundwater level measurements for one year along with a hydrogeological study for the site, provided under a separate cover. Additional details pertaining to groundwater at the site is provided in the hydrogeological study and the seasonally high ground water levels will be determined through the monthly monitoring and summarized in a separate letter report.

4.5 Site Servicing

4.5.1 Bedding

The type of material and depth of granular bedding below the pipe will, to some extent, depend on the method of construction used by the contractor. Pipe bedding for flexible pipes should follow the requirements in Ontario Provincial Standard Drawing 802.010 or 802.013 or applicable municipal standards. Pipe bedding for rigid pipes should follow the requirements in Ontario Provincial Standard Drawings 802.030 to 802.033 or applicable municipal standards.

A subgrade consisting of the native soils at the site will provide adequate support for pipes with the bedding requirements as laid out in the above referenced OPS drawings. Where disturbance of the trench base has occurred from groundwater seepage, construction traffic, etc., the disturbed soils should be sub-excavated and replaced with suitably compacted granular fill. If weak zones are encountered, additional bedding materials and differing construction practices may be required and should be determined during construction.

Regardless of whether flexible or rigid pipes are implemented, granular bedding and cover material should consist of a well graded, free draining material, such as Granular “A” (OPSS.MUNI 1010). All granular bedding must be compacted to a minimum of 98% SPMDD. Clear stone or high-performance bedding is permitted at this site provided it is fully wrapped in a non-woven filter fabric to prevent the migration of fines and loss of pipe support.

4.5.2 Backfill

Excavated native soils may be used as backfill in trenches provided the moisture content is within 2% of optimum. The backfill should be compacted to a minimum of 95% SPMDD. In confined areas the layer thickness will have to be reduced to utilize smaller compaction equipment efficiently or by using granular material instead of locally sourced fill. Any backfill that is frozen, contains a high percentage of organic material (topsoil, peat, etc.) or moisture, or has otherwise unsuitable deleterious inclusion should not be used as backfill. The maximum cobble or boulder size should not exceed half of the loose lift thickness (i.e. all particles with a diameter greater than 100 mm should be removed).

Where trenches are within the traveled portions of a roadway, backfill within the frost penetration depth of 1.2 metres should consist of native, non-organic, excavated material consistent with the soils surrounding the trench. If this technique is not undertaken, then frequently problems arise with yearly differential frost heave movements between the trench backfill and the adjacent native soil. This would occur, for example, if imported granular fill was used to backfill the trenches. Alternatively, if different soil is used as the backfill due to issues with achieving compaction, a frost taper of 3H:1V can be implemented to help mitigate the potential for differential settlement and frost heave.

4.6 Pavement Design

4.6.1 Subgrade Preparation

Final grading plans were not provided to CEE at the time of writing this report, but a new roadway will be made through the development (Street A on Figure 2B). It is expected that subgrade soils below a pavement structure at the site will consist of cohesionless sand or cohesive glacial till. These soils will be an adequate subgrade for the support of a pavement structure, provided a subgrade consisting of the sand is surface compacted to 98% SPMDD, a subgrade consisting of glacial till is cut neat, and both potential subgrades are inspected and approved by a geotechnical engineer at the time of construction and do not contain excessive amounts of organics or deleterious materials. The topsoil is not a suitable subgrade and must be removed. Any fill placed as the pavement subgrade must be compacted to 98% SPMDD.

The modulus of subgrade reaction appropriate for design of a pavement structure on the upper loose sand (provided it is surface compacted to 98% SPMDD) is 20,000 kPa/m. The modulus of subgrade reaction appropriate for design of a pavement structure on the undisturbed glacial till is 30,000 kPa/m.

The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures must be maintained to ensure that uniform subgrade moisture and density conditions are achieved as much as possible when fill is placed, and the natural subgrade is not disturbed or weakened after it is exposed.

4.6.2 Drainage

Control of surface water is an important factor in achieving a good pavement life. The need for adequate subgrade drainage cannot be over-emphasized. The subgrade must be free of depressions and sloped (at a minimum grade of 3 percent) to provide effective drainage toward subgrade drains. Grading adjacent to pavement areas should be designed to ensure that water is not allowed to pond adjacent to the outside edges of the pavement.

Continuous pavement subdrains should be provided along both sides of the roadways and parking area and drained into respective catchbasins to facilitate drainage of the subgrade and the granular materials. The subdrain invert should be maintained at least 0.3 metres below subgrade level. To minimize the problems of differential movement between the pavement and catchbasins/manhole due to frost action, the backfill around the structures should consist of free-draining granulars. Typical pavement drainage details are included in Appendix C.

4.6.3 Pavement Structure

The industry pavement design methods are based on a design life of 15 to 20 years for typical weather conditions depending on actual traffic volumes. The following pavement thickness design is provided on the above noted considerations and subgrade basis and meets the recommended pavement minimums for a local road per the Town of Innisfil document, *"Engineering Design Standards and Specifications Manual,"* Revision 5 dated May 2019. The minimum pavement design for a local road in the manual is considered adequate for this site. It is expected that new roadway to be constructed at the site will be assumed by the Town.

Pavement Layer	**Compaction Requirements**	**Minimum Component Thickness**
Surface Course Asphaltic Concrete: HL3 (OPSS.MUNI 1150) with **PG 58-34** Asphalt Cement (OPSS.MUNI 1101)	OPSS.MUNI 310	40 mm
Binder Course Asphaltic Concrete: HL8 (OPSS.MUNI 1150) with **PG 58-34** Asphalt Cement (OPSS.MUNI 1101)		60 mm
Base Course: Granular A (OPSS.MUNI 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm
Subbase Course: Granular B Type I (OPSS.MUNI 1010)		400 mm

The granular materials should be placed in lifts 200 mm thick or less and must be compacted to a minimum of 100% SPMDD for both granular base and granular subbase. Asphalt materials should be rolled and compacted as per OPSS.MUNI 310. The granular and asphalt pavement materials and their placement should conform to OPSS.MUNI 310, 501, 1010 and 1150.

If the pavement construction occurs in wet, winter or inclement weather, it may be necessary to provide additional subgrade support for heavy construction traffic by increasing the thickness of the granular subbase, base or both. Further, traffic areas for construction equipment may experience unstable subgrade conditions. These areas may be stabilized utilizing additional thickness of granular materials.

It should be noted that in addition to adherence of the above pavement design recommendations, a close control on the pavement construction process will also be required in order to obtain the desired pavement life. Therefore, it is recommended that regular inspection and testing should be conducted during the pavement construction to confirm material quality, thickness, and to ensure adequate compaction.

5 Constructability Considerations

5.1 Excavations

Excavations must be carried out in accordance with the Occupational Health and Safety Act, Ontario Regulation 213/91 (as amended), Construction Projects, Part III - Excavations, Section 222 through 242.

Where workers must enter a trench or excavation the soil must be suitably sloped and/or braced in accordance with the OHSA. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety. The regulation stipulates safe slopes of excavation as follows based on the soils encountered at this site:

- *Type 2 Soils – The cohesive clayey silt glacial till and clayey silt deposits:* Trench sidewalls can be vertical from the base of the excavation to a height of 1.2 metres from the base of the excavation, after which the sidewalls must be constructed no steeper than 1 horizontal to 1 vertical.
- *Type 3 Soils – The cohesionless sand and silt deposits above the groundwater table or when dewatered:* Trench sidewalls to be constructed no steeper than 1 horizontal to 1 vertical from the base of the excavation.
- *Type 4 Soils – The cohesionless sand and silt deposits within the groundwater table:* Trench sidewalls to be constructed no steeper than 3 horizontal to 1 vertical from the base of the excavation.

If more than one soil type is encountered in an excavation, the most conservative soil type must be followed for sloping the sidewalls of the excavation. It is expected that the majority of excavations will be completed considering a Type 3 soil.

Minimum support system requirements for steeper excavations are stipulated in Sections 235 through 238 and 241 of the OHSA and include provisions for timbering, shoring and moveable trench boxes. In order to reduce the potential for instability of the trench excavations, materials excavated from the service trenches and/or other fill materials or heavy equipment should not be placed near the crest of the trench excavations.

Cobbles and boulders embedded in the glacial till may be encountered in construction excavations. It is important to note that soils encountered in the construction excavations may vary significantly across the site. Our preliminary soil classifications are based solely on the materials encountered in widely spaced boreholes advanced on site. The contractor should verify that similar conditions exist throughout the proposed area of excavation. If different subsurface conditions are encountered at the time of construction, we recommend that CEE be contacted immediately to evaluate the conditions encountered.

5.2 Temporary Groundwater Control

Grading plans were not provided to CEE. It is unknown if excavations or utilities will extend below the prevailing groundwater table. During times of high precipitation, some water may also collect at the base of excavations.

It is expected that the groundwater table is located on the order of about 2 metres below grade in the western part of the site (Boreholes 1 and 4), and on the order of about 3 metres below grade in the eastern part of the site (Borehole 2). The cohesionless soils at the site are relatively permeable and will allow for the free flow of water when wet, whereas the clayey silt glacial till and clayey silt deposits have a significantly lower permeability, precluding free flow of water.

Due to a relatively low permeability, excavations made below the groundwater table within the cohesive glacial till or clayey silt will likely not encounter free-flowing water. Temporary groundwater control within the cohesive soils can likely be accomplished using local sump and pump systems.

Although the site soils are predominantly cohesive, the glacial till in Borehole 1 was interbedded with a discontinuous layer of wet silty sand located in the groundwater table. The silt interbedded within the glacial till in Borehole 2 was above the groundwater table. There may be other discontinuous zones of wet cohesionless soils at the site that were not captured by the boreholes. Free flowing water will enter excavations made into the cohesionless layers below the groundwater table.

It is expected that the groundwater inflows from these interbedded cohesionless deposits will not yield significant amounts of groundwater, and can likely be controlled in this scenario by either:

- Sumps placed at the base of the excavation; or
- Sumps created with a corrugated steel pipe filled with gravel which allows the water to enter the sumps and continuously pumping the sumps until all the water stored within the cohesionless soils are drained.

The exact scenario where these groundwater control techniques will work are estimates only and are directly correlated to how coarse/fine the native soils are in an excavation, and both the lateral and vertical extent of the cohesionless deposits encountered. If the groundwater table is not controlled during construction, the base of the excavations will probably be unstable, leading to difficulties in excavating and placement of pipes or footings. Additional details on temporary groundwater control are provided in the hydrogeological study by CEE provided under a separate cover.

5.3 Compaction Specifications

Standard Proctor Maximum Dry Density (SPMDD) is the level to which a soil or aggregate is compacted. To achieve the specified SPMDD as indicated in this report, all soils or aggregates must be placed in lift thicknesses no greater than 200 mm. If this is not the case, only the upper portion of the lift will be adequately compacted, and the lower portion of the lift has a high probability of not meeting compaction specifications. In addition, industry standard equipment used to determine the degree of compaction consists of nuclear densometers. These devices have an inherent limitation in that they cannot test beyond 300 mm in depth, and so the degree of compaction beyond this depth cannot be quantitatively determined.

Along with lift thickness, ensuring that the soil or aggregate is within 2% of its optimum moisture content ensures that the specified compaction can be reached. If the soil or aggregate is too dry/wet, it is either very difficult or impossible to reach the specified compaction. This is especially true for when higher compaction specifications such as 98% and 100% SPMDD are required.

The soil below the groundwater table at this site is generally wet of optimum and should not be re-used as fill at the site. Based on our review of the soil types encountered in the boreholes with associated moisture contents, the soils at this site (above the groundwater table) are considered as follows:

- Half of in-situ soil above the groundwater table: At or near optimum moisture content.

- Quarter of in-situ soil above the groundwater table: Above optimum moisture content.
- Quarter of in-situ soil above the groundwater table: Below optimum moisture content.

The zones with higher moisture content will require moisture conditioning prior to re-use in areas that require compaction. Moisture could be reduced by tilling the soil, spreading the soil out, or blending it with drier material. Soil that is dry of optimum could be blended with wetter soil or have water added prior to re-use. It must be also noted that the above percentages can change significantly based on the time of year in which construction occurs, as the prevailing weather can have a significant effect on the moisture content of stockpiled and in-situ soil.

In addition to the above compaction specifications, in any areas where compacted fill will be placed over the exposed native soil subgrade, any loose, soft, wet or unstable areas should be sub-excavated, and backfilled with clean earth fill of Granular ‘B’ (OPSS.MUNI 1010) compacted to a minimum of 98% SPMDD. This recommendation applies to site servicing and pavement subgrades.

5.4 Quality Verification Services

On-site quality verification services are an integral part of the geotechnical design function, and for foundations and retaining walls, are required under the Ontario Building Code. Quality verification services are used to confirm that construction is being conducted in general conformance with the requirements as outlined in the drawings, reports and specifications prepared for the proposed development.

Central Earth Engineering can provide all the on-site quality verification services outlined below:

- The subgrade for shallow residential dwelling and townhouse foundations may be field reviewed by the geotechnical engineer as required by the municipal regulating authority.
- Installation of retaining structures over 1.0 metres high and related backfilling operations must be field reviewed on a continuous basis by the geotechnical engineer as required in the OBC.
- Part-time monitoring of the subgrade support capabilities (i.e. proof-roll), material quality, lift thickness, moisture content, degree of compaction, etc. is recommended for the following areas to ensure the recommendations within this report are followed and they perform adequately in the long-term:
  - Slab-on-grades;
  - Pavement structure (granulars and asphalt); and
  - Bedding/backfilling of site servicing.
- Testing of the concrete (compressive strength, slump, air content, etc.) and testing of the asphalt (asphalt content and gradation) are recommended to ensure that the quality of the materials being brought to site meet the requirements of the project.

5.5 Site Work

The soils found at this site may become weakened when subjected to traffic, particularly when wet. If there is site work carried out during periods of wet weather, then it can be expected that the subgrade will be disturbed unless an adequate granular working surface is provided to protect the integrity of the subgrade soils from construction traffic. Subgrade preparation works cannot be adequately accomplished during wet weather and the project must be scheduled accordingly. The disturbance caused by the traffic can result in the removal of disturbed soil and use of granular fill material for site restoration or underfloor fill that is not intrinsic to the project requirements.

The most severe loading conditions on the subgrade may occur during construction. Consequently, special provisions such as end dumping and forward spreading of earth and aggregate fills, restricted construction lanes, and half-loads during paving and other work may be required, especially if construction is carried out during unfavourable weather.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the founding subgrade and concrete must be provided. The soil at this site is highly susceptible to frost damage. Consideration must be given to frost effects, such as heave or softening, on exposed soil surfaces in the context of this particular project development.

6 Limitations and Conclusion

6.1 Limitations

The recommendations and comments provided are necessarily on-going as new information of underground conditions becomes available. More specific information with respect to the conditions between samples, or the lateral and vertical extent of materials may become apparent during excavation operations. The interpretation of the borehole information must, therefore, be validated during excavation operations. Consequently, conditions not observed during this investigation may become apparent. Should this occur, CEE should be contacted to assess the situation and additional testing and reporting may be required.

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

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

This report was prepared by CEE for the account of Soheil and Mohamed Fayaz. 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. Central Earth Engineering Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this project.

6.2 Conclusion

It is recognized that municipal/regional governing bodies, in their capacity as the planning and building authority under Provincial statues, will make use of and rely upon this report, cognizant of the limitations thereof, both as are expressed and implied.

We trust this report is complete within our terms of reference, and the information presented is sufficient for your present purposes. If you have any questions, or when we may be of further assistance, please do not hesitate to contact our office.

Yours Truly,

Central Earth Engineering Inc.

Alexander Winkelmann, P.Eng.
President, Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
A.WINKELMANN
100150146
Dec. 3, 2019
PROVINCE OF ONTARIO

Russell Wiginton, P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
R. M. WIGINTON
100193069
Dec. 3, 2019
PROVINCE OF ONTARIO

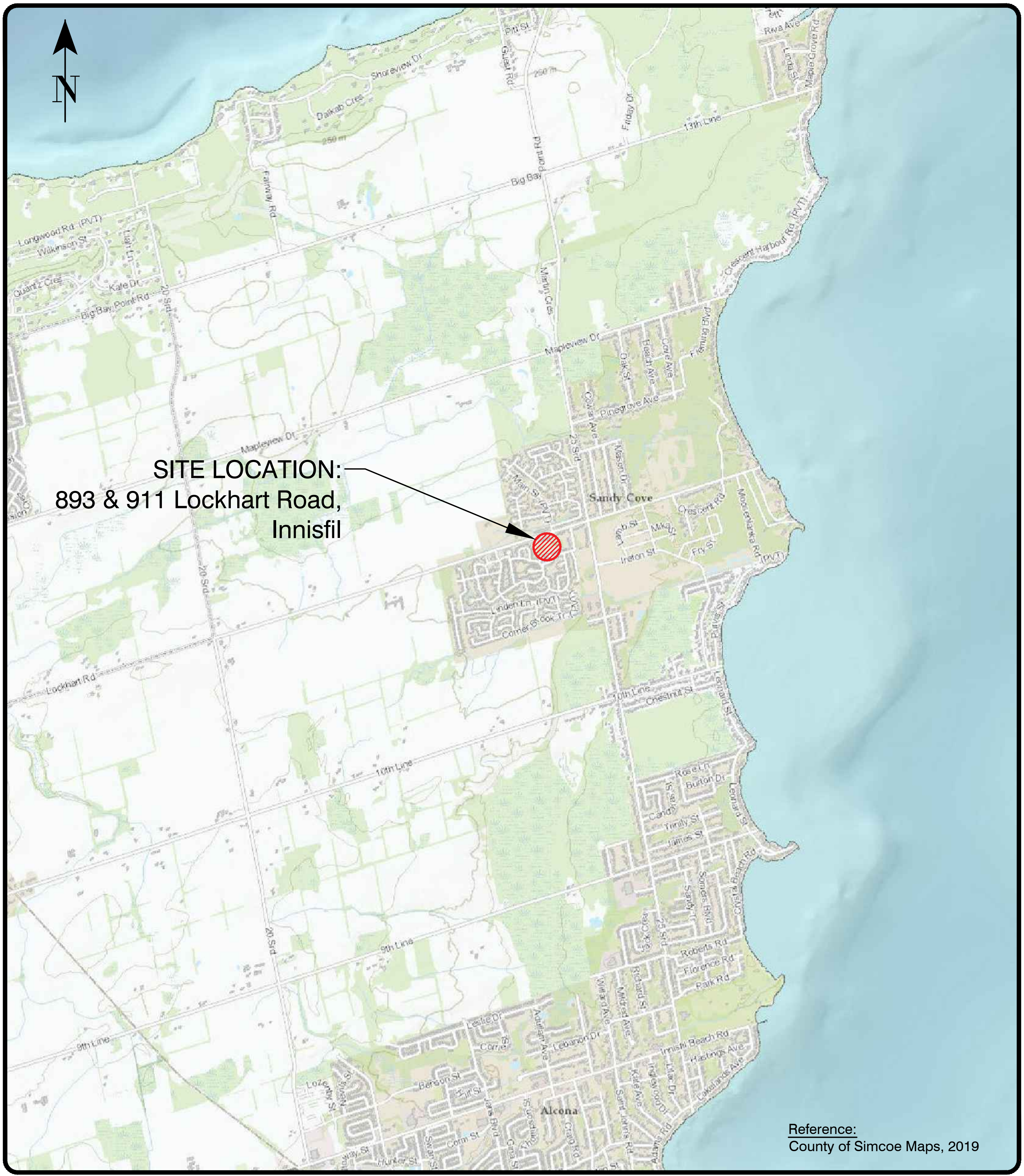
Figures –

SITE LOCATION PLAN

BOREHOLE LOCATION PLAN (AERIAL IMAGE)

BOREHOLE LOCATION PLAN (PROPOSED SITE)

SUBSURFACE PROFILE



SITE LOCATION:
893 & 911 Lockhart Road,
Innisfil

Reference:
County of Simcoe Maps, 2019

Central Earth
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Geotechnical Engineering and Construction
Materials Testing & Inspection

647 Welham Rd, Unit 14, Barrie, ON, L4N 0B7
P: (705) 719-7994 | E: info@centralearth.ca

Project:	893 & 911 Lockhart Road, Innisfil				
Title:	SITE LOCATION PLAN				
Approved by:	A.W.	Date:	November 2019	Project No.:	19-1171A
Drawn by:	R.W.	Scale:	1:40000	Figure No.:	1

Legend:

● Approximate Borehole Location

Reference:
"Concept Plan 9a," Project No. FAY-19035, dated August 15, 2019, by Jones Consulting Group Ltd.

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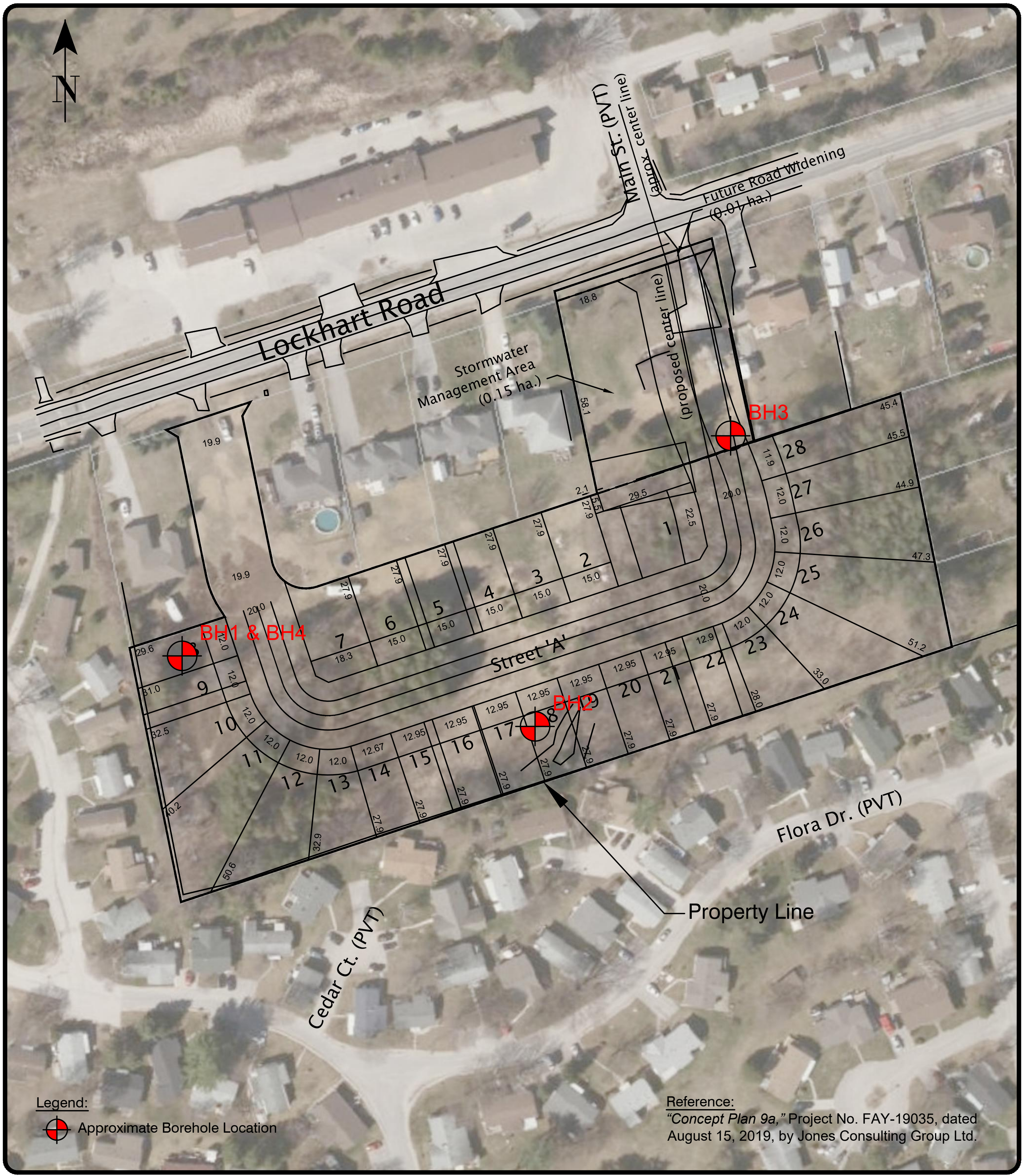
Project:	893 & 911 Lockhart Road, Innisfil				
Title:	BOREHOLE LOCATION PLAN (AERIAL IMAGE)				
Approved by:	A.W.	Date:	November 2019	Project No.:	19-1171A
Drawn by:	R.W.	Scale:	1:1500	Figure No.:	2A



Legend:

● Approximate Borehole Location

Reference:

"Concept Plan 9a," Project No. FAY-19035, dated August 15, 2019, by Jones Consulting Group Ltd.

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647 Welham Rd, Unit 14, Barrie, ON, L4N 0B7
P: (705) 719-7994 | E: info@centralearth.ca

Project: 893 & 911 Lockhart Road, Innisfil

Title: BOREHOLE LOCATION PLAN (PROPOSED SITE)

Approved by: A.W.

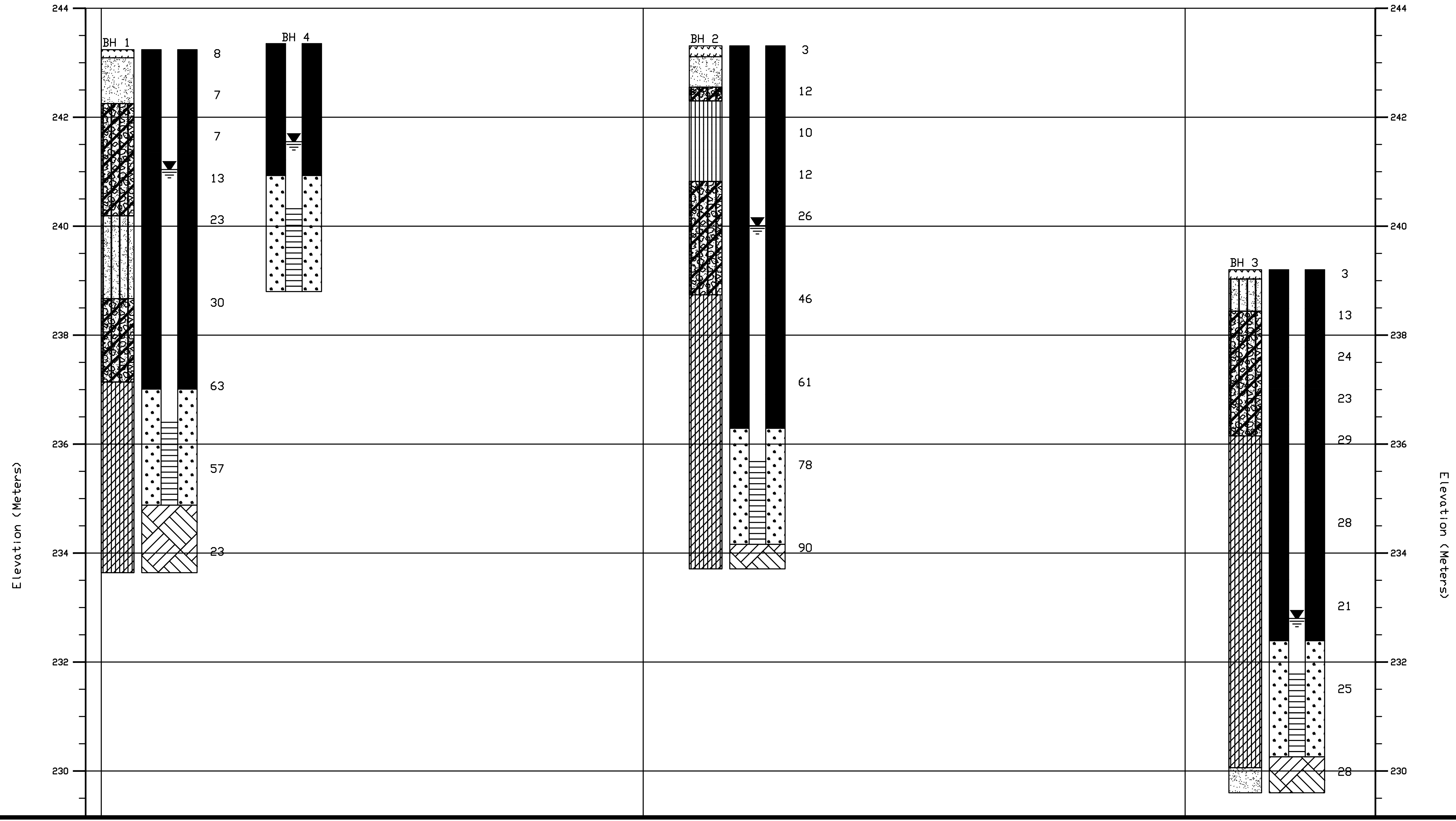
Date: November 2019

Project No.: 19-1171A

Drawn by: R.W.

Scale: 1:1500

Figure No.: 2B



Legend:

- ▨ Topsoil
- ▨ Sand
- ▨ Clayey & Sandy Silt Glacial Till
- ▨ Silty Sand
- ▨ Clayey Silt
- ▨ Silt
- ▼ Water Level in Monitoring Well

Notes:

1. Horizontal distances not to scale, boreholes spaced for visualization purposes.
2. Numbers beside boreholes indicate SPT "N" Values obtained in the borehole.

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Project: 893 & 911 Lockhart Road, Innisfil

Title: SUBSURFACE PROFILE

Approved by: A.W.
Date: November 2019
Project No.: 19-1171A
Drawn by: R.W.
Scale: As Shown
Figure No.: 3

Appendix A –

BOREHOLE LOGS

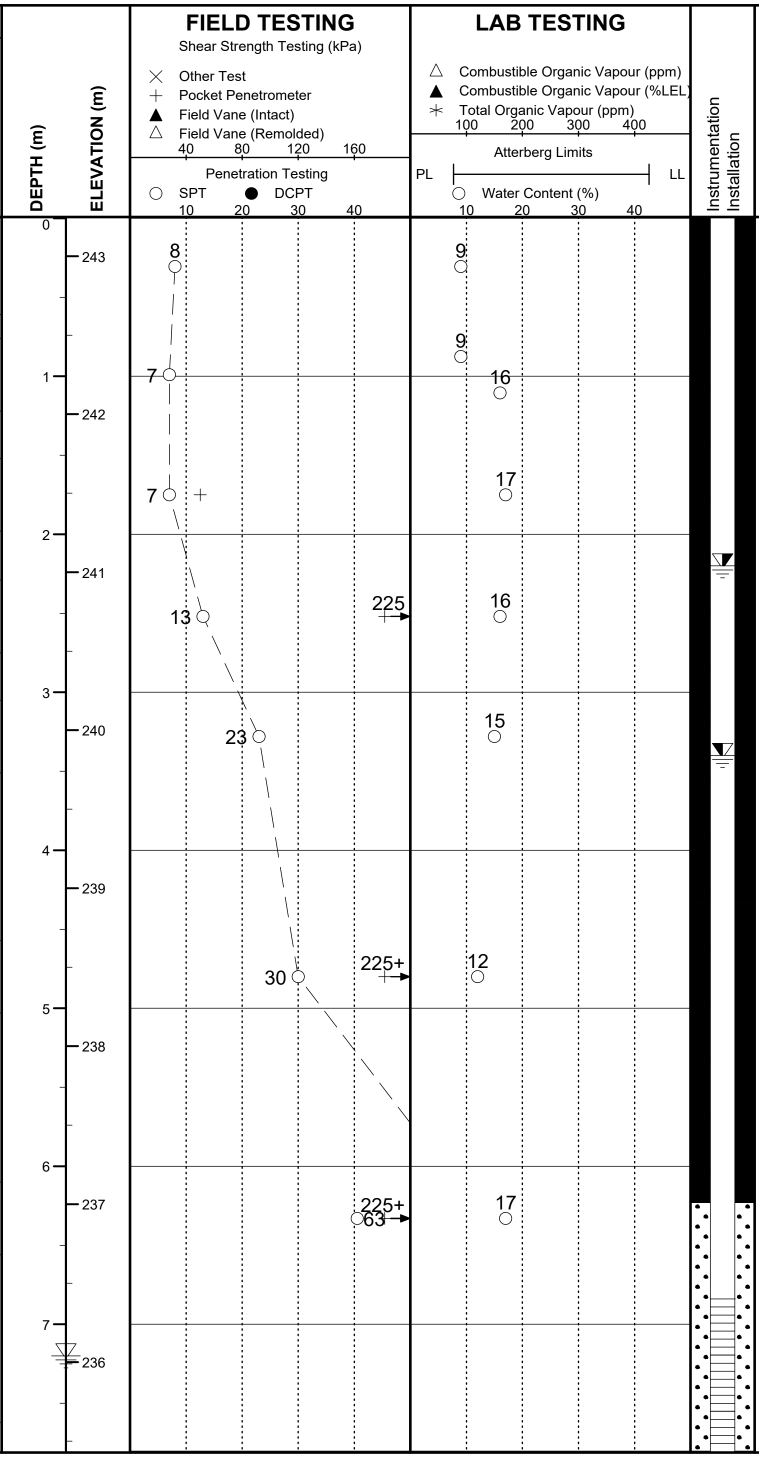
RECORD OF BOREHOLE No. 1

Project Number: **19-1171A**
Project Client: **Soheil & Mohamed Fayaz**
Project Name: **893 & 911 Lockhart Road**
Project Location: **Innisfil, Ontario**
Drilling Location: **Directly behind 913 Lockhart Rd.**

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Track Mount**
Logged By: **RD**
Reviewed By: **AW**
Northing: **4911952**
Easting: **615209**
Date Started: **2019-10-24**
Date Completed: **2019-10-24**

LITHOLOGY PROFILE / SOIL SAMPLING / COMMENTS & GRAIN SIZE DISTRIBUTION (%)

Depth (m)	Elevation (m)	DESCRIPTION (Geodetic 243.24m)	Sample Type	Sample Number	Recovery (%)	SPT "N" Value	GR	SA	SI	CL
0.2	243.1	Topsoil = 150mm								
		SAND, Some Silt, Trace Rootlets, Loose, Reddish Brown, Moist	SS	1	100	8				
		- - - Brown - - -								
1.0	242.3	CLAYEY SILT GLACIAL TILL, Sandy, Firm, Brown, Moist	SS	2A 2B	44	7				
		- - - Greyish Brown - - -	SS	3	89	7				
		- - - Hard - - -	SS	4	100	13	0	24	45	31
3.1	240.2	SILTY SAND, Trace Gravel, Compact, Brown, Wet	SS	5	100	23				
4.6	238.7	CLAYEY SILT GLACIAL TILL, Sandy, Hard, Greyish Brown, Moist	SS	6	94	30				
6.1	237.1	CLAYEY SILT, Some Sand, Hard, Grey, Moist	SS	7	100	63				



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Barrie, Ontario L4N 0B8
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W: centralearth.com

Groundwater depth encountered on completion of drilling: **7.2m**

Cave depth after auger removal: **8.5m**

Groundwater depth observed on **Oct. 30/19** at a depth of: **3.4m**

Observed on **Nov. 15/19** at a depth of: **2.2m**

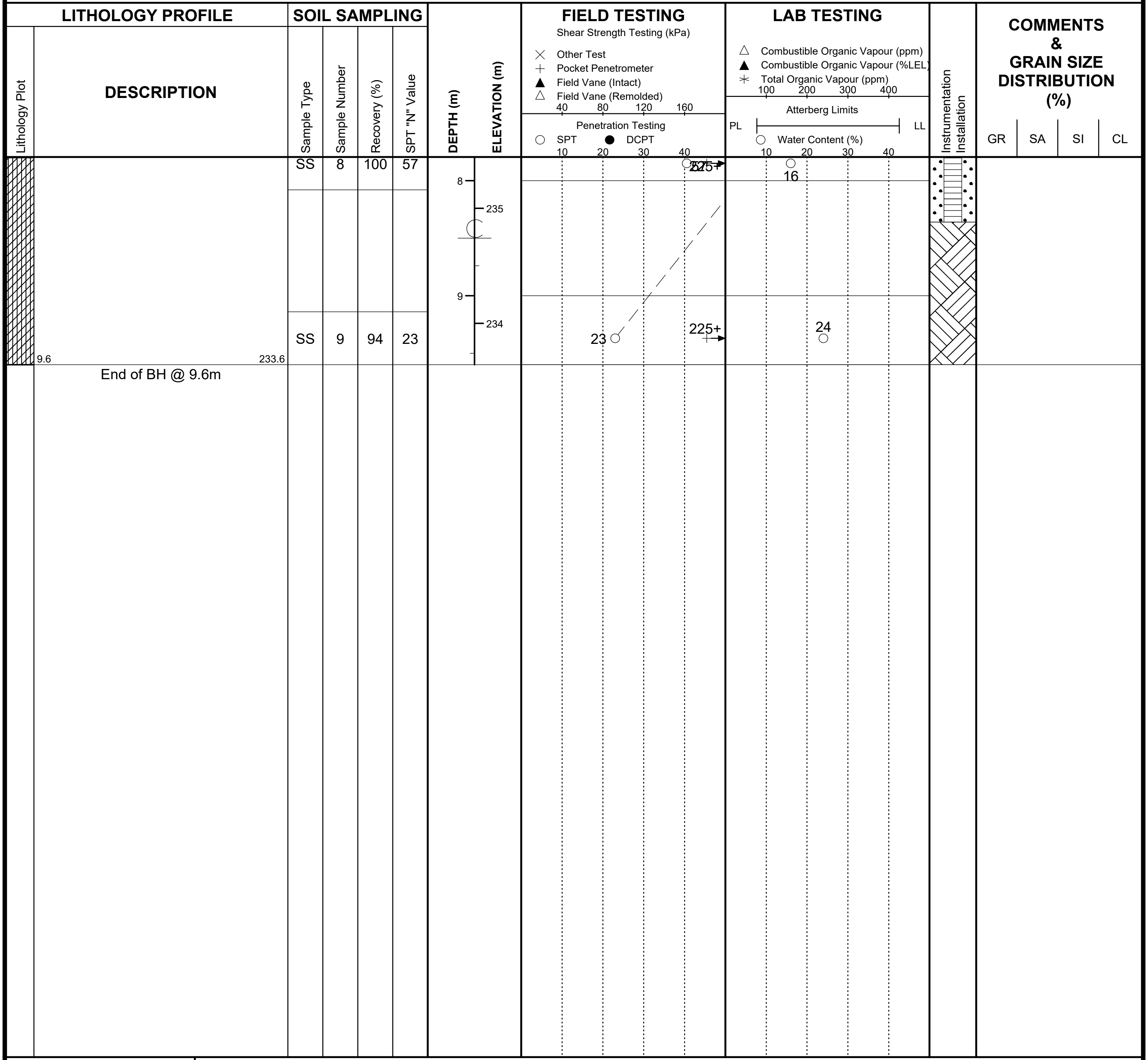
Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: **1 :50**

RECORD OF BOREHOLE No. 1

Project Number: **19-1171A**
Project Client: **Soheil & Mohamed Fayaz**
Project Name: **893 & 911 Lockhart Road**
Project Location: **Innisfil, Ontario**
Drilling Location: **Directly behind 913 Lockhart Rd.**

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Track Mount**
Logged By: **RD**
Reviewed By: **AW**
Northing: **4911952**
Easting: **615209**
Date Started: **2019-10-24**
Date Completed: **2019-10-24**



Sample Type	Sample Number	Recovery (%)	SPT "N" Value	Water Content (%)
SS	8	100	57	16
SS	9	94	23	24

9.6 233.6

End of BH @ 9.6m

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Groundwater depth encountered on completion of drilling: **7.2m**

Cave depth after auger removal: **8.5m**

Groundwater depth observed on **Oct. 30/19** at a depth of: **3.4m**

Observed on **Nov. 15/19** at a depth of: **2.2m**

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: **1 :50**

RECORD OF BOREHOLE No. 2

Project Number: **19-1171A**
Project Client: **Soheil & Mohamed Fayaz**
Project Name: **893 & 911 Lockhart Road**
Project Location: **Innisfil, Ontario**
Drilling Location: **Forested area 70m behind 901 Lockhart Rd.**

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Track Mount**
Logged By: **RD**
Northing: **4911934**
Date Started: **2019-10-24**
Reviewed By: **AW**
Easting: **615316**
Date Completed: **2019-10-24**

LITHOLOGY PROFILE

Geodetic 243.31m

Depth (m)	Elevation (m)	DESCRIPTION
0.2	243.1	Topsoil = 200mm
0.8	242.6	SAND, Some Silt, Very Loose, Reddish Brown, Moist
1.0	242.3	CLAYEY SILT GLACIAL TILL, Sandy, Stiff, Mottled Brown, Moist
2.5	240.8	SILT, Some Sand, Trace Clay, Compact, Brown, Moist
4.6	238.7	CLAYEY SILT GLACIAL TILL, Sandy, Very Stiff, Brown, Moist - - - Hard - - -
		CLAYEY SILT, Some Sand, Hard, Grey, Moist

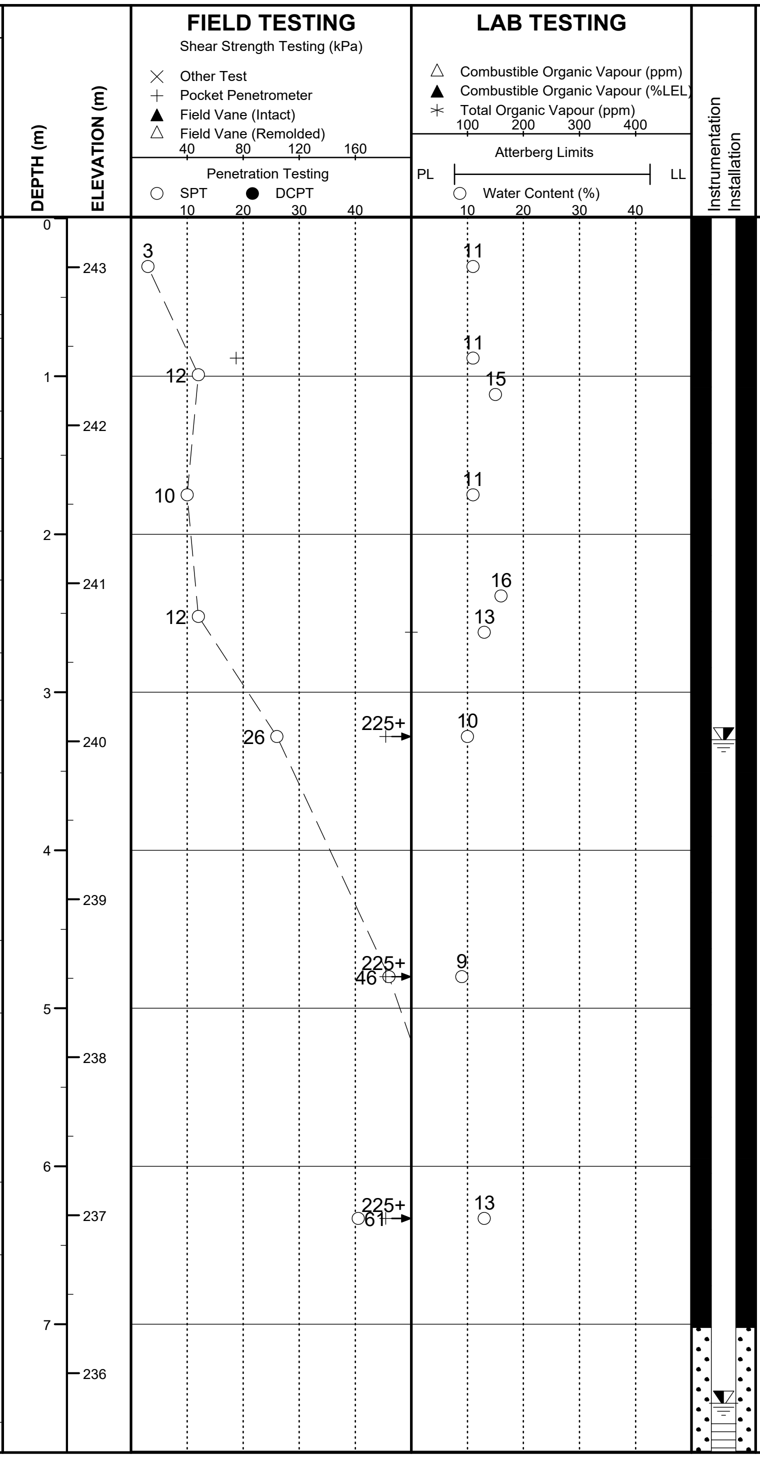
SOIL SAMPLING

Sample Type	Sample Number	Recovery (%)	SPT "N" Value	Water Content (%)	GR	SA	SI	CL
SS	1	83	3	11				
SS	2A	100	12	11				
	2B			15				
SS	3	72	10	11	0	19	73	8
SS	4A	100	12	16				
	4B			13				
SS	5	100	26	10				
SS	6	100	46	9				
SS	7	100	61	13				

FIELD TESTING

LAB TESTING

COMMENTS & GRAIN SIZE DISTRIBUTION (%)



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Groundwater depth encountered on completion of drilling: **Dry**
Cave depth after auger removal: **Open**
Groundwater depth observed on **Oct. 30/19** at a depth of: **7.5m**
Observed on **Nov. 15/19** at a depth of: **3.3m**

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

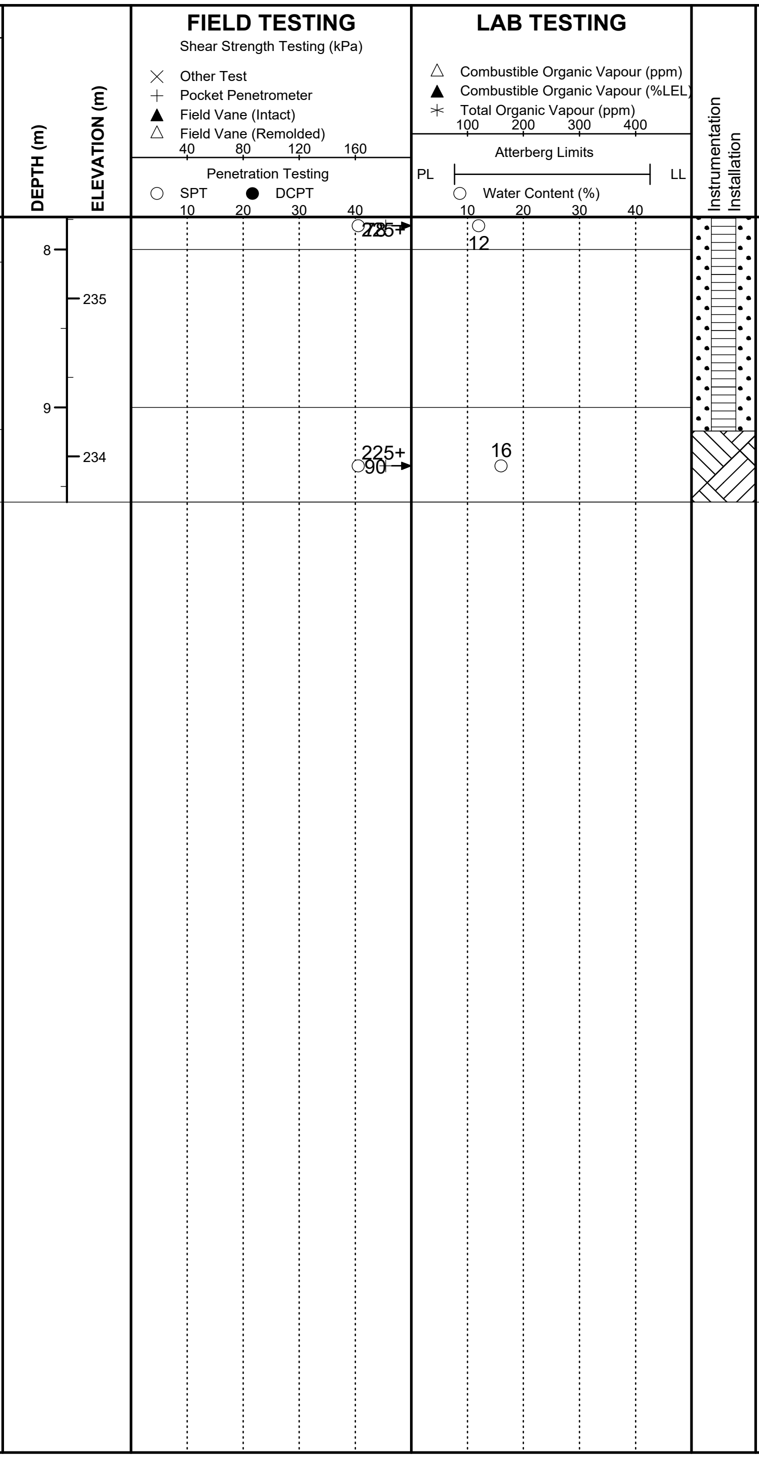
Scale: **1 :50**

RECORD OF BOREHOLE No. 2

Project Number: **19-1171A**
Project Client: **Soheil & Mohamed Fayaz**
Project Name: **893 & 911 Lockhart Road**
Project Location: **Innisfil, Ontario**
Drilling Location: **Forested area 70m behind 901 Lockhart Rd.**

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Track Mount**
Logged By: **RD**
Northing: **4911934**
Date Started: **2019-10-24**
Reviewed By: **AW**
Easting: **615316**
Date Completed: **2019-10-24**

LITHOLOGY PROFILE		SOIL SAMPLING				COMMENTS & GRAIN SIZE DISTRIBUTION (%)			
Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT "N" Value	GR	SA	SI	CL
▨		SS	8	100	78	0	11	62	27
▨		SS	9	100	90				
	9.6 233.7								
	End of BH @ 9.6m								



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W: centralearth.com

Groundwater depth encountered on completion of drilling: **Dry**

Cave depth after auger removal: **Open**

Groundwater depth observed on **Oct. 30/19** at a depth of: **7.5m**

Observed on **Nov. 15/19** at a depth of: **3.3m**

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: **1 :50**

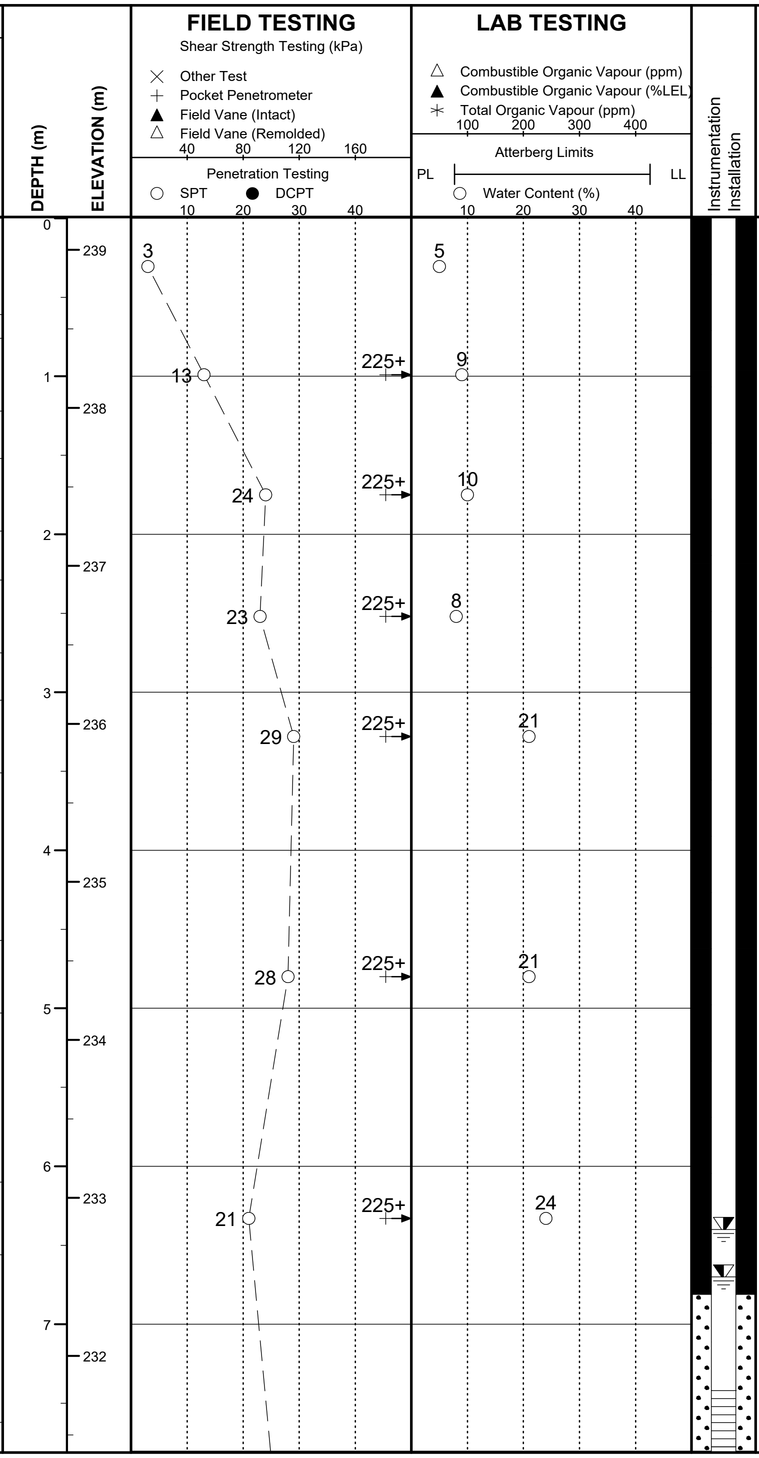
RECORD OF BOREHOLE No. 3

Project Number: **19-1171A**
Project Client: **Soheil & Mohamed Fayaz**
Project Name: **893 & 911 Lockhart Road**
Project Location: **Innisfil, Ontario**
Drilling Location: **Backyard of 893 Lockhart Rd.**

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Track Mount**
Logged By: **RD**
Northing: **4912017**
Date Started: **2019-10-24**
Reviewed By: **AW**
Easting: **615371**
Date Completed: **2019-10-24**

LITHOLOGY PROFILE / SOIL SAMPLING

Depth (m)	Elevation (m)	DESCRIPTION (Geodetic 239.20m)	Sample Type	Sample Number	Recovery (%)	SPT "N" Value	Pocket Penetrometer (kPa)	Water Content (%)
0.2	239.0	Topsoil = 170mm						
0.8	238.4	SILTY SAND, Very Loose, Brown, Moist	SS	1	75	3		5
3.1	236.2	CLAYEY SILT GLACIAL TILL, Sandy, Hard, Brown, Moist	SS	2	100	13	225+	9
			SS	3	100	24	225+	10
			SS	4	100	23	225+	8
		CLAYEY SILT, Some Sand, Hard, Brownish Grey, Moist	SS	5	100	29	225+	21
		- - - Grey - - -	SS	6	100	28	225+	21
			SS	7	100	21	225+	24
		- - - Very Stiff - - -						



COMMENTS & GRAIN SIZE DISTRIBUTION (%)

GR	SA	SI	CL

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Groundwater depth encountered on completion of drilling: **6.7m**

Cave depth after auger removal: **Open**

Groundwater depth observed on **Oct. 30/19** at a depth of: **6.7m**

Observed on **Nov. 15/19** at a depth of: **6.4m**

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: **1 :50**

RECORD OF BOREHOLE No. 3

Project Number: **19-1171A**
Project Client: **Soheil & Mohamed Fayaz**
Project Name: **893 & 911 Lockhart Road**
Project Location: **Innisfil, Ontario**
Drilling Location: **Backyard of 893 Lockhart Rd.**

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Track Mount**
Logged By: **RD**
Northing: **4912017**
Date Started: **2019-10-24**
Reviewed By: **AW**
Easting: **615371**
Date Completed: **2019-10-24**

FIELD TESTING
Shear Strength Testing (kPa)
× Other Test
+ Pocket Penetrometer
▲ Field Vane (Intact)
△ Field Vane (Remolded)
40 80 120 160
Penetration Testing
○ SPT ● DCPT
10 20 30 40

LAB TESTING
△ Combustible Organic Vapour (ppm)
▲ Combustible Organic Vapour (%LEL)
✳ Total Organic Vapour (ppm)
100 200 300 400
Atterberg Limits
PL LL
○ Water Content (%)
10 20 30 40

LITHOLOGY PROFILE		SOIL SAMPLING									COMMENTS & GRAIN SIZE DISTRIBUTION (%)			
Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT "N" Value	DEPTH (m)	ELEVATION (m)	FIELD TESTING	LAB TESTING	Instrumentation Installation	GR	SA	SI	CL
▨	9.1 230.1	SS	8	100	25	8	231	25 ○ +	○ 26					
▨	FINE TO MEDIUM SAND, Trace Silt, Compact, Greyish Brown, Wet 9.6 229.6	SS	9	100	28	9	230	28 ○	17 ○		0	97	(3)	
	End of BH @ 9.6m													

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Groundwater depth encountered on completion of drilling: **6.7m**

Cave depth after auger removal: **Open**

Groundwater depth observed on **Oct. 30/19** at a depth of: **6.7m**

Observed on **Nov. 15/19** at a depth of: **6.4m**

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

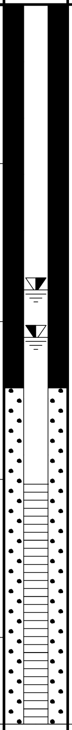
Scale: **1 :50**

RECORD OF BOREHOLE No. 4

Project Number: **19-1171A**
Project Client: **Soheil & Mohamed Fayaz**
Project Name: **893 & 911 Lockhart Road**
Project Location: **Innisfil, Ontario**
Drilling Location: **Immediately Adjacent to BH1**

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Track Mount**
Logged By: **RD**
Reviewed By: **AW**
Northing: **4911953**
Easting: **615211**
Date Started: **2019-10-24**
Date Completed: **2019-10-24**

LITHOLOGY PROFILE		SOIL SAMPLING						FIELD TESTING	LAB TESTING		COMMENTS & GRAIN SIZE DISTRIBUTION (%)			
Lithology Plot	DESCRIPTION Geodetic 243.35m	Sample Type	Sample Number	Recovery (%)	SPT "N" Value	DEPTH (m)	ELEVATION (m)	Shear Strength Testing (kPa) × Other Test + Pocket Penetrometer ▲ Field Vane (Intact) △ Field Vane (Remolded) 40 80 120 160 Penetration Testing ○ SPT ● DCPT 10 20 30 40	△ Combustible Organic Vapour (ppm) ▲ Combustible Organic Vapour (%LEL) ✳ Total Organic Vapour (ppm) 100 200 300 400 Atterberg Limits PL LL ○ Water Content (%) 10 20 30 40	Instrumentation Installation	GR	SA	SI	CL
	No Sampling Conducted - Only for Shallow Monitoring Well Installation Adjacent to Borehole 1 See Stratigraphic Conditions in Borehole 1 for More Details 4.6 238.8					0 1 2 3 4	243 242 241 240 239							
	End of BH @ 4.6m													



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Groundwater depth encountered on completion of drilling: **2.3m**

Cave depth after auger removal: **N/A**

Groundwater depth observed on **Oct. 30/19** at a depth of: **2.1m**

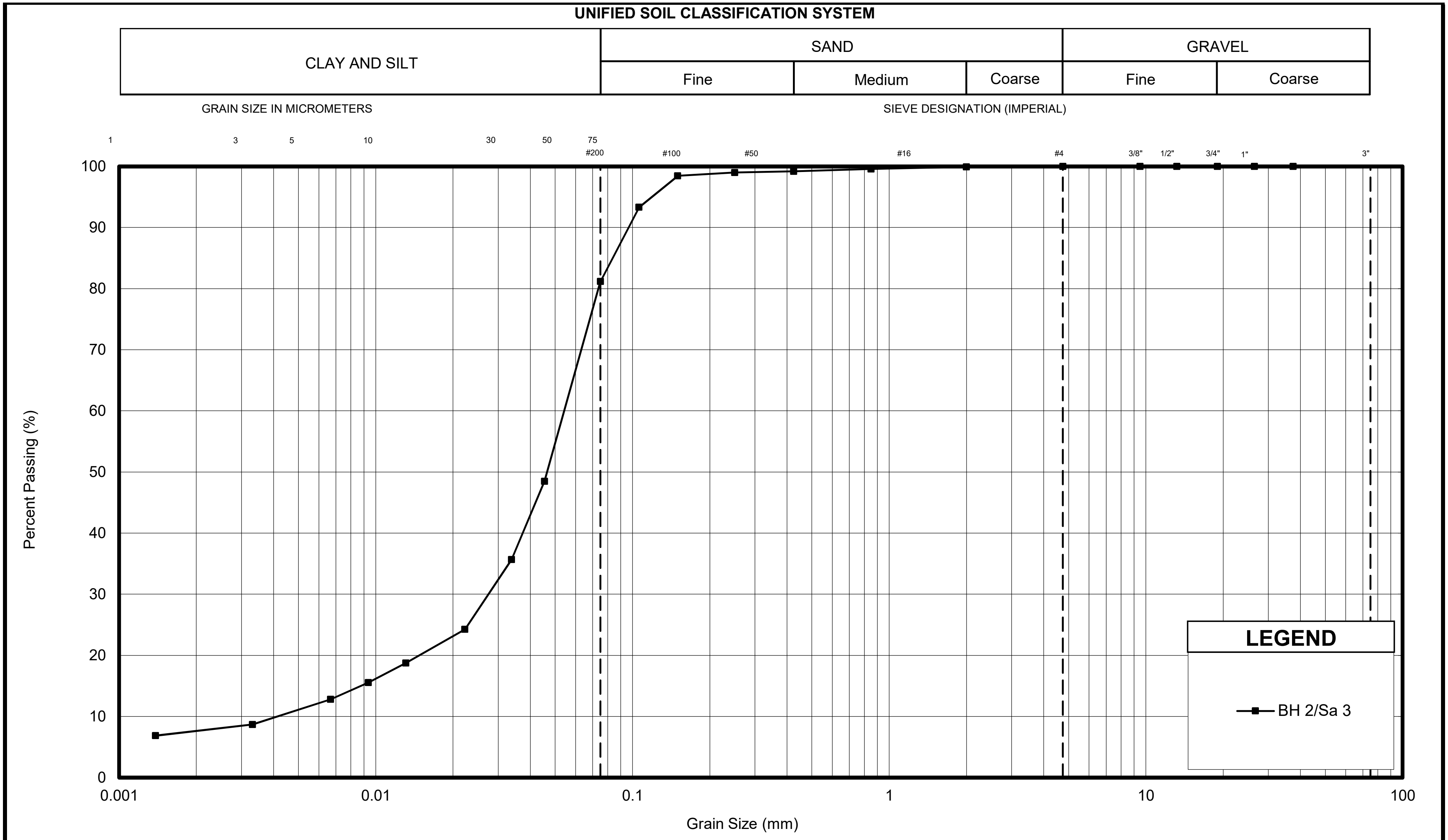
Observed on **Nov. 15/19** at a depth of: **1.8m**

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: **1 :50**

Appendix B –

GEOTECHNICAL LABORATORY DATA



Sample	**Description**	**Gr.**	**Sa.**	**Si.**	**Cl.**	**D_{10}**	**D_{30}**	**D_{60}**	**C_u**	**C_c**
BH 2, Sa 3	SILT, Some Sand, Trace Clay	0	19	73	8	0.004	0.027	0.054	13.1	3.4

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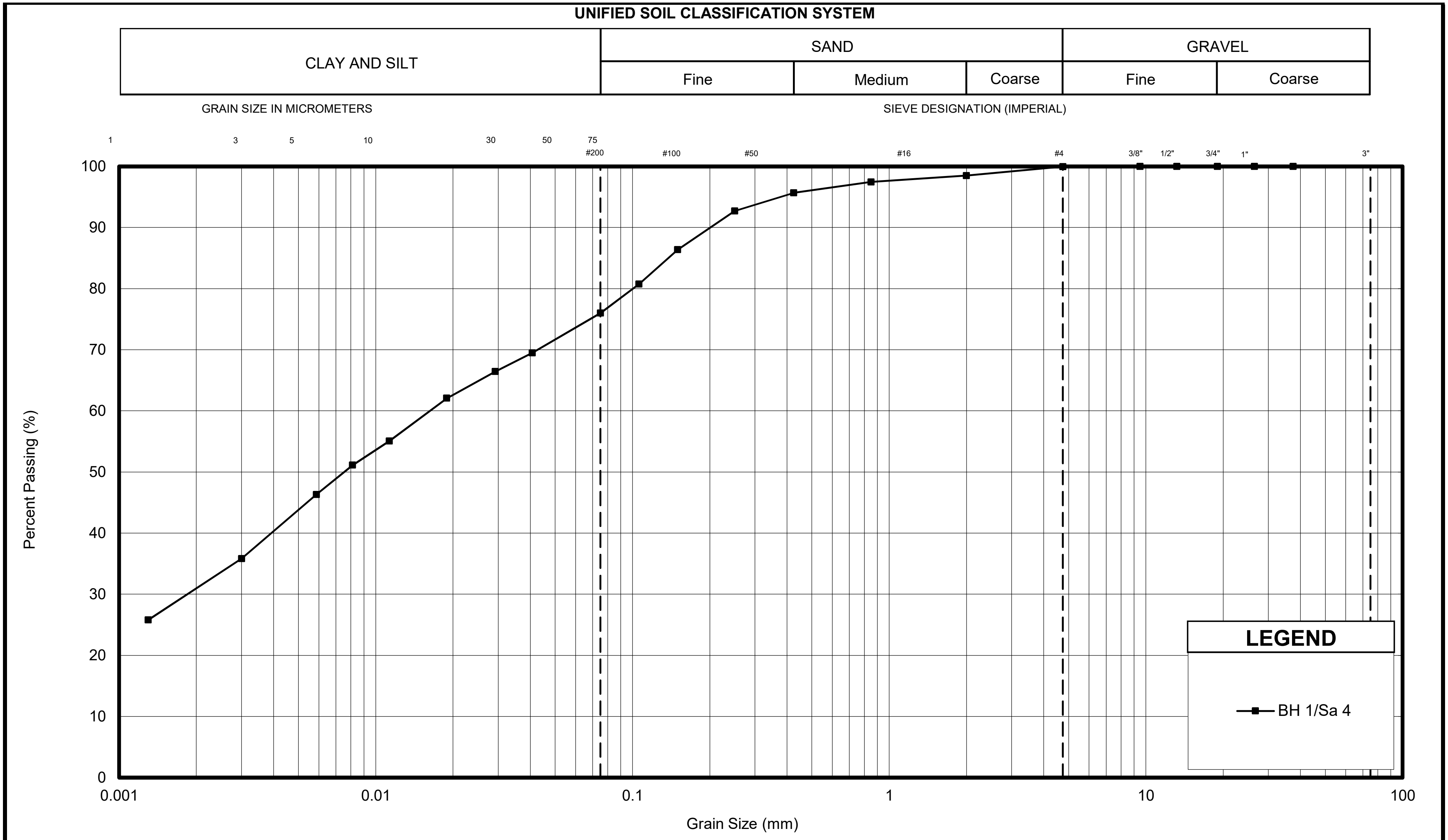
GRAIN SIZE DISTRIBUTION

SILT

FIGURE No.	B1
REF. No.	19-1171A
DATE	November 2019



Sample	Description	Gr.	Sa.	Si.	Cl.	D_{10}	D_{30}	D_{60}	C_u	C_c
BH 1, Sa 4	CLAYEY SILT GLACIAL TILL, Sandy	0	24	45	31	-	0.002	0.016	-	-

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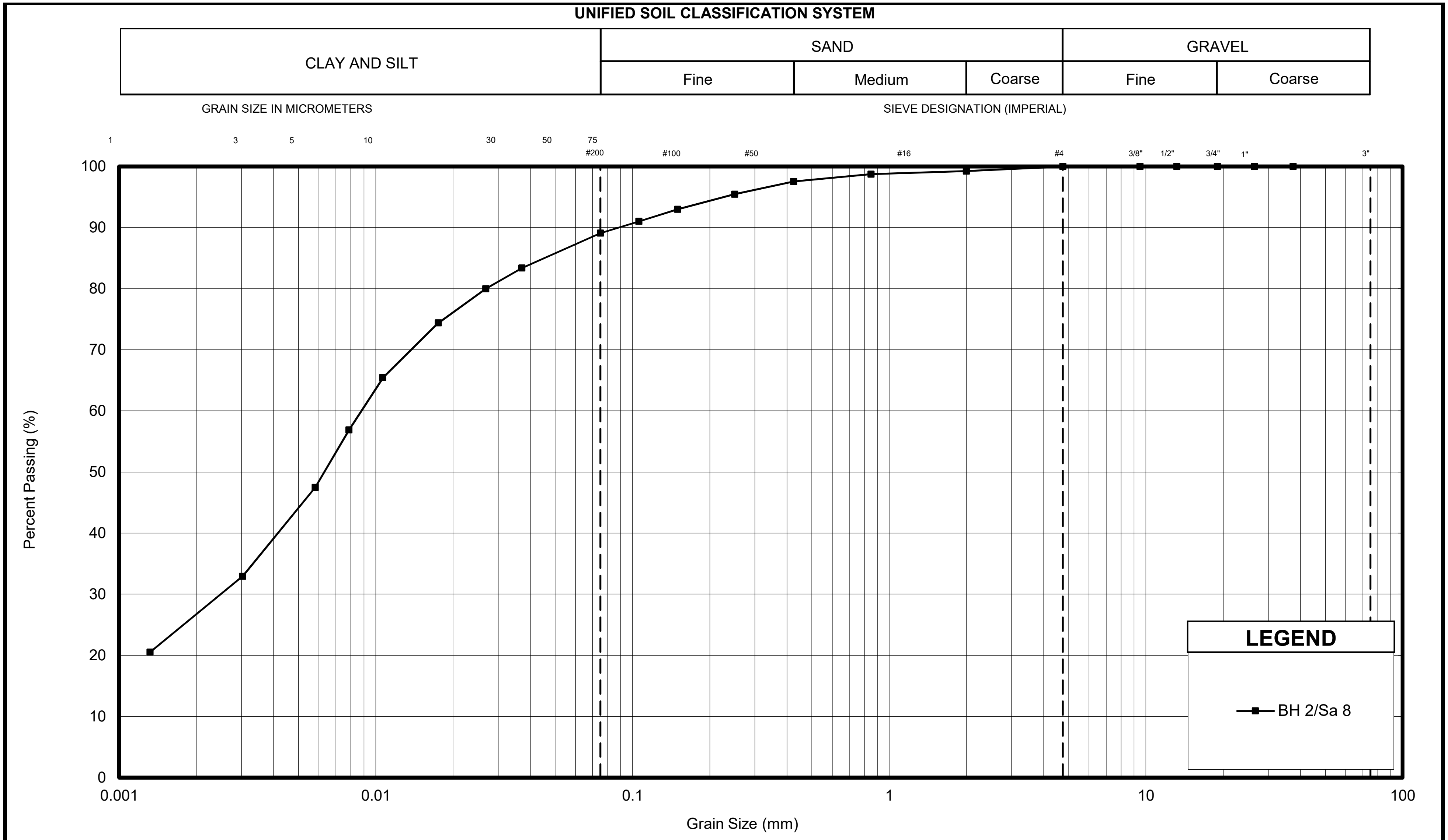
GRAIN SIZE DISTRIBUTION

CLAYEY SILT GLACIAL TILL

FIGURE No.	B2
REF. No.	19-1171A
DATE	November 2019



Sample	Description	Gr.	Sa.	Si.	Cl.	D_{10}	D_{30}	D_{60}	C_u	C_c
BH 2, Sa 8	CLAYEY SILT, Some Sand	0	11	62	27	-	0.002	0.009	-	-

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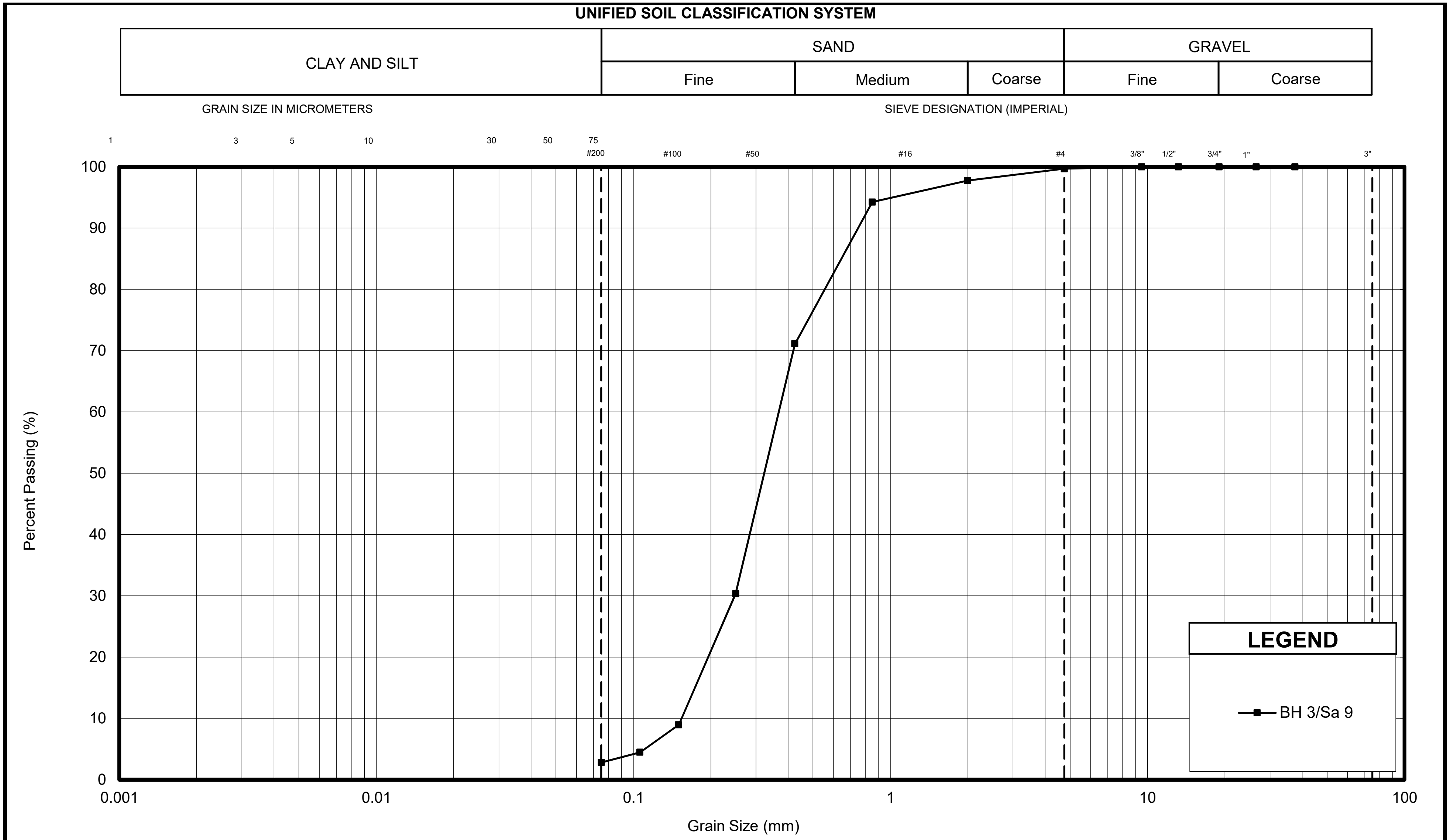
GRAIN SIZE DISTRIBUTION

CLAYEY SILT

FIGURE No.	B3
REF. No.	19-1171A
DATE	November 2019



Sample	Description	Gr.	Sa.	Si.	Cl.	D_{10}	D_{30}	D_{60}	C_u	C_c
BH 3, Sa 9	FINE TO MEDIUM SAND, Trace Silt	0	97	3		0.15	0.25	0.37	2.4	1.1

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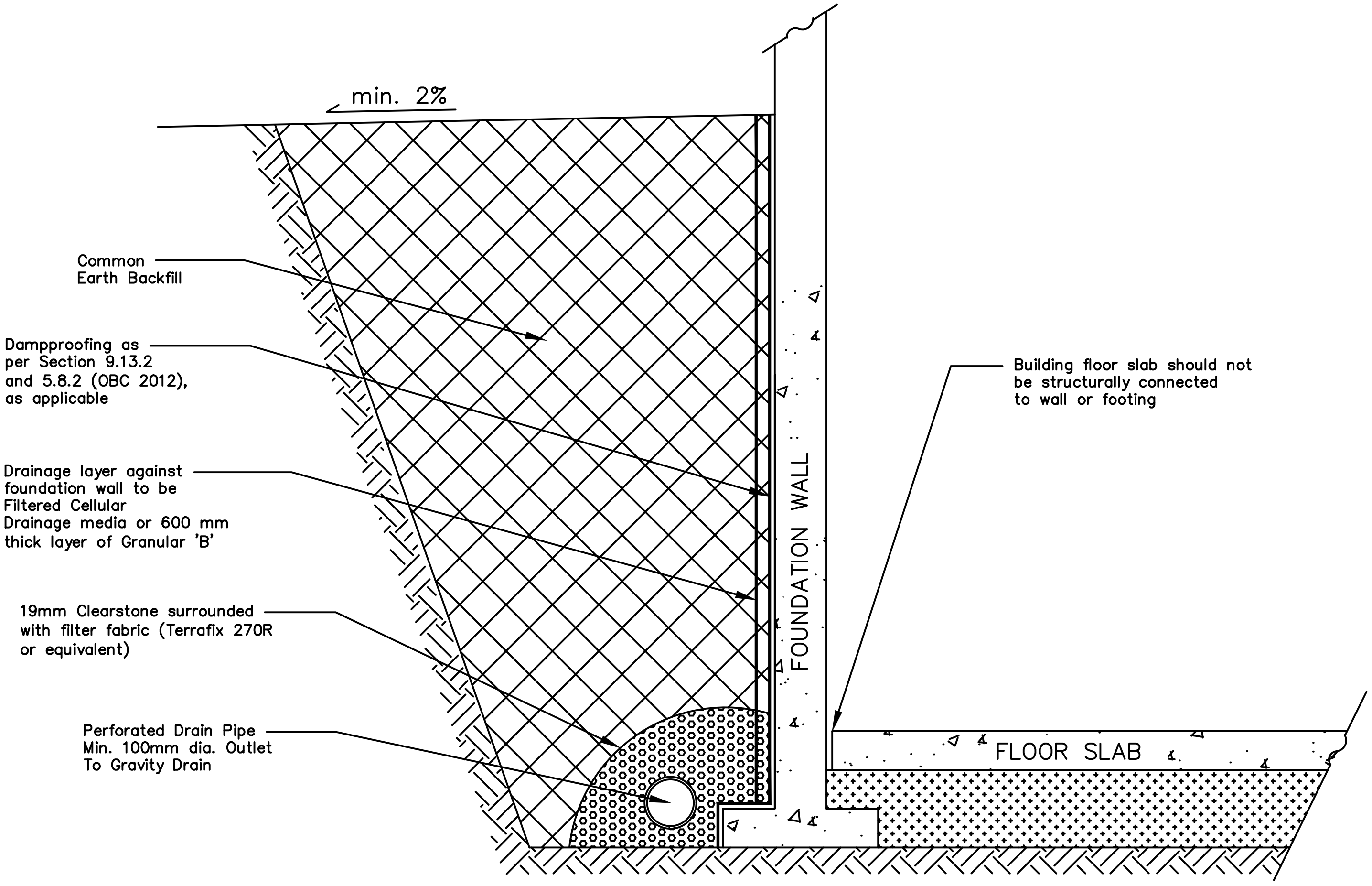
GRAIN SIZE DISTRIBUTION

FINE TO MEDIUM SAND

FIGURE No.	B4
REF. No.	19-1171A
DATE	November 2019

Appendix C –

TYPICAL DETAILS



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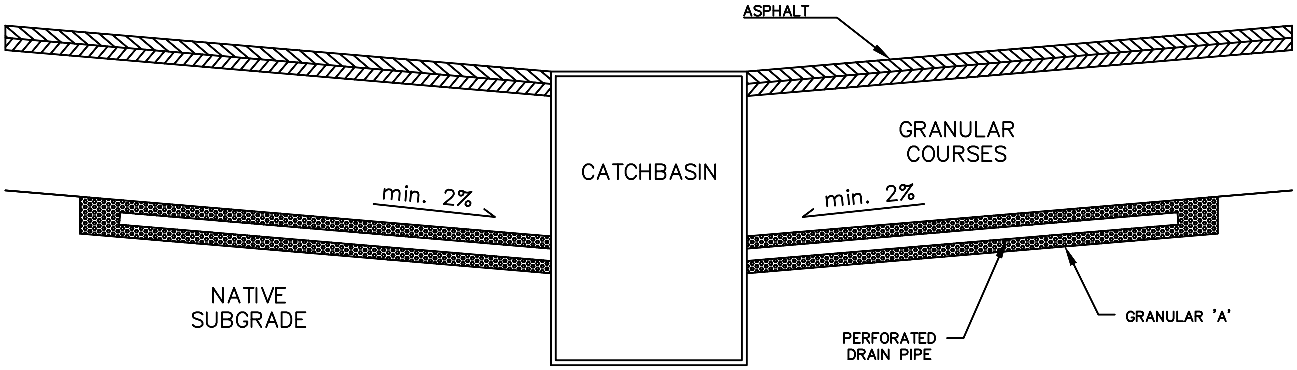
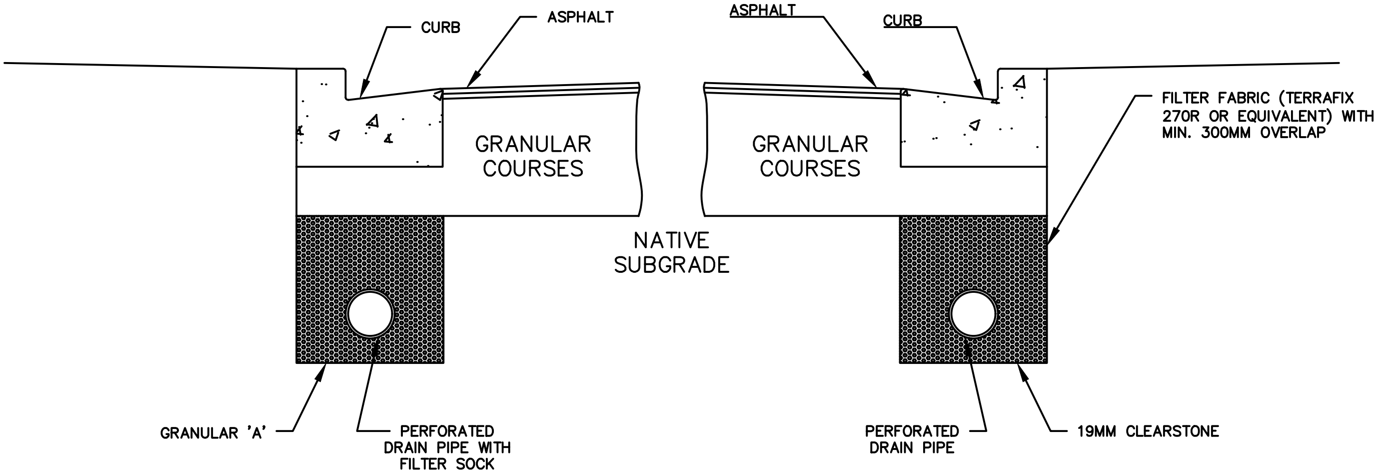
BASEMENT FOUNDATION WALL DRAINAGE TYPICAL DETAIL

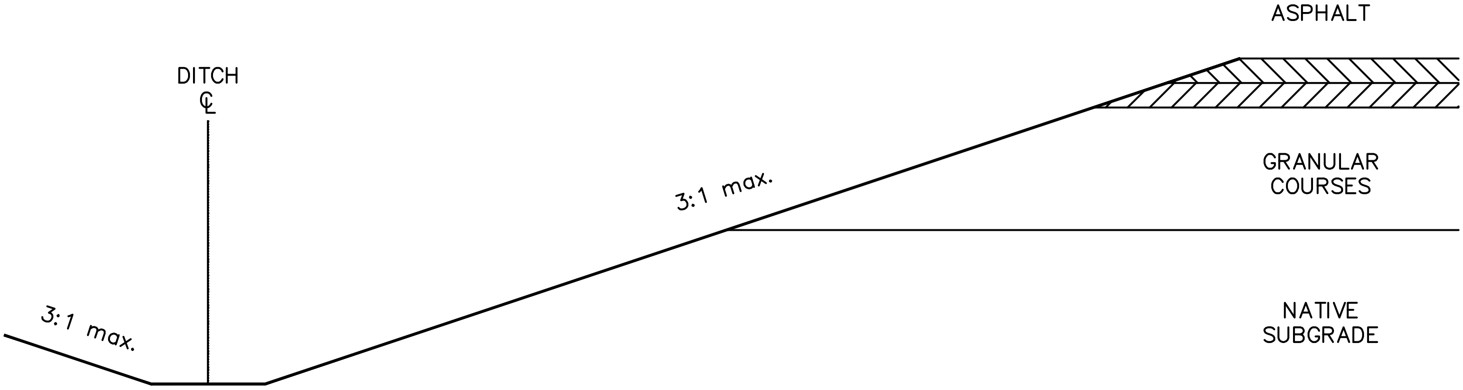




Unwrapped Trench

Wrapped Trench

Urban Cross Sections



Rural Cross Section

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PAVEMENT DRAINAGE ALTERNATIVES TYPICAL DETAILS