GEOTECHNICAL INVESTIGATION REPORT (R1)

4Q COMMERCIAL WP INC.

PROPOSED MIXED-USE DEVELOPMENT

1580 and 1650 Dundas Street East
Mississauga, Ontario

November 21, 2023

Terrapex Environmental Ltd.
90 Scarsdale Road
Toronto, Ontario, M3B 2R7
Telephone: (416) 245-0011
Website: www.terrapex.com

DISTRIBUTION: **4Q Commercial WP Inc.**

PROJECT # CA20-149

TABLE OF CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

Terrapex Environmental Ltd. **(Terrapex)** has been retained by 4Q Commercial WP Inc. (the Applicant) to carry out a geotechnical investigation for the proposed mixed-use development located at 1580 and 1650 Dundas Street East in Mississauga, Ontario (the Site). Authorization to proceed with this study was given by the Applicant.

The Site is located on the south side of Dundas Street East; approximately 600 m east of Dixie Road in Mississauga. It consists of two rectangular parcels of land separated by Mattawa Avenue. The parcel located on the west side of Mattawa Avenue (the West Parcel) has the municipal address of 1580 Dundas Street East and approximate dimensions of 400 m by 80 m; the parcel located on the east side of Mattawa Avenue (East Parcel) has the municipal address of 1650 Dundas Street East and approximate dimensions of 400 m by 90 m. The northern sections of the two Parcels are developed with large 2-storey commercial buildings and a smaller restaurant building. The remaining area of the site is paved with asphaltic concrete and used for parking. Little Etobicoke Creek runs parallel to the west side of the West Parcel.

We understand that it is proposed to demolish the existing buildings and redevelop the two Parcels with a total of 15 buildings. Four buildings ranging in height from 15 to 41 storeys are proposed for the West Parcel (at Blocks A and C), and nine buildings ranging in height from 3 to 18 storeys are proposed for construction on the East Parcel (at Blocks E, F and G). The buildings will be constructed over two underground parking garage levels. The southern approximately 100 m long section of the West Parcel will be developed as a park.

The fieldwork for this geotechnical investigation was conducted in conjunction with the fieldwork for hydrological review. The hydrological condition at the Site is reported under separate cover**.**

The purpose of this investigation was to characterize the subsurface conditions, to determine the engineering properties of encountered soils and bedrock, and based on this date to provide geotechnical engineering recommendations pertaining to the proposed development.

This report presents the results of the investigation performed in accordance with the general terms of reference outlined above and is intended for the guidance of the client and the design architects or engineers only. It is assumed that the design will be in accordance with the applicable building codes and standards.

It should be noted that our scope for this investigation was limited to the proposed development being constructed over two underground parking garage levels. We were subsequently advised that for Blocks A and C; the number of underground levels have been increased to three. Block E located on the East Parcel near Dundas Street East will have two underground parking garage levels and Block F and G will only contain single underground parking garage levels. Block B located on the southern end of the West Parcel will be developed as a public park. Additional investigation consisting of deeper

coring of the bedrock was undertaken for Blocks A and C, and a supplementary report prepared for the two Blocks.

2.0 FIELD WORK

The fieldwork for this investigation was carried out during the period between December 15 and 23, 2020. It consisted of thirteen (13) boreholes advanced by a drilling contractor commissioned by **Terrapex**. The locations of the boreholes are shown on Figure 1 (Borehole Location Plan) attached to this report; chosen by **Terrapex** to collect the necessary information for the geotechnical investigation and hydrogeological review.

Three (3) of the boreholes; designated as BH105, MW107, and MW112, were extended to approximate depths ranging from 3.5 to 8.0 mbgs and subsequently cored using HQ coring equipment to approximate depths of 11.1 to 12.5 mbgs.

Borehole BH103 was terminated in fill material due to auger refusal on probable concrete blocks within the fill, preventing further advancement of the boreholes.

The remaining nine (9) boreholes were extended to bedrock at approximate depths ranging from 3.9 to 7.7 mbgs.

Monitoring wells were installed in six (6) of the boreholes; Boreholes MW101, MW102, MW307, MW111, MW112, and MW113, to determine the long term groundwater table at the site, and for use for the hydrogeological review.

Standard penetration tests were carried out in the course of advancing the boreholes through the overburden to take representative soil samples and to measure penetration index values (N-values) to characterize the condition of the various soil materials. The number of blows of the striking hammer required to drive the split spoon sampler to 300 mm depth was recorded and these are presented on the logs as penetration index values. Results of SPT are shown on the borehole log sheets in Appendix I of this report.

Groundwater measurements were made in the monitoring wells on January 8, 2021. The results of the groundwater measurements are discussed in Section 4.8 of this report.

The ground surface elevations at the locations of the boreholes were extrapolated from the survey drawing titled “Dunnwyn Centre, 1580-1650 Dundas Street East, City of Mississauga”, dated March 23, 2020, prepared by R. Avis Surveying Inc, and provided to us by Applicant.

The fieldwork for this project was carried out under the supervision of an experienced engineer from this office who laid out the positions of the boreholes in the field; arranged locates of buried services; effected the drilling, sampling and in situ testing; observed groundwater conditions; and prepared field borehole log sheets.

3.0 LABORATORY TESTS

The soil samples retained from the split spoon sampler were properly sealed, labelled and brought to our laboratory. They were visually classified and water content tests were conducted on all soil samples retained from Boreholes MW101, BH104, MW107, BH108, BH110. The results of the classification, water contents, and Standard Penetration Tests are presented on the borehole log sheets attached in Appendix I of this report.

Grain-size analyses were carried out on two (2) native soil samples. The test results are presented as Figure II-1 and II-2 in Appendix II.

In addition, four (4) rock core samples were subjected to compression testing, as well as wet unit weight determinations. The test results are discussed in Section 4.7 of this report.

4.0 SITE AND SUBSURFACE CONDITIONS

Full details of the subsurface soil and groundwater conditions at the site are given on the borehole Log sheets attached in Appendix I of this report.

The following paragraphs present a description of the site and a commentary on the engineering properties of the various soil materials contacted in the boreholes advanced at the Site by **Terrapex.**

It should be noted that the boundaries of soil types indicated on the borehole logs are inferred from non-continuous soil sampling and observations made during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design, and therefore, should not be construed as exact planes of geological change.

4.1 SITE DESCRIPTION

The Site is located on the south side of Dundas Street East; approximately 600 m east of Dixie Road in Mississauga. It is bounded by residential developments to the east, and Mattawa Avenue to the south with an industrial building beyond. Little Etobicoke Creek runs parallel to the west side of the West Parcel. For the purpose of this report, Dundas Street East is considered to be oriented in an east-west direction.

The Site consists of two rectangular parcels of land separated by Mattawa Avenue. The parcel located on the west side of Mattawa Avenue (the West Parcel) has the municipal address of 1580 Dundas Street East and approximate dimensions of 400 m by 80 m; the parcel located on the east side of Mattawa Avenue (East Parcel) has the municipal address of 1650 Dundas Street East and approximate dimensions of 400 m by 90 m.

The northern sections of the two Parcels are developed with large 2-storey commercial buildings and a smaller restaurant building. The remaining area of the site is paved with asphaltic concrete and used for parking.

The ground surface topography of the Site is higher on the north and grades down to the south. The ground surface elevations at the borehole locations ranged between 119.8 m at Borehole MW111 and 115.5 m at Borehole BH104.

4.2 ASPHALTIC CONCRETE

All boreholes with the exception of Borehole MW102 and BH103, were advanced through the asphaltic concrete pavement. The boreholes revealed that thickness of the asphaltic concrete ranges from 75 to 125 mm.

4.3 GRANULAR BASE COURSE

At the location of the boreholes, the base course supporting the asphaltic concrete consists of pit run sand and gravel. The thickness of the granular base ranges from approximately 150 to 500 mm.

4.4 TOPSOIL

Topsoil was encountered in Boreholes MW102 and BH103. The thickness of the topsoil was about 450 mm at the borehole locations.

It should be noted that the topsoil thickness will vary between boreholes. Thicker topsoil than that found in the boreholes may be present.

4.5 FILL MATERIAL

Fill material is present below the granular base course/topsoil in all boreholes. At the borehole locations, the fill material extends to approximate depths ranging from 0.7 to 6.0 mbgs. The fill material consists of various clayey silt, sandy silt, and gravelly sand soils. It contains trace organics, brick, concrete, asphalt, and wood pieces at various locations.

SPT carried out in the clayey fill provided N-values ranging from 3 to 25, indicating soft to very stiff consistency. SPT carried out in the sandy/gravelly fill provided N-values ranging from 5 to 15, indicating that its compactness condition is loose to compact.

The fill material is generally brown and dark brown in color. The water content of the fill samples from Boreholes MW101, BH104, MW107, BH108, BH110 ranges from 9 to 25% by weight; generally being damp to moist in appearance.

4.6 NATIVE SOILS

4.6.1. Sandy Silt (TILL) AND Clayey Silt (TILL)

Sandy silt (till) and clayey silt (till) units are present below a clayey silt layer in Borehole MW107 and below the fill material in all remaining boreholes with the exception of Boreholes MW101 and

BH103. The sandy silt (till) and clayey silt (till) units are glacial deposits and consist of a random mixture of soil particles ranging from clay to gravel, with the silt and sand/clay being the predominant fractions. Cobbles and boulders are probably present but would not be representatively sampled with the equipment used in this investigation.

SPT in the sandy silt (till) provided N-values ranging from 30 to 50/125 mm penetration, indicating dense to very dense compactness condition; generally being dense to very dense. SPT in the clayey silt (till) provided N-values ranging from 19 to 50/75 mm penetration, indicating very stiff to hard consistency.

The sandy silt (till) and clayey silt (till) units are generally brown to greyish brown in color. The water content of the tested samples of the till units from Boreholes MW101, BH104, MW107, BH108, BH110 ranges from approximately 6 to 17%; generally being damp to moist in appearance.

Sieve and hydrometer grain size analyses were carried out on two (2) representative samples. The test results are enclosed in Appendix II as Figures II-1 and II-2, and summarized below.

Borehole Location	Sample Depth (mbgs) and No.	Sample Description	Gravel %	Sand %	Silt %	Clay %
MW102	4.6 (7)	Sand and Silt, trace gravel, trace clay	9	38	45	8
BH108	2.3 (4)	Silt and Clay, some sand, trace gravel	9	12	48	31

Based on the results of the grain size analyses, the coefficient of permeability (k value) of the till units is estimated to be less than 10^{-5} cm/sec, corresponding to low to very low relative permeability.

4.6.2. Clayey Silt

A clayey silt layer is present below the fill material in Borehole MW107 and within the till unit in Borehole MW111; extending to an approximate depth of 2.2 mbgs.

SPT in the clayey silt soil measured N-values of 10 and 50/200 mm penetration; indicating stiff and hard consistency.

This unit is brown in color and moist in appearance. The water content of the tested sample of the clayey silt from Borehole MW107 was about 19% by weight.

4.6.3. Sand, silty sand, and Silt

A sand and silt unit is present below the till unit in Boreholes MW107 and BH108 and within the till unit in Borehole BH109. This unit contains variable proportions of silt classifying the soil as sand with trace silt, silty sand, and silt with trace sand.

Standard penetration resistance in the sand/ silt unit provided N-values ranging from 53 to 55, indicating that its compactness condition is very dense.

The sand and silt unit is generally brown to greyish brown in color. The water content of the tested samples of the sand/ silt from Boreholes MW107 and BH108 was about 14% by weight; generally being wet in appearance.

4.6.4. Shale/Till Complex

A mixture of clayey till and shale fragments (shale/till complex) is present below the fill material in Borehole MW101, underneath the sandy/silty layer in Boreholes MW107 and BH108, and between the till soils and shale bedrock in the remaining boreholes with the exception of Boreholes BH103 and BH104.

SPT in the shale/till complex provided N-values ranging from 28 to 50/50 mm penetration, indicating that its consistency is hard.

This unit is grey in color and damp in appearance. The water content of the tested samples of this unit from Boreholes MW107, BH108, BH110 was about 5 to 9% by weight.

4.7 SHALE BEDROCK

Shale bedrock of the Georgian Bay formation was encountered in all boreholes with the exception of Borehole BH103 below the shale/till complex unit and underneath the till in Borehole BH104; positioned below approximate depths ranging from 3.0 to 7.5 mbgs; corresponding to approximate elevations ranging from 109.5 to 116.0 m.

Standard penetration resistance in the weathered upper unit of the shale provided N-values ranging from 90/200 mm to 50/50 mm penetration.

The bedrock was cored in three (3) of the boreholes; in Boreholes BH105, MW107, and MW112 from approximate depths of 3.5 to 8.0 mbgs to depths of 11.1 to 12.5 mbgs.

The shale bedrock is grey and fine grained. Based on our examination of the rock core samples, the top section is generally intensely fractured and very thin bedded, becoming thinly bedded and moderately fractured with increasing depth. The shale has occasional very thin to thin limestone beds and occasional very thin to thin clay seams.

Rock Quality Designation (RQD) values of the bedrock are shown on the borehole log sheets. The RQD values of the recovered cores range from 10 to 58%. Based on Table 3.10 of the Canadian Foundation Engineering Manual (CFEM) 4th Edition, the bedrock is classified as “very poor to good quality”; generally being “**very poor to fair**”.

The strength of the shale bedrock was assessed by peeling the rock specimens with a pocket knife. Using a geological hammer, most of the rock specimens required a single blow for their fracturing. Using Table 3.5 of the CFEM (4th Edition), and based on the above strength index tests, the strength of shale bedrock at the Site is termed “**medium strong**”.

Uniaxial compressive strength (UCS) tests and bulk unit weight (γ_W) determinations were carried out on four (4) rock core samples. The UCS and γ_W values of the tested rock samples are given below.

Borehole No.	Ground Elevation (m)	Sample Depth (mbgs) / Elevation (m)	UCS (MPa)	γw (kN/m³)
BH105	119.1	8.58 /110.5	20.2	24.9
MW107	115.7	5.64 /110.1	18.0	26.6
MW107	115.7	8.67 /107.0	18.8	25.1
MW112	117.1	7.77 /109.3	16.8	25.6

Based on the UCS test results, the bedrock is “**weak**” and its hardness grade is R2 according to Table 3.5 of the CFEM (4th Edition).

Combining the strength index tests, the UCS tests and our observations of the bedrock quality, our assessment is that the bedrock at the site is “**weak to medium strong**”.

Photographic record of the extracted rock cores from Boreholes BH105, MW107, and MW112 is enclosed in Appendix III.

4.8 GROUNDWATER

Groundwater measurements were made in the monitoring wells on January 8, 2021. The groundwater monitoring results are shown in the following table.

Borehole No.	Ground Elevation (m)	Approx. Depth of Monitoring Well Bottom (mbgs)	Groundwater Depth (mbgs)	Groundwater Elevation (m)
MW101	117.4	6.7	6.06	111.21
MW102	117.1	7.6	4.31	112.81
MW107	115.7	9.1	3.33	112.45
MW111	119.9	3.0	Dry	-
MW112	117.1	9.1	2.72	114.35
MW113	116.0	3.0	3.05	112.84

* Note: D and S denote deep and shallow nested monitoring wells.

It should be noted that groundwater levels are subject to seasonal fluctuations. A higher groundwater level condition will likely develop in the spring and following significant rainfall events.

4.9 SUMMARY OF SOIL UNITS

The table below summarizes the soil units encountered at the borehole locations.

Borehole Location	BH No.	Ground Elevation (m)	Asphaltic Concrete/ Topsoil (mm)	Granular Base (mm)	Fill (mbgs)	Sandy Silt (till) (mbgs)	Clayey Silt (till) (mbgs)	Clayey Silt (mbgs)	Sandy/Silty layer (mbgs)	Shale/Till complex (mbgs)	Shale Bedrock (mbgs)
Top of the Slope at 1595 Dundas St E	MW101	117.4	75	200	6.0	-	-	-	-	6.0-7.5	7.5
	MW102	117.1	450	-	2.1	2.1-6.0	-	-	-	6.0-7.5	7.5
	BH103	115.7	450	-	2.7	-					
	BH104	115.6	80	150	3.8	-	3.8-6.0	-	-	-	6.0
	BH105	119.1	80	150	1.2	4.5-6.0	1.2-4.5	-	-	6.0-7.5	7.5

1595 Dundas St E	BH106	117.3	75	225	1.5	-	1.5-3.4	-	-	3.4-4.5	4.5
	MW107	115.7	105	500	1.7	2.2-3.0	-	1.7-2.2	3.0-3.6	3.6-4.5	4.5
1650 Dundas St E	BH108	119.3	?	?	2.0	3.3-5.2	2.0-3.3	-	5.2-5.8	5.7-6.1	6.1
	BH109	116.7	100	200	1.2	1.2-2.1 3.2-3.8	-	-	2.1-3.2	3.8-4.2	4.2
	BH110	116.0	100	250	1.7	-	1.7-2.5	-	-	2.5-4.0	4.0
	MW111	119.9	100	150	1.2	-	1.2-3.1	-	-	3.1-4.0	4.0
	MW112	117.1	125	225	0.7	-	0.7-2.1	-	-	2.1-3.0	3.0
	MW113	116.0	80	200	1.0	-	1.4-2.2	1.5-2.2	-	2.2-3.7	3.7

5.0 DISCUSSION AND RECOMMENDATIONS

The following discussions and recommendations are based on the factual data obtained from the boreholes advanced at the Site by **Terrapex** and are intended for use by the client and design architects and engineers only.

The investigation has revealed that the subsurface stratigraphy generally comprises asphaltic concrete pavement and fill material extending to approximate depths ranging from 0.7 to 6.0 mbgs, underlain by clayey silt (till) and sandy silt (till), followed by shale/till complex which extends to bedrock. Shale bedrock was encountered at approximate depths ranging from 3.0 to 7.5 mbgs.

We understand that it is proposed to demolish the existing buildings, and redevelop the two Parcels with a total of 13 buildings. Four buildings ranging in height from 15 to 41 storeys are proposed for the West Parcel, and nine buildings ranging in height from 3 to 18 storeys are proposed for construction on the East Parcel. The buildings will be constructed over two underground parking garage levels. The southern approximately 100 m long section of the West Parcel will be developed as a Park.

As stated in Section 1.0 of this report, Blocks A and C will contain three underground parking garage levels, the bedrock within these Blocks was cored and a supplementary report including recommendations for three underground parking garage levels are provided in a separate supplementary report. Recommendations for one and two underground parking garage levels are included in this report.

Contractors bidding on this project or conducting work associated with this project should make their own interpretation of the factual data and/or carry out their own investigations.

5.1 EXCAVATION

Based on the borehole findings, excavation for basement and foundations will be carried out through fill, native soils, and shale bedrock.

Excavation of the shale can be made with conventional hydraulic excavators equipped with ripping teeth. Locally, excavations may require the use of a ripper and a hydraulic hammer due to bedrock hardness especially if limestone layers of considerable thickness are encountered. Provisions should be made in the excavation contract to include the use of these equipment for excavation in bedrock.

Excavations for foundations should be carried out so as to minimize the disturbance of shale at the design founding elevations. In this regard, it may be necessary to use a hydraulic hammer for foundation excavations.

All excavations must be carried out in accordance with Occupational Health and Safety Act (OHSA). With respect to OHSA, the near surface fill can be classified as Type 3. The native dense to very dense sandy silt (till), very stiff to hard clayey silt (till), and hard shale/till complex should conform to Type 2 classification. The shale bedrock is classified as Type 1. The wet sandy layers which are positioned below the groundwater table are classified as Type 4 soils

Temporary excavations for slopes in Type 3 soils should not exceed 1.0 horizontal to 1.0 vertical. Excavations in Type 2 soil may be cut with vertical side-walls within the lower 1.2 m height of excavation and 1.0 horizontal to 1.0 vertical above this height. Locally, where loose or soft soil is encountered at shallow depths or within zones of persistent seepage, it may be necessary to flatten the side slopes as necessary to achieve stable conditions. Side slopes of excavations extending below the water table in the wet sandy layers (Type 4 soil) should not be any steeper than 3 horizontal to 1 vertical.

Vertical cuts into the shale bedrock will be possible. However, the exposed rock surface should be inspected by the Geotechnical Engineer to ensure stability, particularly at areas where groundwater seepage occurs from the rock. Remedial works such as steel mesh, shotcrete should be implemented if deemed necessary.

It is well recognized that the shale rocks found in Southern Ontario, including the Georgian Bay shale, exhibits time dependent deformation (TDD) when stress changes (e.g. deep excavation) occur in rock. The locked-in stresses in the bedrock stratum are expected to result in a lateral movement in the sides of excavations. The locked-in stresses diminish over a period of time following stress relief by excavation. In addition to the stress induced deformations, the Georgian Bay shale may also exhibits swelling potential. Allowance should therefore be made for this long term TDD characteristics of the bedrock. In this regard, one method to mitigate the effects of rock squeeze is to delay the installation of the permanent structures until sufficient rock deformation has occurred, so that any further deformation after construction is within manageable limits. However, it may not be possible to delay the construction of permanent structure sufficient to allow for a suitable diminution of rock pressure. Alternatively, a crushable material may be installed at the rock-structure interface, (such as 50 mm thick foam) that would allow for any subsequent rock deformation to occur within the foam layer, therefore preventing the build-up of deleterious pressures on the walls of the structures.

Where workers must enter excavations extending deeper than 1.2 m below grade, the excavation side-walls must be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulation for Construction Projects.

It is anticipated that the basement walls of the proposed development will extend to the property limits and sufficient space will not be available to slope the sidewalls of the basement excavation; as such it will be necessary to shore the basement excavation walls within the overburden soil; above the bedrock. A soldier-pile and wood lagging or a secant pile wall may be used as the shoring system. Shoring recommendations are provided in Section 5.8 of this report.

Where space permits, temporary open cut may be used for basement excavations. The safe side slope angle for open excavations should conform to the Occupational Health and Safety Act requirements.

5.2 REUSE OF ON-SITE EXCAVATED SOIL

On-site excavated inorganic soils are considered suitable for reuse as backfill material or engineered fill, provided their water content is within 2% of their optimum water contents (OWC) as determined by Standard Proctor test, and the materials are effectively compacted with a heavy smooth drum compaction roller.

Based on the findings from the geotechnical investigation, the majority of the existing fill material on site contains organics or debris (i.e., concrete pieces, bricks, glass etc.), which was considered not suitable for backfill. While the quality of the native soils is considered suitable for backfilling, the moisture content of the soils and the lift thickness for compaction must be properly controlled during the backfilling. Alternatively, imported suitable material should be used.

The spoil resulting from excavation through the bedrock will contain a large amount of hard rock slabs which will be virtually impossible to compact.

It is recommended that service trenches be backfilled with on-site native soils compacted to 95% of Standard Proctor Maximum Dry Density (SPMDD). Lift thicknesses should not exceed 200 mm in a loose state and the excavated site material should be compacted using heavy, vibratory pad-type rollers.

In areas of narrow trenches or confined spaces such as around manholes, catch basins, etc., imported sand or OPSS Granular 'B' should be used and compacted to the specified SPMDD.

5.3 GROUNDWATER CONTROL

Based on observations made during the drilling of the boreholes, and close examination of the soil and rock samples extracted from the boreholes, significant groundwater seepage is not expected to occur in the basement or footing excavations. It is anticipated that adequate control of any groundwater seepage can be achieved by pumping from properly filtered sumps in the base of the excavations.

It will be necessary to determine the construction dewatering requirements and to collect the information required for the application for Permit to Take Water (PTTW). The hydrogeological report should be referred to in this regard.

The contractor should make his own assessment for temporary control of groundwater seepage into the excavation. Surface water should be directed away from open excavations.

5.4 FOUNDATION DESIGN

We understand that the proposed development will be constructed over one and two underground parking garage levels. Accordingly, the basement floors will be situated at approximate depths of 3.0 and 6.0 mbgs, and the building foundations will be founded approximately 4.5 and 7.5 mbgs.

Based on the borehole findings, the bearing stratum at these depths at all boreholes with the exception of Borehole BH108 will consist of or bedrock which is suitable for support of conventional strip and spread footing foundations. Bedrock is situated at an approximate depth of 6 m below grade at BH108. It is recommended that in the vicinity of this borehole, and in areas where bedrock is situated deeper than the proposed founding level of the foundations, the foundations be lowered to contact bedrock. Foundations founded on the bedrock may be designed for bearing resistances at Ultimate Limit States (ULS) of 3.0 MPa, for vertical and centric loads.

Settlement of building foundations resting on bedrock is small; accordingly the design of the foundations are based on ULS. The total and differential settlements of foundations founded in the shale are expected to be negligible.

All footing subgrade must be evaluated by the Geotechnical Engineer prior to placing formwork and foundation concrete to ensure that the surface exposed at the excavation base is consistent with the design geotechnical bearing resistance.

Where necessary, the stepping of the footings at different elevations should be carried out at an angle no steeper than 1.4 horizontal (clear horizontal distance between footings) to 1 vertical (difference in elevation).

Rainwater or groundwater seepage entering the foundation excavation must be pumped away (not allowed to pond). The foundation subgrade should be protected from freezing, inundation and equipment traffic at all times.

The bedrock tends to weather and deteriorate rapidly on exposure to atmosphere or surface water. **Terrapex** recommends that footings placed on the exposed soil should be poured on the same day as they are excavated, after removal of all unsuitable founding materials and approval of the bearing surface. Alternatively, a concrete mud slab may be used to protect a bearing surface where footing construction is to be delayed.

5.5 CONCRETE SLAB-ON-GRADE

In is anticipated that the basement floors will be situated at approximate depths of 3 mbgs for single basement level and 6.0 mbgs for two underground levels. The subgrade below the proposed basement floor slab will generally consist of shale bedrock or shale/till complex, and sandy silt till at BH108, which are adequate to support a slab on grade construction.

Subgrade preparation should include the removal of any weak and disturbed rock and soil. Any loose and disturbed soil / rock should be sub-excavated and replaced with suitable approved earth fill material compacted to at least 98% of Standard Proctor Maximum Dry Density (SPMDD).

Where new fill is required to raise the grade, excavated native soil from the Site or similar clean imported granular material may be used. Oversize particles (cobbles, boulders) larger than 100 mm should be discarded from the fill material. The fill material should not be frozen and should not be too wet for efficient compaction (moisture content at optimum or 2% points greater than optimum).

It is recommended that a combined moisture barrier and a leveling course, having a minimum thickness of 200 mm and comprised of free draining material using 19 mm clear stone be provided as a base for the slab-on-grade. The base material should be compacted to a dense condition.

Provided the subgrade, underfloor fill and granular base are prepared in accordance with the above recommendations, the recommended Modulus of Subgrade Reaction (Ks) for slab design will be 30,000 kPa/m.

Sub-floor weeping pipes 100 mm in diameter must be placed under the slab-on-grade at a maximum spacing of 8 m (subject to confirmation at the time of construction). The weeping tiles must be wrapped with filter fabric and covered with a minimum of 150 mm of clear stone. They should be placed a minimum of 0.5 m below the basement floor slab, above the founding level of the footings.

In the event that the exterior basement walls of the proposed building will be poured up against the shored or, grouted or foamed walls of the excavation, prefabricated drainage sheets (Terradrain 600 or equivalent) must be placed continuously against the excavation / shoring walls. These should drain through drainage ports in the walls into a perimeter solid pipe and channel all the water into the sump pits in the buildings. The maximum spacing of the drainage ports must not exceed 6 m, subject to confirmation at the time of construction.

The perimeter foundation and sub-floor drains must be connected to a positive frost free outlet from which the water can be removed, or connected to a sump located in the lowest level of the basement. The water from the sump must be pumped out to a suitable discharge point.

Typical details of perimeter and sub-floor drainage systems are included in Appendix IV of this report. The installation of the perimeter and sub-floor drains as well as the outlet must conform to the applicable plumbing code requirements.

The near surface soils at this site are susceptible to frost effects which would have the potential to deform hard landscaping adjacent to the building. At locations where the building is expected to have flush entrances, care must be taken in detailing the exterior slabs / sidewalks, providing insulation / drainage / non-frost susceptible backfill to maintain the flush threshold during freezing weather conditions.

5.6 ELEVATOR PITS

Elevator pit(s) in general are constructed at a minimum depth of 1.5 m below the lowest basement floor slab. As the elevator pits will extend below the groundwater table, it is not recommended to install permanent dewatering systems (weeping tiles) surrounding the bases of the elevators, due to continuous dewatering requirements. It is recommended to waterproof the bases and the walls of the elevator pits, and design the pits for hydrostatic uplift and lateral hydrostatic pressures.

5.7 LATERAL EARTH PRESSURE

Parameters used in the determination of earth pressure acting on temporary shoring walls are defined below.

Parameter	Definition	Units
Φ’	angle of internal friction	degrees
γ	bulk unit weight of soil	KN/m^3
Ka	active earth pressure coefficient (Rankine)	dimensionless
Ko	at-rest earth pressure coefficient (Rankine)	dimensionless
Kp	passive earth pressure coefficient (Rankine)	dimensionless

The appropriate un-factored values for use in the design of structures subject to unbalanced earth pressures at this site are tabulated as follows:

Soil	Parameter				
	Φ’	γ	Ka	Kp	Ko
Fill Materials	30°	19.0	0.36	2.77	0.53
very stiff to hard Clayey Silt (till)	32°	21.0	0.31	3.25	0.47
dense to very dense Sandy Silt (till)	36°	21.0	0.26	3.88	0.40
Shale/till complex	36°	22.0	0.26	3.88	0.40
Shale Bedrock	26°	25.0	n/a	n/a	n/a

Subsurface walls subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following formula:

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

Where P = lateral pressure in kPa acting at a depth h (m) below ground surface
K = applicable lateral earth pressure coefficient
γ = bulk unit weight of backfill (kN/m^3)
h = height at any point along the interface (m)
q = the complete surcharge loading (kPa)

This equation assumes that free-draining backfill and positive drainage is provided to ensure that there is no hydrostatic pressure acting in conjunction with the earth pressure.

The coefficient of earth pressure at rest (K_o) should be used in the calculation of the earth pressure on the basement walls.

Subsurface walls that are subject to unbalanced earth and hydrostatic pressures must be designed to resist a pressure that can be calculated based on the following formula:

$$\mathbf{P = K\,[\gamma\,(h - h_w) + \gamma' h_w + q] + \gamma_w h_w}$$

where P = lateral pressure in kPa acting at a depth h (m) below ground surface
K = applicable lateral earth pressure coefficient
H = height at any point along the interface (m)
hw = depth below the groundwater level at point of interest (m)
γ = bulk unit weight of backfill (kN/m^3)
γ' = the submerged unit weight (kN/m^3) of exterior soil ($\gamma' = \gamma - \gamma w$)
γw = unit weight of water, assume a value of 9.8 kN/m^3
q = the complete surcharge loading (kPa)

Resistance to sliding of earth retaining structures is developed by friction between the base of the footing and the soil. This friction (R) depends on the normal load on the soil contact (N) and the frictional resistance of the soil (tan Φ') expressed as: R = N tan Φ'. This is an ultimate resistance value and does not contain a factor of safety.

5.8 SHORING DESIGN

Assuming that the basement of the proposed buildings will extend close to the property limits, it will not be possible to slope the banks of the excavation. In this regard it will be necessary to shore the excavation walls within the overburden soil; above the bedrock. The shoring system may be comprised of soldier piles and timber lagging.

Where space permits, temporary open cut may be used for basement excavations. The safe side slope angle for open excavations should conform to the Occupational Health and Safety Act requirements.

The design of temporary shoring for the support of the subsoils must account for the presence of structures and buried services on the adjacent properties, and the existing subsurface conditions at the site.

Vertical cuts into the shale bedrock will be possible. However, the exposed rock surface should be inspected by the Geotechnical Engineer to ensure stability, particularly at areas where groundwater seepage occurs from the rock. Remedial works such as steel mesh, shotcrete should be implemented if deemed necessary.

The lateral restraining force for the shoring system may be provided by employing either rakers or tieback anchors. The latter is favorable because they do not protrude into the excavations as is the case with rakers. The use of tieback anchors will depend on whether permission is obtained to extend the anchors to the required distance on to the neighboring properties.

The shoring design should be based on the procedure detailed in the latest edition of the Canadian Foundation Engineering Manual.

The active earth pressure coefficient; Ka to be used for the design of the shoring system, should be as follows:

= 0.4 where adjacent building footings or buried services fall within an envelope formed by a 60^0 line drawn from the base of the excavation wall to the ground surface.

= 0.3 where adjacent building footings or buried services fall outside an envelope formed by a 60^0 line drawn from the base of the excavation wall to the ground surface.

= 0.25 where adjacent building footings or buried services are outside an envelope formed by a 45^0 line drawn from the base of the excavation wall to the ground surface.

Anchors extended into the shale bedrock may be designed based on skin friction values of 200 kPa within the top 1 m zone of the rock and 600 kPa below this depth. These values depend on the anchor installation method and grouting procedures. Gravity poured concrete can result in low bond values, while pressure grouted anchors will give higher values and produce a more satisfactory anchor.

It will be necessary to perform load tests on the tiebacks to confirm the bond stresses assumed in the design of anchors.

Movement of the shoring system is inevitable. Vertical movements will result from the vertical loads on the soldier piles resulting from the inclined tiebacks and inward horizontal movement will result from the earth and water pressures. The magnitude of this movement can be controlled by sound construction practices. The lateral and vertical movement of the shoring system must be monitored especially at locations in which settlement sensitive structures are present, to ensure that movements are kept within acceptable range.

5.9 EARTHQUAKE DESIGN PARAMETERS

The 2012 Ontario Building Code (OBC) stipulates the methodology for earthquake design analysis, as set out in Subsection 4.1.8.7. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification.

The parameters for determination of the Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A of the 2012 OBC. The classification is based on the determination of the average shear wave velocity in the top 30 metres of the site stratigraphy, where shear wave velocity (vs) measurements have been taken. In the absence of such measurements, the classification is estimated on the basis of empirical analysis of undrained shear strength or penetration resistance. The applicable penetration resistance is that which has been corrected to a rod energy efficiency of 60% of the theoretical maximum or the (N_{60}) value.

Based on the borehole information, the subsurface stratigraphy as revealed in the boreholes generally comprises surficial layer of fill materials underlain by very stiff to hard clayey silt (till) and dense to very dense sandy silt (till), followed by hard shale/till complex which extends to bedrock. Shale bedrock was encountered at approximate depths ranging from 3.0 to 7.5 mbgs. Based on the above, the site has been classified as "**Class C**" for Seismic Site Response in accordance with table 4.1.8.4.A of the 2012 OBC.

The site specific 5% damped spectral acceleration coefficients, and the peak ground acceleration factors are provided in the 2012 Ontario Building Code - Supplementary Standards SB-1 (September 14, 2012), Table 1.2, location Mississauga, Ontario.

6.0 PAVEMENT DESIGN AND SITE SERVICES

It is understood that the site will be served by driveway and parking lots. A municipal road will also be constructed at the site.

6.1 PAVEMENT DESIGN FOR ON-GROUND PARKING

The pavement above the parking garage roof slab may be comprised of a minimum of 75 mm thick layer of granular 'A' topped with asphaltic concrete having a minimum thickness of 80 mm (40 mm HL8 and 40 mm HL3). The asphaltic concrete materials should be rolled and compacted in accordance with OPSS 310 requirements.

The gradation and physical properties of HL-3 and HL-8 asphaltic concrete, and Granular 'A' shall conform to the OPSS standards.

The critical section of pavement will be at the transition between the pavement on grade and the pavement above the garage roof slab. In order to alleviate the detrimental effects of dynamic loading / settlement / pavement depression in the backfill to the rigid garage roof structure, it is recommended that an approach type slab be constructed at the entrance/exit points, by extending the granular sub-base to greater depths along the exterior garage wall.

The granular courses of the pavement should be placed in lifts not exceeding 150 mm thick and be compacted to a minimum of 100% SPMDD.

6.2 ROAD

Boreholes BH105 to BH110 were drilled in the close vicinity of the proposed municipal road. The investigation has shown that the predominant subgrade soil after stripping off the topsoil and pavement structure will generally consist of stiff to very stiff clayey silt (fill) and compact sandy silt (fill), sand and silt as well as silty sand deposit. The frost susceptibility factor is considered as II in according to City of Mississauga Standard 2220.020.

Based on above and assuming that traffic usage will be residential local or residential collector, the following minimum pavement thickness is recommended:

For residential local roads

40 mm HL3 Asphaltic Concrete

85 mm HL8 Asphaltic Concrete

200 mm Granular ‘A’

235 mm Granular ‘B’

For residential collector roads, the following minimum pavement thickness is recommended:

40 mm HL3 Asphaltic Concrete

100 mm HL Asphaltic Concrete

200 mm Granular ‘A’

325 mm Granular ‘B’

The site subgrade and weather conditions (i.e. if wet) at the time of construction may necessitate the placement of geogrid/filter fabric and/or thicker granular sub-base layer in order to facilitate the construction. Furthermore, heavy construction equipment may have to be kept off the newly constructed roads before the placement of asphalt and/or immediately thereafter, to avoid damaging the weak subgrade by heavy truck traffic.

6.2.1 Stripping, Sub-excavation and Grading

The site should be stripped off all topsoil (if any), loose fill and any organic or otherwise unsuitable soils to the full depth of the roads, both in cut and fill areas under roads.

Following stripping, the site should be graded to the subgrade level and approved. The subgrade should then be proof-rolled in the presence of the Geotechnical Engineer, by at least several passes of a heavy compactor having a rated capacity of at least 8 tonnes. Any soft spots thus exposed should be removed and replaced by select fill material, similar to the existing subgrade soil and approved by the Geotechnical Engineer. The subgrade should then be re-compacted from the surface to at least 98% of its Standard Proctor Maximum Dry Density (SPMDD). The final subgrade should be cambered or otherwise shaped properly to facilitate rapid drainage and to prevent the formation of local depressions in which water could accumulate.

Due to the clayey (i.e. impervious) nature of the subsoil in the upper portions, proper cambering and allowing the water to escape towards the sides (where it can be removed by means of sub-drains) is considered to be beneficial for this project. Otherwise, any water collected in the granular sub-base materials could be trapped thus causing problems due to softened subgrade, differential frost heave, etc. For the same reason damaging the subgrade during and after placement of the granular materials by heavy construction traffic should be avoided. If the moisture content of the local material cannot be maintained at ±2% of the optimum moisture content, imported granular material may be required.

Any fill required for regrading the site or backfill should be select, clean material, free of topsoil, organic or other foreign and unsuitable matter. The fill should be placed in layers and compacted to at least 95% of its SPMDD. The degree of compaction should be increased to 98% within the top 1.0 m of the subgrade. The compaction of the new fill should be checked by sufficient number of field compaction tests.

6.2.2 Construction

Once the subgrade has been inspected and approved, the granular base and sub-base course materials should be placed in layers not exceeding 200 mm (uncompacted thickness) and should be compacted to at least 100% of their respective SPMDD. The grading of the material should conform to current OPS Specifications.

The placing, spreading and rolling of the asphalt should be in accordance with OPS Specifications or, as required by the local authorities.

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

6.2.3 Drainage

Installation of full-length sub-drains is required on all roads. The sub-drains should be properly filtered to prevent the loss of (and clogging by) soil fines.

All paved surfaces should be sloped to provide satisfactory drainage towards Catch Basins. As discussed in Section 6.2, by means of good planning any water trapped in the granular sub-base materials should be drained rapidly towards sub-drains or other interceptors.

6.3 SEWERS

As a part of the site development, a network of new storm and sanitary sewers is to be constructed.

6.3.1 Trenching

As no detail drawing is available for us at the time of writing this report, we estimated that trenches will probably be 3 m to 4 m below the existing ground levels,

As indicated in the boreholes, the trenches will be dug through the fill and sandy clayey silt, sandy silt, silty sand and possible bedrock. Based on the borehole information, groundwater seepage is anticipated during construction in trench to a maximum depth of 4 m. Groundwater control may be established by the use of conventional pumping from collection sumps and ditches for most excavation. However, the effectiveness of this method can only be proven by field pump testing. Or otherwise, a positive dewatering system should be adopted. Please refer to Hydrogeology Study report for detail of the groundwater control.

It should be noted that the till is a non-sorted sediment and therefore may contain boulders. Possible large obstructions such as buried concrete pieces are also anticipated in the fill material. Provisions must be made in the excavation contract for the removal of possible boulders in the till or obstructions in the fill material.

Any loose fill or other unsuitable material below the pipe invert level must be removed and replaced with inorganic material compacted to at least 95% of its Standard Proctor Maximum Dry Density (SPMDD) and to 98% of SPMDD within 0.5 m below the pipe invert level.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the fill and compact to dense sandy soil above the water table can be classified as Type 3 soils. The very stiff to hard clayey silt and very dense sandy silt till/silty sand till deposits above the water table are classified as Type 2 soils.

6.3.2 Bedding

The undisturbed very stiff to hard clayey silt, very dense sandy silt, silt and silty sand as described in Section 4 of this report will provide adequate support for the sewer pipes and allow the use of normal Class B type bedding. Sewer bedding and cover material shall conform to City Standard 2112.090 and 2112.100 respectively. If water is present in the trench excavation then 19 mm clear stone or 6 mm washed crushed gravel is to be used for bedding in accordance with City Standards 2112.110 and 2112.140, respectively. The recommended minimum thickness of granular bedding below the invert of the pipes is 150 mm. The thickness of the bedding may, however, have to be increased depending on the pipe diameter or if wet or weak subgrade conditions are encountered. The bedding material should consist of well graded granular material such as Granular ‘A’ or equivalent. After installing the pipe on the bedding, a granular surround of approved bedding material, which extends at least 300mm above the obvert of the pipe, or as set out by the local Authority, should be placed.

To avoid the loss of soil fines from the subgrade, uniformly graded clear stone should not be used unless, below the granular bedding material, a suitable, approved filter fabric (geotextile) is placed. The geotextile should extend along the sides of the trench and should be wrapped all around the poorly (i.e. uniformly) graded bedding material.

6.3.3 Backfilling of Trenches

Based on visual and tactile examination, the on-site excavated organic free clayey silt, sandy silt, silty sand and silt deposits can generally be re-used as backfill in the service trenches provided their moisture contents at the time of construction are at or near optimum.

The clayey silt is likely to be excavated in cohesive chunks or blocks and will be difficult to compact in confined areas. For use as backfill, the clayey material will have to pulverized and placed in thin layers. The clayey soils will have to be compacted using heavy equipment suitable for these soils which may be difficult to operate in the narrow confines of the trenches. Unless the clayey materials are properly pulverized and compacted in sufficiently thin lifts post-construction settlements could occur.

Trench backfilling shall comply with the City of Mississauga Engineering Policy Statement 4.02.06. The backfill should be placed in maximum 200 mm thick layers at or near (±2%) their optimum moisture content, and each layer should be compacted to at least 95% SPMDD. The degree of compaction should be increased to 98% SPMDD for the top 1 m of the subgrade.

The on-site excavated soils, especially the clayey soils should not be used in confined areas (e.g. around catch basins and laterals under roadways) where heavy compaction equipment cannot be operated. The use of imported granular fill together with an appropriate frost taper would be preferable in confined areas and around structures, such as catch basins.

7.0 SLOPE STABILITY ASSESSMENT

Little Etobicoke Creek runs parallel to the west side of the Site. The Credit Valley Conservation Authority (CVCA) requires that a slope stability analysis be carried out for the ravine slope to determine the development setback limits for the site.

The information obtained from the boreholes was used to analyse the stability of the slope.

On the basis of our fieldwork, laboratory tests and other pertinent information supplied by the client, the following analysis and discussions are made.

7.1 EROSION HAZARD LIMIT

The Ontario Ministry of Natural Resources (MNR) Natural Hazards Policies provided general guidelines for the determination of erosion setback for new developments adjacent to the crest of steep natural slopes (shoreline bluffs, river valleys, ravines). The 100 year Erosion Limit (also known as the Erosion Hazard Limit) is defined as a horizontal line located near the crest of steep slopes which limits the proximity of any new developments to natural hazard areas. The erosion hazard limit (or setback) is determined from the following two (2) setback allowances, measured from the slope toe towards the slope crest:

1. Stable Slope Allowance (stability component of setback)
2. Toe Erosion Allowance (erosion component of setback)

The sum of the stable slope allowance and toe erosion allowance determine the Long Term Stable Top of Slope (LTSTOS) line. An erosion access allowance is added to LTSTOS line to determine the development setback limit.

The stable slope allowance for the subject slopes will be determined by analytical methods to derive the stable slope inclination and the setback distance that is required from the top of slope. For land development and planning, a minimum Factor of Safety of 1.5 is required for engineering design of slopes for stability. This safety factor is used to determine the stability component of the Long Term Stable Top of Slope Line for the subject slope.

The toe erosion allowance is applicable to slopes situated within 15 m distance from a water course.

The erosion access allowance is to ensure that there is enough safety zone along the top of a slope for people and vehicles to enter or exit an area during emergency. However, this allowance may extend into development area provided that the area is not built upon. The erosion access allowance is typically determined by CVCA.

7.2 SLOPE CONDITION

The slopes were inspected by Vic Nersesian, P.Eng. on December 11, 2020. General information pertaining to the existing slope features such as slope profile, slope drainage, water course features, vegetation cover, and erosion features were examined during the inspections.

The height of the slope ranges from approximately 3 to 6 m. Little Etobicoke Creek is situated at/ near the base of the ravine. The bankfull width of the creek ranges from approximately 6 to 7 m. The tableland along the top of the study slope has a relatively flat profile.

The slope is covered with dense vegetation and is well treed with predominantly young and some mature trees. There are some young trees on the slope bank that are slightly inclined away from the slope. Our visual examination of the slope reveals that there is active erosion of the slope face on the central section of the slope, but there is no signs of instability such as tension cracks at the slope crest, slope movements, soil creep, and subsidence on the other sections. There is no bulging or heave close to the toe of the slope. During our inspection of the slope, the slope face was dry with no groundwater seepage visible on the slope face. There is evidence of previous seepage paths from the slope bank.

An armour stone retaining wall is present adjacent to the headwall of the culvert that extends under Dundas Street.

View of the slope at Cross Section A shown on Figure 2.

View of Cross Section B shown on Figure 2- a gabion wall approximately 60 m long is present at the bottom of the slope

The trees are predominantly young mixed with some mature. Some young trees on the slope bank are inclined towards the creek

View of the creek

The creek is generally situated at/near the base of the ravine slope. The bankfull width of the creek is close to 6 to 7 m, and the creek banks are predominantly covered with vegetation with sections being bare and near vertical with evidence of erosion.

View of active erosion of the slope face on the central section of the slope. Note shale bedrock at the bottom 0.5 m height of the slope.

7.3 PROFILES OF EXISTING SLOPE

Based on the survey drawing titled “Top of Bank Survey, West Side of 1580 Dundas Street East, City of Mississauga”, dated January 13, 2021, prepared by R. Avis Surveying Inc., five (5) slope sections were prepared by **Terrapex**. The locations of the slope sections are shown on Figure 2 attached to this report. The following table summarizes the vertical height and inclination angle at each slope section.

Vertical Height and Inclination Angles of Slope Sections

Slope Section	Approximate vertical height of slope	Overall Slope Inclination / degrees to horizontal
A	3.3	1V:2.42H / 22.4
B	5.0	1V:1.3H / 37.6
C	4.8	1V:0.77H / 52.4
D	6.3	1V:1.96H / 27.3
E	2.7	1V:1.52H/ 33.3

7.4 SOIL PROPERTIES AND GROUNDWATER LEVELS USED FOR STABILITY ANALYSES

Soil strength parameters used in the slope stability analyses were based on the results of the in situ Standard Penetration Tests, together with an assessment on the soil type using the results of the Grain Size Analyses.

For analysis of long term stability, the drained shear strength of the soil is expressed in terms of angle of internal friction; not utilizing effective cohesion.

Based on the field tests and laboratory test results, the following soil properties were utilized in the slope stability analyses:

Soil Properties used in the Slope Stability Analyses

Soil Type	Unit Weight (kN/m³)	Angle of Internal Friction (degrees)
Fill	19	28
very stiff to hard Clayey Silt (till)	21	32
dense to very dense Sandy Silt (till)	21	36
hard Shale/Till complex	22	36
Shale Bedrock	25	45

Based on our field observations of the water content of the various soil units, the change in soil colour from brown to grey, and the groundwater measurements from the monitoring wells on January 8, 2021, the groundwater level was considered to be situated at approximate depths ranging from 3.0 to 6.0 mbgs at the borehole locations.

7.5 SLOPE STABILITY ANALYSIS

The stable slope allowance for the subject slope was determined by analytical methods to derive the stable slope inclination and the setback distance that is required from the top of slope. For land development and planning, a minimum Factor of Safety of 1.5 is required for engineering design of slopes for stability. This safety factor is used to determine the stability component of the LTSTOS for the subject slope.

The analysis was carried out using Geo5 Slope Analysis (Version18) software package. The program calculates the minimum factor of safety for moment equilibrium assuming circular failure surfaces. The Bishop method employing effective stress was used to calculate the minimum factor of safety against circular failure. The analyses indicate the safety factors of the slope sections A, B, C, and E are below 1.5 with respect to sliding failure. In this regard, it was necessary to establish the setback distance required from the top of slope for the slope sections A, B, C, and E. Results of the slope stability analyses on slope cross sections are contained in Appendix V and summarized below:

Slope Section	Existing Slope	Stable Slope		Stable Slope Inclination / degrees to horizontal
	Factor of Safety	Set Back from the top of slope(m)	Factor of Safety	
A	1.40	1.4	1.51	1V:2.85H / 19.3
B	1.13	4.1	1.51	1V:2.12H / 25.3
C	0.79	7.2	1.51	1V:2.28H / 23.7
D	1.58	-	-	Existing slope section is stable
E	1.07	3.0	1.54	1V:2.63H / 20.8

7.6 TOE EROSION ALLOWANCE

The toe erosion allowance is applicable to slopes situated within 15 m distance from a water course. Given that the Little Etobicoke Creek is situated near the toe of the subject slope, there is evidence of active erosion of the bank, 5 to 8 m toe erosion allowance is recommended by the Ministry of Natural Resources to be applied for the soil type comprising the slope embankments. Due to good vegetation cover, the absence of significant meanders in the creek, the presence of bedrock at or near the base of the creek, and its narrow width, a toe erosion allowance of 5 m should suffice for the slope.

7.7 EROSION ACCESS ALLOWANCE

The erosion access allowance is to ensure that there is enough safety zone along the top of a slope for people and vehicles to enter or exit an area during emergency or maintenance work. The erosion access allowance is typically determined by CVCA.

7.8 CONCLUSION

Using the stable slope inclinations and toe erosion allowance of 6 m, the LTSTOS Line is plotted on Figure 2 attached to this report. The erosion access allowance determined by CVCA will have to be added to the LTSTOS Line.

8.0 CLOSURE

The conclusion and recommendations in this report are based on information determined at the inspection locations. Soil and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the soil investigation. 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 details of alignment and elevations stated in the report. Since all details of the design may not be known to us, in our analysis certain assumptions had to be made as set out in this report. The actual conditions may, however, vary from those assumed, in which case changes and modifications may be required to our recommendations.

This report was prepared for 4Q Commercial WP Inc. by Terrapex Environmental Ltd. The material in it reflects Terrapex Environmental Ltd. judgement in light of the information available to it at the time of preparation. Any use which a Third Party makes of this report, or any reliance on decisions which the Third Party may make based on it, are the sole responsibility of such Third Parties.

We recommend, therefore, that we be retained during the final design stage to review the design drawings and to verify that they are consistent with our recommendations or the assumptions made in our analysis. We recommend also that we be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the test holes. In cases when these recommendations are not followed, the company's responsibility is limited to accurately interpreting the conditions encountered at the test holes, only.

The comments given in this report on potential construction problems and possible methods are intended for the guidance of the design engineer, only. The number of inspection locations may not be sufficient to determine all the factors that may affect construction methods and costs. 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.

Respectfully submitted,

TERRAPEX ENVIRONMENTAL LTD.

LICENSED PROFESSIONAL ENGINEER
TY
T. YAN
100208861
Nov. 21, 2023
PROVINCE OF ONTARIO

Meysam Najari

Thomas Yan., P.Eng.
Senior Geotechnical Engineer

Meysam Najari, PhD
Vice President, Geotechnical Services

FIGURES

TERRAPEX

BOREHOLE LOCATION PLAN

1590-1650 DUNDAS STREET EAST
MISSISSAUGA, ON

CLIENT

HAZELVIEW INVESTMENTS

LEGEND

- APPROXIMATE SITE BOUNDARY
- MONITORING WELL
- BOREHOLE

0 50m 100m

SOURCE: VUMAP FIRST BASE SOLUTIONS, 2019 IMAGERY. CLIENT PROVIDED DESIGN PLAN.

PROJECT # CA20-149
SCALE AS SHOWN
DATE DECEMBER 2020
DRAWN SW
CHECKED
DRAWING #
FIGURE 1

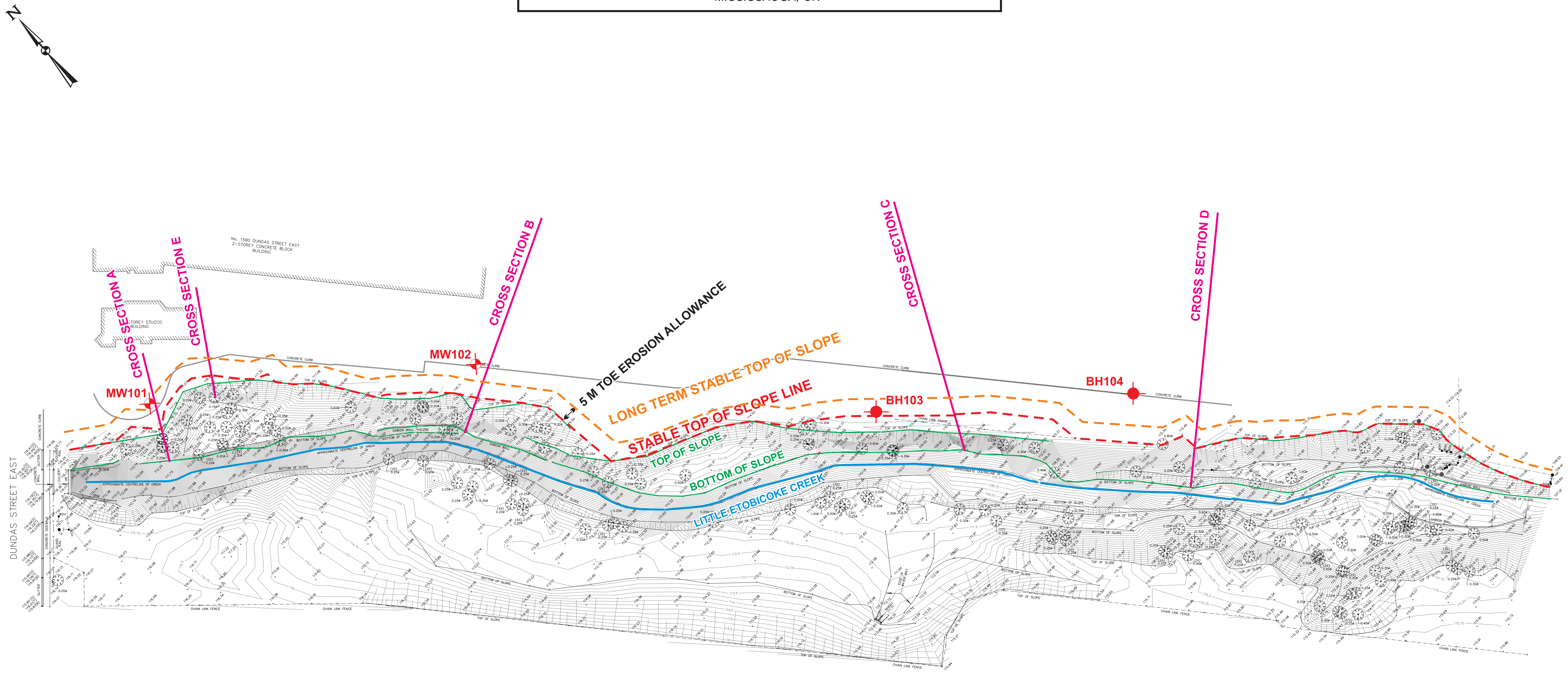
TERRAPEX

SLOPE STABILITY ASSESSMENT

1590-1650 DUNDAS STREET EAST
MISSISSAUGA, ON

CLIENT

HAZELVIEW INVESTMENTS



LEGEND

-·-·- APPROXIMATE SITE BOUNDARY

⌖ MONITORING WELL

● BOREHOLE

SOURCE: CLIENT PROVIDED SURVEY PLAN.

0 30m 60m

(APPROXIMATE)

PROJECT #	CA20-149
SCALE	AS SHOWN
DATE	FEBRUARY 2021
DRAWN	SA/SW
CHECKED	
DRAWING #	FIGURE 2

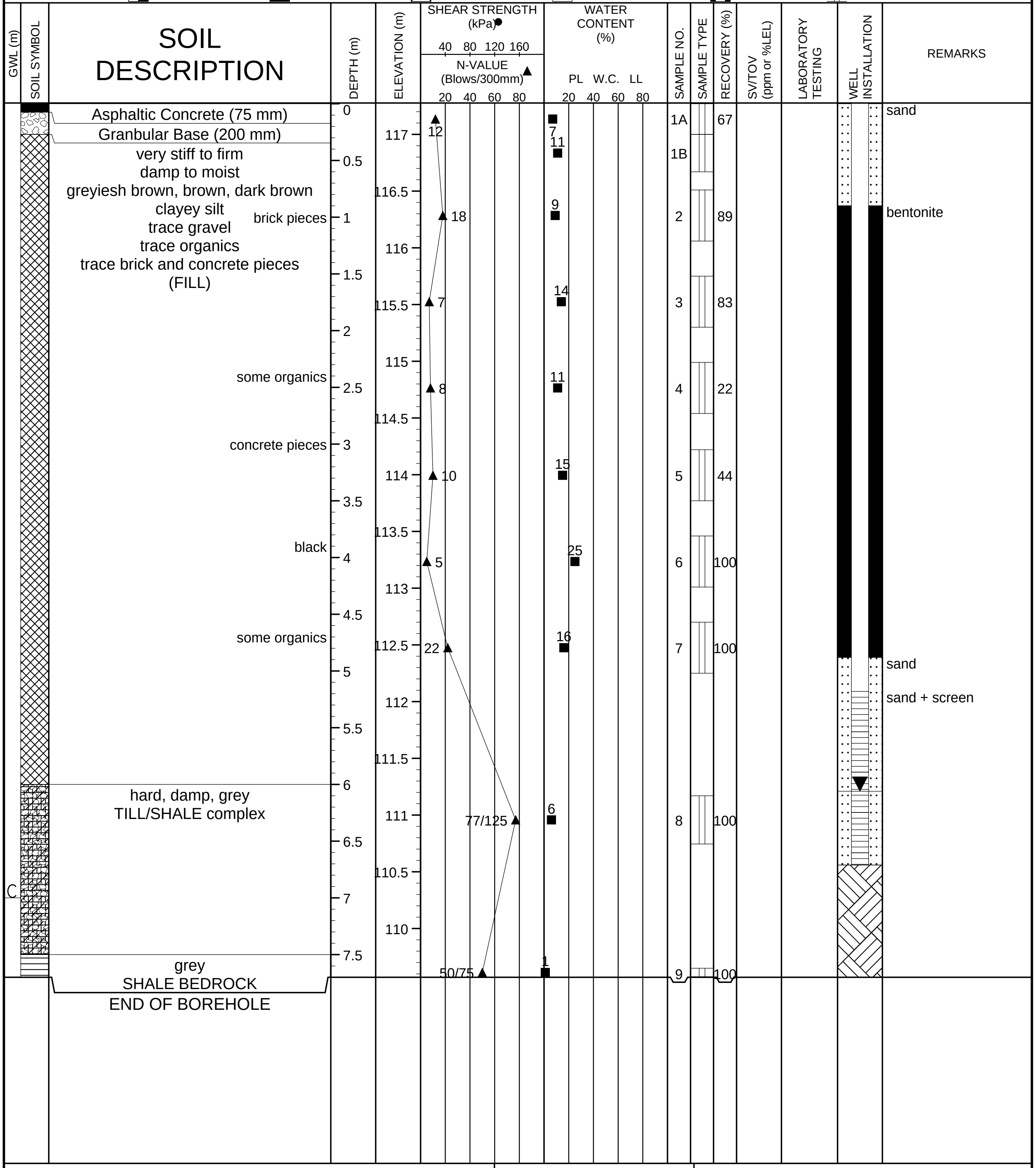
APPENDIX I

BOREHOLE LOG SHEETS

CLIENT: Hazleview Investment
PROJECT NO.: CA20-149
ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 117.27
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Solid Stem Auger and Split SPoon Sampling
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm): 5
SCREEN SLOT #: 10
SAND TYPE: 2
SEALANT TYPE: Holeplug

RECORD OF:
MW101

SAMPLE TYPE ▐ AUGER ◢ DRIVEN ⧓ CORING ▀ DYNAMIC CONE ▌▐ SHELBY ‖ SPLIT SPOON



Asphaltic Concrete (75 mm)

Granbular Base (200 mm)

very stiff to firm
damp to moist
greyiesh brown, brown, dark brown
clayey silt
trace gravel
trace organics
trace brick and concrete pieces
(FILL)

brick pieces

some organics

concrete pieces

black

some organics

hard, damp, grey
TILL/SHALE complex

grey
SHALE BEDROCK

END OF BOREHOLE

SAMPLE NO.	N-VALUE	WATER CONTENT (%)	RECOVERY (%)
1A	12	7	67
1B		11	
2	18	9	89
3	7	14	83
4	8	11	22
5	10	15	44
6	5	25	100
7	22	16	100
8	77/125	6	100
9	50/75	1	100

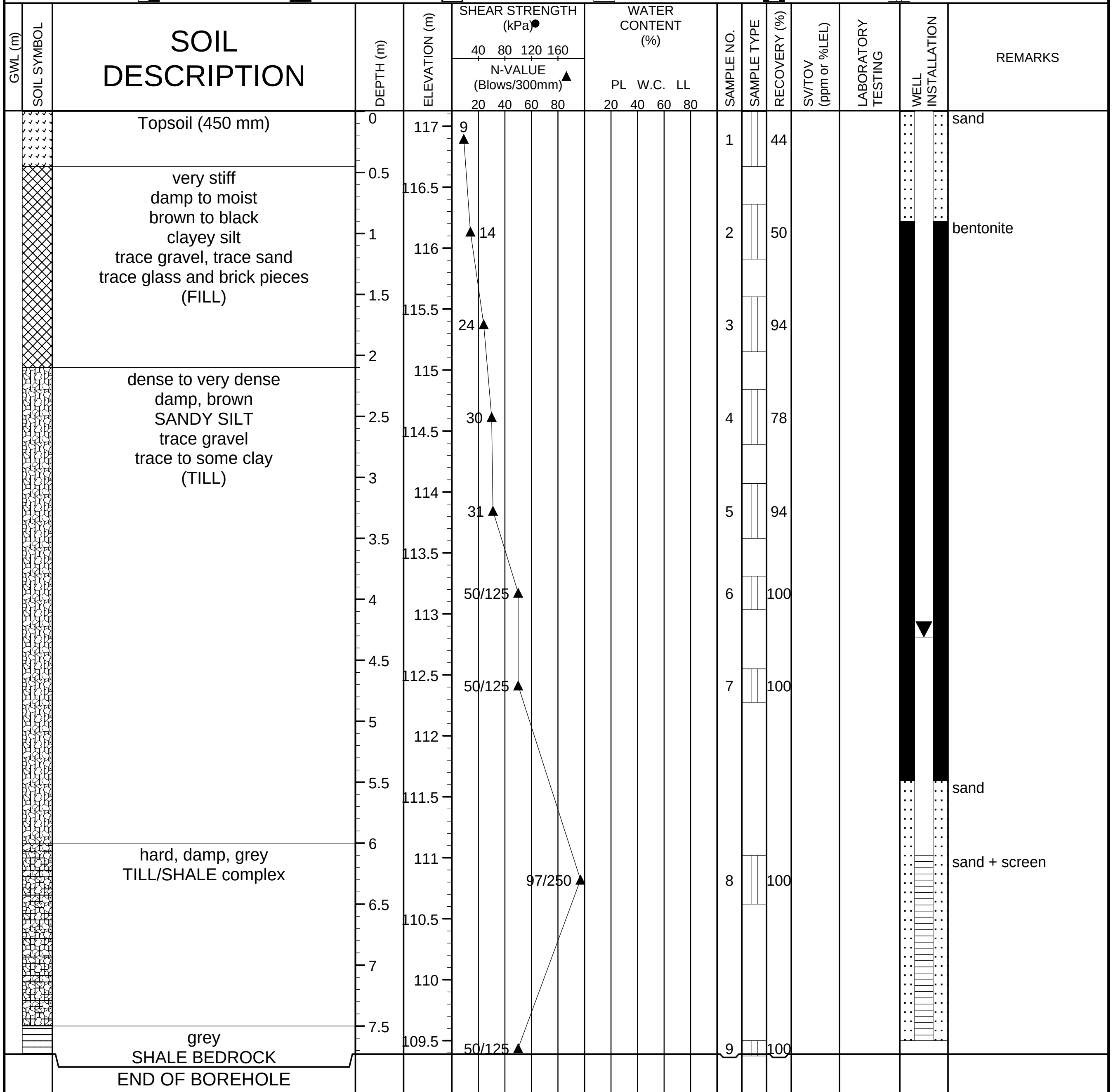
REMARKS:
sand
bentonite
sand
sand + screen

TERRAPEX

LOGGED BY: RG
DRILLING DATE: 16-Dec-2020
INPUT BY: SA
MONITORING DATE: 08-Jan-2021
REVIEWED BY: VN
PAGE 1 OF 1

CLIENT: Hazleview Investment	PROJECT NO.: CA20-149	**RECORD OF:**
ADDRESS: 1590 & 1650 Dundas Street East	STATION:	**MW102**
CITY/PROVINCE: Toronto, Ontario	NORTHING (m): EASTING (m):	ELEV. (m) 117.12
CONTRACTOR: Pontil Drilling Services Inc.	METHOD: Hollow Stem Auger and Split SPoon Sampling	
BOREHOLE DIAMETER (cm): 15	WELL DIAMETER (cm): 5	SCREEN SLOT #: 10 SAND TYPE: 2 SEALANT TYPE: Holeplug

SAMPLE TYPE ▐ AUGER ◢ DRIVEN ⧗ CORING ▀ DYNAMIC CONE ▐▌ SHELBY ‖ SPLIT SPOON



SOIL DESCRIPTION
Topsoil (450 mm)
very stiff damp to moist brown to black clayey silt trace gravel, trace sand trace glass and brick pieces (FILL)
dense to very dense damp, brown SANDY SILT trace gravel trace to some clay (TILL)
hard, damp, grey TILL/SHALE complex
grey SHALE BEDROCK

END OF BOREHOLE

SAMPLE NO.	N-VALUE	RECOVERY (%)
1	9	44
2	14	50
3	24	94
4	30	78
5	31	94
6	50/125	100
7	50/125	100
8	97/250	100
9	50/125	100

REMARKS:
sand
bentonite
sand
sand + screen

LOGGED BY: RG	DRILLING DATE: 16-Dec-2020
INPUT BY: SA	MONITORING DATE: 08-Jan-2021
REVIEWED BY: VN	PAGE 1 OF 1

CLIENT: Hazleview Investment
PROJECT NO.: CA20-149

RECORD OF:
BH103

ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 115.56
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Hollow Stem Auger and Split SPoon Sampling
BOREHOLE DIAMETER (cm): 20
WELL DIAMETER (cm):
SCREEN SLOT #:
SAND TYPE: 2
SEALANT TYPE: Holeplug

SAMPLE TYPE ▐ AUGER ◢ DRIVEN ⧗ CORING ▀ DYNAMIC CONE █ SHELBY ‖ SPLIT SPOON

GWL (m)	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)	ELEVATION (m)	SHEAR STRENGTH (kPa) ● 40 80 120 160 N-VALUE (Blows/300mm) ▲ 20 40 60 80	WATER CONTENT (%) PL W.C. LL 20 40 60 80	SAMPLE NO.	SAMPLE TYPE	RECOVERY (%)	SV/TOV (ppm or %LEL)	LABORATORY TESTING	WELL INSTALLATION	REMARKS
	⁘	Topsoil (450 mm)	0	115.5	▲ 6		1	‖	100				
	▩	loose to compact damp to moist brown sand and gravel some concrete pieces (FILL)	0.5	115									
	▩		1	114.5	▲ 8		2	‖	39				
	▩		1.5	114	▲ 6		3	‖	22				
	▩		2	113.5									
	▩		2.5	113	▲ 12		4	‖	-				
		END OF BOREHOLE											Auger Refusal due to concrete

TERRAPEX

LOGGED BY: RG
DRILLING DATE: 21-Dec-2020
INPUT BY: SA
MONITORING DATE:
REVIEWED BY: VN
PAGE 1 OF 1

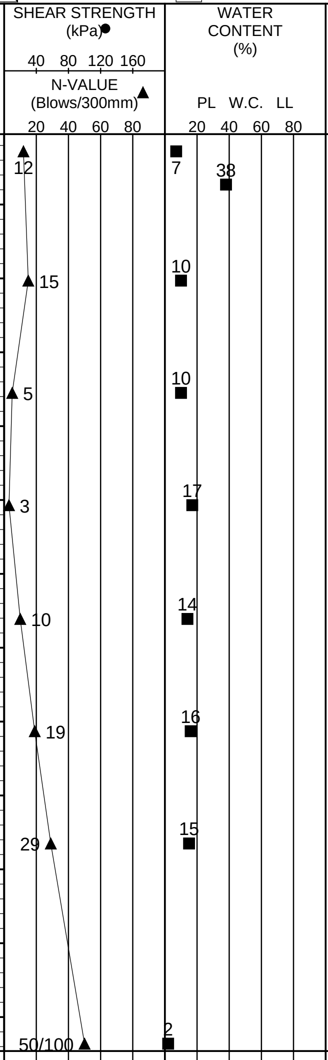
CLIENT: Hazleview Investment
PROJECT NO.: CA20-149

RECORD OF:
BH104

ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 115.47
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Solid Stem Auger and Split SPoon Sampling
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm):
SCREEN SLOT #:
SAND TYPE:
SEALANT TYPE: Holeplug
SAMPLE TYPE: AUGER, DRIVEN, CORING, DYNAMIC CONE, SHELBY, SPLIT SPOON

SOIL DESCRIPTION
Asphaltic Concrete (80 mm)
Granular Base (150 mm)
compact to loose damp to moist greyish brown sandy silt (FILL)
soft to stiff moist dark brown to black clayey silt trace organics (FILL)
very stiff damp, brown CLAYEY SILT trace gravel, trace sand (TILL)
grey SHALE BEDROCK
END OF BOREHOLE

DEPTH (m)	ELEVATION (m)
0	
0.5	115
1	114.5
1.5	114
2	113.5
2.5	113
3	112.5
3.5	112
4	111.5
4.5	111
5	110.5
5.5	110
6	109.5



SAMPLE NO.	N-VALUE (Blows/300mm)	WATER CONTENT (%)	RECOVERY (%)
1A	12	7	72
1B		38	
2	15	10	55
3	5	10	44
4	3	17	67
5	10	14	22
6	19	16	78
7	29	15	78
8	50/100	2	100

GWL (m) | SOIL SYMBOL | SAMPLE TYPE | SV/TOV (ppm or %LEL) | LABORATORY TESTING | WELL INSTALLATION | REMARKS

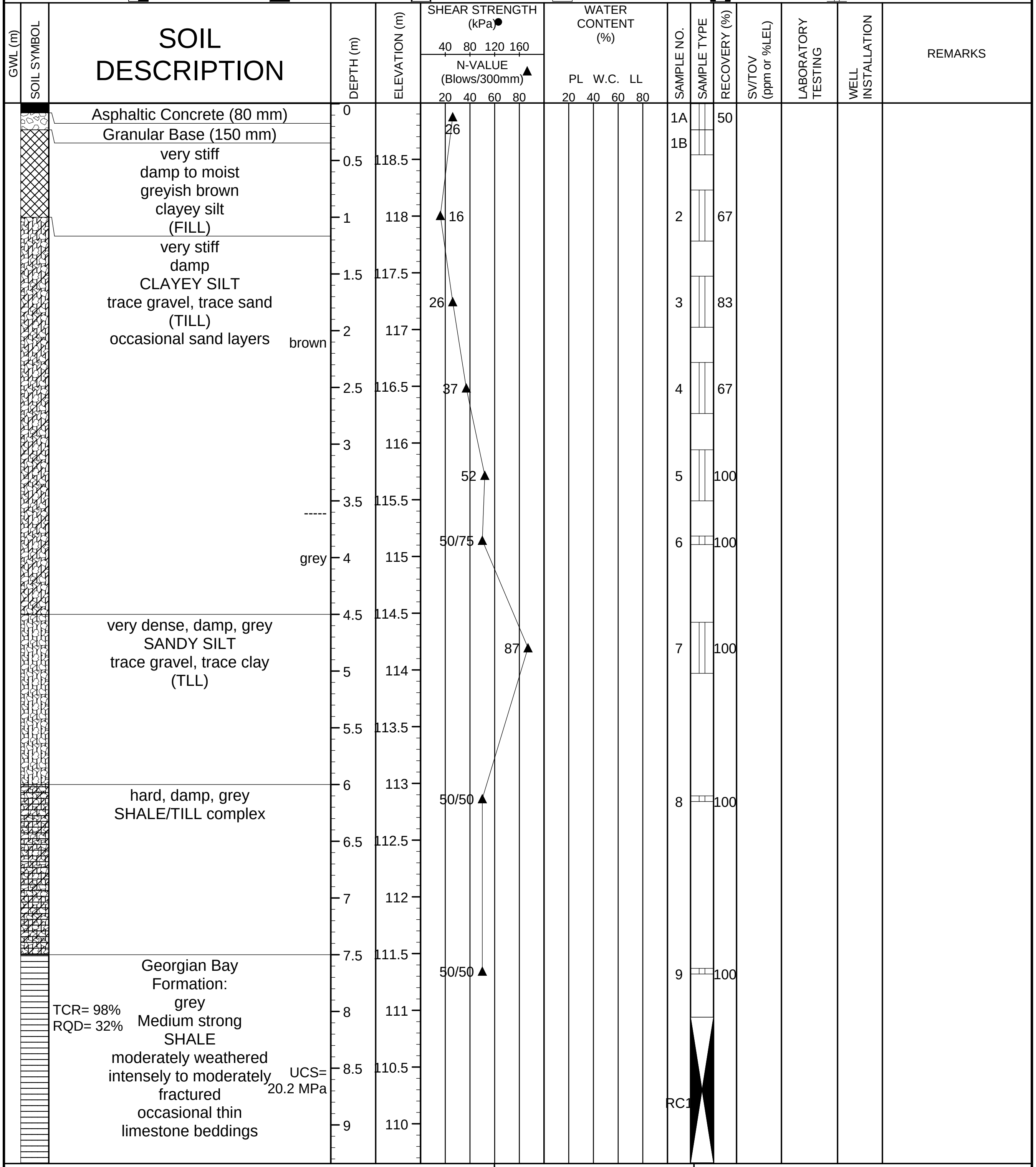
TERRAPEX

LOGGED BY: RG
DRILLING DATE: 16-Dec-2020
INPUT BY: SA
MONITORING DATE:
REVIEWED BY: VN
PAGE 1 OF 1

CLIENT: Hazleview Investment
PROJECT NO.: CA20-149

RECORD OF:
BH105

ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 118.99
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Solid Stem Auger and Split SPoon Sampling
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm):
SCREEN SLOT #:
SAND TYPE:
SEALANT TYPE: Holeplug
SAMPLE TYPE ▮ AUGER ◪ DRIVEN ⧓ CORING ▬ DYNAMIC CONE ▯ SHELBY ⌶ SPLIT SPOON



DEPTH (m)	ELEVATION (m)
0	
0.5	118.5
1	118
1.5	117.5
2	117
2.5	116.5
3	116
3.5	115.5
4	115
4.5	114.5
5	114
5.5	113.5
6	113
6.5	112.5
7	112
7.5	111.5
8	111
8.5	110.5
9	110

SOIL DESCRIPTION

Asphaltic Concrete (80 mm)

Granular Base (150 mm)

very stiff
damp to moist
greyish brown
clayey silt
(FILL)

very stiff
damp
CLAYEY SILT
trace gravel, trace sand
(TILL)
occasional sand layers

brown
-----
grey

very dense, damp, grey
SANDY SILT
trace gravel, trace clay
(TLL)

hard, damp, grey
SHALE/TILL complex

Georgian Bay
Formation:
grey
Medium strong
SHALE
moderately weathered
intensely to moderately
fractured
occasional thin
limestone beddings

TCR= 98%
RQD= 32%

UCS=
20.2 MPa

SAMPLE NO.	N-VALUE	RECOVERY (%)
1A	26	50
1B		
2	16	67
3	26	83
4	37	67
5	52	100
6	50/75	100
7	87	100
8	50/50	100
9	50/50	100
RC1		

TERRAPEX

LOGGED BY: RG
DRILLING DATE: 22-Dec-2020
INPUT BY: SA
MONITORING DATE:
REVIEWED BY: VN
PAGE 1 OF 2

CLIENT: Hazleview Investment
PROJECT NO.: CA20-149
RECORD OF:
BH105
ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 118.99
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Solid Stem Auger and Split SPoon Sampling
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm):
SCREEN SLOT #:
SAND TYPE:
SEALANT TYPE: Holeplug
SAMPLE TYPE ▮ AUGER ◢ DRIVEN ⋈ CORING ▀ DYNAMIC CONE ▯ SHELBY ‖ SPLIT SPOON

GWL (m)	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)	ELEVATION (m)	SHEAR STRENGTH (kPa) 40 80 120 160 N-VALUE (Blows/300mm) 20 40 60 80	▲ WATER CONTENT (%) PL W.C. LL 20 40 60 80	SAMPLE NO.	SAMPLE TYPE	RECOVERY (%)	SV/TOV (ppm or %LEL)	LABORATORY TESTING	WELL INSTALLATION	REMARKS
	▤	TCR= 98% RQD= 41% Georgian Bay Formation: grey Medium strong SHALE moderately weathered intensely to moderately fractured occasional thin limestone beddings occasional thin clay seams	9.5	109.5				⋈					
	▤		10	109				⋈					
	▤		10.5	108.5			RC2	⋈					
	▤	TCR= 100% RQD= 58%	11	108				⋈					
	▤		11.5	107.5				⋈					
	▤		12	107			RC3	⋈					
	▤		12.5	106.5				⋈					
		END OF BOREHOLE											

TERRAPEX
LOGGED BY: RG
DRILLING DATE: 22-Dec-2020
INPUT BY: SA
MONITORING DATE:
REVIEWED BY: VN

CLIENT: Hazleview Investment
PROJECT NO.: CA20-149

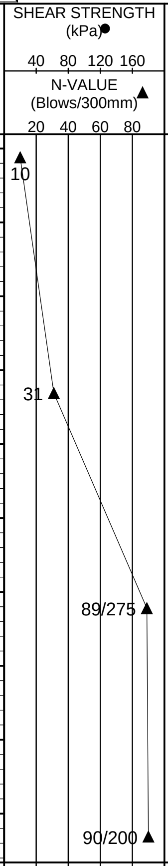
RECORD OF:
BH106

ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 117.09
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Solid Stem Auger and Split SPoon Sampling
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm):
SCREEN SLOT #:
SAND TYPE:
SEALANT TYPE: Holeplug

SAMPLE TYPE ▌ AUGER ◢ DRIVEN ⧓ CORING ▀ DYNAMIC CONE ▐▌ SHELBY ‖ SPLIT SPOON

GWL (m)	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)	ELEVATION (m)	N-VALUE (Blows/300mm) ▲	SAMPLE NO.	SAMPLE TYPE	RECOVERY (%)	SV/TOV (ppm or %LEL)	LABORATORY TESTING	WELL INSTALLATION	REMARKS
		Asphaltic Concrete (75 mm)	0	117	10	1A	‖	67				
		Granular Base (225 mm)				1B	‖					
		stiff damp to moist greyish brown clayey silt (FILL)	0.5 1	116.5 116								
		hard damp, brown CLAYEY SILT trace gravel, trace sand (TILL)	1.5 2 2.5 3	115.5 115 114.5 114	31 89/275	2 3A	‖ ‖	61 94				
		hard, damp, grey SHALE/TILL complex	3.5 4	113.5 113		3B	‖					
		grey SHALE BEDROCK	4.5	112.5	90/200	4	‖	57				

END OF BOREHOLE



WATER CONTENT (%)
PL W.C. LL
20 40 60 80

TERRAPEX

LOGGED BY: RG
DRILLING DATE: 23-Dec-2020
INPUT BY: SA
MONITORING DATE:
REVIEWED BY: VN
PAGE 1 OF 1

CLIENT: Hazleview Investment	PROJECT NO.: CA20-149	**RECORD OF:**
ADDRESS: 1590 & 1650 Dundas Street East	STATION:	**MW107**
CITY/PROVINCE: Toronto, Ontario	NORTHING (m): EASTING (m):	ELEV. (m) 115.78
CONTRACTOR: Pontil Drilling Services Inc.	METHOD: Hollow Stem Auger and Split SPoon Sampling	
BOREHOLE DIAMETER (cm): 15 WELL DIAMETER (cm): 5	SCREEN SLOT #: 10 SAND TYPE: 2	SEALANT TYPE: Holeplug

SAMPLE TYPE: AUGER, DRIVEN, CORING, DYNAMIC CONE, SHELBY, SPLIT SPOON

GWL (m) | SOIL SYMBOL | SOIL DESCRIPTION | DEPTH (m) | ELEVATION (m)

Asphaltic Concrete (105 mm)

Granular Base (500 mm)

stiff, damp to moist
grey to black
clayey silt
trace organic (FILL)

stiff, damp, greyish brown
CLAYEY SILT
trace organics

dense, damp, brown
SANDY SILT
trace gravel, trace clay
(TILL)

very dense, damp, grey
SILT, trace sand

hard, damp, grey
TILL/SHALE complex

Georgian Bay
Formation:
grey
Medium strong
SHALE
moderately weathered
intensely to moderately
fractured
occasional thin
limestone beddings
occasional thin
clay seams

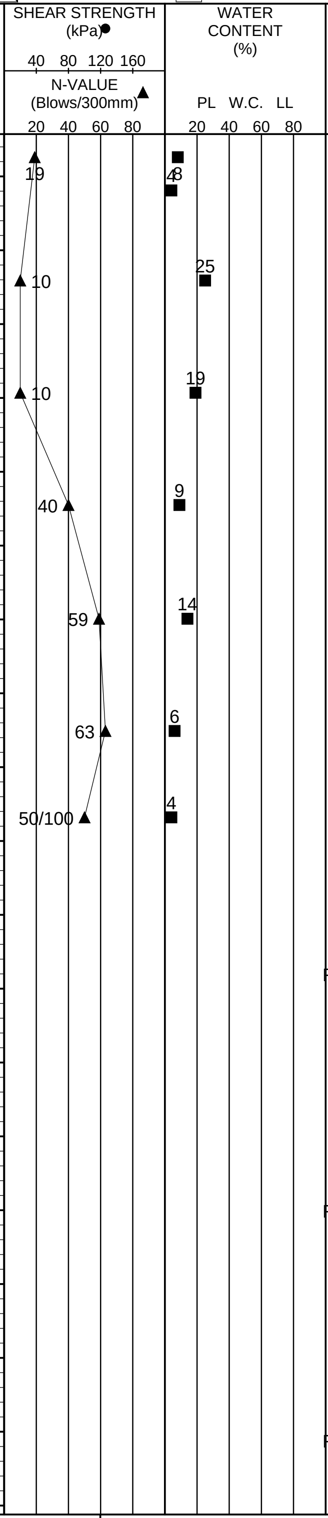
TCR= 100%
RCD= 23%

UCS=
18.0 MPa

TCR= 100%
RCD= 23%

TCR= 100%
RCD= 72%

UCS=
18.8 MPa



SAMPLE NO.	N-VALUE	W.C.	RECOVERY (%)
1A	19	8	67
1B		4	
2	10	25	67
3	10	19	55
4	40	9	94
5	59	14	59
6	63	6	33
7	50/100	4	100
RC1			
RC2			
RC3			

SV/TOV (ppm or %LEL) | LABORATORY TESTING | WELL INSTALLATION | REMARKS

sand

bentonite

sand

sand + screen

LOGGED BY: RG	DRILLING DATE: 21-Dec-2020
INPUT BY: SA	MONITORING DATE: 08-Jan-2021
REVIEWED BY: VN	PAGE 1 OF 2

TERRAPEX

CLIENT: Hazleview Investment
PROJECT NO.: CA20-149

RECORD OF:
MW107

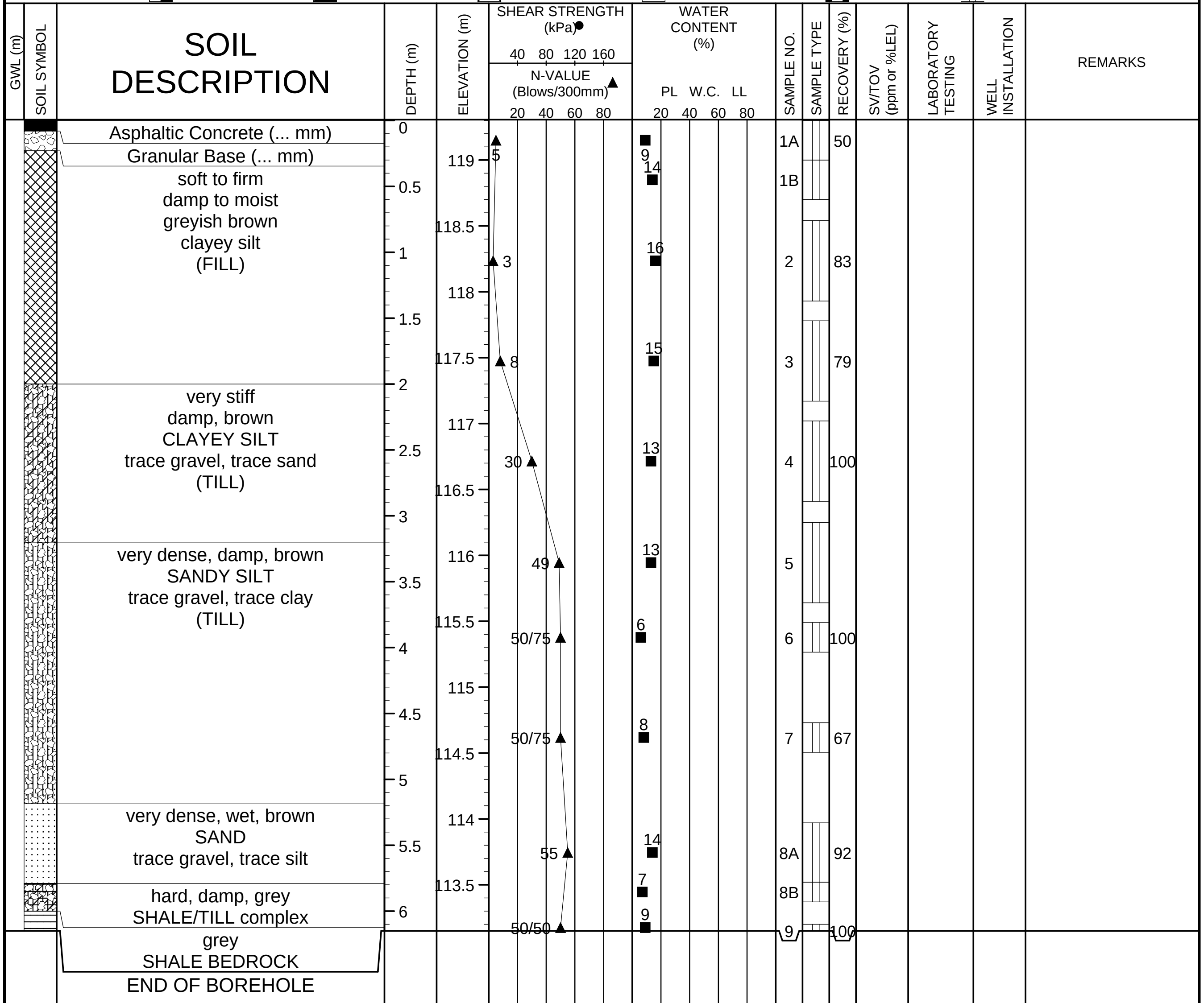
ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 115.78
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Hollow Stem Auger and Split SPoon Sampling
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm): 5
SCREEN SLOT #: 10
SAND TYPE: 2
SEALANT TYPE: Holeplug
SAMPLE TYPE: AUGER; DRIVEN; CORING; DYNAMIC CONE; SHELBY; SPLIT SPOON

GWL (m)	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)	ELEVATION (m)	SHEAR STRENGTH (kPa) 40 80 120 160 N-VALUE (Blows/300mm) 20 40 60 80	▲ WATER CONTENT (%) PL W.C. LL 20 40 60 80	SAMPLE NO.	SAMPLE TYPE	RECOVERY (%)	SV/TOV (ppm or %LEL)	LABORATORY TESTING	WELL INSTALLATION	REMARKS
		TCR= 100% RCD= 71% Georgian Bay Formation: grey Medium strong SHALE moderately weathered intensely to moderately fractured occasional thin limestone beddings occasional thin clay seams	9.5 10 10.5 11	106 105.5 105			RC4	CORING					
		END OF BOREHOLE											

LOGGED BY: RG
DRILLING DATE: 21-Dec-2020
INPUT BY: SA
MONITORING DATE: 08-Jan-2021
REVIEWED BY: VN

CLIENT: Hazleview Investment	PROJECT NO.: CA20-149	**RECORD OF:** **BH108**
ADDRESS: 1590 & 1650 Dundas Street East	STATION:	
CITY/PROVINCE: Toronto, Ontario	NORTHING (m): EASTING (m):	ELEV. (m) 119.30
CONTRACTOR: Pontil Drilling Services Inc.	METHOD: Solid Stem Auger and Split SPoon Sampling	
BOREHOLE DIAMETER (cm): 15	WELL DIAMETER (cm): SCREEN SLOT #: SAND TYPE:	SEALANT TYPE: Holeplug

SAMPLE TYPE: AUGER, DRIVEN, CORING, DYNAMIC CONE, SHELBY, SPLIT SPOON



SOIL DESCRIPTION

- Asphaltic Concrete (... mm)
- Granular Base (... mm)
- soft to firm damp to moist greyish brown clayey silt (FILL)
- very stiff damp, brown CLAYEY SILT trace gravel, trace sand (TILL)
- very dense, damp, brown SANDY SILT trace gravel, trace clay (TILL)
- very dense, wet, brown SAND trace gravel, trace silt
- hard, damp, grey SHALE/TILL complex
- grey SHALE BEDROCK
- END OF BOREHOLE

SAMPLE NO.	N-VALUE	WATER CONTENT (%)	RECOVERY (%)
1A	5	9	50
1B		14	
2	3	16	83
3	8	15	79
4	30	13	100
5	49	13	
6	50/75	6	100
7	50/75	8	67
8A	55	14	92
8B		7	
9	50/50	9	100

TERRAPEX

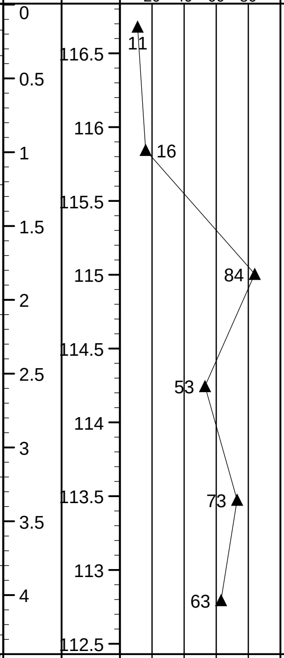
LOGGED BY: RG	DRILLING DATE: 15-Dec-2020
INPUT BY: SA	MONITORING DATE:
REVIEWED BY: VN	PAGE 1 OF 1

CLIENT: Hazleview Investment
PROJECT NO.: CA20-149

RECORD OF:
BH109

ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 116.83
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Solid Stem Auger and Split SPoon Sampling
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm):
SCREEN SLOT #:
SAND TYPE:
SEALANT TYPE: Holeplug
SAMPLE TYPE: AUGER, DRIVEN, CORING, DYNAMIC CONE, SHELBY, SPLIT SPOON

GWL (m)	SOIL DESCRIPTION	DEPTH (m)	ELEVATION (m)
	Asphaltic Concrete (100 mm)	0	116.83
	Granular Base (200 mm)		
	stiff to very stiff damp to moist greyish brown clayey silt trace asphalt pieces (FILL)		
	very dense damp, brown SANDY SILT trace gravel, trace clay (TILL)		
	very dense, wet greyish brown SILTY SAND		
	very dense, damp, brown SANDY SILT trace gravel, trace clay (TILL)		
	hard, damp, grey SHALE/TILL complex		
	grey SHALE BEDROCK END OF BOREHOLE		



WATER CONTENT (%): PL W.C. LL — 20 40 60 80

SAMPLE NO.	SAMPLE TYPE	RECOVERY (%)	N-VALUE	SV/TOV (ppm or %LEL)	LABORATORY TESTING	WELL INSTALLATION	REMARKS
1A		58	11				
1B							
2A		83	16				
2B							
3		83	84				
4		92	53				
5		100	73				
6A		92	63				
6B							

TERRAPEX

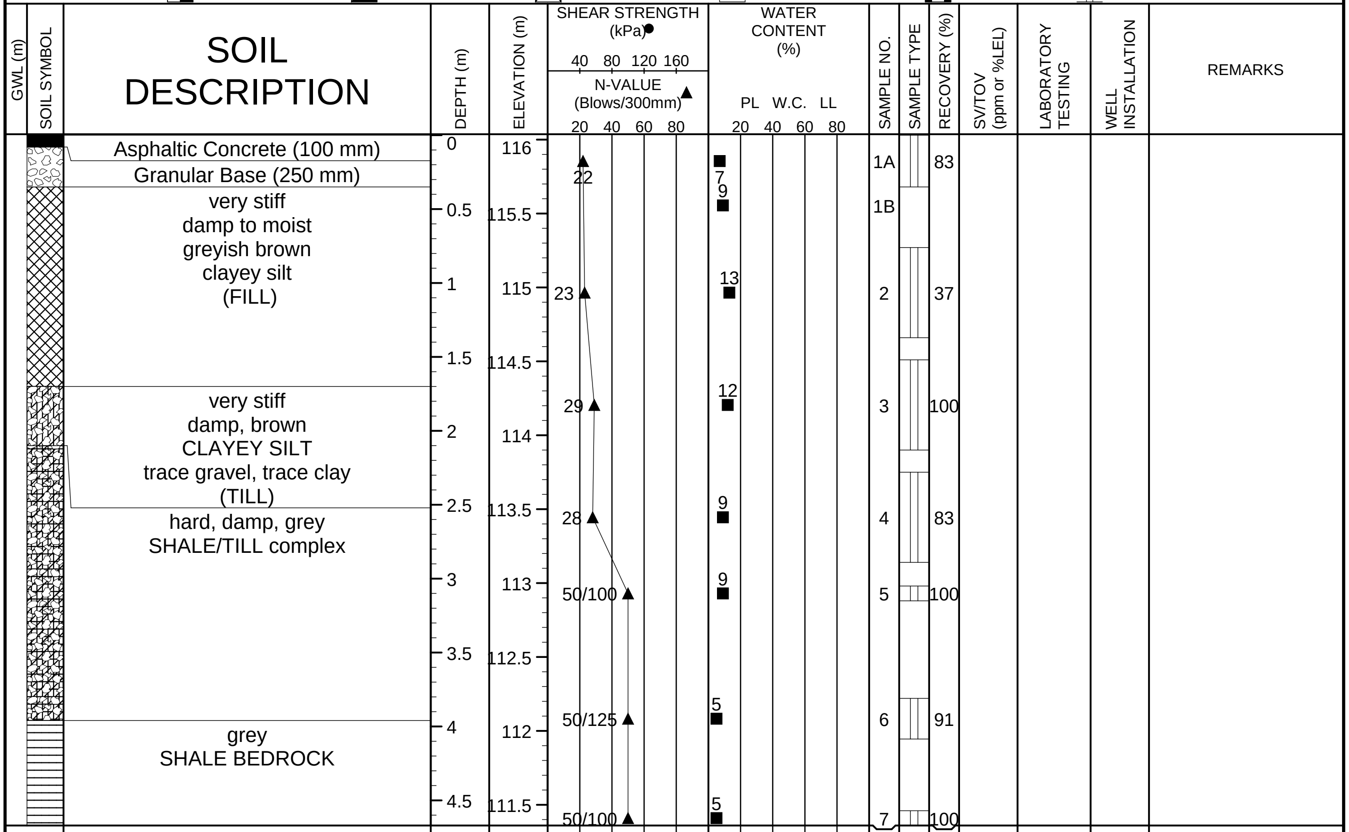
LOGGED BY: RG
DRILLING DATE: 15-Dec-2020
INPUT BY: SA
MONITORING DATE:
REVIEWED BY: VN
PAGE 1 OF 1

CLIENT: Hazleview Investment
PROJECT NO.: CA20-149

RECORD OF:
BH110

ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 116.03
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Solid Stem Auger and Split SPoon Sampling
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm):
SCREEN SLOT #:
SAND TYPE:
SEALANT TYPE: Holeplug

SAMPLE TYPE ▮ AUGER ◢ DRIVEN ⧗ CORING ▀ DYNAMIC CONE ▯ SHELBY ‖ SPLIT SPOON



SOIL DESCRIPTION
Asphaltic Concrete (100 mm)
Granular Base (250 mm)
very stiff damp to moist greyish brown clayey silt (FILL)
very stiff damp, brown CLAYEY SILT trace gravel, trace clay (TILL)
hard, damp, grey SHALE/TILL complex
grey SHALE BEDROCK
END OF BOREHOLE

SAMPLE NO.	N-VALUE (Blows/300mm)	WATER CONTENT (%)	RECOVERY (%)
1A	22	7	83
1B		9	
2	23	13	37
3	29	12	100
4	28	9	83
5	50/100	9	100
6	50/125	5	91
7	50/100	5	100

TERRAPEX

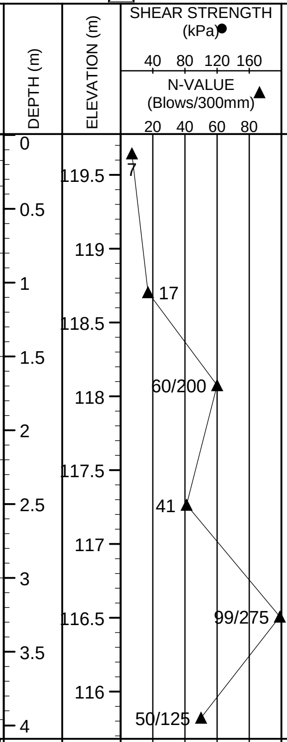
LOGGED BY: RG
DRILLING DATE: 15-Dec-2020
INPUT BY: SA
MONITORING DATE:
REVIEWED BY: VN
PAGE 1 OF 1

CLIENT: Hazleview Investment
PROJECT NO.: CA20-149

RECORD OF:
MW111

ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 119.77
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Solid Stem Auger and Split SPoon Sampling
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm):
SCREEN SLOT #:
SAND TYPE:
SEALANT TYPE: Holeplug
SAMPLE TYPE: AUGER, DRIVEN, CORING, DYNAMIC CONE, SHELBY, SPLIT SPOON

SOIL DESCRIPTION	SAMPLE NO.	RECOVERY (%)	N-VALUE (Blows/300mm)	REMARKS
Asphaltic Concrete (100 mm)	1A	62	7	bentonite
Granular Base (150 mm)				
firm, damp to moist, greyish brown clayey silt (FILL)	1B			
very stiff, damp, greyish brown CLAYEY SILT trace gravel, trace sand (TILL)	2	83	17	sand
hard, damp, greyish brown CLAYEY SILT	3	100	60/200	sand + screen
hard damp, greyish brown CLAYEY SILT trace gravel, trace sand (TILL)	4	100	41	
hard, damp, grey SHALE/TILL complex	5	100	99/275	
	6	100	50/125	
grey SHALE BEDROCK				
END OF BOREHOLE				



LOGGED BY: RG
DRILLING DATE: 16-Dec-2020
INPUT BY: SA
MONITORING DATE:
REVIEWED BY: VN
PAGE 1 OF 1

CLIENT: Hazleview Investment
PROJECT NO.: CA20-149

RECORD OF:
MW112

ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 117.06
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Solid Stem Auger and Split SPoon Sampling
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm): 5
SCREEN SLOT #: 10
SAND TYPE: 2
SEALANT TYPE: Holeplug
SAMPLE TYPE ▮ AUGER ◢ DRIVEN ⧓ CORING ▀ DYNAMIC CONE ▮ SHELBY ▯ SPLIT SPOON

SOIL DESCRIPTION

Asphaltic Concrete (125 mm)

Granbular Base (225 mm)

firm, damp, brown
clayey silt (FILL)

very stiff, damp, brown
CLAYEY SILT
trace gravel, trace sand
(TILL)

hard, damp, grey
TILL/SHALE complex

Georgian Bay
Formation:
grey
Medium strong
SHALE
moderately weathered
intensely to moderately
fractured
occasional thin
limestone beddings
occasional thin
clay seams

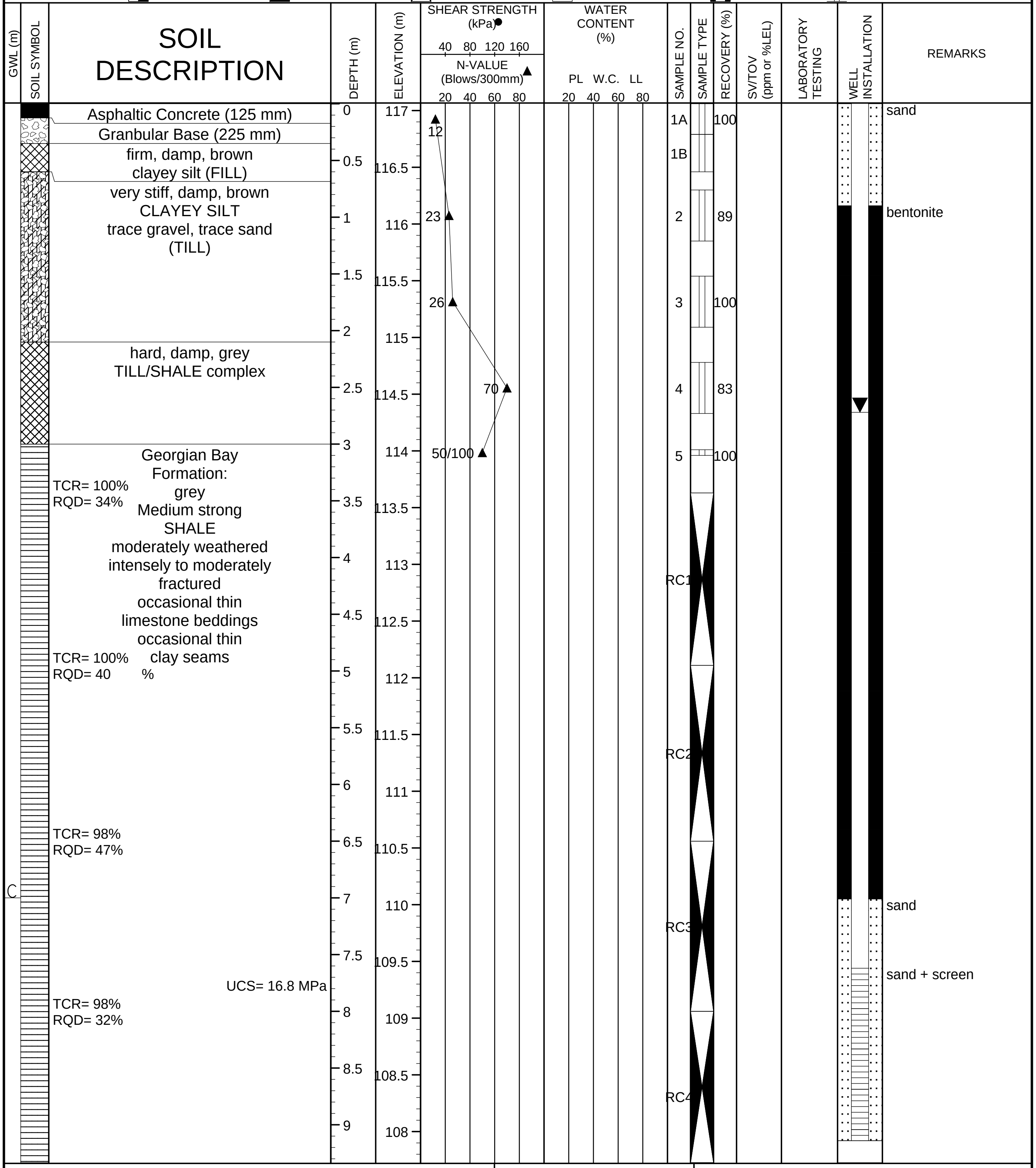
TCR= 100%
RQD= 34%

TCR= 100%
RQD= 40 %

TCR= 98%
RQD= 47%

UCS= 16.8 MPa

TCR= 98%
RQD= 32%



LOGGED BY: RG
DRILLING DATE: 16-Dec-2020
INPUT BY: SA
MONITORING DATE: 08-Jan-2021
REVIEWED BY: VN
PAGE 1 OF 2

CLIENT: Hazleview Investment

PROJECT NO.: CA20-149

RECORD OF:
MW112

ADDRESS: 1590 & 1650 Dundas Street East

STATION:

CITY/PROVINCE: Toronto, Ontario

NORTHING (m):

EASTING (m):

ELEV. (m) 117.06

CONTRACTOR: Pontil Drilling Services Inc.

METHOD: Solid Stem Auger and Split SPoon Sampling

BOREHOLE DIAMETER (cm): 15

WELL DIAMETER (cm): 5

SCREEN SLOT #: 10

SAND TYPE: 2

SEALANT TYPE: Holeplug

SAMPLE TYPE ◧ AUGER ◢ DRIVEN ⋈ CORING ⬒ DYNAMIC CONE ▯ SHELBY ǁ SPLIT SPOON

GWL (m)	SOIL SYMBOL	SOIL DESCRIPTION	DEPTH (m)	ELEVATION (m)	SHEAR STRENGTH (kPa) 40 80 120 160 / N-VALUE (Blows/300mm) 20 40 60 80	▲ WATER CONTENT (%) PL W.C. LL 20 40 60 80	SAMPLE NO.	SAMPLE TYPE	RECOVERY (%)	SV/TOV (ppm or %LEL)	LABORATORY TESTING	WELL INSTALLATION	REMARKS
	▤	TCR= 100% RQD= 32% Georgian Bay Formation: grey Medium strong SHALE moderately weathered intensely to moderately fractured occasional thin limestone beddings occasional thin clay seams TCR= 87% RQD= 37%	9.5 10 10.5 11 11.5 12	107.5 107 106.5 106 105.5 105			RC5 RC6	⋈ ⋈					
		END OF BOREHOLE											

TERRAPEX

LOGGED BY: RG

DRILLING DATE: 16-Dec-2020

INPUT BY: SA

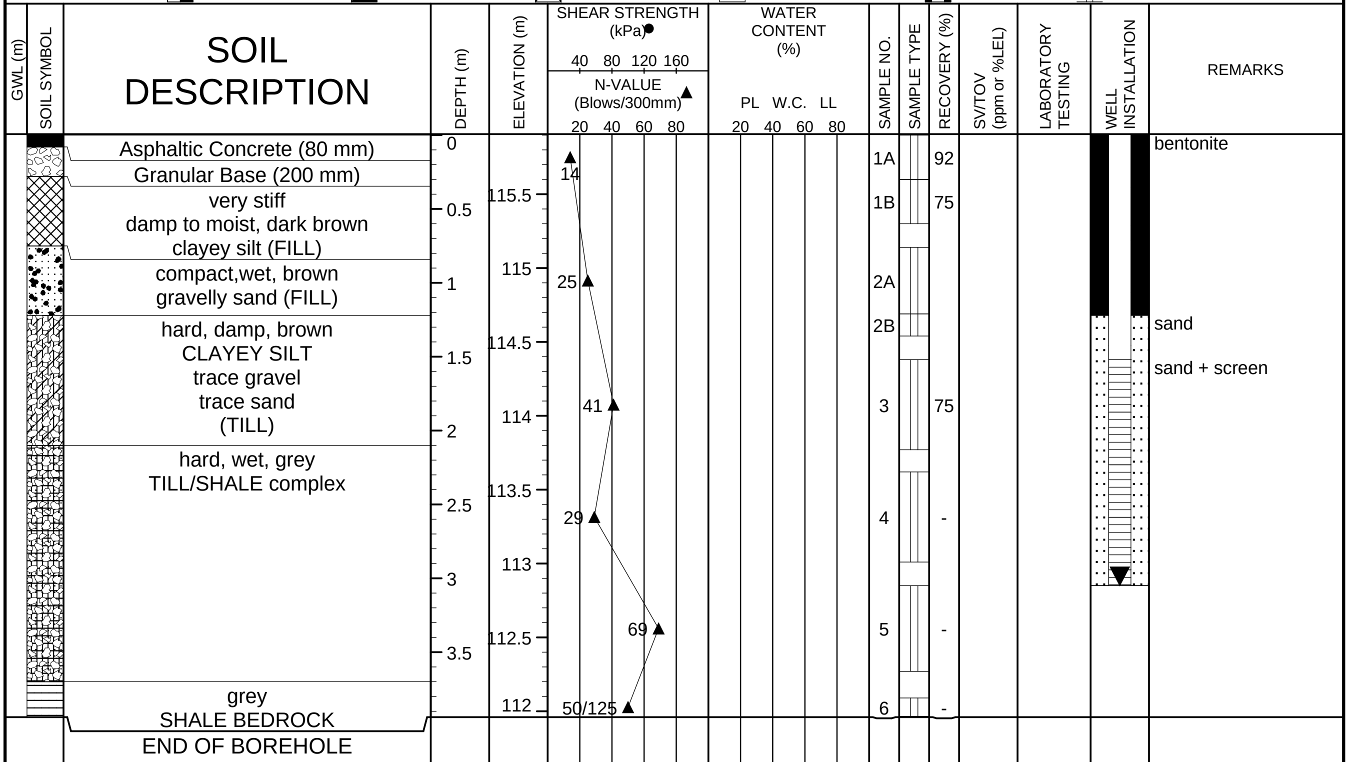
MONITORING DATE: 08-Jan-2021

REVIEWED BY: VN

CLIENT: Hazleview Investment
PROJECT NO.: CA20-149

RECORD OF:
MW113

ADDRESS: 1590 & 1650 Dundas Street East
STATION:
CITY/PROVINCE: Toronto, Ontario
NORTHING (m):
EASTING (m):
ELEV. (m) 115.90
CONTRACTOR: Pontil Drilling Services Inc.
METHOD: Hollow Stem Auger and Split Spoon Sampling
BOREHOLE DIAMETER (cm): 15
WELL DIAMETER (cm): 5
SCREEN SLOT #: 10
SAND TYPE: 2
SEALANT TYPE: Holeplug
SAMPLE TYPE: AUGER, DRIVEN, CORING, DYNAMIC CONE, SHELBY, SPLIT SPOON



SOIL DESCRIPTION

Asphaltic Concrete (80 mm)

Granular Base (200 mm)

very stiff
damp to moist, dark brown
clayey silt (FILL)

compact,wet, brown
gravelly sand (FILL)

hard, damp, brown
CLAYEY SILT
trace gravel
trace sand
(TILL)

hard, wet, grey
TILL/SHALE complex

grey
SHALE BEDROCK

END OF BOREHOLE

SAMPLE NO.	N-VALUE (Blows/300mm)	RECOVERY (%)
1A	14	92
1B		75
2A	25	
2B		
3	41	75
4	29	-
5	69	-
6	50/125	-

REMARKS
bentonite
sand
sand + screen

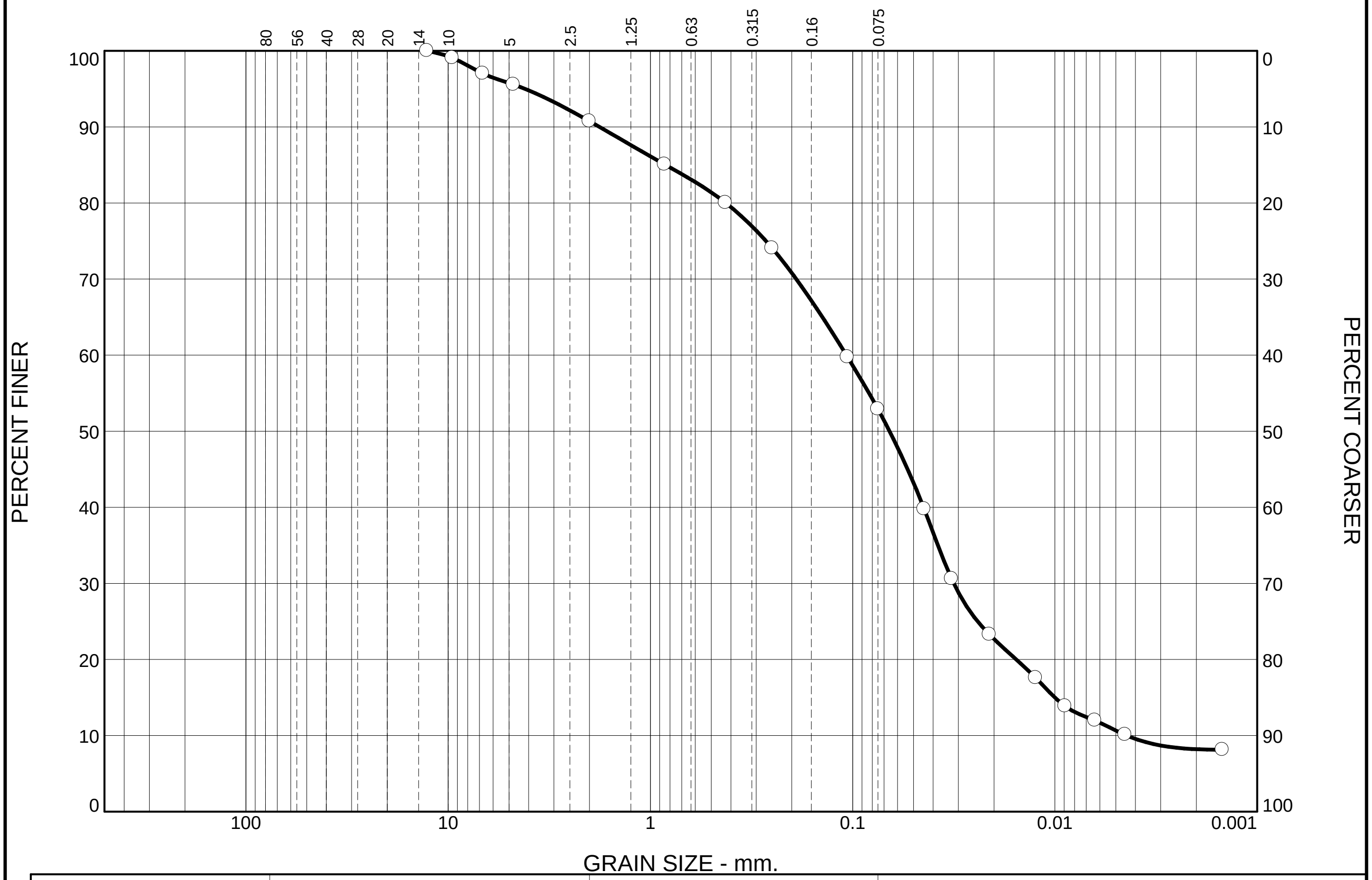
TERRAPEX

LOGGED BY: RG
DRILLING DATE: 15-Dec-2020
INPUT BY: SA
MONITORING DATE: 08-Jan-2021
REVIEWED BY: VN
PAGE 1 OF 1

APPENDIX II

LABORATORY TEST RESULT

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	9.3	10.7	27.1	44.7	8.2

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.8422	0.1074	0.0657	0.0316	0.0100	0.0044	2.11	24.36

Material Description	USCS	AASHTO
○ SAND and SILT trace gravel trace clay		

Project No. CA20-149 **Client:** Hazelview Investments Inc

Project: 1530-1650 Dundas St East Mississauga

○ **Sample Number:** MW102, SS7

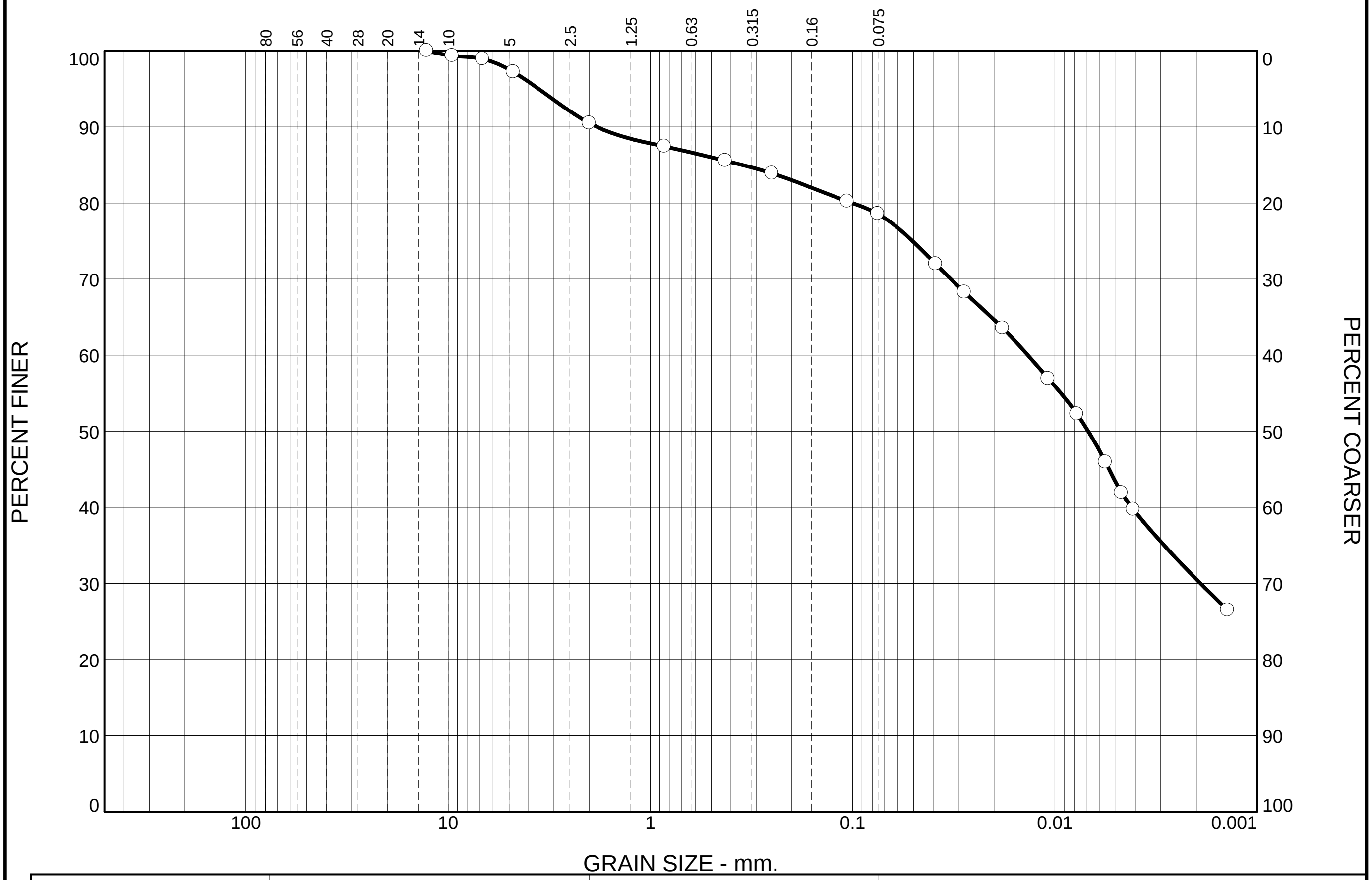
Remarks:

Terrapex

Figure 1

Tested By: AM **Checked By:** DM

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	9.5	5.0	6.9	48.0	30.6

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.3518	0.0136	0.0068	0.0019				

Material Description	USCS	AASHTO
○ CLAYEY SILT some sand trace gravel		

Project No. CA20-149 **Client:** Hazelview Investments Inc

Project: 1530-1650 Dundas St East Mississauga

○ **Sample Number:** BH108, SS4

Remarks:

Terrapex

Figure 2

Tested By: AM **Checked By:** DM

APPENDIX III

PHOTOGRAPHIC RECORDS OF ROCK CORES

Extracted rock cores from Borehole **BH105; 8.1 to 12.6** m below ground surface

Extracted rock cores from Borehole **MW107; 4.9 to 11.1** m below ground surface

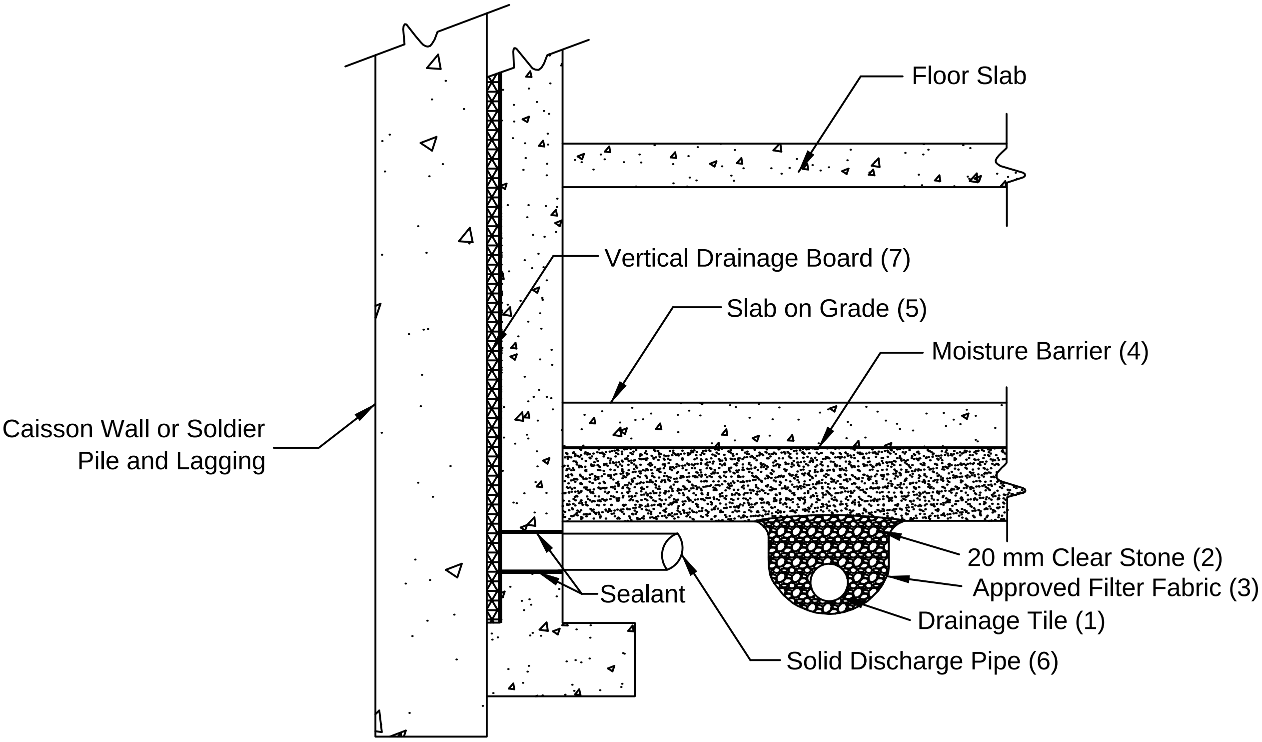
Extracted rock cores from Borehole **MW112; 3.4 to 12.5** m below ground surface

APPENDIX IV

RECOMMENDED DRAINAGE SYSTEM

TERRAPEX

DRAINAGE DETAILS



EXTERIOR FOOTING

Notes

1. Drainage tile to consist of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet
2. 20 mm (3/4") Clear Stone - 150 mm (6") top and side of drain, 100 mm (4") of stone below drain.
3. Wrap the clear stone with an approved filter membrane (Terrafix 270R or equivalent).
4. Moisture barrier to be at least 200 mm (8") of compacted clear 20 mm (3/4") stone or equivalent free draining material. A vapour barrier may be required for special floors.
5. Do not connect the underfloor drains to the perimeter drains.
6. Solid discharge pipe outletting into a solid pipe leading to a sump.
7. Vertical drainage board Terradrain 600 or equivalent with filter fabric should be continuous from bottom to 1.2 m below exterior finished grade.
8. Review the geotechnical report for specific details. Final detail must be approved before system is considered acceptable.

DRAINAGE RECOMMENDATIONS
Shored Basement Wall with Underfloor Drainage System
(Not to Scale)

APPENDIX V

SLOPE STABILITY ASSESSMENT

Slope stability analysis

Input data

Project

Task :	Slope Stability Assessment
Part :	Cross Section A
Description :	1590 Dundas Street East, Mississauga
Customer :	Hazelview Investments
Author :	SA
Date :	2021-02-03
Project number :	CA20-149

Settings

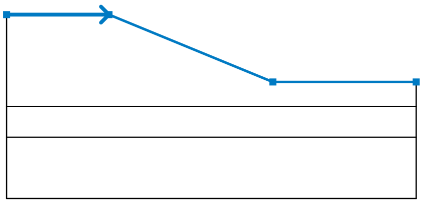
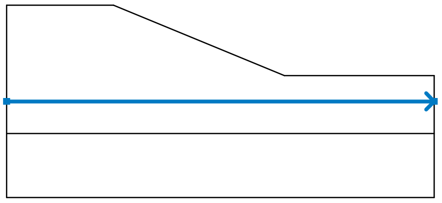
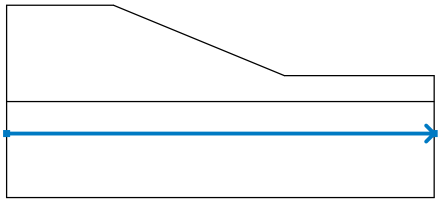
Standard - safety factors

Stability analysis

Earthquake analysis :	Standard
Verification methodology :	Safety factors (ASD)

Safety factors			
Permanent design situation			
Safety factor :	SF_s =	1.50	[–]

Interface

No.	Interface location	Coordinates of interface points [m]					
		x	z	x	z	x	z
1		0.00	0.00	5.00	0.00	13.00	-3.30
		20.00	-3.30				
2		0.00	-4.50	20.00	-4.50		
3		0.00	-6.00	20.00	-6.00		

Soil parameters - effective stress state

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
1	Fill	▨	28.00	0.00	19.00

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
2	Sandy Silt (Till)	🟩	36.00	0.00	21.00
3	Shale/Till Complex	▨	36.00	0.00	22.00
4	Shale Bedrock	▤	45.00	0.00	25.00

Soil parameters - uplift

No.	Name	Pattern	γ_{sat} [kN/m^3]	γ_s [kN/m^3]	n [–]
1	Fill	▦	19.00		
2	Sandy Silt (Till)	🟩	21.00		
3	Shale/Till Complex	▨	22.00		
4	Shale Bedrock	▤	25.00		

Soil parameters

Fill
Unit weight : γ = 19.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 28.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 19.00 kN/m^3

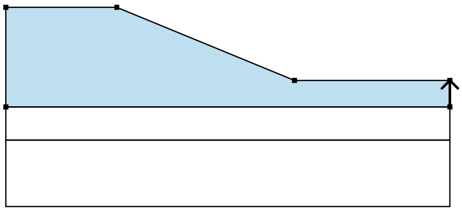
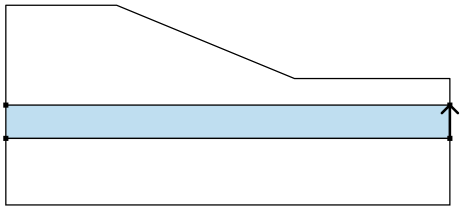
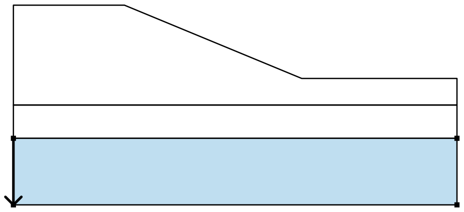
Sandy Silt (Till)
Unit weight : γ = 21.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 36.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 21.00 kN/m^3

Shale/Till Complex
Unit weight : γ = 22.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 36.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 22.00 kN/m^3

Shale Bedrock

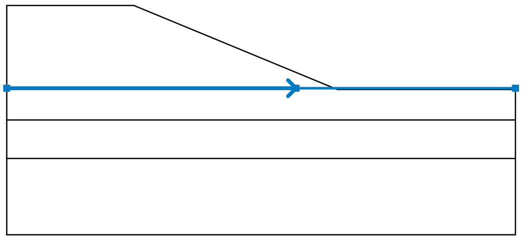
Unit weight : γ = 25.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 45.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 25.00 kN/m^3

Assigning and surfaces

No.	Surface position	Coordinates of surface points [m]				Assigned soil
		x	z	x	z	
1		20.00	-4.50	20.00	-3.30	Fill ▨
		13.00	-3.30	5.00	0.00	
		0.00	0.00	0.00	-4.50	
2		20.00	-6.00	20.00	-4.50	Shale/Till Complex ▨
		0.00	-4.50	0.00	-6.00	
3		0.00	-6.00	0.00	-9.00	Shale Bedrock ▤
		20.00	-9.00	20.00	-6.00	

Water

Water type : GWT

No.	GWT location	Coordinates of GWT points [m]					
		x	z	x	z	x	z
1		0.00	-3.25	11.37	-3.25	20.00	-3.25

Tensile crack

Tensile crack not input.

Earthquake

Earthquake not included.

Settings of the stage of construction

Design situation : permanent

Results (Stage of construction 1)

Analysis 1

Circular slip surface

Slip surface parameters							
Center :	x =	12.79	[m]	Angles :	α_1 =	-45.61	[°]
	z =	8.03	[m]		α_2 =	9.27	[°]
Radius :	R =	11.48	[m]				
The slip surface after optimization.							

Segments restricting slip surface

No.	First point		Second point	
	x [m]	z [m]	x [m]	z [m]
1	13.01	-3.12	12.95	-3.43
2	13.05	-3.44	4.95	-0.14
3	4.98	0.29	4.99	-0.39
4	4.92	0.14	13.05	-3.19
5	11.86	-2.53	11.65	-2.97
6	9.62	-1.72	9.46	-2.09
7	8.06	-1.02	7.95	-1.43
8	6.43	-0.39	6.34	-0.80

The restrictions of points of circular slip surface

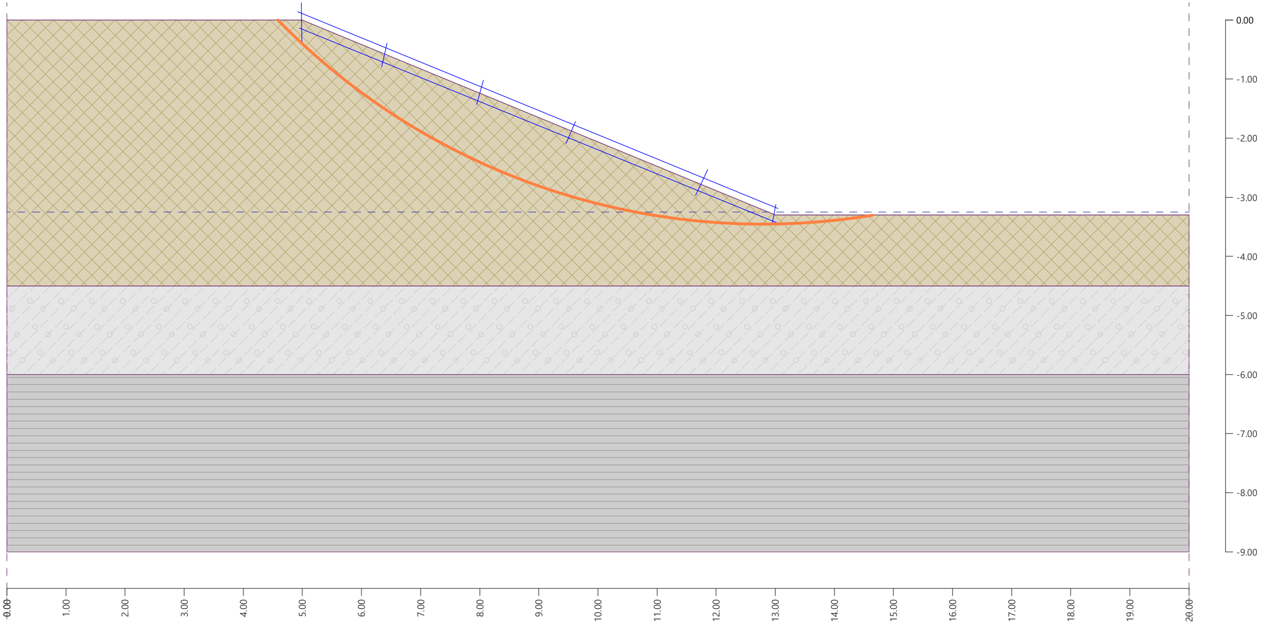
Slope stability verification (Bishop)

Sum of active forces : F_a = 47.43 kN/m
Sum of passive forces : F_p = 66.57 kN/m

Sliding moment : M_a = 544.48 kNm/m
Resisting moment : M_p = 764.20 kNm/m
Factor of safety = 1.40 < 1.50

Slope stability NOT ACCEPTABLE

Name : Analysis | **Stage - analysis : 1 - 1**



Analysis 2

Circular slip surface

Slip surface parameters							
Center :	x =	12.50	[m]	Angles :	α_1 =	-43.22	[°]
	z =	9.67	[m]		α_2 =	12.21	[°]
Radius :	R =	13.27	[m]				
The slip surface after optimization.							

Segments restricting slip surface

No.	First point		Second point	
	x [m]	z [m]	x [m]	z [m]
1	13.01	-3.12	12.95	-3.43
2	13.05	-3.44	4.95	-0.14
3	4.98	0.29	4.99	-0.39
4	4.92	0.14	13.05	-3.19
5	11.86	-2.53	11.65	-2.97
6	9.62	-1.72	9.46	-2.09
7	8.06	-1.02	7.95	-1.43
8	6.43	-0.39	6.34	-0.80
9	3.60	0.20	3.61	-0.18
10	3.55	-0.07	5.07	-0.06
11	3.50	0.12	5.11	0.12
12	4.19	0.20	4.22	-0.16

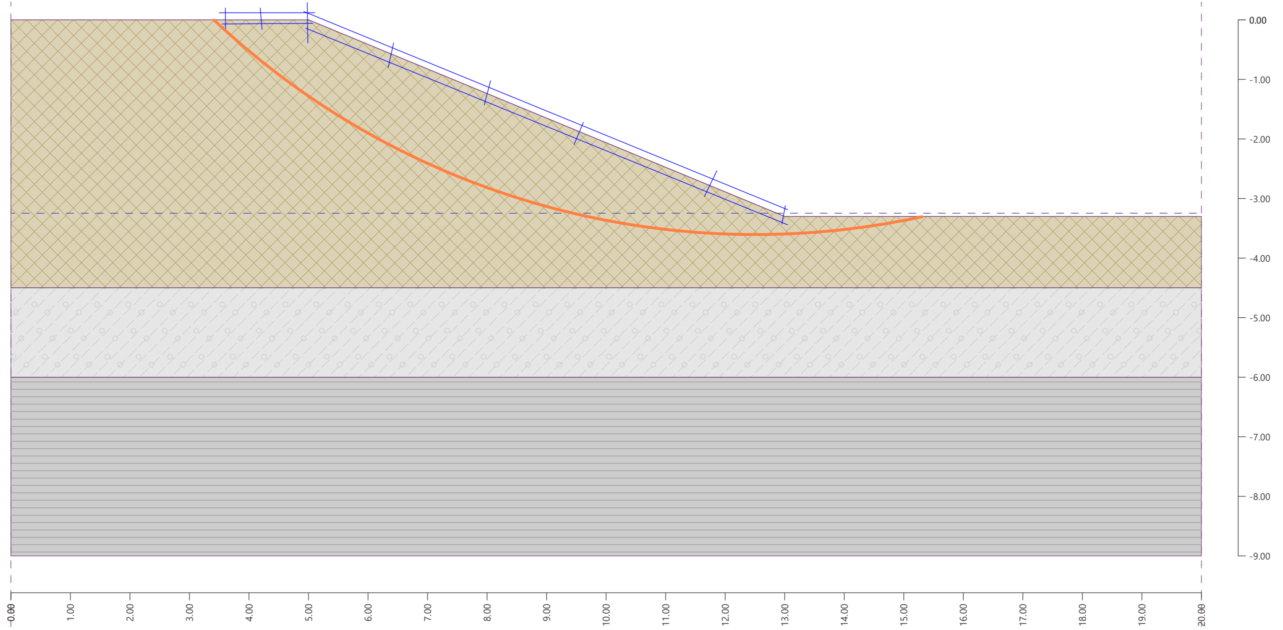
The restrictions of points of circular slip surface

Slope stability verification (Bishop)

Sum of active forces : F_a = 69.20 kN/m
Sum of passive forces : F_p = 104.58 kN/m

Sliding moment : M_a = 918.30 kNm/m
Resisting moment : M_p = 1387.83 kNm/m
Factor of safety = 1.51 > 1.50
Slope stability ACCEPTABLE

Name : Analysis **Stage - analysis : 1 - 2**



Slope stability analysis

Input data

Project

Task :	Slope Stability Assessment
Part :	Cross Section B
Description :	1590 Dundas Street East, Mississauga
Customer :	Hazelview Investments
Author :	SA
Date :	2021-02-03
Project number :	CA20-149

Settings

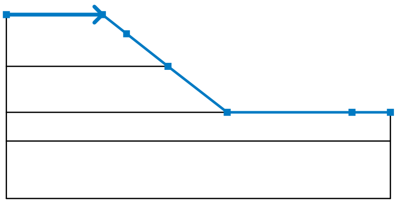
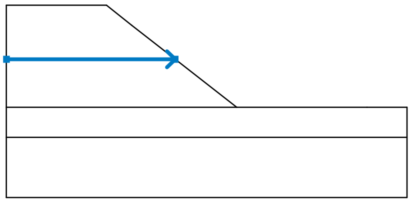
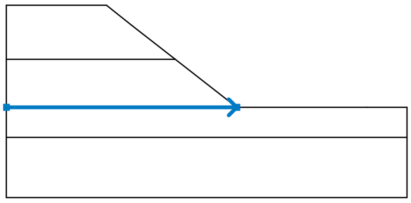
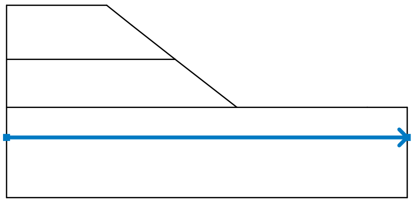
Standard - safety factors

Stability analysis

Earthquake analysis :	Standard
Verification methodology :	Safety factors (ASD)

Safety factors			
Permanent design situation			
Safety factor :	SF_s =	1.50	[–]

Interface

No.	Interface location	Coordinates of interface points [m]					
		x	z	x	z	x	z
1		0.00	0.00	5.00	0.00	6.26	-1.00
		8.42	-2.70	11.50	-5.10	18.00	-5.10
		20.00	-5.10				
2		0.00	-2.70	8.42	-2.70		
3		0.00	-5.10	11.50	-5.10		
4		0.00	-6.60	20.00	-6.60		

Soil parameters - effective stress state

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
1	Fill	▩	28.00	0.00	19.00
2	Sandy Silt (Till)	🟩	36.00	0.00	21.00
3	Shale/Till Complex	▨	36.00	0.00	22.00
4	Shale Bedrock	▤	45.00	0.00	25.00

Soil parameters - uplift

No.	Name	Pattern	γ_{sat} [kN/m^3]	γ_s [kN/m^3]	n [–]
1	Fill	▩	19.00		
2	Sandy Silt (Till)	🟩	21.00		
3	Shale/Till Complex	▨	22.00		
4	Shale Bedrock	▤	25.00		

Soil parameters

Fill
Unit weight : γ = 19.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 28.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 19.00 kN/m^3

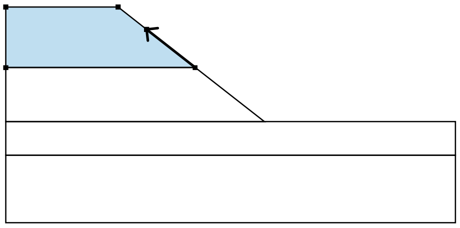
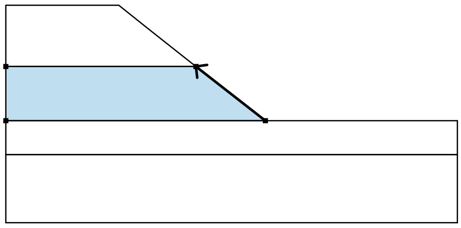
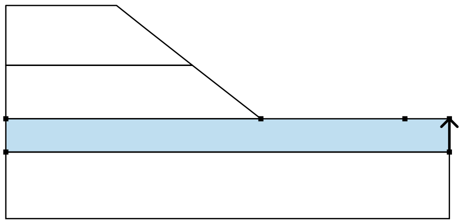
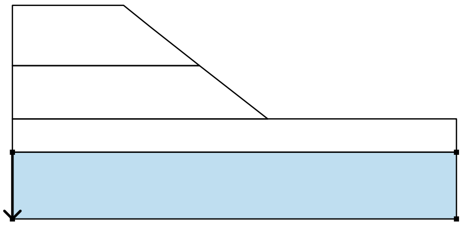
Sandy Silt (Till)
Unit weight : γ = 21.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 36.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 21.00 kN/m^3

Shale/Till Complex
Unit weight : γ = 22.00 kN/m^3
Stress-state : effective

Angle of internal friction : φ_{ef} = 36.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 22.00 kN/m^3

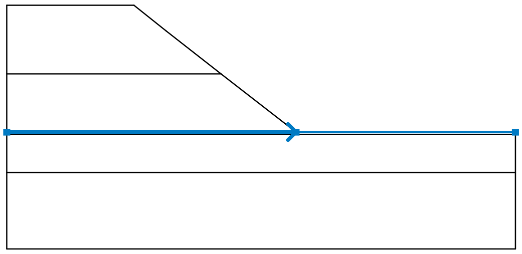
Shale Bedrock
Unit weight : γ = 25.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 45.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 25.00 kN/m^3

Assigning and surfaces

No.	Surface position	Coordinates of surface points [m]				Assigned soil
		x	z	x	z	
1		8.42	-2.70	6.26	-1.00	Fill
		5.00	0.00	0.00	0.00	▨
		0.00	-2.70			
2		11.50	-5.10	8.42	-2.70	Sandy Silt (Till)
		0.00	-2.70	0.00	-5.10	▨
3		20.00	-6.60	20.00	-5.10	Shale/Till Complex
		18.00	-5.10	11.50	-5.10	▨
		0.00	-5.10	0.00	-6.60	
4		0.00	-6.60	0.00	-9.60	Shale Bedrock
		20.00	-9.60	20.00	-6.60	▨

Water

Water type : GWT

No.	GWT location	Coordinates of GWT points [m]					
		x	z	x	z	x	z
1		0.00	-5.00	11.37	-5.00	20.00	-5.00

Tensile crack

Tensile crack not input.

Earthquake

Earthquake not included.

Settings of the stage of construction

Design situation : permanent

Results (Stage of construction 1)

Analysis 1

Circular slip surface

Slip surface parameters							
Center :	x =	12.77	[m]	Angles :	α_1 =	-66.74	[°]
	z =	3.46	[m]		α_2 =	12.27	[°]
Radius :	R =	8.76	[m]				
The slip surface after optimization.							

Segments restricting slip surface

No.	First point		Second point	
	x [m]	z [m]	x [m]	z [m]
1	5.06	0.13	4.96	-0.13
2	4.93	0.13	11.57	-5.01
3	11.63	-4.88	11.39	-5.19
4	5.86	-0.37	5.66	-0.78
5	7.44	-1.69	7.27	-2.05
6	9.10	-2.96	8.95	-3.30
7	10.53	-4.08	10.36	-4.38
8	6.62	-1.07	6.44	-1.33
9	5.40	-0.15	5.18	-0.40
10	8.31	-2.41	8.16	-2.63
11	9.80	-3.55	9.64	-3.78
12	11.19	-4.59	11.07	-4.87
13	7.99	-1.94	7.82	-2.43
14	4.89	-0.01	11.51	-5.18

The restrictions of points of circular slip surface

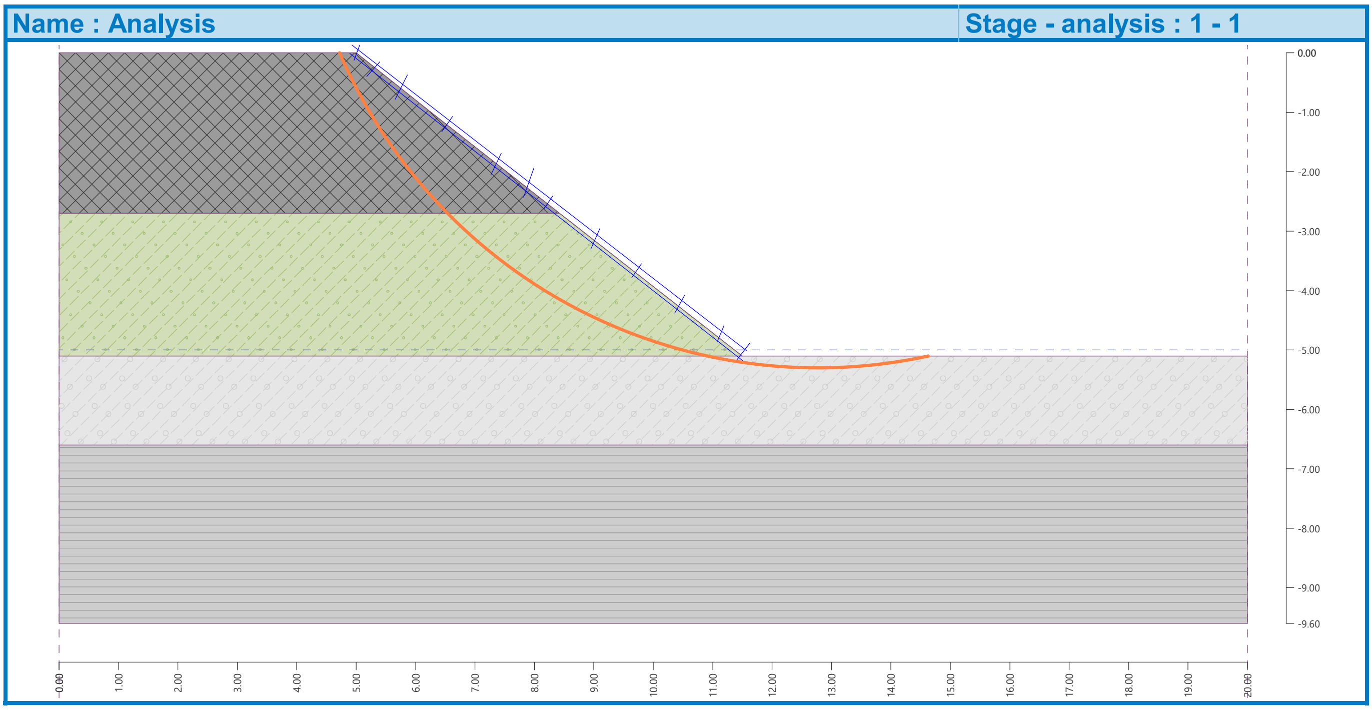
Slope stability verification (Bishop)

Sum of active forces : F_a = 82.40 kN/m
Sum of passive forces : F_p = 92.94 kN/m

Sliding moment : M_a = 721.83 kNm/m

Resisting moment : M_p = 814.17 kNm/m
Factor of safety = 1.13 < 1.50
Slope stability NOT ACCEPTABLE

Name : Analysis **Stage - analysis : 1 - 1**



Analysis 2

Circular slip surface

Slip surface parameters							
Center :	x =	13.61	[m]	Angles :	α_1 =	-45.57	[°]
	z =	12.44	[m]		α_2 =	9.23	[°]
Radius :	R =	17.77	[m]				
The slip surface after optimization.							

Segments restricting slip surface

No.	First point		Second point	
	x [m]	z [m]	x [m]	z [m]
1	5.06	0.13	4.96	-0.13
2	4.93	0.13	11.57	-5.01
3	11.63	-4.88	11.39	-5.19
4	5.86	-0.37	5.66	-0.78
5	7.44	-1.69	7.27	-2.05
6	9.10	-2.96	8.95	-3.30
7	10.53	-4.08	10.36	-4.38
8	6.62	-1.07	6.44	-1.33
9	5.40	-0.15	5.18	-0.40
10	8.31	-2.41	8.16	-2.63
11	9.80	-3.55	9.64	-3.78
12	11.19	-4.59	11.07	-4.87
13	7.99	-1.94	7.82	-2.43
14	4.89	-0.01	11.51	-5.18
15	1.51	0.18	1.56	-0.06
16	1.45	0.14	5.14	0.05
17	1.51	-0.04	5.06	-0.09

No.	First point		Second point	
	x [m]	z [m]	x [m]	z [m]
18	2.34	0.13	2.36	-0.07
19	3.23	0.13	3.26	-0.11
20	4.14	0.10	4.17	-0.11
21	1.84	0.15	1.89	-0.06
22	4.55	0.09	4.52	-0.12
23	1.56	-0.04	1.21	-0.04
24	1.53	0.15	1.20	0.15
25	1.22	0.17	1.23	-0.07
26	1.01	0.21	1.02	-0.09
27	0.97	-0.03	1.25	-0.02
28	0.97	0.15	1.23	0.14

The restrictions of points of circular slip surface

Slope stability verification (Bishop)

Sum of active forces : $F_a =$ 158.11 kN/m

Sum of passive forces : $F_p =$ 238.66 kN/m

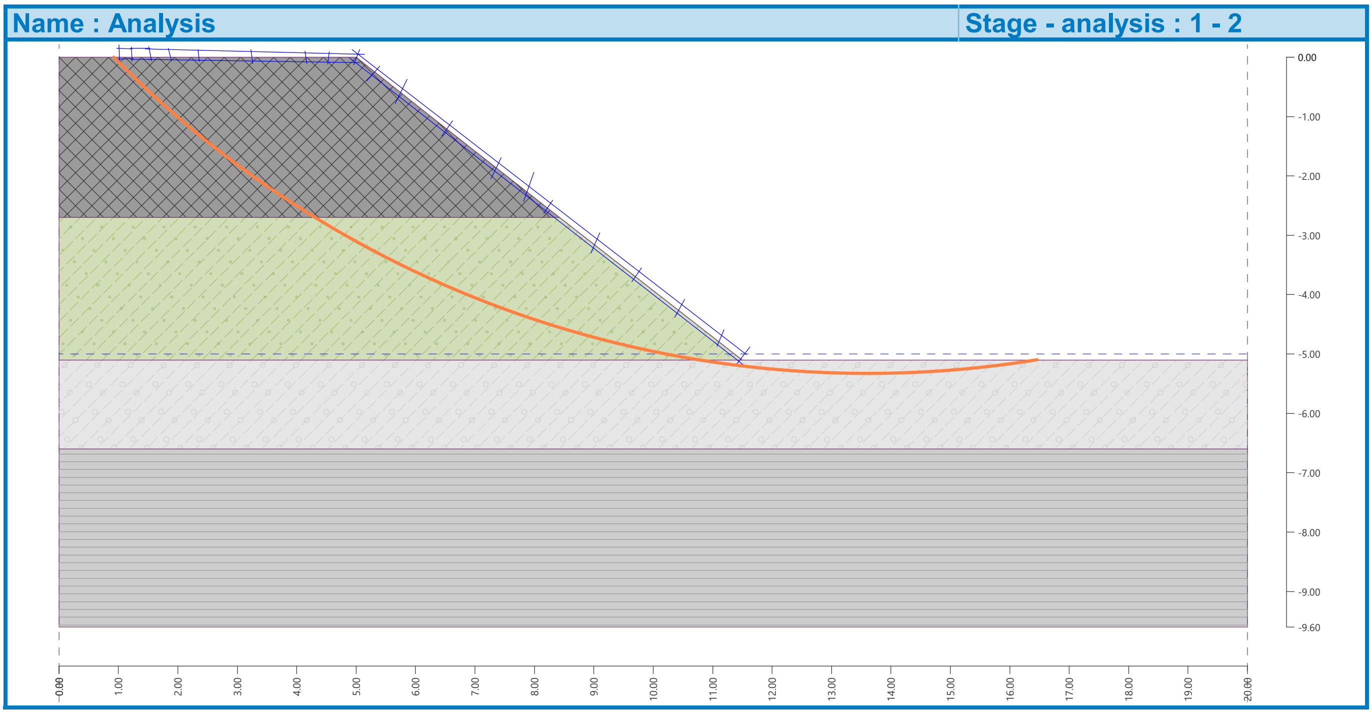
Sliding moment : $M_a =$ 2809.64 kNm/m

Resisting moment : $M_p =$ 4241.02 kNm/m

Factor of safety = 1.51 > 1.50

Slope stability ACCEPTABLE

Name : Analysis **Stage - analysis : 1 - 2**



Slope stability analysis

Input data

Project

Task : Slope Stability Assessment
Part : Cross Section C
Description : 1590 Dundas Street East, Mississauga
Customer : Hazelview Investments
Author : SA
Date : 2021-01-15
Project number : CA20-149

Settings

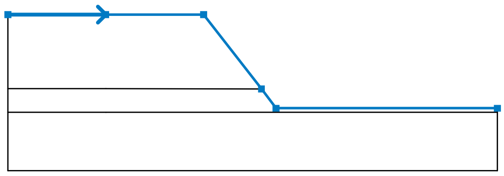
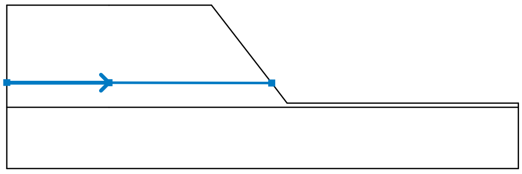
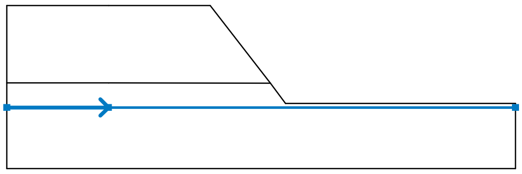
Standard - safety factors

Stability analysis

Earthquake analysis : Standard
Verification methodology : Safety factors (ASD)

Safety factors			
Permanent design situation			
Safety factor :	SF_s =	1.50	[–]

Interface

No.	Interface location	Coordinates of interface points [m]					
		x	z	x	z	x	z
1		-5.00	0.00	0.00	0.00	5.00	0.00
		7.95	-3.81	8.70	-4.80	20.00	-4.80
2		-5.00	-3.79	0.00	-3.80	7.95	-3.81
3		-5.00	-5.00	0.00	-5.00	20.00	-5.00

Soil parameters - effective stress state

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
1	Fill	▨	28.00	0.00	19.00

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
2	Clayey Silt (Till)	🟦	32.00	0.00	21.00
3	Shale/Till Complex	▨	36.00	0.00	22.00
4	Shale Bedrock	▤	45.00	0.00	25.00

Soil parameters - uplift

No.	Name	Pattern	γ_{sat} [kN/m^3]	γ_s [kN/m^3]	n [–]
1	Fill	▦	19.00		
2	Clayey Silt (Till)	🟦	21.00		
3	Shale/Till Complex	▨	22.00		
4	Shale Bedrock	▤	25.00		

Soil parameters

Fill
Unit weight : γ = 19.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 28.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 19.00 kN/m^3

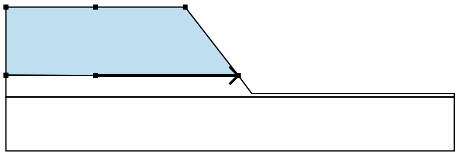
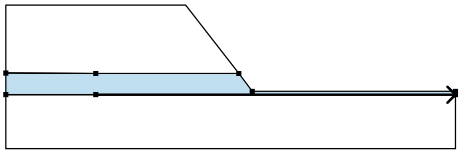
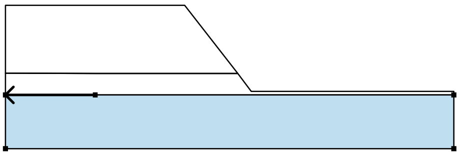
Clayey Silt (Till)
Unit weight : γ = 21.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 32.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 21.00 kN/m^3

Shale/Till Complex
Unit weight : γ = 22.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 36.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 22.00 kN/m^3

Shale Bedrock

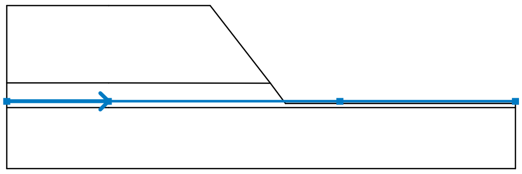
Unit weight : γ = 25.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 45.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 25.00 kN/m^3

Assigning and surfaces

No.	Surface position	Coordinates of surface points [m]				Assigned soil
		x	z	x	z	
1		0.00	-3.80	7.95	-3.81	Fill
		5.00	0.00	0.00	0.00	▨
		-5.00	0.00	-5.00	-3.79	
2		0.00	-5.00	20.00	-5.00	Clayey Silt (Till)
		20.00	-4.80	8.70	-4.80	▤
		7.95	-3.81	0.00	-3.80	
		-5.00	-3.79	-5.00	-5.00	
3		0.00	-5.00	-5.00	-5.00	Shale Bedrock
		-5.00	-8.00	20.00	-8.00	▤
		20.00	-5.00			

Water

Water type : GWT

No.	GWT location	Coordinates of GWT points [m]					
		x	z	x	z	x	z
1		-5.00	-4.70	0.00	-4.70	11.37	-4.70
		20.00	-4.70				

Tensile crack

Tensile crack not input.

Earthquake

Earthquake not included.

Settings of the stage of construction

Design situation : permanent

Results (Stage of construction 1)

Analysis 1

Circular slip surface

Slip surface parameters

Center :	x =	9.47	[m]	Angles :	α_1 =	-89.32	[°]
	z =	0.06	[m]		α_2 =	14.94	[°]
Radius :	R =	5.03	[m]				

The slip surface after optimization.

Segments restricting slip surface

No.	First point		Second point	
	x [m]	z [m]	x [m]	z [m]
1	5.12	0.21	4.93	-0.18
2	4.95	0.13	8.79	-4.74
3	8.85	-4.58	8.59	-4.89
4	8.70	-4.91	4.87	0.01
5	5.83	-0.81	5.56	-1.20
6	6.90	-2.07	6.66	-2.50
7	7.85	-3.41	7.61	-3.63
8	8.40	-4.08	8.06	-4.34
9	5.50	-0.33	5.22	-0.62

The restrictions of points of circular slip surface

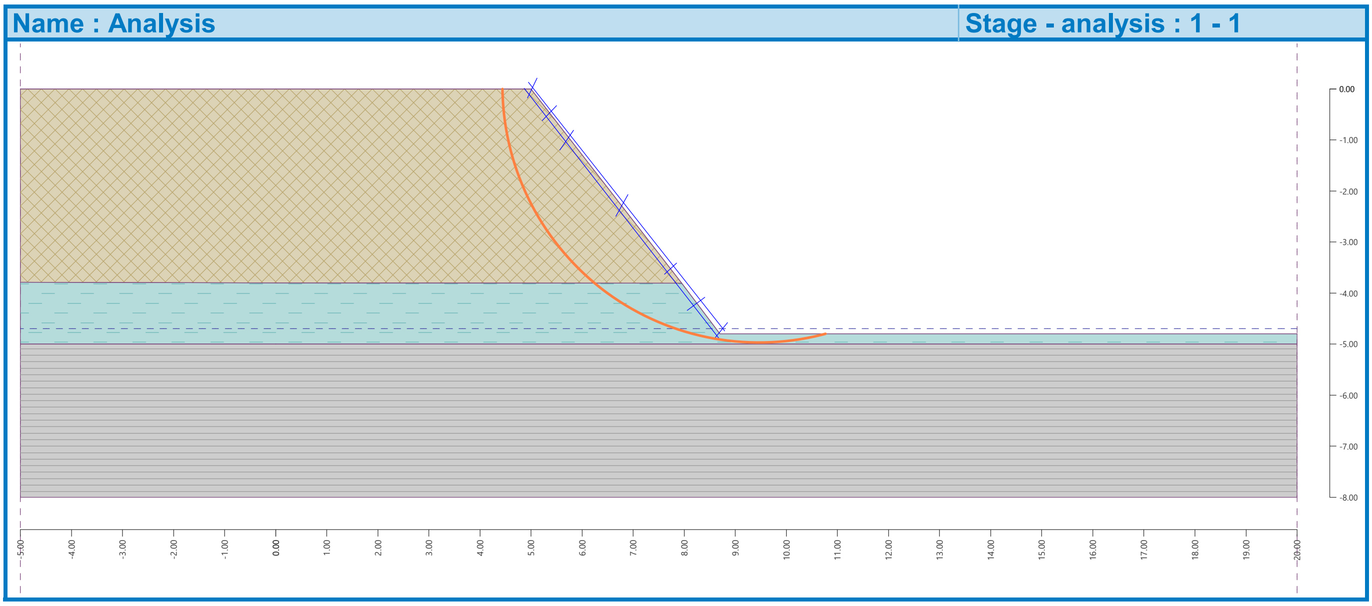
Slope stability verification (Bishop)

Sum of active forces : F_a = 83.67 kN/m
Sum of passive forces : F_p = 66.33 kN/m

Sliding moment : M_a = 420.86 kNm/m
Resisting moment : M_p = 333.63 kNm/m
Factor of safety = 0.79 < 1.50

Slope stability NOT ACCEPTABLE

Name : Analysis	**Stage - analysis : 1 - 1**



Analysis 2

Circular slip surface

Slip surface parameters								
Center :	x =	10.95	[m]	Angles :	α_1 =	-41.99	[°]	
	z =	14.59	[m]		α_2 =	8.97	[°]	
Radius :	R =	19.63	[m]					
The slip surface after optimization.								

Segments restricting slip surface

No.	First point		Second point	
	x [m]	z [m]	x [m]	z [m]
1	5.12	0.21	4.93	-0.18
2	4.95	0.13	8.79	-4.74
3	8.85	-4.58	8.59	-4.89
4	8.70	-4.91	4.87	0.01
5	5.83	-0.81	5.56	-1.20
6	6.90	-2.07	6.66	-2.50
7	7.85	-3.41	7.61	-3.63
8	8.40	-4.08	8.06	-4.34
9	5.50	-0.33	5.22	-0.62
10	5.22	0.11	1.01	0.13
11	5.05	-0.10	1.09	-0.13
12	1.13	0.21	1.21	-0.25
13	2.46	0.17	2.61	-0.22
14	3.66	0.22	3.81	-0.19
15	-0.64	0.30	-0.69	-0.24
16	0.31	0.21	0.38	-0.19
17	-1.48	0.27	-1.43	-0.22
18	-1.65	0.17	1.52	0.14
19	-1.60	-0.15	1.32	-0.18
20	-1.44	0.16	-1.78	0.19
21	-1.77	0.31	-1.72	-0.27
22	-1.74	-0.17	-1.38	-0.17
23	-1.89	0.26	-1.87	-0.27
24	-1.91	-0.16	-1.43	-0.17
25	-1.90	0.18	-1.43	0.17

The restrictions of points of circular slip surface

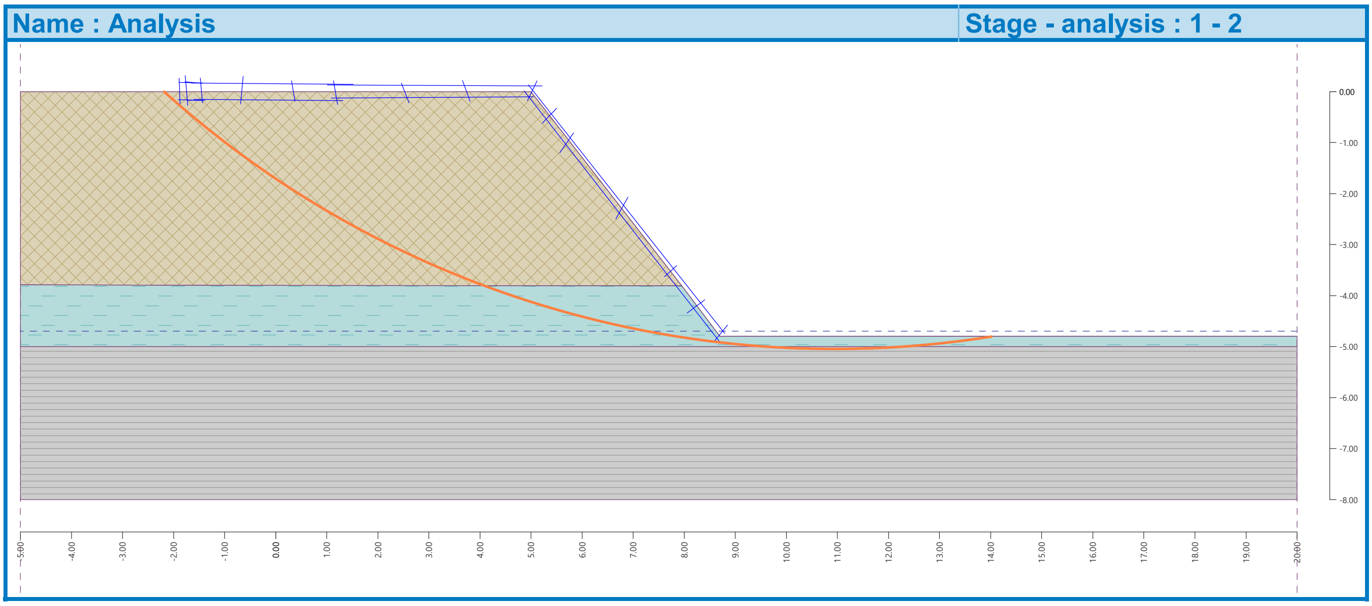
Slope stability verification (Bishop)

Sum of active forces : F_a = 179.10 kN/m
Sum of passive forces : F_p = 271.28 kN/m

Sliding moment : M_a = 3515.68 kNm/m
Resisting moment : M_p = 5325.23 kNm/m
Factor of safety = 1.51 > 1.50
Slope stability ACCEPTABLE

Name : Analysis

Stage - analysis : 1 - 2



Slope stability analysis

Input data

Project

Task : Slope Stability Assessment
Part : Cross Section D
Description : 1590 Dundas Street East, Mississauga
Customer : Hazelview Investments
Author : SA
Date : 2021-02-03
Project number : CA20-149

Settings

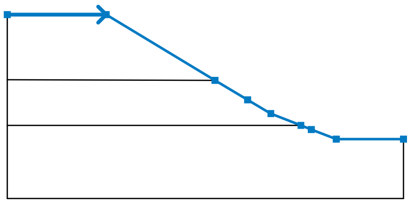
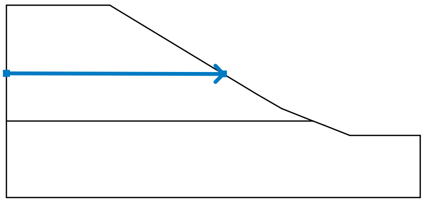
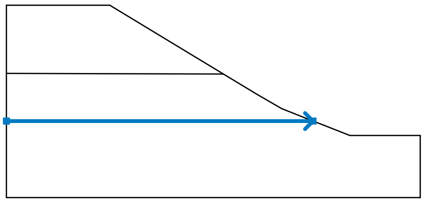
Standard - safety factors

Stability analysis

Earthquake analysis : Standard
Verification methodology : Safety factors (ASD)

Safety factors			
Permanent design situation			
Safety factor :	SF_s =	1.50	[–]

Interface

No.	Interface location	Coordinates of interface points [m]					
		x	z	x	z	x	z
1		0.00	0.00	5.00	0.00	10.49	-3.32
		12.13	-4.31	13.30	-5.00	14.82	-5.60
		15.35	-5.81	16.60	-6.30	20.00	-6.30
2		0.00	-3.30	10.49	-3.32		
3		0.00	-5.60	14.82	-5.60		

Soil parameters - effective stress state

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
1	Fill	▨	28.00	0.00	19.00

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
2	Clayey Silt (Till)	🟦	32.00	0.00	21.00
3	Shale/Till Complex	▨	36.00	0.00	22.00
4	Shale Bedrock	▤	45.00	0.00	25.00

Soil parameters - uplift

No.	Name	Pattern	γ_{sat} [kN/m^3]	γ_s [kN/m^3]	n [–]
1	Fill	▩	19.00		
2	Clayey Silt (Till)	🟦	21.00		
3	Shale/Till Complex	▨	22.00		
4	Shale Bedrock	▤	25.00		

Soil parameters

Fill

Unit weight : γ = 19.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 28.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 19.00 kN/m^3

Clayey Silt (Till)

Unit weight : γ = 21.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 32.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 21.00 kN/m^3

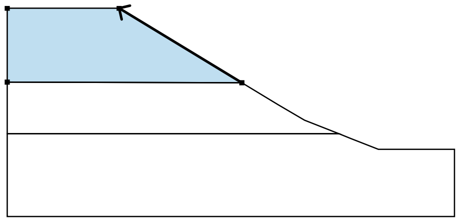
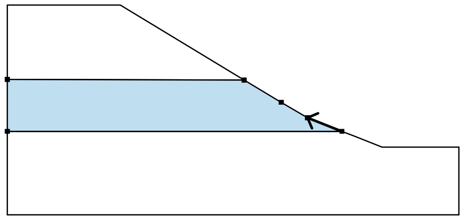
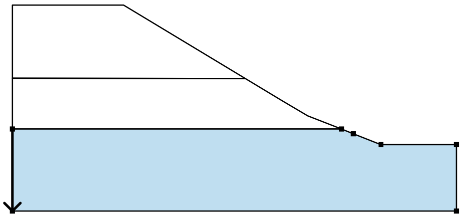
Shale/Till Complex

Unit weight : γ = 22.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 36.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 22.00 kN/m^3

Shale Bedrock

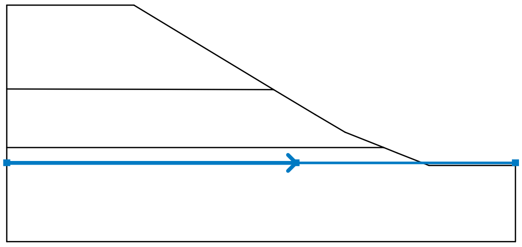
Unit weight : γ = 25.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 45.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 25.00 kN/m^3

Assigning and surfaces

No.	Surface position	Coordinates of surface points [m]				Assigned soil
		x	z	x	z	
1		10.49	-3.32	5.00	0.00	Fill ▨
		0.00	0.00	0.00	-3.30	
2		14.82	-5.60	13.30	-5.00	Clayey Silt (Till) ▨
		12.13	-4.31	10.49	-3.32	
		0.00	-3.30	0.00	-5.60	
3		0.00	-5.60	0.00	-9.30	Shale Bedrock ▤
		20.00	-9.30	20.00	-6.30	
		16.60	-6.30	15.35	-5.81	
		14.82	-5.60			

Water

Water type : GWT

No.	GWT location	Coordinates of GWT points [m]					
		x	z	x	z	x	z
1		0.00	-6.20	11.37	-6.20	20.00	-6.20

Tensile crack

Tensile crack not input.

Earthquake

Earthquake not included.

Settings of the stage of construction

Design situation : permanent

Results (Stage of construction 1)

Analysis 1

Circular slip surface

Slip surface parameters							
Center :	x =	17.02	[m]	Angles :	α_1 =	-54.70	[°]
	z =	8.98	[m]		α_2 =	10.50	[°]
Radius :	R =	15.54	[m]				
The slip surface after optimization.							

Segments restricting slip surface

No.	First point		Second point	
	x [m]	z [m]	x [m]	z [m]
1	16.71	-6.02	16.51	-6.55
2	16.79	-6.43	12.58	-4.92
3	13.27	-5.31	4.89	-0.08
4	5.10	0.16	4.91	-0.24
5	6.26	-0.30	6.07	-0.93
6	8.01	-1.60	7.99	-2.19
7	9.74	-2.64	9.55	-3.19
8	11.59	-3.78	11.38	-4.23
9	13.38	-4.78	12.95	-5.31
10	15.35	-5.67	15.23	-6.00
11	16.79	-6.14	12.99	-4.76
12	15.35	-5.47	15.33	-5.98
13	13.42	-4.98	4.83	0.25

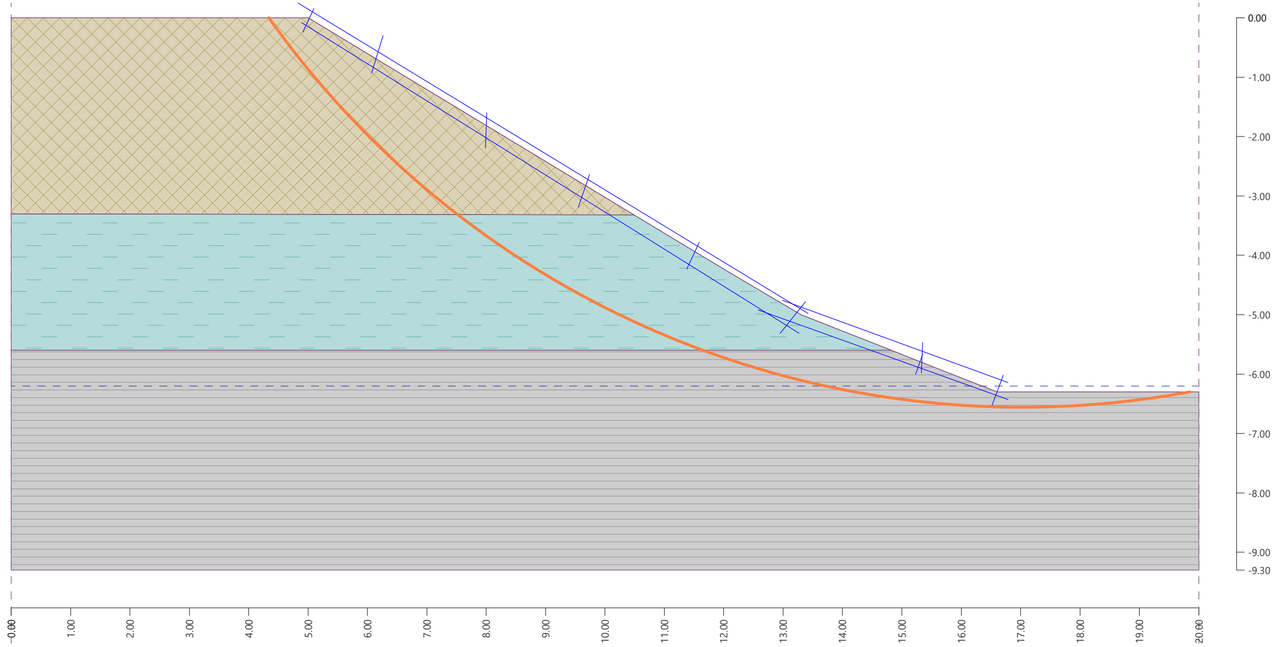
The restrictions of points of circular slip surface

Slope stability verification (Bishop)

Sum of active forces : F_a = 145.82 kN/m
Sum of passive forces : F_p = 230.29 kN/m

Sliding moment : M_a = 2266.02 kNm/m
Resisting moment : M_p = 3578.69 kNm/m
Factor of safety = 1.58 > 1.50
Slope stability ACCEPTABLE

Name : Analysis **Stage - analysis : 1 - 1**



Slope stability analysis

Input data

Project

Task : Slope Stability Assessment
Part : Cross Section E
Description : 1590 Dundas Street East, Mississauga
Customer : Hazelview Investments
Author : SA
Date : 2021-02-03
Project number : CA20-149

Settings

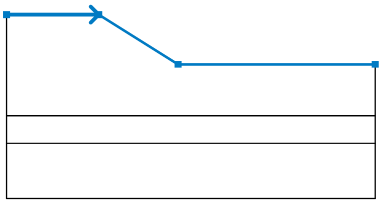
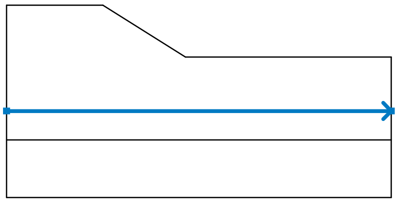
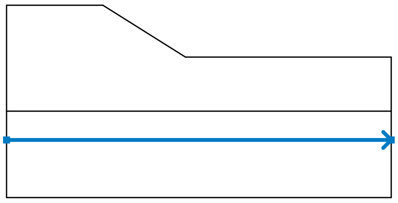
Standard - safety factors

Stability analysis

Earthquake analysis : Standard
Verification methodology : Safety factors (ASD)

Safety factors			
Permanent design situation			
Safety factor :	SF_s =	1.50	[–]

Interface

No.	Interface location	Coordinates of interface points [m]					
		x	z	x	z	x	z
1		0.00	0.00	5.00	0.00	9.30	-2.70
		20.00	-2.70				
2		0.00	-5.50	20.00	-5.50		
3		0.00	-7.00	20.00	-7.00		

Soil parameters - effective stress state

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m³]
1	Fill	▨	28.00	0.00	19.00

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
2	Sandy Silt (Till)	🟩	36.00	0.00	21.00
3	Shale/Till Complex	▨	36.00	0.00	22.00
4	Shale Bedrock	▤	45.00	0.00	25.00

Soil parameters - uplift

No.	Name	Pattern	γ_{sat} [kN/m^3]	γ_s [kN/m^3]	n [–]
1	Fill	▩	19.00		
2	Sandy Silt (Till)	🟩	21.00		
3	Shale/Till Complex	▨	22.00		
4	Shale Bedrock	▤	25.00		

Soil parameters

Fill
Unit weight : γ = 19.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 28.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 19.00 kN/m^3

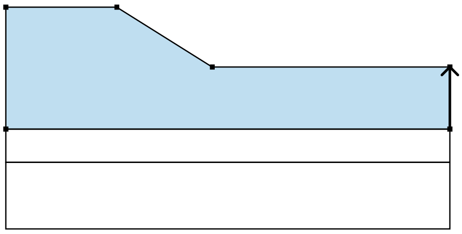
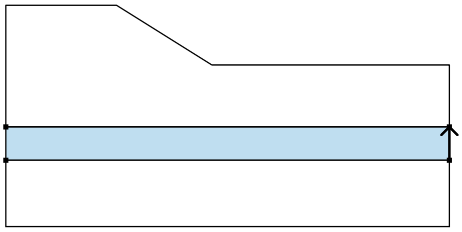
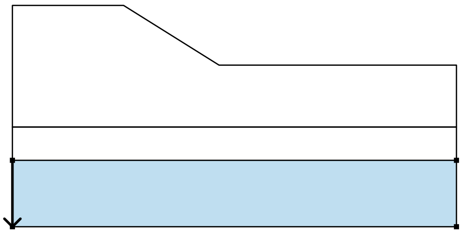
Sandy Silt (Till)
Unit weight : γ = 21.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 36.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 21.00 kN/m^3

Shale/Till Complex
Unit weight : γ = 22.00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 36.00 °
Cohesion of soil : c_{ef} = 0.00 kPa
Saturated unit weight : γ_{sat} = 22.00 kN/m^3

Shale Bedrock

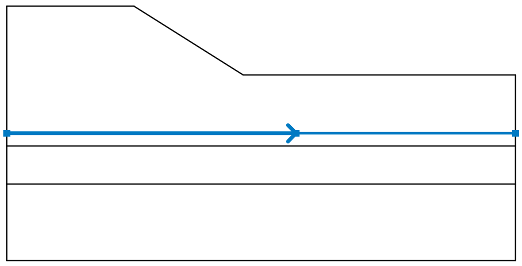
Unit weight :	γ = 25.00 kN/m^3
Stress-state :	effective
Angle of internal friction :	φ_{ef} = 45.00 °
Cohesion of soil :	c_{ef} = 0.00 kPa
Saturated unit weight :	γ_{sat} = 25.00 kN/m^3

Assigning and surfaces

No.	Surface position	Coordinates of surface points [m]				Assigned soil
		x	z	x	z	
1		20.00	-5.50	20.00	-2.70	Fill
		9.30	-2.70	5.00	0.00	▨
		0.00	0.00	0.00	-5.50	
2		20.00	-7.00	20.00	-5.50	Shale/Till Complex
		0.00	-5.50	0.00	-7.00	▨
3		0.00	-7.00	0.00	-10.00	Shale Bedrock
		20.00	-10.00	20.00	-7.00	▨

Water

Water type : GWT

No.	GWT location	Coordinates of GWT points [m]					
		x	z	x	z	x	z
1		0.00	-5.00	11.37	-5.00	20.00	-5.00

Tensile crack

Tensile crack not input.

Earthquake

Earthquake not included.

Settings of the stage of construction

Design situation : permanent

Results (Stage of construction 1)

Analysis 1

Circular slip surface

Slip surface parameters							
Center :	x =	9.75	[m]	Angles :	α_1 =	-59.03	[°]
	z =	3.00	[m]		α_2 =	12.12	[°]
Radius :	R =	5.83	[m]				
The slip surface after optimization.							

Segments restricting slip surface

No.	First point		Second point	
	x [m]	z [m]	x [m]	z [m]
1	5.02	0.12	4.89	-0.22
2	4.88	0.12	9.32	-2.58
3	9.28	-2.42	9.30	-2.81
4	9.43	-2.82	4.78	-0.04
5	5.93	-0.42	5.80	-0.78
6	7.00	-1.10	6.89	-1.43
7	8.26	-1.86	8.20	-2.19
8	7.69	-1.48	7.65	-1.82
9	6.31	-0.68	6.20	-0.97
10	5.51	-0.17	5.35	-0.47
11	8.75	-2.17	8.69	-2.49

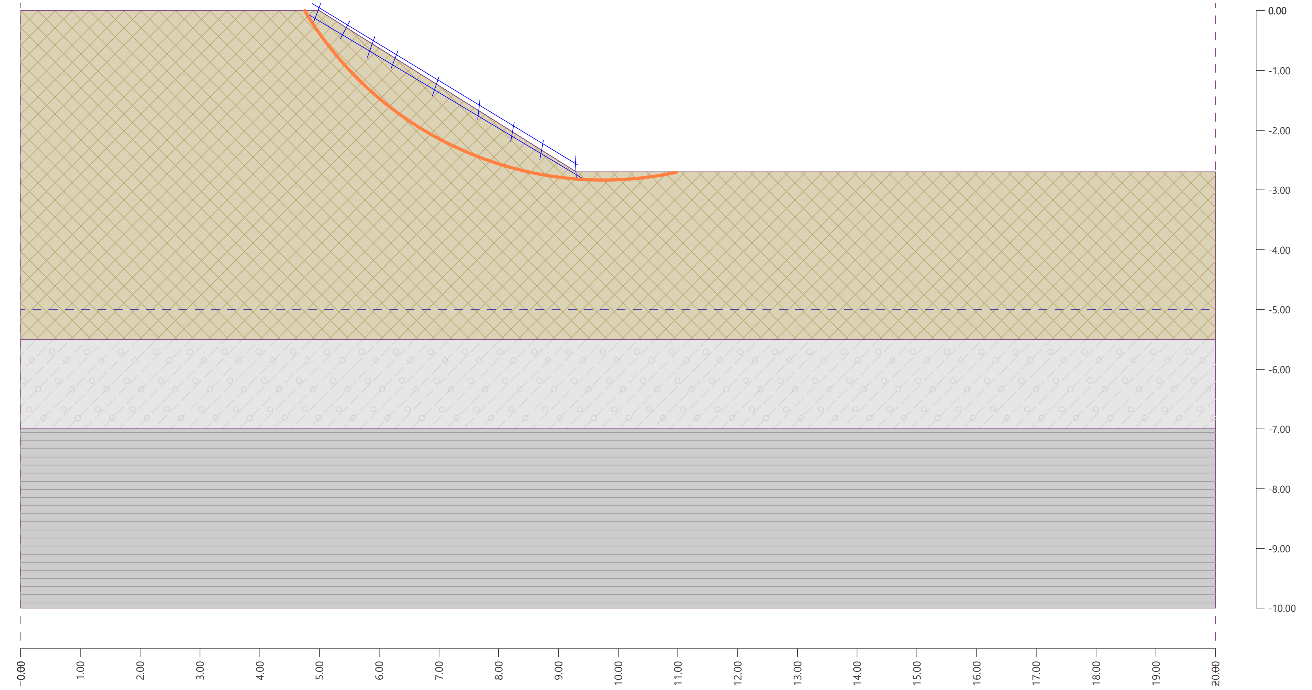
The restrictions of points of circular slip surface

Slope stability verification (Bishop)

Sum of active forces : F_a = 26.90 kN/m
Sum of passive forces : F_p = 28.76 kN/m

Sliding moment : M_a = 156.85 kNm/m
Resisting moment : M_p = 167.66 kNm/m
Factor of safety = 1.07 < 1.50
Slope stability NOT ACCEPTABLE

Name : Analysis **Stage - analysis : 1 - 1**



Analysis 2

Circular slip surface

Slip surface parameters							
Center :	x =	9.84	[m]	Angles :	α_1 =	-39.59	[°]
	z =	9.51	[m]		α_2 =	8.32	[°]
Radius :	R =	12.34	[m]				
The slip surface after optimization.							

Segments restricting slip surface

No.	First point		Second point	
	x [m]	z [m]	x [m]	z [m]
1	5.02	0.12	4.89	-0.22
2	4.88	0.12	9.32	-2.58
3	9.28	-2.42	9.30	-2.81
4	9.43	-2.82	4.78	-0.04
5	5.93	-0.42	5.80	-0.78
6	7.00	-1.10	6.89	-1.43
7	8.26	-1.86	8.20	-2.19
8	7.69	-1.48	7.65	-1.82
9	6.31	-0.68	6.20	-0.97
10	5.51	-0.17	5.35	-0.47
11	8.75	-2.17	8.69	-2.49
12	4.59	0.08	4.61	-0.20
13	2.30	0.35	2.29	-0.25
14	2.22	-0.11	4.96	-0.13
15	2.18	0.14	5.01	0.05
16	3.92	0.12	3.91	-0.17
17	3.31	0.13	3.32	-0.13
18	2.81	0.15	2.81	-0.13
19	2.50	0.15	2.51	-0.15

The restrictions of points of circular slip surface

Slope stability verification (Bishop)

Sum of active forces : F_a = 51.74 kN/m

Sum of passive forces : F_p = 77.90 kN/m

Sliding moment : M_a = 638.44 kNm/m

Resisting moment : M_p = 961.25 kNm/m

Factor of safety = 1.51 > 1.50

Slope stability ACCEPTABLE

Name : Analysis | **Stage - analysis : 1 - 2**