GEOTECHNICAL INVESTIGATION REPORT
PROPOSED SANITARY SEWER
OAK RIDGE DRIVE AND WILDWOOD ROAD
HALTON HILLS, ONTARIO

Prepared for

2147925 ONTARIO INC.

Prepared by

SIRATI & PARTNERS CONSULTANTS LIMITED

Geotechnical Hydrogeological & Environmental Solutions

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1. INTRODUCTION

Sirati & Partners Consultants Limited (SIRATI) was retained by 2147925 Ontario Inc. (the Client) to undertake a geotechnical investigation for the proposed sanitary sewer extending from the proposed subdivision at Part of the West half of Lot 21, Concession 9, Hamlet of Glen Williams, to Oak Ridge Drive and Wildwood Road to approximately 100 m east of Confederation Street.

A copy of the proposed sanitary sewer plan was provided to SIRATI:

Ref. 1: External Sanitary Sewer Plan and Profile, Drawing Nos. 09-015-04 to 09-015-11, dated September 2020, prepared by Condeland Consulting Engineers and Project Managers, prepared for 2147925 Ontario Inc.

According to Ref. 1, the proposed sanitary sewer will have a 200 mm to 300 mm diameter and will be 3 m to 8 m deep. The construction of the sewer will be carried out using open cut method from the subdivision up to SANMH16A. The installation will be continued using trenchless methods to cross two culverts, as well as Credit River.

The purpose of the geotechnical investigation was to obtain information on the general subsurface soil and shallow groundwater conditions at the site by means of boreholes as well as geotechnical laboratory tests, and provide recommendations regarding open cut and trenchless excavations and installation of the new sanitary sewer, as well as the following items:

- A slope stability evaluation for the section of sanitary sewer installation adjacent to 27 and 33 Wildwood Road, identifying the location of the Long-Term Stable Slope Line and recommending a method for constructing the sanitary sewer through this area.

- A geotechnical evaluation for construction adjacent to the two retaining walls opposite to each other at 33 Wildwood Road to verify that the proposed sanitary sewer installation will not have a negative impact on slope stability and the retaining wall structures.

This report is provided based on the terms of reference presented above and, on the assumption, that the design will be in accordance with the applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations of this office can be relied upon.

The site investigation and recommendations follow generally accepted practice for geotechnical consultants in Ontario. The format and contents are guided by client specific needs and economics and do not conform to generalized standards for services. Laboratory testing for most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice.

This report has been prepared for GTAA and its designers. Third party use of this report without SIRATI consent is prohibited. The limitation conditions presented in **Appendix E** form an integral part of the report and they must be considered in conjunction with this report.

2. INVESTIGATION PROCEDURE

A total of twelve (12) boreholes (BH101 to BH107 and BH 109 to BH 113, see **Figures 1** for the borehole location) were drilled by SIRATI during December 18, 2020 and January 07, 2021. The boreholes were advanced 5.2 m (BH 105, BH 106, and BH 110) to 9.6 m (BH 101, BH 112 and BH 113) below existing ground surface (m bgs).

The boreholes were drilled with hollow stem continuous flight auger equipment by a drilling sub-contractor under the direction and full-time supervision of SIRATI personnel. Samples were retrieved at regular intervals with a 50 mm O.D. split-barrel sampler driven with a hammer weighing 624 N and dropping 760 mm in accordance with the Standard Penetration Test (SPT) method. Soil samples were logged in the field and returned to SIRATI laboratory for detailed examination by the project engineer and for laboratory testing.

A total of six (6) monitoring wells were installed at boreholes BH 101, BH 103, BH 105, BH 107, BH 112, and BH113 for post-field work groundwater monitoring purposes at stabilized condition. The monitoring wells were constructed using a 50 mm diameter Schedule 40 polyvinyl chloride (PVC) pipe. The monitoring wells were constructed in accordance with O. Reg. 903 (as amended) by extending a bentonite seal and/or grout from above the well screen to the surface. The monitoring wells were completed by installing a steel protective well cover.

Soil samples were tested for moisture content and selected soil samples were tested for grain size, hydrometer and Atterberg Limits tests. The results of the laboratory tests will be provided in the final report.

Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD), weathering, classification data of the recovered rock core sample in BH 110 were recorded in the field based on visual inspection. The bedrock was sequentially photographed, packed and transported to SIRATI laboratory for further visual examination.

The degree of weathering of the bedrock core sample and the strength classification of the intact rock mass based on field identification are described in accordance with Table B.3 and Table B.6, respectively of the International Society of Rock Mechanics (ISRM, 1985) standard classification system. Classification of the rock mass quality of the bedrock with respect to the Rock Quality Designation (RQD) is described based on Table 3.10 of the Canadian Foundation Engineering Manual 2006 (CFEM), and the strength of the bedrock core samples is based on Table 3.5 of CFEM.

The as-drilled borehole locations and ground surface elevations were established by SIRATI. The borehole locations, ground surface elevations, and termination depths and elevations are presented on the borehole records.

3. SITE AND SUBSURFACE CONDITIONS

The borehole location plan is shown on **Figure 1**. Notes on soil descriptions are presented on **Enclosure 1A**. The subsurface conditions in the boreholes are presented in the individual borehole logs (**Encl. 2 to 13**).

The subsurface conditions in the boreholes are summarized in the following paragraphs.

3.1. SOIL CONDITIONS

The following presents the soil stratigraphy based on the observations of the boreholes drilled by SIRATI (BH 101 to BH 107 and BH 109 to BH 113).

Asphalt: A 100 to 150 mm-thick asphalt was encountered at ground surface at the location of all boreholes, except at BH 112.

Topsoil: A 150 mm-thick topsoil was encountered at ground surface at the location of BH 112.

Granular Fill (Sand and Gravel): A layer of Granular fill composed of sand and gravel was encountered below the asphalt/topsoil in all boreholes. The thickness of granular layer varies between 75 mm and 900 mm.

Fill Material: Below the granular fill, a layer of fill/probable fill material was encountered in BH 103, BH 104, BH 107, and BH 109 to BH 113 underlying the granular fill. The fill material mainly consists of sand/sandy silt/silty sand, trace to some clay to clayey silt with different proportions of gravel. The fill/probable fill material was found to be generally moist and brown in color, extending 1.5 m (BH 104, BH 109 to BH 111) to 3 m (BH 107 and BH 112) below existing ground surface (mbgs). It should be noted that trace of organics and cobbles were observed in the fill layer.

The measured SPT ‘N’ values in fill material ranged from 3 to 41 blows per 300 mm of penetration, indicating a very loosely to densely compacted fill material.

Cohesionless Soil Deposits: Native cohesionless soil deposits were observed in BH 101, BH 102, BH 103, BH 104, BH 106, BH 107, and BH 109 to BH 113 underlying/sandwiched into the glacial till deposit or underlying the fill material. This layer generally comprises silt, sand, gravel or sandy gravel/gravelly sand, and is found to be brown to grey and moist to wet.

The measured SPT ‘N’ values in the cohesionless soil deposit ranged from 10 to over 50 blows per 300 mm penetration, indicating a compact to very dense material.

It should be noted that trace to some cobbles were observed within the stratum at some borehole locations, including BH 106, BH 107, BH 109, BH 112, and BH 113. Also, this stratum is expected to contain boulders which should be taken into consideration by the contractor.

Glacial Deposits: Glacial till deposits were observed in BH101, BH102, BH105, and BH106 underlying the fill/granular fill as well as in BH 103, BH 104, and BH 113 underlying cohesionless soil deposit. This layer generally comprises sandy silt/silty sand, trace clay to clayey silt deposit. The glacial till deposit was found to be generally moist and brown in color.

The measured SPT 'N' values in the sandy silt/silty sand till deposit ranged from 10 to 74 blows per 300 mm penetration, indicating a compact to very dense material.

The measured SPT 'N' values of 9 to 37 were obtained in the cohesive till deposit, indicating a stiff to hard consistency of the layer.

It should be noted that the stratum is believed to contain cobbles as well as boulders which should be taken into consideration by the contractor.

Queenston Formation – Shale Bedrock: Weathered shale bedrock was observed in boreholes BH 110 and BH 111, upon spoon refusal at the Elevations of 227.1 m and 222.6 m, respectively (Geodetic).

The shale bedrock is of the Queenston Formation. The material is reddish brown and features an upper sub-unit. The upper (weak) sub-unit is highly weathered (W4) and fractured, and in a very poor to poor condition. SPT tests carried out in this sub-unit of the weathered shale bedrock measured N-values of more than 50 blows for less than 300 mm sampler penetration.

The highly weathered upper sub-unit of the bedrock can be readily penetrated using solid stem augers which indicates that in all likelihood, the material in upper sub-unit will have the engineering characteristics of a hard clayey till soil.

It should be further noted that the bedrock surface has been inferred from the results of the standard penetration tests, limited samples obtained by the split spoon and observations made on the rock core recovered during coring. These boundaries generally represent transitions from overburden to residual soil or highly weathered shale and should no be inferred to represent an exact plane of the bedrock surface. In general, depths to refusal to further auger, casing, and/or split spoon advancement in boreholes should not be interpreted as a confirmation of bedrock surface but may be inferred to indicate potential proximity to bedrock surface.

It should be noted that bedrock coring was carried out in less weathered sub-unit of bedrock at borehole BH 110 to verify the quality of the bedrock. Based on the examination of the rock core samples retrieved, the bedrock in less weathered sub-unit typically consisted of moderately weathered to slightly weathered (W3 to W2) Shale.

The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the core sample recovered in the borehole BH 110 were found to be 69% and 58%, respectively. The Rock Quality Designation (RQD) was 33%, indicating a rock mass of poor quality, as per Table 3.10 of CFEM.

Approximate depth, length and Rock Quality Designation (R. Q. D.) of the cored sample are presented in the respective borehole log.

3.2. GROUNDWATER CONDITIONS

Monitoring wells were installed in BH 101, BH 103, BH 105, BH 107, BH 112, and BH113, upon completion of drilling. Groundwater (GW) levels in the monitoring wells were measured on January 28, 2021, the results of which are presented in **Table 1**.

Table 1: Groundwater level observations

Borehole/ Monitoring Well	**Ground Elevation (m ASL*)**	**Well Depth (m)**	**January 28, 2021**	
			Depth to GW (m bgs)	**GW Elevation (m ASL*)**
BH 101	273.5	9.1	2.7	270.8
BH 103	270.0	7.6	Dry	N/A
BH 105	258.1	4.7	2.0	256.1
BH 107	242.5	7.6	7.5	235.0
BH 112	229.0	9.1	2.2	226.8
BH 113	229.9	9.1	3.1	226.8

**m ASL – meters above sea level*

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to major weather events.

4. GEOTECHNICAL ENGINEERING RECOMMENDATIONS

This section of the report provides geotechnical recommendations for the installation of the proposed sanitary sewer that will extend from the proposed subdivision at Part of the West half of Lot 21, Concession 9, Hamlet of Glen Williams, to Oak Ridge Drive and Wildwood Road to approximately 100 m east of Confederation Street. The current geotechnical report discusses the installation of the sanitary sewer outside the proposed subdivision.

According to Ref. 1, the proposed sanitary sewer will be approximately 1.4 km long, with 200 mm to 300 mm diameter, extending from the Client's property boundary to Meagan Drive, to Oak Ridge Drive south, to Wildwood Drive, to approximately 100 m east of confederation Street on Main Street. The depth of the proposed sanitary sewer changes from 3 m to 8 m and the depth of the boreholes were selected accordingly.

It is understood that the construction of the proposed sewer will be carried out using open cut excavation, except from SANMH16A towards the end of the pipe at SANMH20A, where trenchless methods should be used to cross two existing culverts, as well as credit river (approximately 260 m).

Where comments are made on construction, they are provided to highlight those aspects which could affect the design of the project, and for which special provision may be required during construction. Those requiring information on aspects of construction should make their own interpretation of the factual information, provided such interpretation may affect selections, proposed construction methods, scheduling and the like.

It should be noted that the glacial till and granular deposits may contain boulders. Possible large obstructions can be anticipated in the fill material. Contractor should be prepared for such conditions during construction.

4.1. TRENCHING AND DEWATERING

According to Ref. 1, it is understood that the proposed sanitary sewer may be constructed using open cut excavation method from the property boundary in Meagan Drive to SANMH16A on Wildwood Road. The depth of the sanitary changes from 2.5 m to 8.0 m.

It is expected that open cut trenches will be dug through fill material and native strata. Groundwater levels observed in the monitoring wells installed at boreholes BH 101, BH 105, BH 107, BH 112, and BH 113 were found to range between the depths of 2.0 m and 7.5 m below the existing grade, (Elev. 226.8 m to 270.8 m). Therefore, water ingress in the trenches should be expected. For any excavation below groundwater table, temporary dewatering will be required. Groundwater level should be lowered to at least 1 m below the excavation level, otherwise it will result in base instability and flowing sides. Given the cohesionless nature of the soil, significant dewatering through point wells will be required.

It is recommended that test pits be undertaken by prospective contractors in order to observe and evaluate soil support and groundwater conditions along the proposed sewer installation to assess preferred means of excavation and groundwater control.

In the planning of trench shoring and excavation, the presence of the adjacent existing buried service pipes should be considered. In addition to the stability of the existing adjacent pipes, which must be maintained without detrimental settlements, the backfill in these trenches and especially the granular bedding surrounding the existing service pipes, manholes, etc. may be a source of water, which, if encountered, must be dealt with.

The earth pressure acting on the sheeting and bracing may be evaluated using the earth pressure diagram given on Drawing No. 15.

Drawing No. 16 indicates zones in which some degree of movement of the ground can be anticipated as a consequence of trench excavation. In this respect, it should also be noted that less ground movements will be experienced outside the excavation if the sides of the excavation are properly supported by tight, braced sheeting than if the sides are unsupported. Ground movements would be further reduced if the bracing were to be pre-stressed.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, fill material, native sand, gravel, silt, silty sand till, sandy gravel, gravelly sand, silty sand and gravel could be classified as Type 3 Soil above the groundwater table and Type 4 soil below the groundwater table. Clayey silt till and sandy silt till could be classified as Type 2 soil above the groundwater table and Type 3 soil below groundwater table.

4.1.1. Use of Trench Box for Open Cut Wall Support

Where permissible under the OSHA, contractors often elect to utilize trench boxes for temporary trench support.

While in many situations the use of trench boxes can result in a high rate of productivity in trenching, it is not without some technical drawbacks. These include increased loss of ground relative to other shoring methods, and reduced ability to compact backfill between the trench wall and trench box. Ground loss, raveling and/or loosening of soils will occur when using a trench box prior to its installation and while moving the box, particularly in pre-existing fill as present at this site.

Granular courses below existing pavements are particularly susceptible to caving and significant undermining can occur. It is important that trenches not be over-excavated to ensure a tight fit between the box and the trench walls. Trench boxes need to be installed expediently. When moving the box, the void space between the outer walls and the trench must be backfilled and compacted. This may require raising the box sequentially prior to sliding it laterally. If this is not done, post-construction settlements will occur along the trench walls.

Where trench boxes are used in the existing roadways, it is prudent to expect pavement structure settlement along both sides of the trench. In such cases, following backfilling of the trench, road reconstruction should include a provision for saw cutting of the asphalt and concrete road base at least 600 mm back from the trench walls, re-compaction of the upper trench backfill and then paving.

4.1.2. Trenching Adjacent to Existing Service Trenches

In areas where the new services impinge on existing utility trenches, unstable trench conditions can occur, particularly where granular backfill, clear stone, or poorly compacted fill of any type are present. In such cases, raveling of the pre-existing fill and high rates of water infiltration through utility bedding can potentially occur which can, in severe cases, put the stability of the adjacent utility in jeopardy. As such, a higher standard of care in shoring is needed where the trench is located closer than 0.75H to an adjacent trench, where H is the depth of the deeper cut. In such instances, the use of trenching boxes is poorly suited since they do not provide adequate intimate lateral support to the sides of the cut and considerable loss of ground can occur prior to insertion of the box. Closed sheeting or other pre-installed shoring measures are more suitable.

4.1.3. Trenching Adjacent to Existing Armourstone Walls

Two armourstone retaining walls are located opposite each other at 33 Wildwood Road. Selected photographs of the walls are presented in Appendix B (see Photos 1 and 2). The maximum heights of the northern and southern retaining walls are approximately 2.5 m to 3.5 m. According to Ref. 1, the proposed sanitary sewer will be constructed at a depth of 3.5 m and an approximate distance of 3.5 m from the northern retaining wall. Existing watermain and storm sewer pipes are also running along the road, at an approximate depth of 2 m. Given the proximity of the retaining walls, the walls may need to be temporarily braced to minimize movement.

The trench walls should be tightly supported using hydraulic trench box. The trench should be advanced in small length, say 6 m to 8 m and advanced only after the previous section is backfilled. The trench can be pre-excavated to a maximum depth of 1 m, or shallower in case of significant caving, for placement of the trench box. Any voids between the back of the panels and the trench wall should be backfilled to avoid unwanted ground movement. The box should then be pushed to the ground as excavation from within the box proceeds. Given the proximity of the existing storm sewer and watermain, backfilling of the trench may become difficult and as such consideration may be given to backfilling with unshrinkable fill (U-Fill). For installation between the two retaining walls, the contractor shall evaluate the logistics of using trench box and potential interference of the excavation with existing services while advancing the trench, since the existing watermain is not parallel to the proposed sanitary sewer and existing storm sewer. Alternatively, the contractor may choose to install the sanitary sewer in this section using trenchless methods.

4.2. BEDDING

The undisturbed native soil as encountered in all boreholes will provide adequate support for proposed sanitary sewer and will allow the use of normal Class B type bedding. The bedding should conform to the current Ontario Provincial Standard specifications and/or standards set by the local municipality. Any loose/soft subgrade material should be sub-excavated and replaced with compacted inorganic granular material.

The minimum bedding thickness should be 150 mm. However, where the subgrade is wet, the minimum bedding thickness should be increased to 250 mm.

In the areas where the pipe is installed within bedrock, a polystyrene layer with a minimum thickness of 50 mm will be required at both sides of the pipe to avoid rock squeezing. The polystyrene layer should extend to at least 0.3 m above the pipe. The rock trench should be wide enough so that the horizontal distance between the pipe side and the cut rock surface is at least 0.3 m at each side.

4.3. BACKFILLING OF OPEN CUT TRENCHES

It is recommended that backfilling of trenches should be completed using approved imported material such as Ontario Provincial Standard Specifications Select Subgrade Material or unshrinkable fill, to reduce the post-construction settlement.

The select inorganic fill materials or native soils free from topsoil, organics, and debris can be used as general construction backfill for the open-cut trenches, provided their moisture contents at the time of construction are at or near optimum.

It is preferable that the native soils be re-used from approximately the position at which they are excavated so that frost response characteristics of the soils after construction remain essentially similar to their surrounding.

Care should be taken to maintain the water content of the soils 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. For fine grained soils, some moisture conditioning may be required if excess pore air and pore water pressures are generated during compaction process. If bulking is noted, delaying the placement of subsequent lifts may be necessary, to allow for the dissipation of such induced excess pressures.

In addition, any boulders or cobbles greater than 150 mm in size should be removed from the trench backfill materials. The native materials are susceptible to wet/freezing temperatures. Care should be taken during construction to protect the site borrow materials from exposure to freezing temperature and wet weather conditions.

Consideration may also be given to backfilling trenches with a well-graded, compacted granular soil such as Granular 'B' material. The use of such material, if thoroughly compacted, would reduce the post construction settlements to a negligible amount and may also expedite the compaction process. In this instance, however, frost response characteristics of non-frost susceptible granular fill and the frost susceptible indigenous soils would be different giving rise to differential frost heave or movement. In this case it would be prudent to use as backfill the on-site excavated, naturally occurring soils to match the existing conditions within the frost zone (i.e. within 1.5 m depth) as well as to provide a frost taper zone (i.e. to provide a zone of taper to prevent a sudden change in frost heave characteristics to reduce the effects of frost heave).

In any case the degree of compaction of the trench backfill should be at least 98% of the material's Standard Proctor Maximum Dry Density (SPMDD), at a placement moisture content within ±2% of the optimum. The granular pavement sub-base and base materials should be compacted to at least 100% of their respective SPMDD.

4.4. OVERVIEW OF TRENCHLESS TECHNOLOGY METHODS

As noted above, the eastern extent of the proposed sanitary sewer, from SANMH16A to SANMH20A, is expected to be constructed using trenchless techniques. The crossings are as follows:

- Two existing culvert crossings between SANMH16A and SANMH17A as well as between SANMH17A and SANMH18A (between the chainage 0+975 and 1+060; see Ref. 1).

- Credit River crossing between SANMH19A and SANMH20A (between the chainage 1+065 and 1+165; see Ref. 1).

For reference, an overview of conventional trenchless methods is provided herein. Taking into consideration the soil and groundwater conditions, the diameter of the product pipe, length of the trenchless installation and depth of the installation, jack and bore, horizontal directional drilling and micro-tunneling were considered. The advantages, disadvantages of the options are provided in the following sections. Nevertheless, early engagement of a specialist trenchless contractor is recommended, to evaluate site constraints, stratigraphy, and groundwater conditions, and undertake a cost comparison of different methods to select the best method of construction. The contractor may consider using different techniques for each crossing.

4.4.1 Jack and Bore (Auger Boring)

Jack and bore is a process of simultaneously jacking casing while removing the spoil material by means of an auger. A rotating cutting head is attached to the leading edge of the auger string. The spoil is transported back by the rotation of auger flights within the steel pipe casing being placed.

The leading edge of the pipe is usually open and its shape allows a small overcut (to reduce friction between the carrier pipe and soil and improve load conditions of the pipe) to direct the soil into the pipe interior instead of compacting it outside the pipe.

Jack and bore is not steerable and therefore once the bore has begun there is little control of the line and grade of the installation. Installation accuracy (vertically and horizontally) is usually about ±1 per cent of the length of the bore, but subsurface obstructions or improperly aligned pipes may result in significant deviations from the desired line and grade.

The recommended dimensions for jacking and receiving pits are a length of 8 to 12 m and a width of 3 to 4 m, with sufficient room for material storage. The drive shaft is a critical part of the process that must be designed and constructed properly. The foundation must be designed to support the jacking process.

Adequate depth of cover above the pipe can minimize ground displacement. A minimum cover of jack and bore installations shall be 1.5 m on secondary roads and 3 m on primary and freeway roadways. Given the proposed sanitary sewer design, this condition will be met.

Jack and bore should not be used below the groundwater table, in running sands, or in soils with large boulders. The soil and groundwater condition encountered at the current site limits the application of jack and bore method for the proposed sewer construction. This should however be further discussed with specialist trenchless contractor.

4.4.2 Horizontal Directional Drilling

Horizontal Directional Drilling (HDD) method uses a small rotating and steerable drill bit that is launched from the surface at a shallow angle and used to drill a pilot hole supported with drilling fluid. Once the pilot bore is complete, the drill head is replaced with a back-reamer or expander which enlarges the drill hole. Once the desired size is reached, the product pipe is attached to the reaming head and pulled through the bore.

During the HDD installation through the overburden, deviation of the pilot hole alignment may occur at the interface of till and sand and gravel/sandy silt layers as well as the interface of bedrock and overburden (see BH110 and BH111). The contractor should select equipment such that the pilot hole and reamers can accommodate and make their way through such materials.

The potential for hydraulic fracturing (frac-out) to occur during directional drilling is influenced by several factors, including the depth of cover below ground surface, the ground conditions (including fractured or high permeability materials), the hydrostatic head acting in the drill hole and the dynamic drilling fluid pressures that occur during the drilling process. For preliminary design purposes and in accordance with Latorre et al (2002), minimum cover depths of 3.0 m should be considered for the 300 mm diameter casings. If HDD method is adopted, detailed hydraulic fracture analysis/evaluation should be completed along the entire bore path.

Directional drilling is best suited for clays. Soft to hard clays are the preferred soils for HDD applications, although its use in cohesionless fine sands and silts is also acceptable (Iseley and Gokhale, 1997). Soils containing more than 50% gravel or loose soils are generally unsuitable (Hair, 1994). Directional boring should not be conducted in soils that contain material with particle diameters greater than 3 in, since these particles are too large to be suspended in the drilling fluid (Gelinas et al., 2010). HDD can be used successfully underwater, in saturated soils, under permafrost, and in a soil that is likely to erode (Hashash, 2011).

Given the above, the existing gravel to sandy gravel layer observed in BHs 110 to 113 will pose a risk to successful installation of the sanitary pipe using HDD method. This should be discussed with specialized trenchless contractor. The contractor may choose to advance the pipe from greater depths to avoid encountering the gravel layer. It should be noted that drilling of the gravel layer during borehole investigation was difficult due to encountering relatively large cobbles.

4.4.3 Micro-tunneling

Micro-tunneling is a method of installing pipes behind a steerable remote-controlled shield that is pressurized with a bentonitic fluid to minimize ground losses. The process is essentially remote-controlled pipe jacking where all operations are controlled from the surface, cuttings are removed by the circulating slurry and personnel entry to the bore is not required. Micro-tunnelling is a precise method of tunneling and there is relatively negligible settlement with this method, if the face pressure and cutting tools are appropriate for the ground and maintained over the length of the drive.

Micro-tunneling is considered feasible at this site. It should be noted that slurry micro-tunneling with high viscous and pressurized bentonite slurry is recommended to counterbalance the earth and water pressures acting at the tunnel face. The slurry is circulated back through the tunnel to transport cuttings to a settling tank. Specialist advice on machine selection should be sought and recommendations regarding the machine design for the given ground conditions should be supported by the manufacturer. In particular, the cutting tools on the face should be selected to cut cobbles and boulders (e.g., disc cutters) as well as excavate soil materials. Internal “crushing chambers” should not be solely relied upon for managing large stones.

In the presence of groundwater, finely abraded particles may mix with the lubricant and set around the pipe. This situation can substantially increase the friction and result in increasingly high jacking forces. To counteract these problems, the slurries should be designed properly to keep the excavated particles in suspension; alternatively foam fillers can be used to prevent abraded materials from filling the gaps. Spikes in jacking loads can result in surging of the pipe string and spikes in the thrust loads of the MTBM cutting tools. To prevent such spikes, the slurry micro-tunnel boring machine should include a telescopic tail to isolate pipe string advancement.

Microtunnelling requires that adequate work space be provided for the jacking equipment. The size of the pit is dependent on the dimensions of the drive shaft. The size typically ranges from 6 m to 12 m wide and 15 m to 30 m long. Circular pits are also a viable option.

A minimum cover of 1.5 m or 3 times the pipe diameter, whichever bigger, is recommended to minimize displacement of the surface.

4.4.4. Receiving/Jacking Shafts

Microtunnelling and jack and bore method will require construction of jacking and receiving shafts, as well as thrust block. Potential location of these shafts on the two sides of the Credit River are shown on Ref. 1.

According to BH112 and 113, the soil stratigraphy at the two sides of the river is expected to be comprised of 2.3 to 3 m of fill material, underlain by a generally cohesionless alluvial deposit comprising sand, gravel, gravelly sand, with cobbles. Layers of sandy silt till and silty sand till where also encountered in BH113. Groundwater elevation was observed to be at 226.8 m in both boreholes, which correspond to depths of 2.2 m and 3.1 m in BH112 and 113, respectively.

Given the groundwater condition and the coarse-grained gravelly nature of the soil, and assuming that the pits will likely be advanced to below the groundwater table, significant dewatering through point wells is expected to be required. Groundwater should be lowered to at least 1 m below the excavation base, otherwise it will result in base instability. The amount of dewatering could however be reduced by means of installing sheet piles or caisson walls. A specialist dewatering contractor should be consulted to evaluate the dewatering requirements. Permit to Take Water will likely be required. This should be addressed by the Hydrogeologist.

Excavation of the entry and exit shafts must be carried out in accordance with the Occupational Health and Safety Act for construction activities. In accordance with OHSA, existing fill material and native cohesionless soil deposit can be classified as Type 3 Soil above the groundwater table and Type 4 soil below the groundwater table. The excavation support system should be designed to resist the lateral earth pressures of the soils. It is common practice for a specialist contractor to design and install the excavation support system. The design of the shoring system should be reviewed by SIRATI from a geotechnical perspective.

The temporary excavation support system should be designed and constructed in accordance with OPSS 539 (Construction Specification for Temporary Protection Systems).

4.5. MANHOLES

4.5.1 General

It is anticipated that as part of the proposed sanitary sewer installation, manholes will be constructed. The following should be adhered to:

- Manholes construction should be carried out according to Ontario Provincial Standard Design OPSD 705.010 to 705.020,
- The depth of the manhole excavation should extend to a minimum of 300 mm below the base of the proposed manholes,
- The base of the excavation should be tested and inspected by a geotechnical engineer to ensure that the native soil can support the manhole load and the soil is free from any other deleterious material,

- Granular A materials should be used below the bottom of the manholes. The materials should be compacted to 98% of the materials SPMDD,

4.5.2 Foundation

The proposed manholes are expected to be founded on undisturbed native compact to very dense cohesionless soil or glacial till deposits, depending on the location and the depth of manholes. Manhole foundations founded on undisturbed native soils can be designed for bearing capacity value of 250 kPa at SLS and factored ULS bearing resistance of 375 kPa. The total settlement for such foundations is expected to be less than 25 mm. Structures founded in shale bedrock, can be designed for an allowable (SLS) bearing resistance of 3000 kPa and factored ULS bearing resistance of 4500 kPa. For the design bearing pressure, total and differential settlements are estimated to be less than 15mm. A lean mix concrete mud slab must be placed on the prepared shale base after inspection and approval by the geotechnical engineer to minimize slaking deterioration.

In order to minimize the development of shrinkage cracks and “rock squeeze” in the base slab of the manhole, it is proposed to incorporate a layer of granular material between the underside of the slab and the mud slab of lean concrete placed on the surface of the bedrock. In this case, we recommend that the thickness of the granular layer should not be more than 200mm. It should consist of Granular “A” material (OPSS 1010) compacted to 100% of standard Proctor maximum dry density (SPMDD).

The manholes should be designed as water-tight structures. The floor-to-wall slab intersections may be fitted with conventional water stops. The floor slabs and walls may be waterproofed using a membrane or other means.

4.5.3 Lateral Earth Pressure

Lateral Earth Pressure in Overburden Soils:

The earth pressure distribution on manholes backfilled with granular “B” fill can be taken as hydrostatic, i.e. which is increasing uniformly with depth according to the formula:

$$P_h = K_o (\gamma.h + q)$$

where

P_h = horizontal pressure at depth h (kN/m²)

γ = unit weight of soil, taken as 21 kN/m^3

h = depth below ground surface (m)

q = surcharge load at ground surface (kPa)

K_o = coefficient of lateral earth pressure at rest, taken as 0.5

Passive earth pressure coefficient for calculation of manhole thrust resistance can be taken as 3, which will govern the design.

Below the water table, the submerged unit weight of the soil should be used, and the full hydrostatic water pressure should be added.

If the ground surface is not horizontal, the uneven portion can be treated as an equivalent surcharge load.

Lateral Earth Pressure in Bedrock:

Structures which extend below the surface of the bedrock and the walls of which are poured in direct contact with the bedrock will be subject to “rock squeeze”. Therefore, a layer of compressible material must be placed between the structure and the rock. This compressible layer could be either a synthetic material (e.g. EPS GeoSpan Compressible Fill) or foamed “cellular grout”. Properties and proposed thicknesses of the compressible material should be submitted to a qualified engineer to evaluate its stiffness and assess its suitability to accommodate the strain without transferring unacceptably high loads onto the structure. Certain rigid polystyrene insulation products are considered to be excessively stiff for this application.

Provided that rock squeeze is allowed to dissipate by delaying construction of permanent concrete walls or by applying a compressible void former, the lateral earth pressures acting on this bedrock portion of the manhole below the overburden can be assumed to be a uniform pressure equal to the maximum overburden lateral earth pressure calculated at the overburden to rock interface, plus the hydrostatic forces even if lean concrete is used as backfill material.

4.6. ROAD RECONSTRUCTION

It is anticipated that re-construction and/or rehabilitation of the excavated areas of the road will be required, following open cut trench excavations.

The excavated area along the proposed sanitary sewer should be backfilled to an elevation approximately 800 mm below the existing pavement surface following installation of the sanitary sewer. Flexible pavement will be considered in the reconstruction of the road. The Flexible pavement structure generally consists of a prepared roadbed, underlying layers of subbase, base, and surface courses. The following are details of the flexible pavement construction.

4.6.1 Preparing Subgrade

Backfilling will be required following installation of the sanitary sewer. Backfilling should extend to a depth of 800 mm below the existing grade. The surface of this layer (after backfilling the excavated area) should be compacted to 100% of the materials SPMDD, and proof-rolled with a loaded tandem truck in the presence of a geotechnical engineer.

The proof rolling should be performed with a loaded tandem axle dump truck, rubber-tired loader with a minimum static weight of 10 tons. The proof rolling should consist of a minimum of six passes per unit area to ensure that any areas exhibiting more than 20 mm deflection, or any soft, wet, or deleterious materials identified at the time of proof rolling should have the top 300 mm removed and replaced with approved drier materials.

Imported fill materials for use at the site shall meet all applicable municipal, provincial, and federal guidelines and regulation associated with environmental characterization of the materials. The materials should also be examined and approved by qualified geotechnical engineer.

4.6.2 Pavement Structure

The production, gradation, and placement of granular materials (base and sub-base) shall be in accordance with the applicable OPSS requirements. The granular base and subbase should be placed in layers not exceeding 200 mm (uncompacted thickness) and should be compacted to at least 100% of their respective SPMDD.

The surface of the pavement should be graded to direct runoff water away from the roadway. The placing, spreading and rolling of the asphalt should be in accordance with OPS Specifications or as required by the client.

The recommended thickness of the asphalt, base and subbase materials for the road are given in **Table 3.**

For reconstruction of the mainline roadway, the granular layers of the proposed pavement section should match the top elevation of the existing granular section. The HMA thickness obtained from the boreholes in the parking areas varies and the binder layers may need to be adjusted to match existing.

Table 3: pavement structure Details for roads

Description	Materials	Wildwood Rd. and Main St.	Meagan Dr. and Oak Ridge Dr.
Asphalt Surface Course (HL3)	Hot Mix Asphalt	50 mm	40 mm
Asphalt Base Course (HL8)	Hot Mis Asphalt	80 mm *	50 mm *

Base Course	19 mm Crushed Limestone or Granular ‘A’	250 mm	150 mm
Sub-base Course	50 mm Crushed Limestone or Granular ‘B’ Type II	400 mm	300 mm

* The thickness of HL8 should be adjusted to match the existing pavement.

The flexible pavement tie-ins at mainline roadways should be milled 50 mm (or depth off the surface course) to 500 mm back from the construction joint. The milled area shall be brush cleaned for coarse materials, air blast cleaned for fine materials and tack coated for adhesion prior to placement of the new surface course. The granular base of the new pavement section should be sloped at 2% prior to placing the final lift(s) of asphalt. the new surface asphalt course shall be extended from the mainline roadway.

Frequent field density tests should be carried out on both the asphalt and granular base materials to ensure that the required degree of compaction is achieved.

4.7. SUITABILITY OF SITE MATERIAL FOR REUSE

The base/subbase materials below the pavement and the fill and native soils encountered below the pavement structure at the location of the boreholes are considered suitable as backfilling materials provided that the moisture content is within the optimum content of 2% and these activities are not undertaken in wet freezing conditions below the pavement structure. Deleterious materials, if any, should be removed prior to reuse on site.

4.8. WINTER CONSTRUCTION

When construction is undertaken during periods of inclement weather, or when freeze-thaw affects are a concern, the backfilling procedures should consider the following comments:

- The preferred backfill material during periods of inclement weather is OPSS Granular B Type1.
- Areas of intended backfill should be clearly identified in the field prior to commencing the work.
- Fill placement should be inspected by qualified field personnel under the supervision of a geotechnical engineer.
- Imported materials that contain ice, snow, or any frozen material should not be accepted for use.
- Overnight frost penetration into the existing subgrade or the fill layer should be prevented by using insulation blankets. Alternatively, the frozen fill can be removed prior to placing subsequent lifts. Approaches such as breaking the frost in-situ is not considered acceptable.

- During periods of cold, where ambient temperature is -5° or less, placement of engineered fill should be protected from freezing.

4.9 INSPECTION AND TESTING

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 guidelines in this report have been adequately interpreted. During construction, sufficient trench excavation inspections and in-situ material 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.

All subgrade areas should be inspected by experienced geotechnical personnel prior to backfilling. The trench backfill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction point of view.

4.10 SEISMIC SITE CLASSIFICATION

Based on the borehole information and according to Table 4.1.8.4.A of OBC 2012, the subject site for the proposed manhole structures can be classified as ‘Class C’ for seismic site response.

5. SLOPE STABILITY ANALYSIS

There is a relatively steep slope, north of the Road, between 27 and 33 Wildwood Road.

The slope stability assessment in this report is based on subsurface conditions in BH107 and BH109, the observations during our site visits, and the slope profiles derived from the topographic map provided by the Client.

Stability analyses of the slopes are carried out using the program SLIDE 8 with the Bishop’s Method. The slope conditions and the results of stability analyses are presented in the following sections.

5.1 SITE AND SLOPE CONDITIONS

A site visit was made by a senior geotechnical engineer of SIRATI on January 28, 2021. The existing slope conditions, including general topography of the slopes, vegetation cover, and any evidence of slope failure and erosion were examined during the site visit. Photographs of the site taken during our site visits are shown in photos 3 to 6 of **Appendix B** of this report.

The topography of the slope includes an armourstone retaining wall and a ground steepness of 3H:1V and flatter. A cross section of the slope was prepared from the topographic survey provided by the

Client, as presented in **Appendix C**. Based on our observations during the site visit, the slope conditions are summarized as follows:

- The elevation difference from the highest elevation to the creek is approximately 3 m.
- The slope surface within the property boundaries is covered with trees. The trees were observed to be straight with no sign of slope movement.
- No sign of movement was observed at the top of the slope, on the road.
- There was no water seepage observed from the slope surface during our site visit.

5.2 SOIL PARAMETERS AND GROUNDWATER

Based on stratigraphy obtained from BH107 and BH109, soil parameters used in the slope stability analyses are given in **Table 4**.

Table 4: Soil Parameters for Slope Stability Analyses

Soil Type	**Soil**	**Drained Strength Parameters**	
(See Section 3 for soil description)	**Density** **(kN/m^3)**	**c'** **(kPa)**	**φ'** **(degree)**
Fill Material	20.0	0	29.0
Dense Native Deposits	21	0	34.0

5.3 STABILITY ANALYSIS OF EXISTING SLOPE

A slope profile at Section A-A’ is presented in **Appendix C** of this report. Slope stability analysis of the existing slope was carried out using the soil conditions listed in **Table 4** and the results are presented in **Appendix D**. All failure slopes with factor of safety of less than 1.5 are presented in the Appendix.

5.4 LONG-TERM STABLE TOP OF SLOPE

Given the results of the slope stability analysis presented in **Appendix D,** and the fact that the proposed sanitary sewer will be constructed at an approximate depth of 3.5 m, the pipe will be behind the long-term stable top of slope with a factor of safety of 1.5, and hence not affected by potential future slope failures.

6. GENERAL COMMENTS ON REPORT

Sirati & Partners Consultants Limited (SIRATI) should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, SIRATI will assume no responsibility for interpretation of the recommendations in the report.

The construction works as part of proposed installation may cause ground movement and/or direct transmission of ground vibrations to adjoining structures. Considerations should be taken for preconstruction survey of the existing structures to be carried out, along with movement monitoring plan.

The comments given in this report are intended only for the guidance of design engineers. 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.

The limitation conditions presented in **Appendix E** form an integral part of the report and they must be considered in conjunction with this report.

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

Yours truly,
SIRATI & PARTNERS CONSULTANTS LIMITED

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Drawings/Enclosures

SIRATI & PARTNERS
Geotechnical Hydrogeological & Environmental Solutions

12700- Keele Street
King City, ON. L7B 1H5
Phone# 905 833 1582, Fax# 905 833 5360

North:

Legend:

⊕ Borehole Location

⊚ Monitoring Well Location

Project Title:

Geotechnical Investigation for Proposed Sanitary Sewer Construction

Site Location:

From Meagan Drive to Oak Ridge Drive to Wildwood Road to Main Street, Halton Hills

Figure Title:

Proposed Boreholes Location for Proposed Sanitary Sewer and Trenchless Crossing

Scale: As Shown

Project Number: SP20-474-10

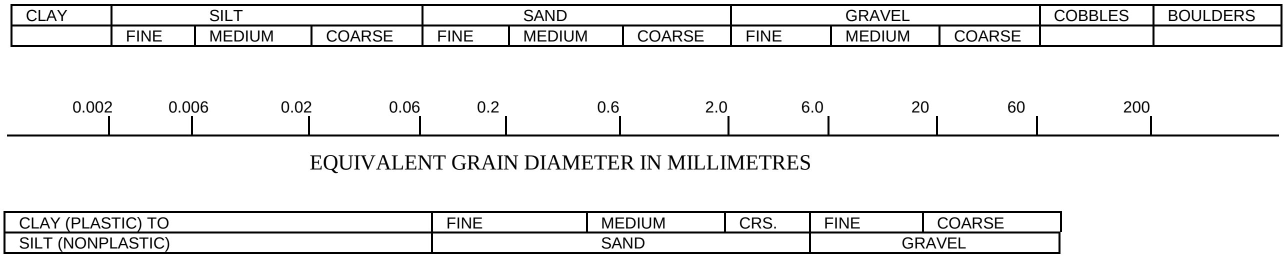
Date: November 2020

Figure Number: 1

Enclosure 1A: Notes on Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Society for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by Sirati & Partners Consultants Limited also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

ISSMFE SOIL CLASSIFICATION



UNIFIED SOIL CLASSIFICATION

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

SIRATI & PARTNERS

LOG OF BOREHOLE BH/MW 101

1 OF 1

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4834950.138 E 585710.804

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Jan-06-2021

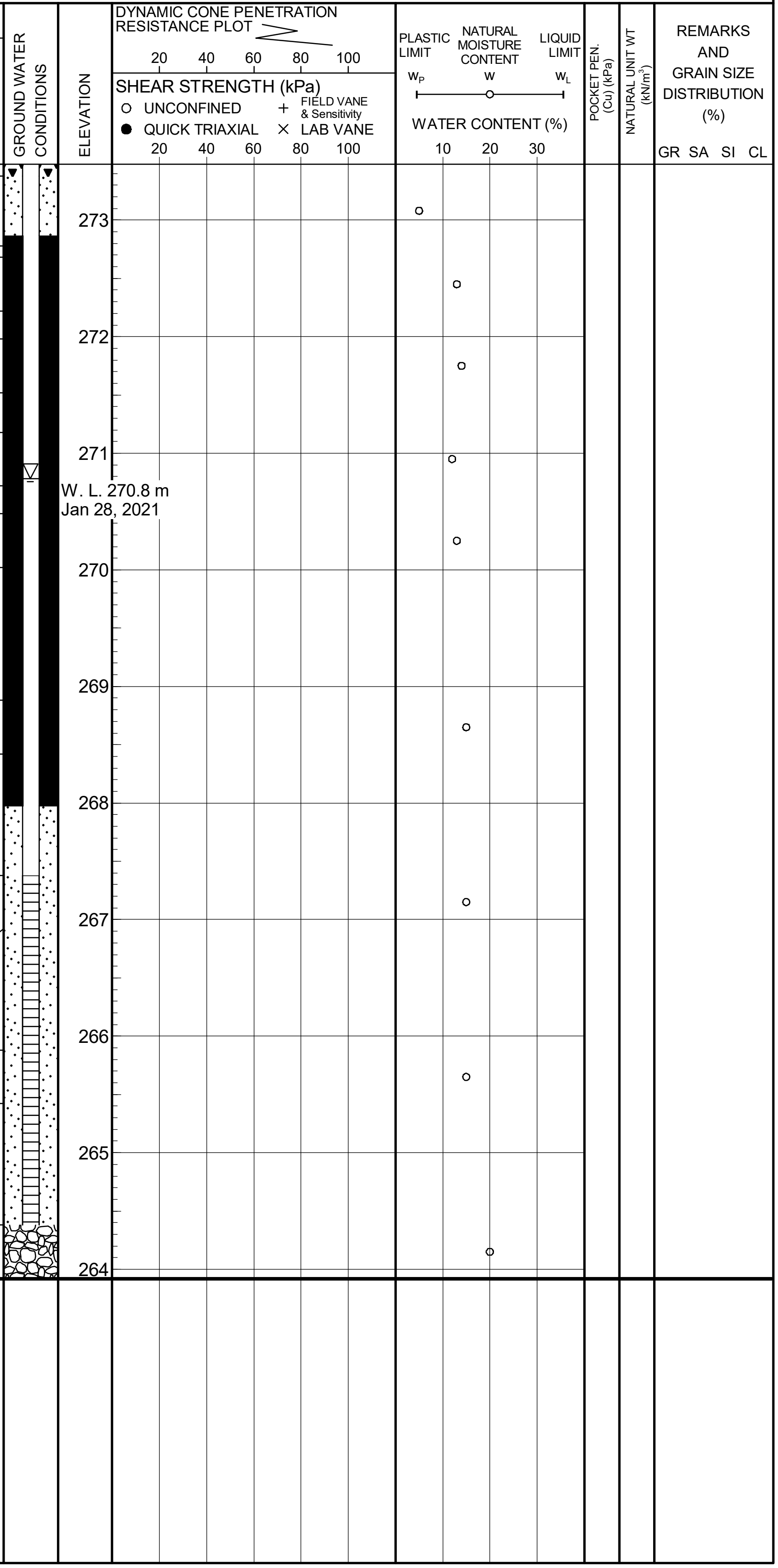
REF. NO.: SP20-747-00
ENCL NO.: 2

SOIL PROFILE		SAMPLES		
(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS / 0.3 m
273.5				
273.4 0.0 0.1	**ASPHALT:** 100 mm thick		AUGER	
273.1 0.4	**GRANULAR FILL:** sand and gravel, 300 mm thick	1	SS	20
	SANDY SILT TILL: trace gravel, trace clay, brown, moist, oxidation stains, compact	2	SS	16
272.0 1.5	**CLAYEY SILT TILL:** trace gravel, trace sand, brown, moist, stiff to very stiff	3	SS	15
271.2 2.3	**SANDY SILT TILL:** trace gravel, trace to some clay, brown, moist, compact to dense	4	SS	12
		5	SS	16
	- trace clay, grey at 4.6 m	6	SS	32
267.4 6.1	**SILT:** trace clay, grey, moist, very dense	7	SS	50/ 50 mm
		8	SS	51
	- wet at 9.1 m	9	SS	57
263.9 9.6	**END OF BOREHOLE:**			

Note:

1. Monitoring well was installed in the borehole from 6.1 m to 9.1 m bgs.
2. Groundwater level observations:

Date	Depth (mbgs)
2021-01-28	2.7



GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

SPCL SOIL LOG /DRAFT SP20-747-00.GPJ SPCL.GDT 21-3-1

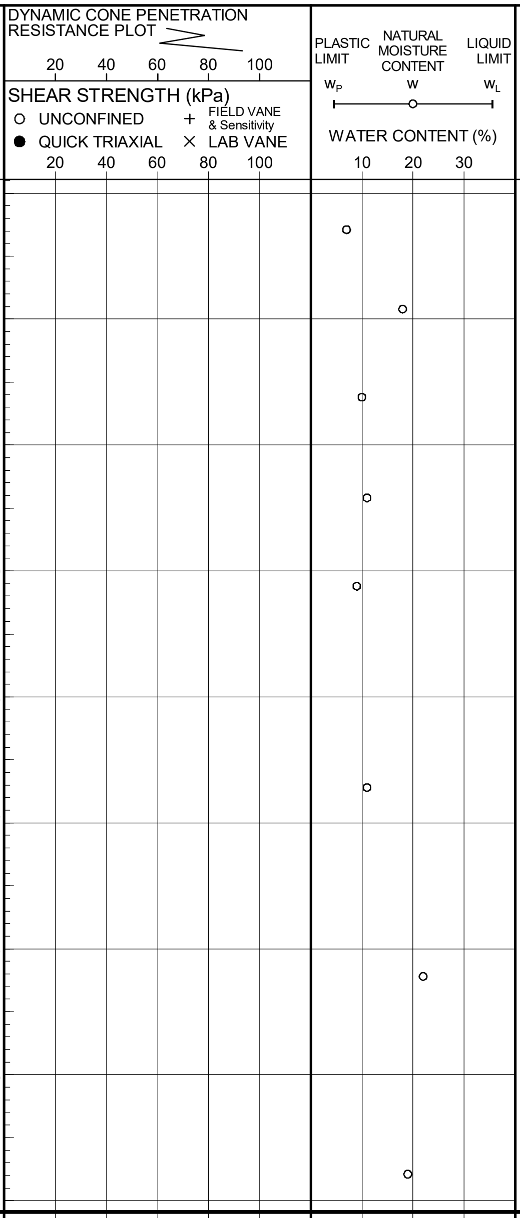
LOG OF BOREHOLE BH 102

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4834937.878 E 585840.738

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Jan-06-2021

REF. NO.: SP20-747-00
ENCL NO.: 3

(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m
272.1				
272.0 0.1	**ASPHALT:** 100 mm thick		AUGER	
271.8 0.4	**GRANULAR FILL:** sand and gravel, 260 mm thick			
	CLAYEY SILT TILL: trace sand, trace gravel, brown, moist, stiff	1	SS	18
		2	SS	9
270.6 1.5	**SAND:** trace gravel, brown, very moist, compact	3	SS	27
269.8 2.3	**CLAYEY SILT TILL:** trace sand, trace gravel, brown, moist, hard	4	SS	37
269.1 3.0	**SANDY SILT TILL:** trace gravel, brown, moist, dense	5	SS	39
267.5 4.6	**SILTY SAND TILL:** trace gravel, trace sand, brown, moist, very dense	6	SS	66
266.0 6.1	**SILT:** trace sand, brown, wet, dense	7	SS	39
	- trace clay, greyish brown, moist at 7.6 m	8	SS	47
263.9 8.2	**END OF BOREHOLE:**			



GROUND WATER CONDITIONS | POCKET PEN. (Cu) (kPa) | NATURAL UNIT WT (kN/m^3) | REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
+ 3, × 3: Numbers refer to Sensitivity
○ ⁸=3% Strain at Failure

SIRATI & PARTNERS

LOG OF BOREHOLE BH/MW 103

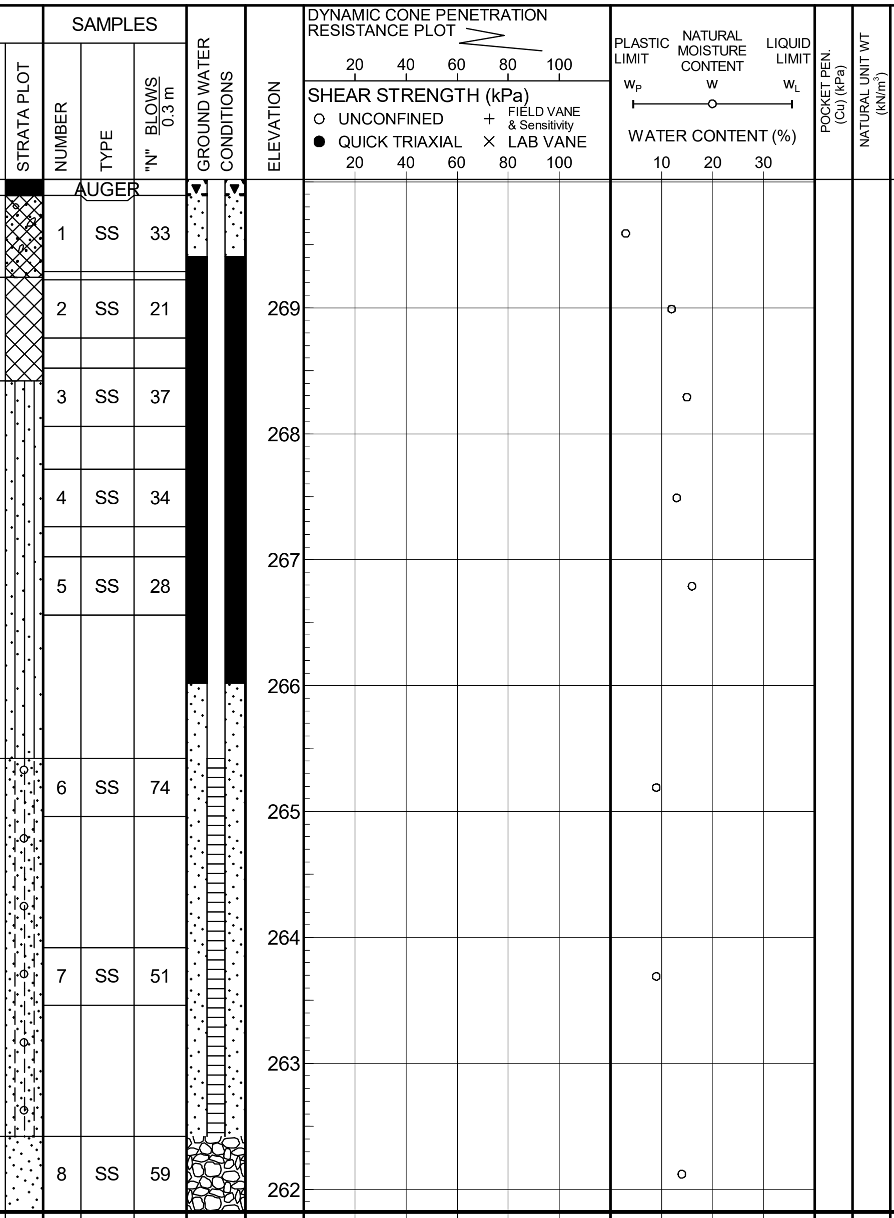
1 OF 1

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4834901.04 E 585953.251

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Dec-21-2020

REF. NO.: SP20-747-00
ENCL NO.: 4

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
270.0 0.0	**ASPHALT:** 130 mm thick		AUGER					
269.9 0.1	**GRANULAR FILL:** sand and gravel, 650 mm thick	1	SS	33				
269.2 0.8	**FILL:** sandy silt, trace to some clay, trace gravel, brown, moist, compact	2	SS	21				
268.4 1.6	**SILT AND SAND:** trace clay, light brown, moist, compact to dense	3	SS	37				
		4	SS	34				
		5	SS	28		44	52	4
					- auger grinding at 3.81 m			
265.4 4.6	**SILTY SAND TILL:** trace gravel, reddish brown, moist, very dense, with cobbles/boulders	6	SS	74				
	- trace cobbles at 6.1 m	7	SS	51				
262.4 7.6	**SAND:** trace silt, very moist to wet, very dense	8	SS	59				
261.8 8.2	**END OF BOREHOLE:**							



Note:

1. Borehole was open and dry upon completion of drilling.
2. Monitoring well was installed in the borehole from 4.6 m to 7.6 m bgs.
3. Groundwater level observations:

Date	Depth (mbgs)
2021-01-28	Dry

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
+ 3, × 3: Numbers refer to Sensitivity
○ 8=3% Strain at Failure

LOG OF BOREHOLE BH 104

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4834979.055 E 586125.347

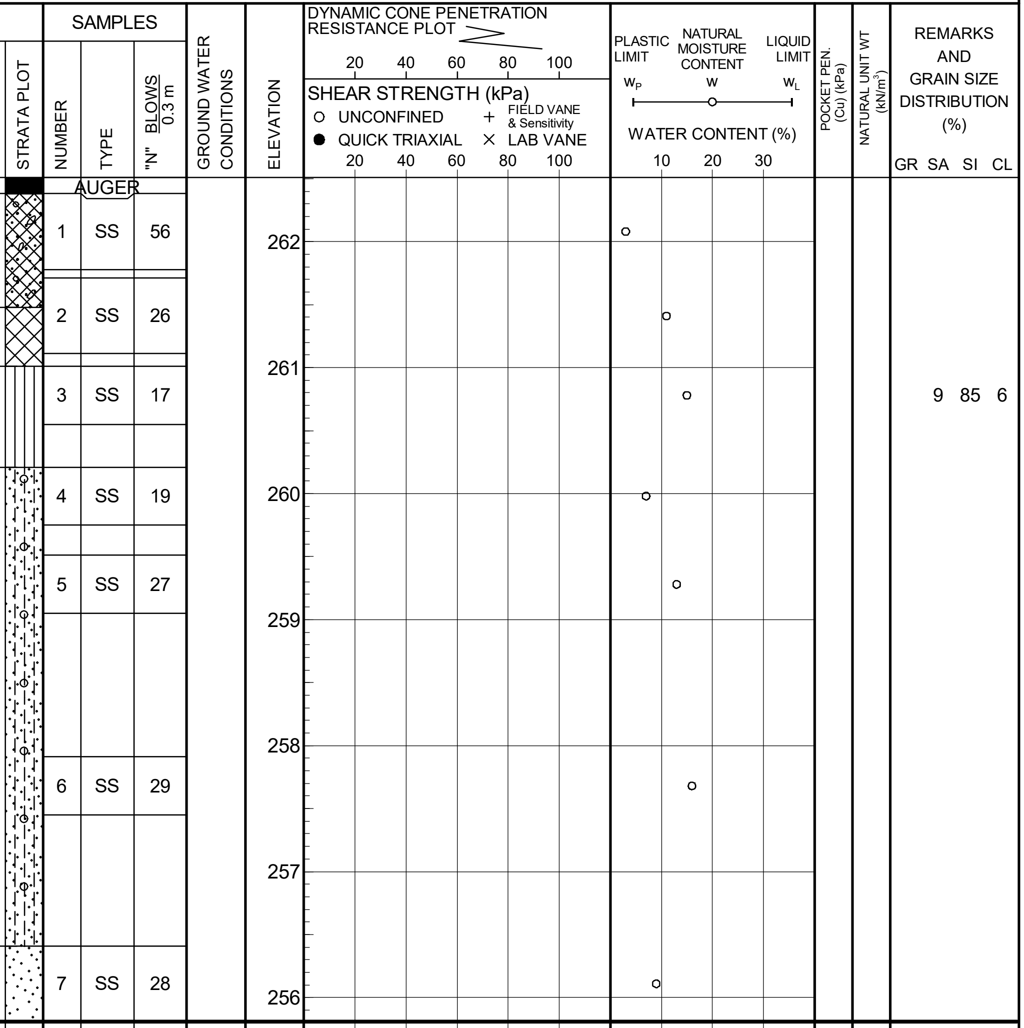
DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Dec-21-2020

REF. NO.: SP20-747-00
ENCL NO.: 5

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m		GR SA SI CL
262.5						
262.4 0.0 0.1	**ASPHALT:** 130 mm thick **GRANULAR FILL:** sand and gravel, 900 mm thick	 1	AUGER SS	 56		
261.5 1.0	**FILL:** sandy silt, reddish brown, moist, compact	2	SS	26		
261.0 1.5	**SILT:** trace sand, trace clay, light brown, moist, compact	3	SS	17		9 85 6
260.2 2.3	**SILTY SAND TILL :** trace gravel, trace cobbles, brown, very moist to wet, compact	4 5	SS SS	19 27		
	- trace cobbles at 4.6 m	6	SS	29		
256.4 6.1	**SAND:** trace silt, trace gravel, brown, wet, compact	7	SS	28		
255.8 6.7	**END OF BOREHOLE:**					

Note:

1. Borehole was open upon completion of drilling.
2. Water was encountered at 6.4 mbgs upon completion of drilling.



GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

SIRATI & PARTNERS

LOG OF BOREHOLE BH/MW 105

1 OF 1

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4835010.915 E 586221.861

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Dec-21-2020

REF. NO.: SP20-747-00
ENCL NO.: 6

(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
258.1						
258.0 0.1	**ASPHALT:** 130 mm thick		AUGER		258	
	GRANULAR FILL: sand and gravel, 790 mm thick	1	SS	54		
257.2 0.9	**SILT AND SAND TILL:** some gravel, trace clay, reddish brown, moist, compact to dense - trace to some cobbles below 1.5 m	2	SS	18	257	
		3	SS	35		14 36 41 9
		4	SS	33		
		5	SS	33	255	
254.3 3.8	**SANDY SILT TILL:** trace gravel, trace clay, grey, moist, compact	6	SS	17	254	
252.9		7	SS	22	253	
5.2	**END OF BOREHOLE:**					

W. L. 256.1 m
Jan 28, 2021

Note:

1. Borehole was open and dry upon completion of drilling.
2. Monitoring well was installed in the borehole from 3.1 m to 4.6 m bgs.
3. Groundwater level observations:

Date	Depth (mbgs)
2021-01-28	2.0

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 10 20 30
POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

SIRATI & PARTNERS

LOG OF BOREHOLE BH 106

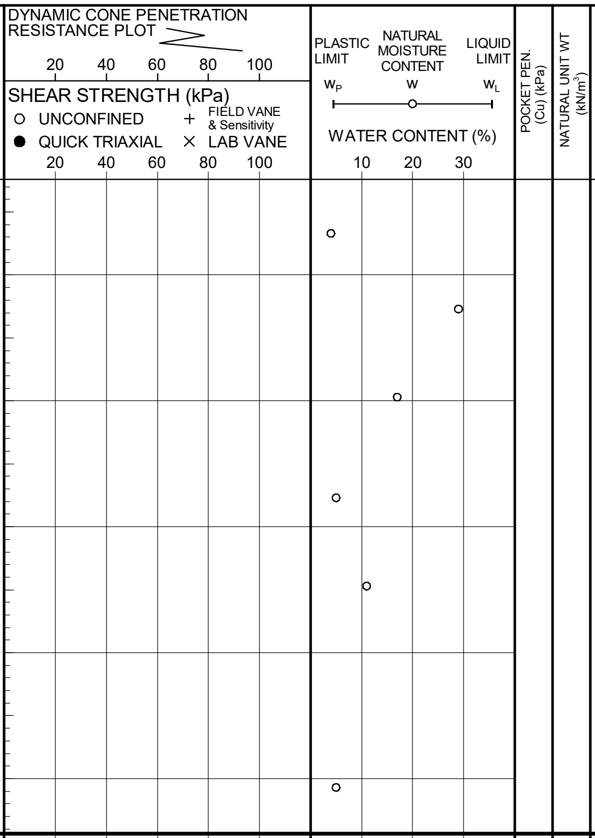
1 OF 1

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4835071.839 E 586346.086

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Dec-18-2020

REF. NO.: SP20-747-00
ENCL NO.: 7

SOIL PROFILE (m) ELEV / DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
250.8							
0.0	**ASPHALT:** 130 mm thick		AUGER				
250.6 / 0.1	**GRANULAR FILL:** sand and gravel, 460 mm thick	1	SS	57			
250.2 / 0.6	**CLAYEY SILT TILL:** trace sand, brown, moist, stiff	2	SS	10		250	
249.3 / 1.5	**SILT:** some sand, trace clay, trace gravel, brown, moist, compact	3	SS	10		249	4 15 71 10
248.4 / 2.3	**SAND:** tarce to some silt, trace gravel, brown, moist, dense	4	SS	38		248	
247.8 / 3.0	**SANDY SILT TILL:** trace to some gravel, brown, moist, compact	5	SS	23		247	
246.2 / 4.6	**SAND:** tarce to some silt, trace gravel, trace cobbles, brown, moist, dense	6	SS	66		246	
245.6 / 5.2	**END OF BOREHOLE:** Note: 1. Borehole was open and dry upon completion of drilling.						



GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity
○ ^{8}=3% Strain at Failure

SPCL SOIL LOG /DRAFT SP20-747-00.GPJ SPCL.GDT 21-3-1

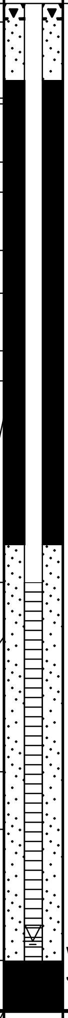
LOG OF BOREHOLE BH/MW 107

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4835144.23 E 586387.58

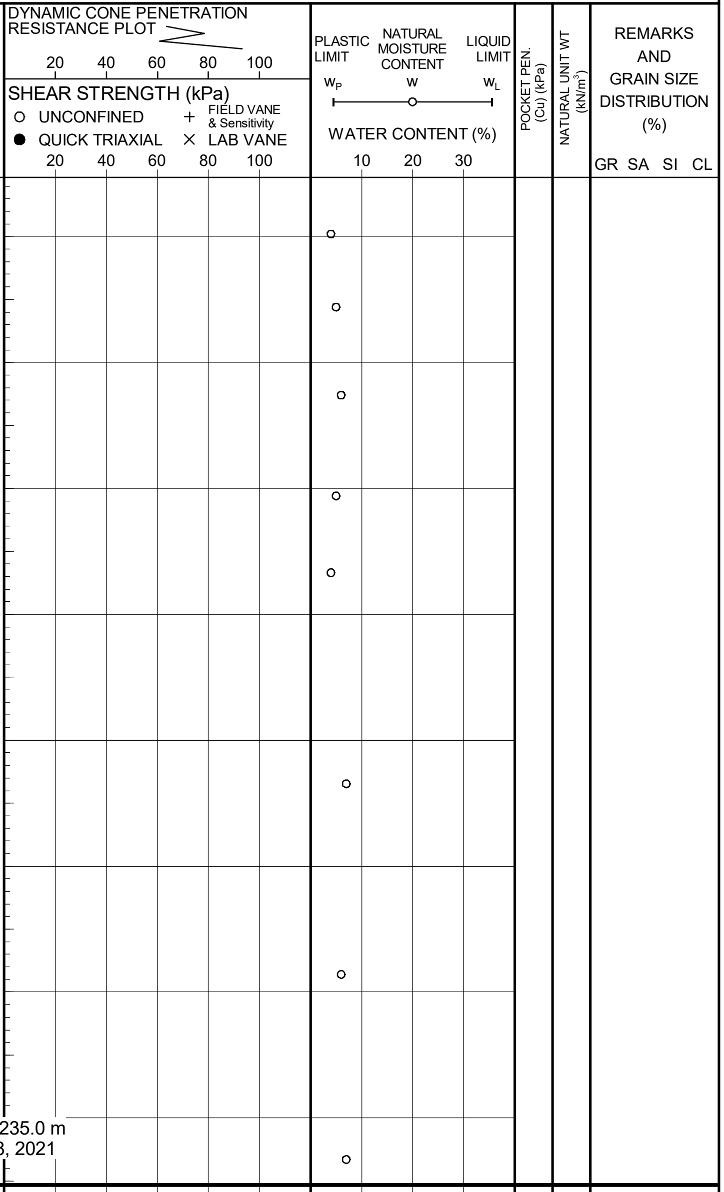
DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Jan-06-2021

REF. NO.: SP20-747-00
ENCL NO.: 8

SOIL PROFILE		SAMPLES			ELEVATION
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	
242.5					
242.3 0.0	**ASPHALT:** 150 mm thick	AUGER			
0.2	**GRANULAR FILL:** sand and gravel, 450 mm thick	1	SS	55	242
241.9 0.6	**PROBABLE FILL:** sand, trace to some gravel, brown, moist, compact to dense	2	SS	12	
	- trace cobbles below 1.5 m	3	SS	28	241
		4	SS	41	240
239.5 3.0	**SANDY GRAVEL:** grey, dry to moist, very dense	5	SS	50/ 130 mm	239
					238
	- trace cobbles at 4.6 m	6	SS	77/ 280 mm	237
236.4 6.1	**GRAVELLY SAND:** brown, moist, very dense	7	SS	60	236
234.9 7.6	**SAND:** trace gravel, brown, very moist, very dense	8	SS	82/ 260 mm	235
234.5 8.0	**END OF BOREHOLE:**				



W. L. 235.0 m
Jan 28, 2021



Note:
1. Monitoring well was installed in the borehole from 4.6 m to 7.6 m bgs.
2. Groundwater level observations:

Date	Depth (mbgs)
2021-01-28	7.5

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3$, $\times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

LOG OF BOREHOLE BH 109

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4835292.783 E 586470.246

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Dec-18-2021

REF. NO.: SP20-747-00
ENCL NO.: 9

(m) ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
230.3					
230.2 0.1	**ASPHALT:** 100 mm thick		AUGER		
	GRANULAR FILL: sand and gravel, 600 mm thick	1	SS	70	
229.5 0.8	**PROBABLE FILL:** sandy silt, some clay, trace organics, dark brown, moist, loose	2	SS	6	
228.8 1.5	**SANDY GRAVEL:** some silt, trace clay, trace to some cobbles, reddish brown, moist to very moist, compact to dense	3	SS	28	50 34 12 4
		4	SS	42	
		5	SS	28	
225.7 4.6	**SAND:** trace silt, trace gravel, brown, very moist to wet, dense	6	SS	33	
224.2 6.1	**GRAVEL:** some sand, trace to some silt, trace cobbles, brown, very moist to wet, very dense	7	SS	86/ 130 mm	
223.8 6.5	**END OF BOREHOLE:**				

Note:

1. Borehole caved at 3.96 mbgs.
2. Water was encountered at 0.9 mbgs upon completion of drilling.

GROUND WATER CONDITIONS

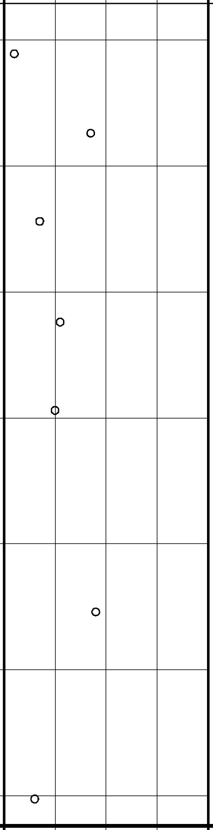
ELEVATION: 230, 229, 228, 227, 226, 225, 224

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH (kPa)
○ UNCONFINED + FIELD VANE & Sensitivity
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%)



POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
+ 3, × 3: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

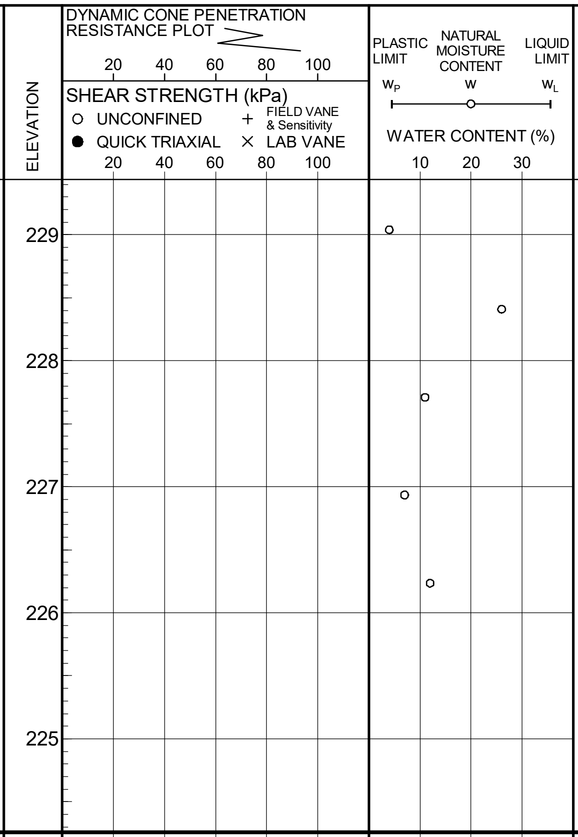
LOG OF BOREHOLE BH 110

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4835393.438 E 586476.564

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Jan-07-2021

REF. NO.: SP20-747-00
ENCL NO.: 10

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
229.4								
229.3 0.1	**ASHALT:** 100 mm thick		AUGER					
	GRANULAR FILL: sand and gravel, 460 mm thick	1	SS	43				
228.9 0.6	**FILL:** clayey silt, trace gravel, tace sand, reddish brown, moist, stiff	2	SS	9				
227.9 1.5	**GRAVEL:** trace to some sand, trace silt, brown, very moist, dense	3	SS	50				
227.1 2.3	**INFERRED BEDROCK, HIGHLY WEATHERED TO MODERATELY WEATHERED SHALE:** queenston formation, reddish brown	4	SS	93/ 280 mm				
		5	SS	92/ 280 mm				
224.9 4.6	**QUEENSTON FORMATION:** moderately weathered to slightly weathered,, reddish brown, SHALE Run=1 TCR=69%, SCR=58%, RQD=33%	1	CORE					
224.3 5.2	**END OF BOREHOLE:**							



GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

SIRATI & PARTNERS

LOG OF BOREHOLE BH 111

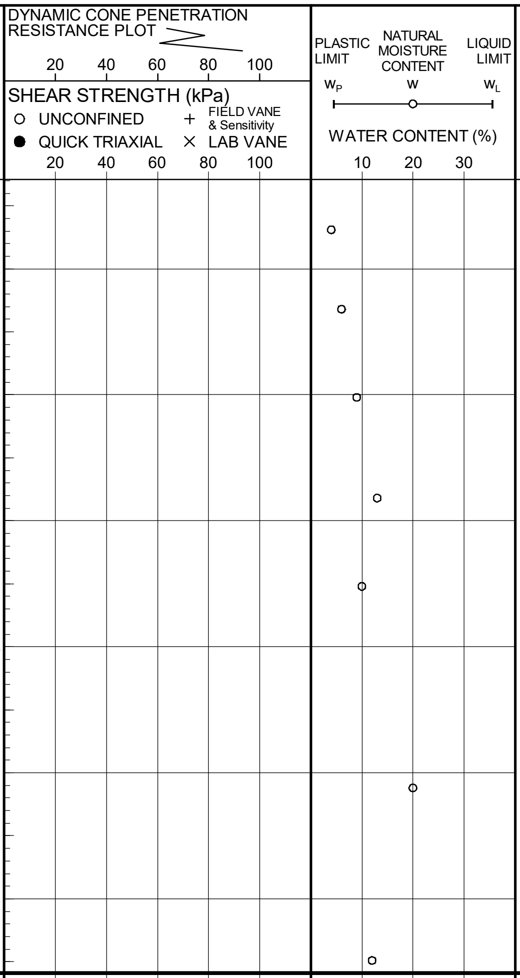
1 OF 1

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4835496.807 E 586491.49

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Jan-05-2021

REF. NO.: SP20-747-00
ENCL NO.: 11

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
228.7									
228.6 0.1	**ASHALT:** 100 mm thick		AUGER						
	GRANULAR FILL: sand and gravel, 460 mm thick	1	SS	86					
228.2 0.6	**FILL:** sand and gravel, trace cobbles, dense	2	SS	32		228			
227.2 1.5	**GRAVEL:** trace to some sand, trace silt, reddish brown, very moist to wet, compact	3	SS	10		227			
		4	SS	14		226			
		5	SS	15		225			
224.1 4.6	**SILT:** trace sand, grey, wet, dense - with sand interbeds	6	SS	32		224 223			
222.6 6.1	**INFERRED BEDROCK, HIGHLY WEATHERED SHALE:** queenston formation, reddish brown	7	SS	50/ 50 mm					
222.4 6.3	**END OF BOREHOLE:**								



GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

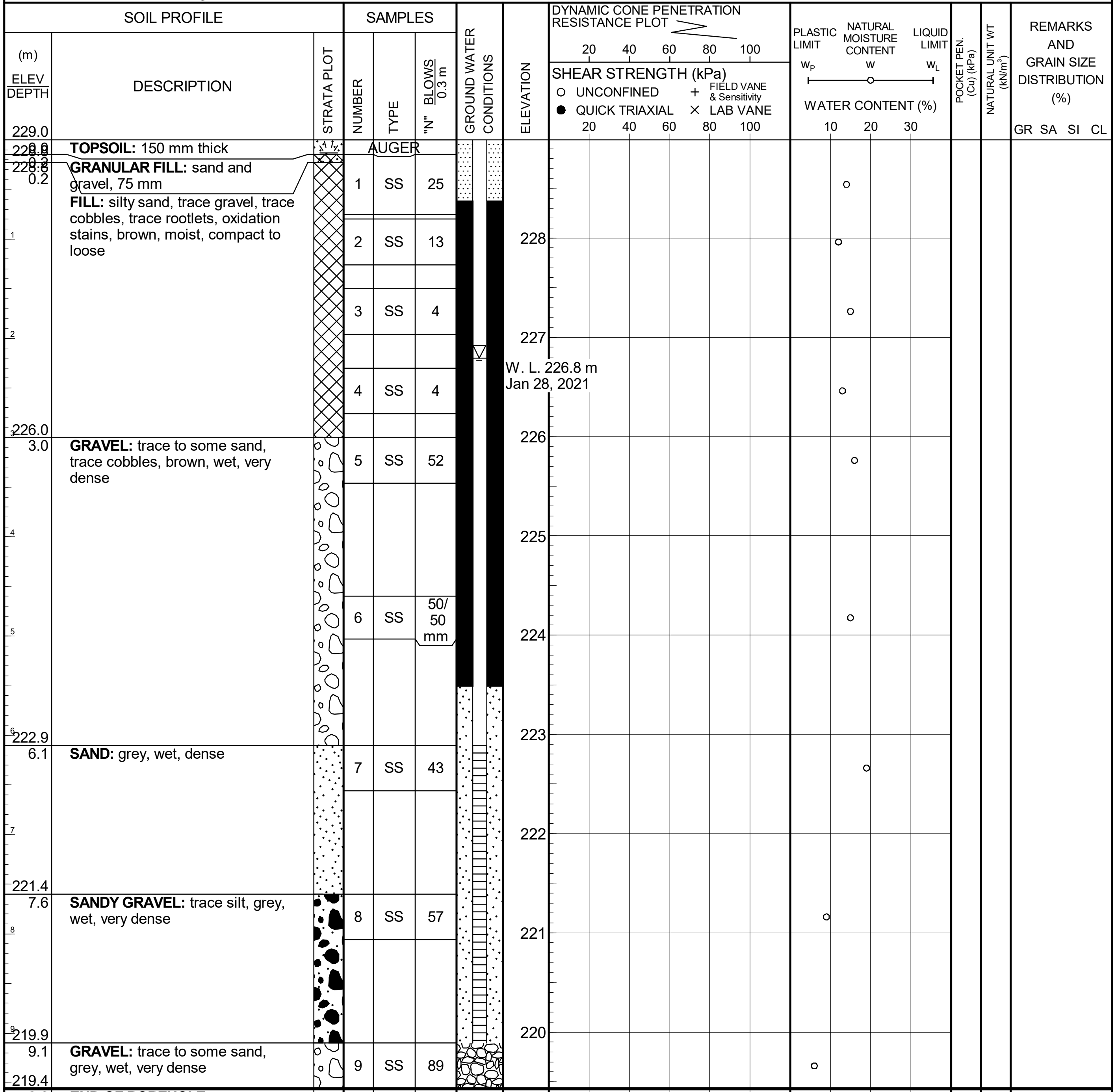
GRAPH NOTES
$+^3$, $\times^3$: Numbers refer to Sensitivity
○ $^{\varepsilon}$=3% Strain at Failure

LOG OF BOREHOLE BH/MW 112

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4835582.8 E 586518.594

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Jan-07-2021

REF. NO.: SP20-747-00
ENCL NO.: 12



ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m
229.0				
0.0	**TOPSOIL:** 150 mm thick		AUGER	
228.8 0.2	**GRANULAR FILL:** sand and gravel, 75 mm			
228.8 0.2	**FILL:** silty sand, trace gravel, trace cobbles, trace rootlets, oxidation stains, brown, moist, compact to loose	1	SS	25
		2	SS	13
		3	SS	4
		4	SS	4
226.0 3.0	**GRAVEL:** trace to some sand, trace cobbles, brown, wet, very dense	5	SS	52
		6	SS	50/ 50 mm
222.9 6.1	**SAND:** grey, wet, dense	7	SS	43
221.4 7.6	**SANDY GRAVEL:** trace silt, grey, wet, very dense	8	SS	57
219.9 9.1	**GRAVEL:** trace to some sand, grey, wet, very dense	9	SS	89
219.4 9.6	**END OF BOREHOLE:**			

Note:

1. Monitoring well was installed in the borehole from 6.1 m to 9.1 m bgs.
2. Groundwater level observations:

Date	Depth (mbgs)
2021-01-28	2.2

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ^{8}=3% Strain at Failure

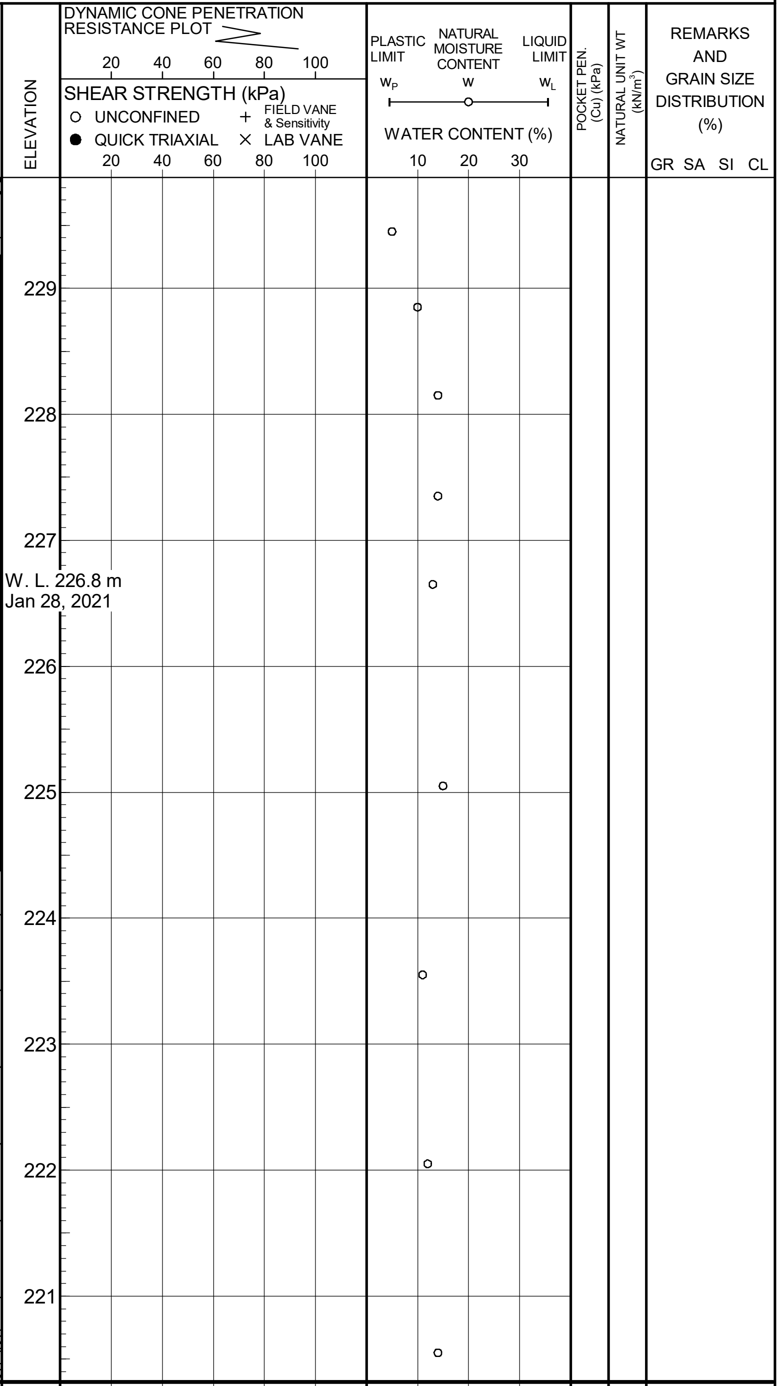
LOG OF BOREHOLE BH/MW 113

PROJECT: Proposed Sanitary Sewer Installation
CLIENT: Condeland Engineering Limited
PROJECT LOCATION: From Meagan Drive to Eildwood Drive in Georgetwon
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4835674.923 E 586516.29

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200 mm
Date: Jan-05-2021

REF. NO.: SP20-747-00
ENCL NO.: 13

(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS
229.9	**ASPHALT:** 130 mm thick		AUGER		
229.8 0.1	**GRANULAR FILL:** sand and gravel, 300 mm thick	1	SS	61	
229.5 0.4	**FILL:** sand, trace gravel, dark brown, moist, dense				
229.1 0.8	**FILL:** silty sand, trace gravel, dark brown, moist, compact	2	SS	21	
228.4 1.5	**PROBABLE FILL:** silty sand, trace gravel, dark brown, moist, very loose	3	SS	3	
227.6 2.3	**SILTY SAND TILL:** trace gravel, trace cobbles, brown, moist, compact	4	SS	10	
226.9 3.0	**GRAVEL:** trace sand, reddish brown, trace cobbles, very moist, compact	5	SS	22	W. L. 226.8 m Jan 28, 2021
225.3 4.6	**SAND:** trace gravel, reddish brown, wet, compact	6	SS	28	
223.8 6.1	**SILTY SAND AND GRAVEL:** reddish brown, wet, dense	7	SS	33	
222.3 7.6	**SANDY SILT TILL:** trace gravel, reddish brown, wet, very dense	8	SS	58	
220.7 9.1	**SILTY SAND TILL:** trace gravel, reddish brown, wet, very dense	9	SS	87	
220.3 9.6	**END OF BOREHOLE:**				



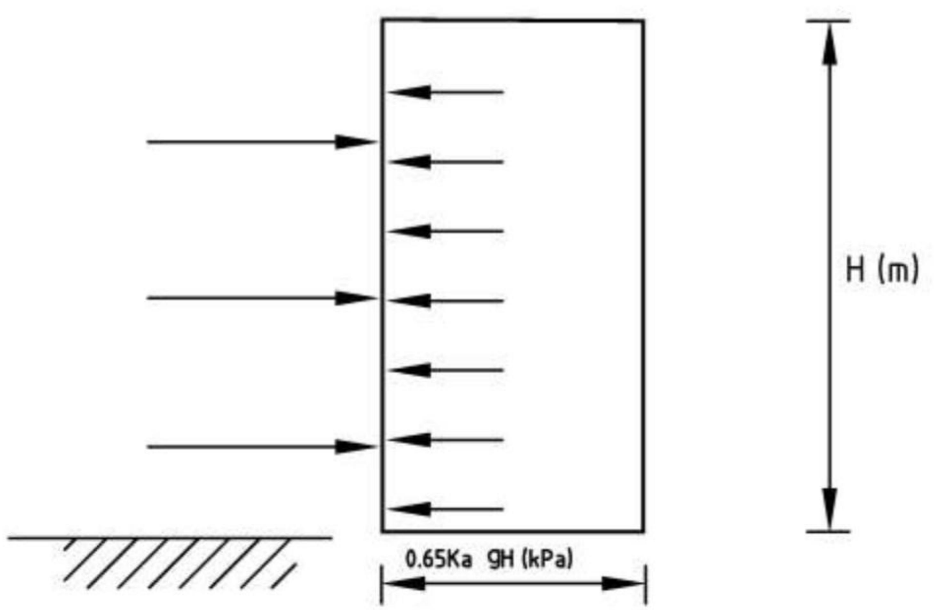
Note:

1. Monitoring well was installed in the borehole from 6.1 m to 9.1 m bgs.
2. Groundwater level observations:

Date	Depth (mbgs)
2021-01-28	3.1

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ 8=3% Strain at Failure

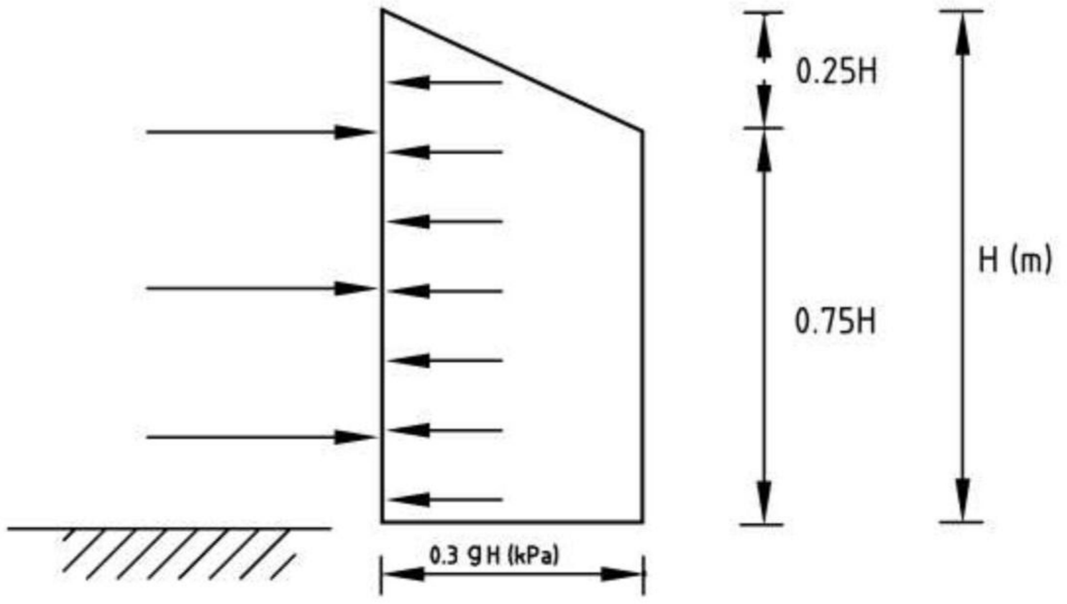


g = unit weight of soil = 21.0 kN/m^3

g' = submerged unit weight of soil (i.e. below ground water level)= 11.2 kN/m^3

Ka = 0.3

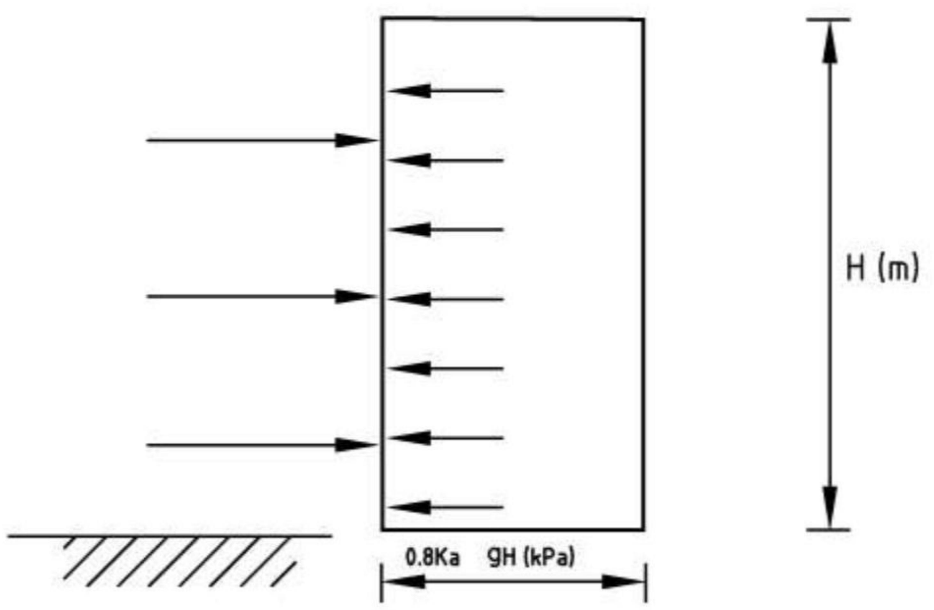
IN COMPACT TO VERY DENSE NON-COHESIVE SOILS (SANDS AND SILTS)



g = unit weight of soil = 21.5 kN/m^3

g' = submerged unit weight of soil (i.e. below ground water level)= 11.7 kN/m^3

IN COHESIVE CLAYS OR CLAYEY SOILS

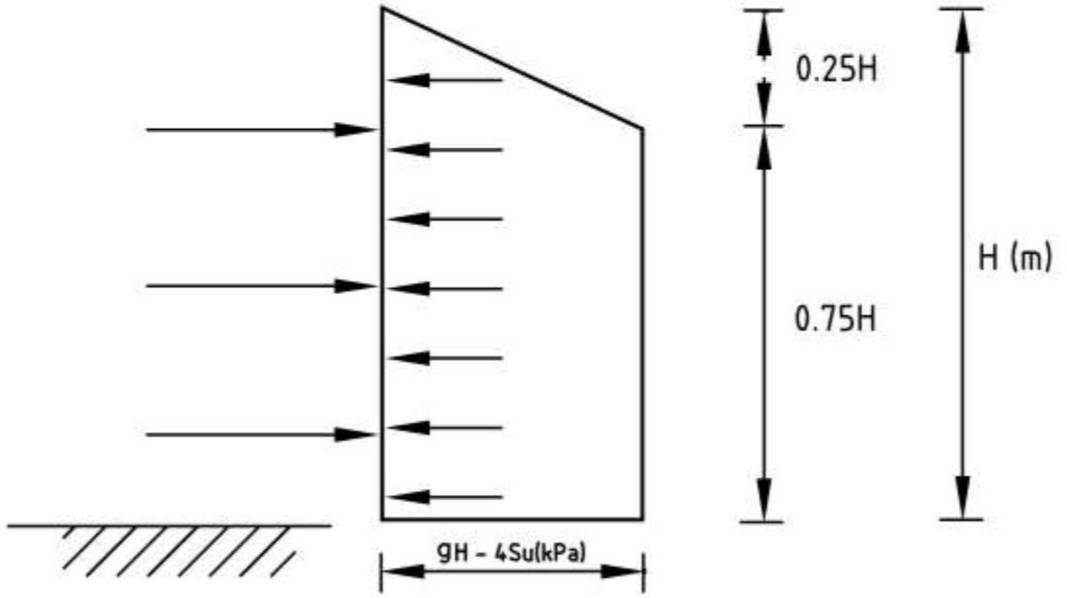


g = unit weight of soil = 19.0 kN/m^3

g' = submerged unit weight of soil (i.e. below ground water level)= 9.2 kN/m^3

Ka = 0.36

IN LOOSE OR DISTURBED NON-COHESIVE SOILS (SANDS AND SILTS)



g = unit weight of soil = 19.0 kN/m^3

g' = submerged unit weight of soil (i.e. below ground water level)= 9.2 kN/m^3

Su = 10 KPa

IN VERY SOFT TO FIRM COHESIVE CLAYS OR CLAYEY SOILS

SIRATI & PARTNERS
Geotechnical Hydrogeological & Environmental Solutions
12700- Keele Street
King City, ON. L7B 1H5
Phone# 905 833 1582, Fax#905 833 5360

North:

Notes:

1. Check system for partial excavation condition.
2. If the free water level is above the base of the excavation, the hydrostatic pressure must be added to the above pressure distribution.
3. If surcharge loadings are present near the excavation, these must be included in the lateral pressure calculation.

Project Title:
Geotechnical Investigation for the Proposed Sanitary Sewer along Oak Ridge Drive and Wildwood Road, Halton Hills, ON.

Site Location:
Oak Ridge Drive and Wildwood Road,
Halton Hills, ON.

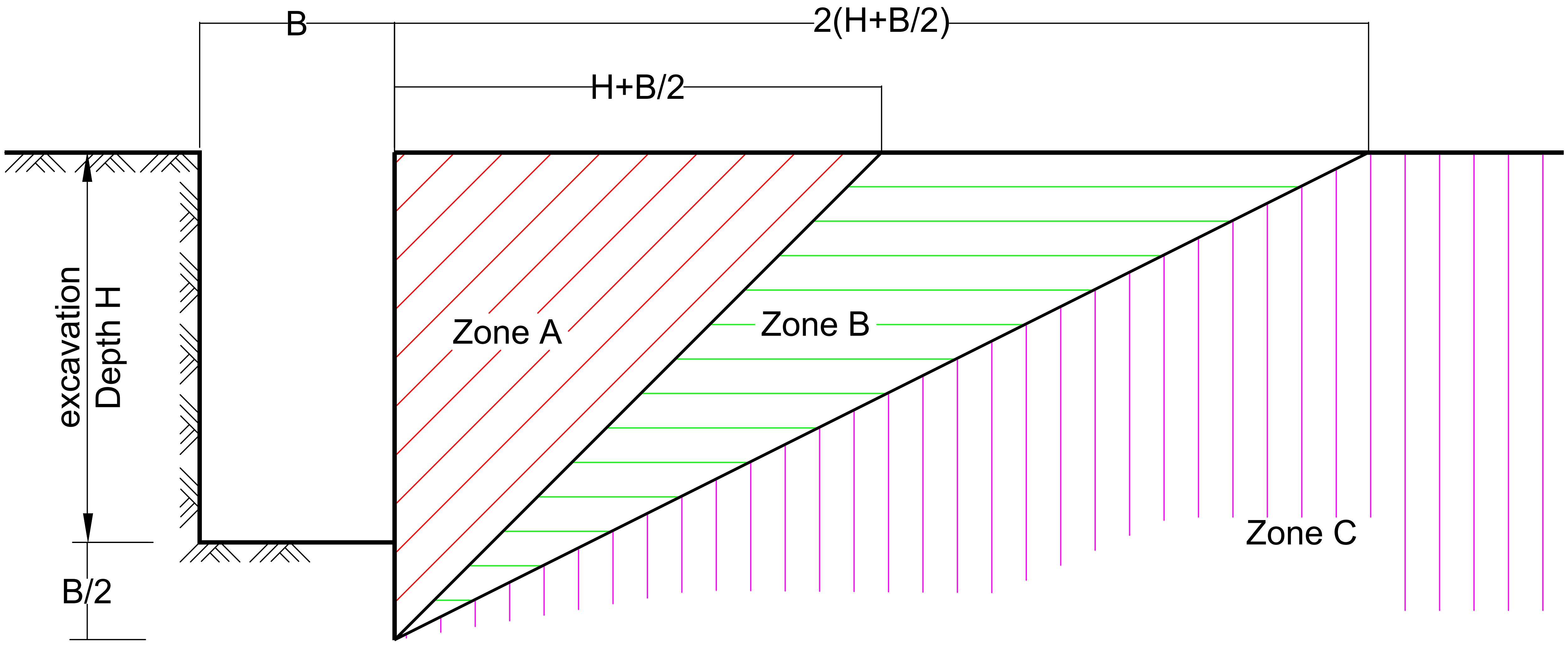
Figure Title:
Earth Pressure Distribution on Braced Excavations

Scale: As Shown

Project Number: SP20-747-10

Date: February 2021

Figure Number: 15



RISK ZONES (after Howe et al., 1980): Zone A is zone of long term risk,Zone B is zone of intermediate risk,Zone C is zone of no risk.

SIRATI & PARTNERS
Geotechnical Hydrogeological & Environmental Solutions
12700- Keele Street
King City, ON. L7B 1H5
Phone# 905 833 1582, Fax#905 833 5360

North:

Project Title:
Geotechnical Investigation for the Proposed Sanitary Sewer along Oak Ridge Drive and Wildwood Road, Halton Hills, ON.

Site Location:
Oak Ridge Drive and Wildwood Road, Halton Hills, ON.

Figure Title:
Excavation Risk Zone

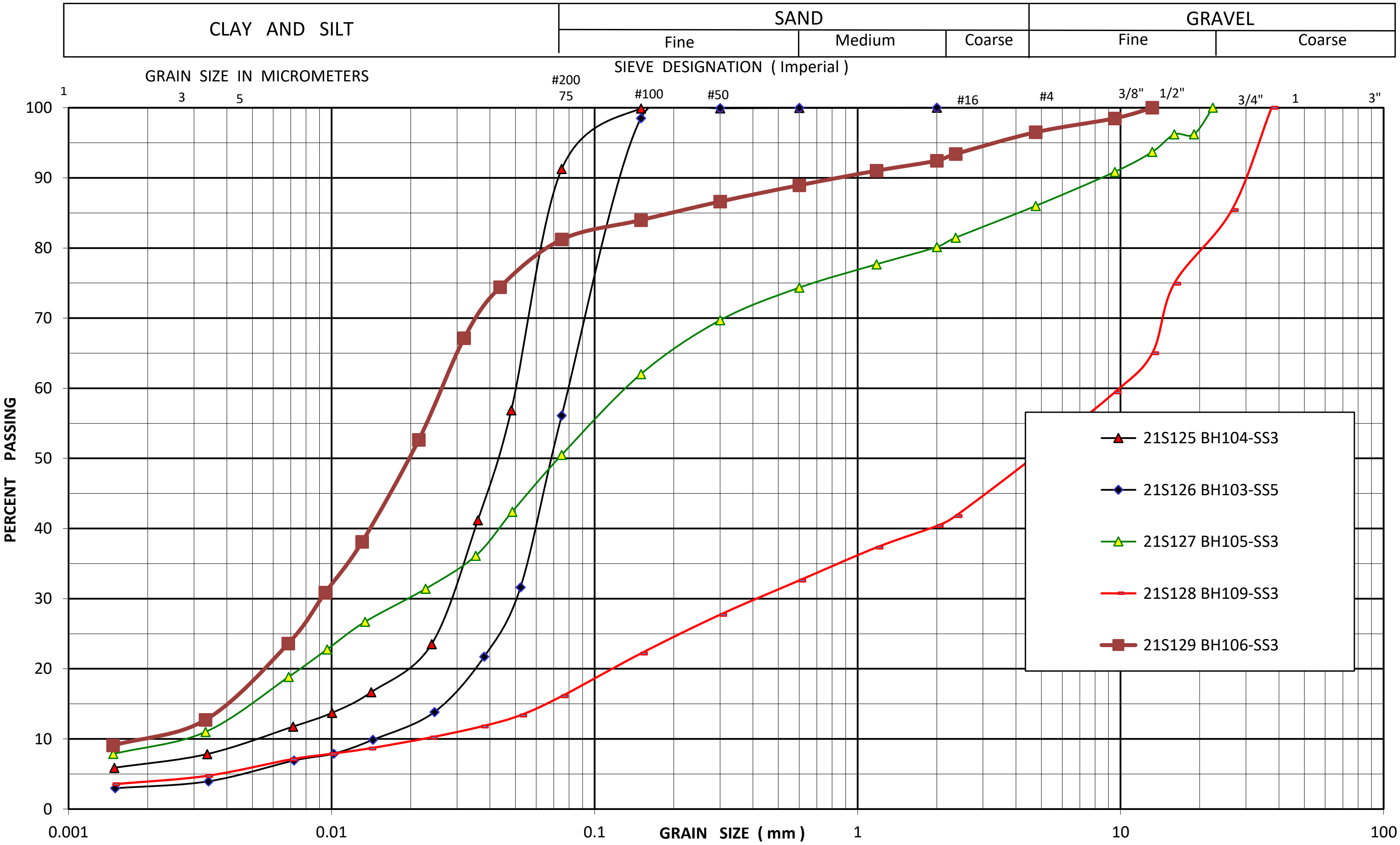
Scale: As Shown	**Project Number:** SP20-747-10
Date: February 2021	**Figure Number:** 16

APPENDIX A: GEOTECHNICAL LABORATORY TEST RESULTS

SIRATI & PARTNERS

GRAIN SIZE DISTRIBUTION

UNIFIED SOIL CLASSIFICATION SYSTEM



Project No.	:	SP20-747-00
Date	:	10 February 2021
Figure No.	:	A-1

APPENDIX B: PHOTOGRAPHS OF THE SITE

Figure 1: Armourstone retaining wall, south of street, 33 Wildwood Road

Figure 2: Armourstone retaining wall, north of street, 33 Wildwood Road

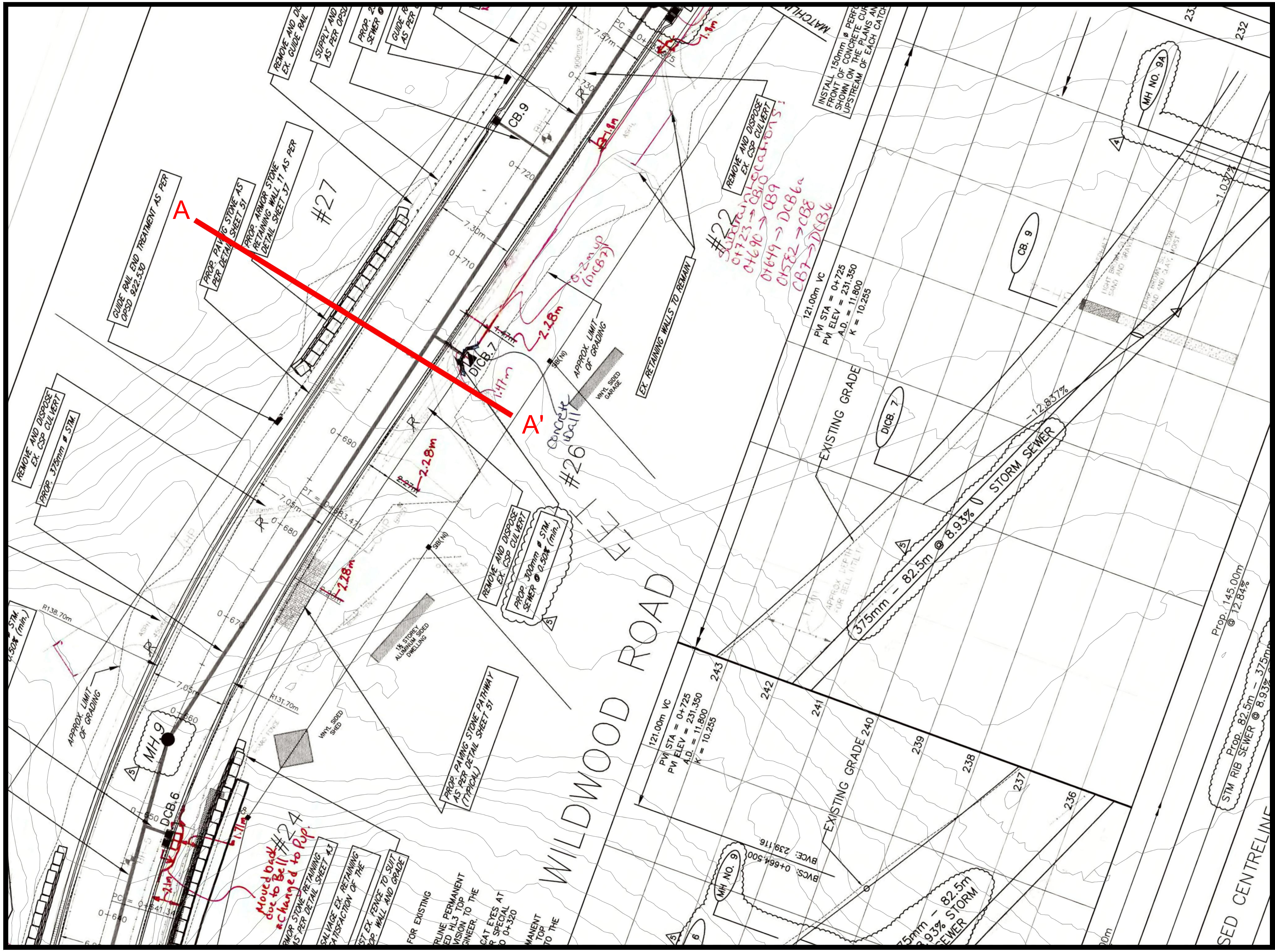
Figure 3: Top of Slope, between 27 and 33 Wildwood Road, looking northeast

Figure 4: Top of Slope, between 27 and 33 Wildwood Road, looking east

Figure 5: Vegetation cover of the slope, between 27 and 33 Wildwood Road, looking east

Figure 6: Armourstone wall, between 27 and 33 Wildwood Road, looking east

APPENDIX C: CROSS SECTION OF THE SLOPE BETWEEN 27 AND 33 WILDWOOD ROAD



SIRATI & PARTNERS

Geotechnical Hydrogeological & Environmental Solutions

12700- Keele Street
King City, ON. L7B 1H5
Phone# 905 833 1582, Fax# 905 833 5360

North:

Legend:

Project Title:

Geotechnical Investigation for Proposed Sanitary Sewer Construction

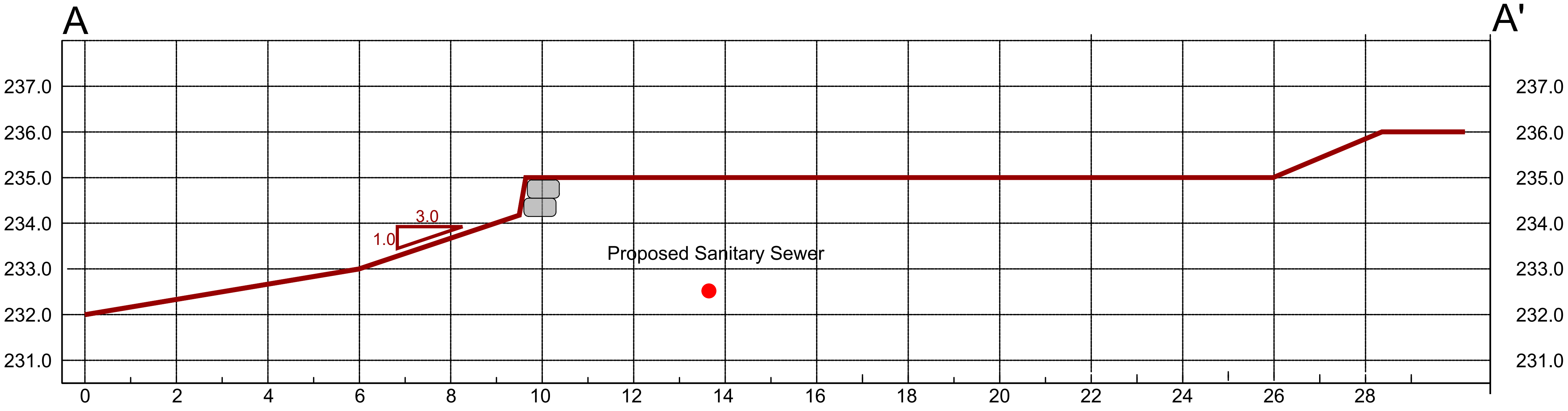
Site Location:

From Meagan Drive to Oak Ridge Drive to Wildwood Road to Main Street, Halton Hills

Figure Title:

Site Topography between 27 and 33 Wildwood Rd.

Scale: As Shown	**Project Number:** SP20-474-10
Date: March, 2021	**Figure Number:** C-1



SIRATI & PARTNERS
Geotechnical Hydrogeological & Environmental Solutions

12700- Keele Street
King City, ON. L7B 1H5
Phone# 905 833 1582, Fax# 905 833 5360

North:

Legend:

Project Title:
Geotechnical Investigation for Proposed
Sanitary Sewer Construction

Site Location:
From Meagan Drive to Oak Ridge Drive to
Wildwood Road to Main Street, Halton Hills

Figure Title:
Cross Section A-A'

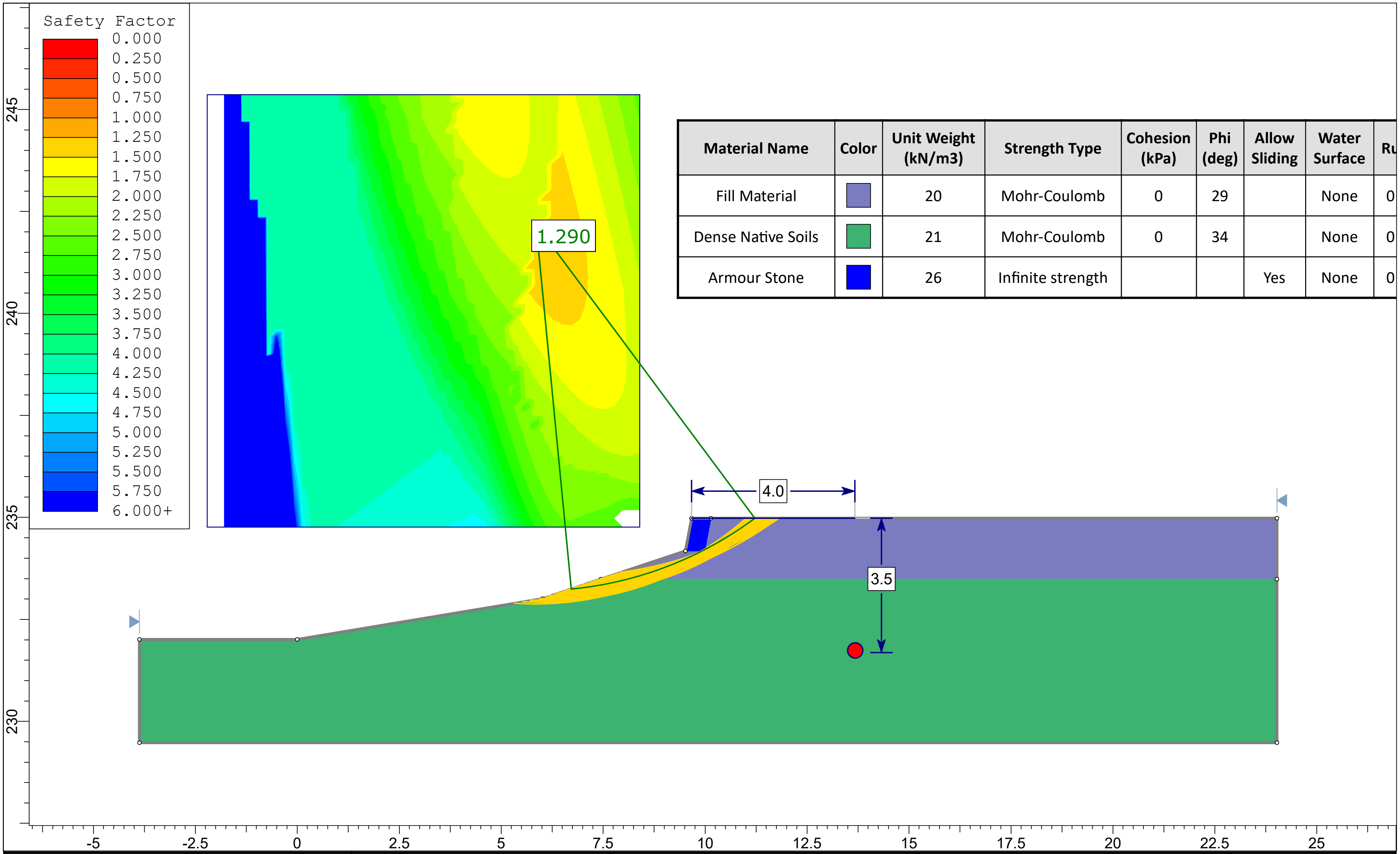
Scale: As Shown

Project Number: SP20-474-10

Date: February 2021

Figure Number: C-2

APPENDIX D: SLOPE STABILITY ANALYSIS RESULT



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding	Water Surface	Ru
Fill Material	🟪	20	Mohr-Coulomb	0	29		None	0
Dense Native Soils	🟩	21	Mohr-Coulomb	0	34		None	0
Armour Stone	🟦	26	Infinite strength			Yes	None	0

rocscience

SLIDEINTERPRET 8.032

Project

SLIDE - An Interactive Slope Stability Program

Analysis Description

Drawn By

Scale 1:132

Company

Date

File Name

Appendix E: Limitation and Use of the Report

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Sirati & Partners Consultants Limited (SIRATI) at the time of preparation. Unless otherwise agreed in writing by SIRATI, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the borehole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the borehole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the borehole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc. Professional judgement was exercised in gathering and analyzing data and formulation of recommendations using current industry guidelines and standards. Similar to all professional persons rendering advice, SIRATI cannot act as absolute insurer of the conclusion we have reached. No additional warranty or representation, expressed or implied, is included or intended in this report other than stated herein the report.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

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. SIRATI accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time. Any user of this report specifically denies any right to claims against the Consultant, Sub-Consultants, their officers, agents and employees in excess of the fee paid for professional services.

SIRATI engagement hereunder is subject to and condition upon, that SIRATI not being required by the Client, or any other third party to provide evidence or testimony in any legal proceedings pertaining to this finding of this report or providing litigations support services which may arise to be required in respect of the work produced herein by SIRATI. It is prohibited to publish, release or disclose to any third party the report produced by SIRATI pursuant to this engagement and such report is produced solely for the Client own internal purposes and which shall remain the confidential proprietary property of SIRATI for use by the Client, within the context of the work agreement. The Client will and does hereby remise and forever absolutely release SIRATI, its directors, officers, agents and shareholders of and from any and all claims, obligations, liabilities, expenses, costs, charges or other demands or requirements of any nature pertaining to the report produced by SIRATI hereunder. The Client will not commence any claims against any Person who may make a claim against SIRATI in respect of work produced under this engagement.