Geotechnical Investigation

Proposed 12 Unit Residential Development
1164-1166 Highcroft Drive
Manotick, Ontario

Prepared for:

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TABLE OF CONTENTS

LIST OF TABLES

APPENDICES

1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by ARK Construction Ltd. to perform a geotechnical investigation for a proposed 12 unit residential development, located at 1164-1166 Highcroft Drive, in Manotick, Ontario.

The purpose of the investigation was to identify the subsurface conditions across the site by the completion of a limited borehole drilling program. Based on the visual and factual information obtained, this report will provide guidelines on the geotechnical engineering aspects of the design of the project, including construction considerations.

This report has been prepared in consideration of the terms and conditions noted above. Should there be any changes in the design features, which may relate to the geotechnical recommendations provided in the report, LRL should be advised in order to review the report recommendations.

2 SITE AND PROJECT DESCRIPTION

The site under investigation is currently located at 1164 – 1166 Highcroft Drive, in Manotick, Ontario. It currently encompasses two (2) single family (detached) residential dwellings. The site is rectangular in shape, having about 61 m of frontage along Highcroft Drive, and a total surface area of about 3,716 m^2 (approximately 0.92 acres). It is sloped downwards from southwest to northeast, with elevations ranging from approximately 95 to 88 m. At the time of the field investigation, the site was snow covered. The site location is presented in Figure 1 included in **Appendix A**.

It is our understanding that the proposed development will consist of demolishing the existing dwellings, and the construction of twelve (12) residential units across the site.

3 PROCEDURE

The fieldwork for this investigation was carried out on January 29, 2019. Prior to the fieldwork, the site was cleared for the presence of any underground services and utilities. A total of six (6) boreholes, labelled BH1 through BH6, were drilled onsite within the proposed residential unit complexes footprints, where it was possible to do so. The approximate locations of the boreholes are shown in Figure 2 included in **Appendix A**.

The boreholes were advanced using a track mount CME 45 drill rig equipped with 200 mm diameter continuous flight hollow stem auger supplied and operated by George Downing Estate Drilling. A “two man” crew experienced with geotechnical drilling operated the drill rig and equipment.

Sampling of the overburden materials encountered in the boreholes was carried out at regular depth intervals using a 50.8 mm diameter drive open conventional spoon sampler in conjunction with standard penetration testing (SPT) “N” values. The SPT were conducted following the method **ASTM D1586** and the results of SPT, in terms of the number of blows per 0.3 m of split-spoon sampler penetration after first 0.15 m designated as “N” value.

The boreholes were advanced to depths from 2.06 and 6.10 m below ground surface (bgs) with all boreholes, with the exclusion of BH3, being terminated after practical auger refusal. Upon completion, the boreholes were backfilled and compacted using the overburden cuttings.

The fieldwork was supervised throughout by a member of our engineering staff who oversaw the drilling activities, cared for the samples obtained and logged the subsurface conditions encountered within each of the boreholes. All soil samples collected from the boreholes were placed and sealed in plastic bags to prevent moisture loss. The recovered soil samples collected from the boreholes were classified based on visual examination of the materials recovered and the results of the in-situ testing.

Furthermore, all boreholes were surveyed and located using a Garmin Etrex Legend GPS (Global Positioning System) receiver using NAD 83 datum (North American Datum). LRL's field personnel determined the existing grade elevations at the borehole locations through a topographic survey carried out using the "Top of a public utility pedestal box located at the northwest property limit of 1166 Highcroft Drive" as a Temporary Bench Mark (TBM). The TBM was assumed to have an elevation of 100.00 m. Ground surface elevations of boring locations are shown on their respective boreholes logs.

4 SUBSURFACE SOIL AND GROUNDWATER CONDITIONS

4.1 General

A review of local surficial geology maps provided by the Department of Energy, Mines and Resources Canada suggest that the surficial geology for this area consists within the transition zone of clay, silty clay, and till material. The till consists of a heterogeneous mixture of material ranging from clay to large boulders, generally sandy, grades downwards into unmodified till.

The subsurface conditions encountered in the boreholes were classified based on visual and tactile examination of the materials recovered from the boreholes and the results of in-situ and laboratory testing. The soil descriptions presented in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil were conducted according to the procedure **ASTM D2487** and judgement, and LRL does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The subsurface soil conditions encountered at boreholes are given in their respective logs presented in **Appendix B**. A greater explanation of the information presented in the borehole logs can be found in **Appendix C** of this report. These logs indicate the subsurface conditions encountered at a specific test location only. Boundaries between zones on the logs are often not distinct, but are rather transitional and have been interpreted as such.

4.2 Topsoil

Topsoil of thickness ranging from 150 to 600 mm was found at all boring locations. It was found to be sandy, with black organic material.

This material was classified as topsoil based on colour and the presence of organic material and is intended as identification for geotechnical purposes only. It does not constitute a statement as to the suitability of this layer for cultivation and sustaining plant growth.

4.3 Silt and Clay

Underlying the topsoil at boring locations BH3 – BH6, a deposit of silt and clay was encountered, and extended to depths of 1.45 and 4.89 m bgs. This material can generally be described as silt and clay, trace sand, and brownish grey to grey in colour.

Standard penetration tests were carried out in the silt and clay and the STP “N” values were found ranging from 3 to 21, indicating the deposit is very stiff, becoming soft with increased depth. The natural moisture contents were found varying between 21 and 45%.

A soil sample was collected from BH4 (SS4-2) between depths 0.76 and 1.37 m bgs for laboratory gradation analyses. The gradation analyses comprised of a sieve and hydrometer were conducted following the procedure **ASTM D422.** Details of laboratory analysis are reflected in **Table 1**.

Table 1: Gradation Analysis Summary – Silt and Clay

Sample Location	Depth (m)	Percent for Each Soil Gradation					Estimated Hydraulic Conductivity K (cm/s)
		Sand			Silt (%)	Clay (%)	
		Coarse (%)	Medium (%)	Fine (%)			
BH4	0.76 – 1.37	0.0	0.7	3.5	44.1	51.7	5×10^{-6}

Atterberg limits and moisture contents were conducted on the spoon soil sample collected between depths 1.52 and 2.13 m in BH5 (SS5-3). Based on the test result, the sample yielded a plastic limit of 19% and corresponding liquid limit of 42%. These values indicate that the subsoil contains inorganic clays of low plasticity. A summary of these values are provided below in **Table 2**.

Table 2: Summary of Atterberg Limits and Water Contents

Sample Location	Parameter					
	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Water Content (%)	USCS Group Symbol
BH2	1.52 – 2.13	42	19	23	45	CL

4.4 Silt and Sand Till

Underlying the topsoil at boring locations BH1 and BH2, and the silt and clay in BH3 – BH6, a deposit of silt and sand till was encountered, and extended to depths ranging from 2.06 to 6.10 (end of explorations depths). It generally consisted of having trace clay, some gravel sized stone, and can be described as greyish brown to brownish grey in colour. The recorded SPT “N” values of this deposit varied from 6 to 83, indicating the deposit is compact to very dense in relative density. The natural moisture content was found varying between 8 and 15%.

A second soil sample was selected for a laboratory gradation analysis, from BH2 (SS2-2) between depths 0.76 – 1.37 m bgs. Details of the laboratory analysis are reflected in **Table 3.**

Table 3: Gradation Analysis Summary – Silt and Sand Till

Sample Location	Depth (m)	Percent for Each Soil Gradation							Estimated Hydraulic Conductivity K (cm/s)
		Gravel		Sand			Fines		
		Coarse (%)	Fine (%)	Coarse (%)	Medium (%)	Fine (%)	Silt (%)	Clay (%)	

BH2	1.5-2.0	8.5	16.7	5.6	10.8	22.8	26.2	9.5	5×10^{-4}

The laboratory reports can be found in **Appendix D** of this report.

4.5 Refusal

Practical auger refusal was encountered at all boring locations, with the exclusion of BH3, at depths ranging from 2.06 – 5.01 m bgs. Auger refusal was encountered over large boulders within the till material, or possible bedrock. Bedrock coring was not part of the scope for this project.

4.6 Groundwater Conditions

Groundwater was carefully monitored during this field investigation. While drilling, water was only encountered in BH4 at about 4.8 m bgs. Upon completion, water was encountered at about 4.3 m bgs in BH4; the remaining boreholes were found to be dry after drilling. No standpipe or piezometer was installed for long term water level observations.

It should be noted that groundwater levels could fluctuate with seasonal weather conditions, (i.e.: rainfall, droughts, spring thawing) and due to construction activities at or in the vicinity of the site.

5 GEOTECHNICAL CONSIDERATIONS

This section of the report provides general geotechnical recommendations for the design aspect of the project based on our interpretation of the information gathered from the boreholes performed at this site and from the project requirements.

This section will detail the specific requirements and limitations with regard to allowable foundation bearing pressure, grade raise and footing size restrictions.

5.1 Foundations

Based on the subsurface soil conditions established at this site, the proposed residential units could be supported on shallow spread footing foundations below the frost penetration depth. It is anticipated the footings will be founded on either silt and clay, silt and sand till, or a combination of both. Therefore, all topsoil/organic material should be removed down to the required founding elevation.

5.2 Shallow Foundation

Conventional strip and column footings founded over the undisturbed native silt and clay or silt and sand till may be designed using a maximum allowable bearing pressure of **90 kPa** for serviceability limit state **(SLS)** and **135 kPa** for ultimate limit state **(ULS)** factored bearing resistance. The factored ULS value includes the geotechnical resistance factor of 0.5. This bearing capacity limits the allowable grade raise to 2.5 m, a maximum strip footing width of 2.0, and a maximum pad footing width of 4.0 m.

In-situ field testing may be required to check the strength and stability of the footings subgrade. Any incompetent subgrade areas as identified from in-situ testing must be sub-excavated and backfilled with approved structural fill. Similarly, any soft or wet areas should also be sub-excavated and backfilled with approved structural fill only. Prior to placing any approved structural fill, the subgrade should be inspected and approved by geotechnical engineer or a qualified geotechnical personnel. The bearing pressure is contingent on the water level being 0.3 m below the underside footing elevation in order to have a stable and dry subgrade during construction.

The footings and foundation walls supported by such footings should be reinforced to bridge anomalies “soft areas” in the material, in consultation with the project structural engineer. Footings and foundation walls shall be reinforced, especially at segments where footings founding soils are comprised of partly structural fill and partly undisturbed native soil.

Prior to pouring footings concrete, the subgrade should be inspected and approved by a geotechnical engineer or a representative of geotechnical engineer.

5.3 Structural Fill

For foundations set over undisturbed native soil and where excavation below the underside of the footings is performed in order to reach a suitable founding stratum, consideration should also be given to support the footings on structural fill. The structural fill should be placed over undisturbed native soils in layers not exceeding 300 mm and compacted to 98% of its Standard Proctor Maximum Dry Density (SPMDD) within ±2% of its optimum moisture content. In order to allow the spread of load beneath the footings and to prevent undermining during construction, the structural fill should extend minimum 0.6 m beyond the outside edges of the footings and then outward and downward at 1 horizontal to 1 vertical profile (or flatter) over a distance equal to the depth of the structural fill below the footing. Furthermore, the structural fill must be tested to ensure that the specified compaction level is achieved.

5.4 Settlement

The estimated total settlement of the shallow foundations, designed using the recommended serviceability limit state capacity value, as well as other recommendations given above, will be less than 25 mm. The differential settlement between adjacent column footings is anticipated to be 15 mm or less.

5.5 Seismic

Based on the information of this geotechnical investigation and in accordance with the Ontario Building Code 2015 (Table 4.1.8.4.A.) and Canadian Foundation Engineering Manual (4^{th} edition), the site can be classified for Seismic Site Response Site Class D.

The above classifications were recommended based on conventional method exercised for Site Classification for Seismic Site Response and in accordance with the generally accepted geotechnical engineering practice. If a greater Site Class is required, this may be achieved by carrying out site specific seismic testing such as a Shear Wave Velocity Test.

5.6 Liquefaction Potential

5.6.1 Silt and Clay

As recommended in the Canadian Foundation Engineering Manual 4^{th} edition (*Bray et al. 2004)*, the following criteria can be used to determine liquefaction susceptibility of fine grained soils.

- $w/w_L \geq 0.85$ and $I_p \leq 12$: Susceptible to liquefaction or cyclic mobility
- $w/w_L \geq 0.8$ and $12 \leq I_p \leq 20$: Moderately susceptible to liquefaction or cyclic mobility

- $w/w_L < 0.8$ and $I_p \leq 20$: No liquefaction or cyclic mobility, but may undergo significant deformations if cyclic shear stress > static undrained shear strength.

Laboratory plasticity test on a split spoon sample collected at an approximate depth of 1.5 – 2.0 m bgs exhibits the ratio of water content and liquid limit of approximately 1.07, and I_p is 23. Based on the test results, the silt and clay deposit is not susceptible to liquefaction.

5.6.2 Silt and Sand Till

The silt and sand till material at this site is not considered to be susceptible to liquefaction based on the SPT "N" values measured within this deposit.

5.7 Frost Protection

All exterior footings located in any unheated portions of the proposed warehouse should be protected against frost heaving by providing a minimum of 1.5 m of earth cover. Areas that are to be cleared of snow (i.e. sidewalks, paved areas, etc.) should be provided with at least 1.8 m of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided using a combination of earth cover and extruded polystyrene insulation. Detailed guidelines for footing insulation frost protection can be provided upon request.

In the event that foundations are to be constructed during winter months, the founding soils are required to be protected from freezing temperatures using suitable construction techniques. The base of all excavations should be insulated from freezing temperatures immediately upon exposure, until heat can be supplied to the building interior and the footings have sufficient soil cover to prevent freezing of the subgrade soils.

5.8 Foundation Drainage

A conventional, perforated corrugated polyethylene drainage pipe (100 mm minimum), pre-wrapped with geotextile knitted sock conforming to **OPSS 1840** should be embedded in a 300 mm layer of 19 mm clear stone and set adjacent to the perimeter footings. The drainage pipe should be connected positively to a suitable outlet, such as a sump pit or storm sewer.

In order to minimize ponding of water adjacent to the foundation walls, roof water should be controlled by a roof drainage system that directs water away from the building to prevent ponding of water adjacent to the foundation wall. The exterior grade should be sloped away from the building to promote water drainage away from the foundation walls.

5.9 Foundation Walls Backfill (Shallow Foundations)

To prevent possible foundation frost jacking and lateral loading, the backfill material against any foundation walls, grade beams, isolated walls, or piers should consist of free draining, non-frost susceptible material such as sand or sand and gravel meeting OPSS Granular B Type II or equivalent grading requirements.

The foundation wall backfill should be compacted to minimum 95% of its SPMDD using light compaction equipment, where no loads will be set over top. The compaction shall be increased to 98% of its SPMDD under walkways, slabs or paved areas close to the foundation or retaining walls. Backfilling against foundation walls should be carried out on both sides of the wall at the same time where applicable.

5.10 Trees

It should be noted that the clayey soils at the site may be sensitive to water depletion by trees of high water demand during periods of dry weather. When trees draw water from the clay, it undergoes shrinkage which can result in settlement of adjacent structures. The zone of influence of a tree is considered to be approximately equal to the mature height of the tree. Therefore, trees which have a high water demand should not be planted closer to structures than the ultimate height of the trees. **Appendix E** provides a list of the common trees in decreasing order of water demand and, accordingly, decreasing risk of potential effects on structures

5.11 Slab-on-grade Construction

Concrete slab-on-grade should rest over compacted, free draining and well graded structural fill only. Therefore, all fill including organic or otherwise deleterious material shall be removed from the proposed building's footprint. The exposed undisturbed native subgrade should then be inspected and approved by qualified geotechnical personnel.

Any underfloor fill needed to raise the general floor grade shall consist of OPSS Granular B Type II material or an approved equivalent, compacted to 95% of its SPMDD. The final lift shall be compacted to 98% of its SPMDD. A 200 mm Granular A meeting the **OPSS 1010** shall be placed underneath the slab and compacted to 100% of its SPMDD. Alternatively, if wet condition persists, 200 mm thickness of 19 mm clear stone meeting the **OPSS 1004** requirements shall be used instead of Granular A.

It is also recommended that any area exterior slab-on-grade (sidewalks, ramp etc.) shall be constructed using Granular B subbase of thickness 300 mm and Granular A base of thickness 150 mm with incorporating subdrain facilities. The modulus of subgrade reaction (ks) for the design of the slabs set over competent native soil/structural fill is **22 MPa/m.**

Drainage tile consisting of 100 mm diameter weeping tile wrapped with a filter cloth is also recommended to install underneath the floor slab with invert to be at least 300 mm below underside of the floor slab in parallel rows of 5.0 m spacing in one direction. In order to further minimize and control cracking, the floor slab shall be provided with wire or fibre mesh reinforcement and construction or control joints. The construction or control joints should be spaced equal distance in both directions and should not exceed 4.5 m. The wire or fibre mesh reinforcement shall be carried out through the joints.

If any areas of the proposed building area are to remain unheated during the winter period, thermal protection of the slab on grade may be required. The "Guide for Concrete Floor and Slab Construction", **ACI 302.1R-04** is recommended to follow for the design and construction of vapour retarders below the floor slab. Further details on the insulation requirements could be provided, if necessary.

5.12 Retaining Walls and Shoring

The following **Table 4** below provides the suggested soil parameters for the design of retaining wall and/or shoring systems. For excavations near existing services and structures, the coefficient of earth pressure at rest (K_o) should be used. Material properties for shoring and permanent wall design (static) are shown in details in **Table 4**.

Table 4: Material Properties for Shoring and Permanent Wall Design (Static)

Type of Material	Bulk Density (kN/m^3)	Friction Angle (Φ)	Pressure Coefficient		
			At Rest (K_0)	Active (K_A)	Passive (K_P)

Granular A	23.0	34	0.44	0.28	3.53
Granular B Type I	20.0	31	0.49	0.32	3.12
Granular B Type II	23.0	32	0.47	0.31	3.25
Silt and Clay	18.5	33	0.46	0.29	3.34
Silt and Sand Till	20.5	40	0.36	0.22	4.56

The above values are for a flat surface behind the wall, a straight wall and a wall friction angle of 0°. The designer should consider any difference between these coefficients, and make appropriate corrections for a sloped surface behind the wall, angled wall or wall friction as required. The bearing capacity for the design of a retaining wall are the same as provided for the building structure provided it is founded over the same soil stratum.

Retaining walls should also be designed to resist the earth pressures produces under seismic conditions. The total active thrust (P_{AE}) in seismic condition includes both a static component (P_A) and a dynamic component (ΔP_{AE}), and can be calculated as follows:

The active thrust, $P_{AE} = P_A + \Delta P_{AE}$

Where

$P_A = ½ K_A \gamma H^2$

(K_A = 0.31 for Granular B Type II. For other material, use relevant value for K_A from the above Table 4)

H = Total height of the wall (m)

γ = Unit weight of the backfill material (kN/m^3)

These dynamic thrust (ΔP_{AE}) can be calculated from

$\Delta P_{AE}, = 0.375 (a_c \gamma H^2/g)$

Where

$a_c = (1.45 – a_{max}/g)a_{max}$

The peak ground acceleration (PGA) or a_{max}, for the area is 0.28g according to 2015 National Building Code Seismic Hazard Calculation and acceleration of gravity, g = 9.81 m/s^2. The seismic coefficient in the vertical direction is assumed to be negligible. The total active thrust P_{AE} may be considered to act at a height, h (m), from the base of the wall,

$h = [P (H/3) + \Delta P_{AE} (0.6H)]/ P_{AE}$

Internal force acting on the reinforced zone, $P_{IR} = a_c \gamma_r HL/g$

Where

γ_r is the unit weight of reinforced zone.

Add P_{AE} and 0.5 P_{IR} to check the stability. Factor of safety (Seismic) ≥ 0.75 Factor of safety (Static)

6 EXCAVATION AND BACKFILLING REQUIREMENTS

6.1 Excavation

It is anticipated that the depth of excavation for the residential units and any associated services will not extend below 2.4 m bgs. Most of the excavation being carried out will be through silt and clay and silt and sand till. Excavation must be carried out in accordance with Occupational Health and Safety Act and Regulations for Construction Projects.

According to the Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 213/91 and its amendments, the surficial overburden expected to be excavated into at this site can be classified as Type 3. Therefore, shallow temporary excavations in overburden soil classified as Type 3 can be cut at 1 horizontal to 1 vertical (1H: 1V), for a fully drained excavation starting at the base of the excavation and as per requirements of the OHSA regulations.

In the event that the aforementioned slopes are not possible to achieve due to space restrictions, the excavation shall be shored according to OHSA O. Reg. 213/91 and its amendments. A geotechnical engineer shall design and approve the shoring and establish the shoring depth under the excavation profile. Refer to the parameters provided in **Table 4** in **Section 5.12** for use in the design of any shoring structures.

Any excavated material stockpiled near an excavation or trench should be stored at a distance equal to or greater than the depth of the excavation/trench and construction equipment, traffic should be limited near open excavation.

6.2 Groundwater Control

Based on the subsurface conditions encountered at this site, very minor groundwater seepage or infiltration from the native soils, if any, can be expected into the shallow temporary excavations during construction. It is anticipated that pumping from open sumps will be sufficient to control any groundwater inflow through the vertical face of excavations. Surface water runoff into the excavation should be minimized and diverted away from the excavation if possible.

A permit to take water (PTTW) is required from Ministry of Environment and Climate Change (MOECC), Ontario Reg. 387/04, if more than 400,000 litres per day of groundwater will be pumped during a construction period less than 30 days. Registration in the Environmental Activity and Sector Registry (EASR) is required when the takings of ground water and storm water for the purpose of dewatering construction projects range between 50,000 and 400,000 litres per day.

Based on the field investigation through localized borings, it is anticipated that pumping of groundwater will not exceed 50,000 litres per day. As such, no PTTW nor registration in the EASR is anticipated to be required for the construction of this development.

6.3 Pipe Bedding Requirements

It is anticipated that if any underground services are required as part of this project, they will be founded over silt and clay or silt and sand till. Alternately, underground services may be founded over properly prepared and approved structural fill, where excavation below the invert is required. Consequently all organic material should be removed down to a suitable bearing layer. Any sub-excavation of disturbed soil should be removed and replaced with a Granular B Type II or approved equivalent, laid in loose lifts of thickness not exceeding 300 mm and compacted to 95% of its SPMDD. Bedding, thickness of cover material and compaction requirements for watermains and sewer pipes should conform to the manufacturers design requirements and to the detailed installations outlined in the Ontario Provincial Standard Specifications (OPSS) and any applicable standards or requirements from the City of Ottawa.

As an alternative to Granular A bedding and only where wet conditions are encountered, the use of “clear stone” bedding, such as 19 mm clear stone, **OPSS 1004**, may be considered only in conjunction with a suitable geotextile filter (such as terrafix 270R or approved equivalent). Without proper filtering, there may be entry of fines from native soils and trench backfill into the bedding, which could result in loss of support to the pipes and possible surface settlements. The sub-bedding, bedding and cover materials should be compacted in maximum 200 mm thick lifts to at least 95% of its SPMDD within ±2% of its optimum moisture content using suitable vibratory compaction equipment.

6.4 Trench Backfill

All service trenches should be backfilled using compactable material, free of organics, debris and large cobbles or boulders. Acceptable native materials (if encountered and where possible) should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetrations (i.e. 1.8 m below finished grade) in order to reduce the potential for differential frost heaving between the new excavated trench and the adjacent section of roadway. Where native backfill is used, it should match the native materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type II. Any boulders larger than 150 mm in size should not be used as trench backfill.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadway, the trench should be compacted in maximum 300 mm thick lifts to at least 95% of its SPMDD. The specified density may be reduced where the trench backfill is not located within or in close proximity to existing roadways or any other structures.

For trenches carried out in existing paved areas, transitions should be constructed to ensure that proper compaction is achieved between any new pavement structure and the existing pavement structure to minimize potential future differential settlement between the existing and new pavement structure. The transition should start at the subgrade level and extend to the underside of the asphaltic concrete level (if any) at a 1 horizontal to 1 vertical slope. This is especially important where trench boxes are used and where no side slopes is provided to the excavation. Where asphaltic concrete is present, it should be cut back to a minimum of 150 mm from the edge of the excavation to allow for proper compaction between the new and existing pavement structures.

7 Slope Stability Analysis

7.1 Slope Description

The slope under review is located throughout the site, and slopes downwards from the southwest to northeast. Overall, the slope has a relatively constant slope profile of about 10H:1V, having a total height of approximately 6 m. The slope profile was determined using a combination of field measurements, and a grading plan, developed by D.B. Gray Engineering Inc (dated: Oct 15, 2018) provided to LRL by Ark Construction.

Due to the seasonal timing of this field investigation, the slope was not able to be inspected for any signs of former slope failure. Based on aerial photos, it is assumed the site is covered with manicured grasses, with the exception of the existing residential houses and pavement structures.

7.2 Slope Stability Results

The slope modelling program, Slide 5.0 (Rocscience), was used to implement the Bishop simplified method of slices. The slope profile for the existing slope conditions, as measured in the field and the proposed slope profile including the residential units was used and modeled to check the conditions of the slope. The slope was analyzed under both the undrained (short term failure) and drained (long term failure) conditions.

The field measurements in conjunction with known published data of the materials encountered onsite were used for selection of appropriate soil modelling parameters in the slope stability analyses.

The results of the analyses are potentially dependent on the assumption of groundwater condition. During the development of this report, no information on the groundwater level was available throughout the year. However, as a conservative approach the analysis was completed assuming full saturation throughout the slope profile. The location of the slope profile A-A is shown on Figure 2, attached in **Appendix A**.

Table 5: Soil Parameters used in Slope Stability Analysis

Soil Type	Effective cohesion (c') - KPa	Angle of internal friction (ϕ') - degrees	Bulk unit weight (γ_B) – KN/m^3
Drained Parameters (Long Term)			
Granular Material (Grade Raise)	1	34	22.0
Silt and Clay	5	33	18.0
Silt and Sand Till	2	40	20.5
Undrained Parameters (Short Term)			
Granular Material (Grade Raise)	1	34	22.0
Silt and Clay	100	-	18.0
Silt and Sand Till	2	40	20.5

The factor of safety (FoS) against slope failure was run with the additional loading for the residential units for the undrained and drained condition. The FoS values were found ranging between 2.07 and 5.41. A FoS of 1.5 or greater is considered to be safe with regards to slope stability. The designed load was not provided (design bearing pressure at serviceability limit state) while generating this report. However a typical value assumed to be 75 kPa for similar type of residential construction was included within the model.

These results indicate that the proposed development will not have a negative effect on the stability of the slope; in both the long and short term.

The model results are included in **Appendix E.**

7.3 Conclusion

Based on the information presented herein, the proposed development may be constructed safely given the conceptual plan does not differ than what is illustrated on the Grading Plan, submitted to LRL.

If any additional structures are considered to be constructed beyond what was indicated/proposed, LRL should be consulted to ensure that the results of this report are still valid.

8 Reuse of On-Site Soils

The existing surficial overburden soils consist mostly of silt and clay and silt and sand till. The silt and clay is considered to be frost susceptible and should not be used as backfill material directly against foundation walls or underneath unheated concrete slabs. However, it could be reused as general backfill material (service trenches, general landscaping/backfilling) if it can be compacted according to the specifications outlined herein at the time of construction and found free from any waste, organics and debris. The silt and sand till is non frost susceptible, and could potentially be used as backfill material given further laboratory testing is carried out on this material (ie: Proctor and sieve analysis). Any imported material shall conform to OPSS Granular B – Type II or approved equivalent.

It should be noted that the adequacy of any material for reuse as backfill will depend on its water content at the time of its use and on the weather conditions prevailing prior to and during that time. Therefore, all excavated materials to be reused shall be stockpiled in a manner that will prevent any significant changes in their moisture content, especially during wet conditions. Any excavated materials proposed for reuse should be stockpiled in a manner to promote drying and should be inspected and approved for reuse by a geotechnical engineer.

9 Recommended Pavement Structure

It is anticipated that the subgrade soils for the new parking and access lanes will consist of either silt and clay and/or silt and sand till. The construction of access lanes and parking areas will be acceptable over these undisturbed materials once all debris, organic material, or otherwise deleterious material are removed from the subgrade area. Furthermore, the subgrade must be compacted using a suitable heavy duty compacting equipment and approved by a geotechnical engineer prior to placing any granular base material.

The following **Table 6** presents the recommended pavement structures to be constructed over a stable subgrade along the proposed parking areas and access lane or driveway as part of this project.

Table 6: Recommended Pavement Structure

Course	Material	Thickness (mm)	
		Light Duty Parking Area (mm)	Heavy Duty Parking Area (Access Roads, Fire Routes and Trucks) (mm)
GBE		450	615
Surface	HL3 A/C	50	50

Binder	HL8 A/C	-	50
Base course	Granular A	150	150
Sub base	Granular B Type II	300	400
Total:		500	650

Performance Graded Asphaltic Cement (PGAC) **58-34** is recommended for this project.

The base and subbase granular materials shall conform to **OPSS 1010** material specifications. Any proposed materials shall be tested and approved by a geotechnical engineer prior to delivery to the site and shall be compacted to 98% of its SPMDD. Asphaltic concrete shall conform to **OPSS 1150** and be placed and compacted to at least 93% of the Marshall Density. The mix and its constituents shall be reviewed, tested and approved by a geotechnical engineer prior to delivery to the site.

9.1 Paved Areas & Subgrade Preparation

The access lanes and parking areas shall be stripped of vegetation, debris and other obvious objectionable material. Following the backfilling and satisfactory compaction of any underground service trenches up to the subgrade level, the subgrade shall be shaped, crowned and proof-rolled. A tandem axle, dual wheel dump truck or heavy duty smooth drum roller shall be used for proof-rolling. Any resulting loose/soft areas should be sub-excavated down to an adequate bearing layer and replaced with approved backfill.

The preparation of the subgrade shall be scheduled and carried out in a manner so that a protective cover of overlying granular material (if required) is placed as quickly as possible in order to avoid unnecessary circulation by heavy equipment, except on unexcavated or protected surfaces. Frost protection of the surface shall be implemented if works are carried out during the winter season.

The performance of the pavement structure is highly dependent on the subsurface groundwater conditions and maintaining the subgrade and pavement structure in a dry condition. To intercept excess subsurface water within the pavement structure granular materials, sub-drains with suitable outlets should be installed below the pavement area's subgrade if adequate overland flow drainage is not provided (i.e. ditches). The surface of the pavement should be properly graded to direct runoff water towards suitable drainage features. It is recommended that the lateral extent of the subbase and base layers not be terminated vertically immediately behind the curb/edge of pavement line but be extended beyond the curb.

10 INSPECTION SERVICES

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed site do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design.

All footing areas and any structural fill areas for the proposed structures should be inspected by LRL to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundations and slab-on-grade should be inspected to ensure that the materials used conform to the grading and compaction specifications.

The subgrade for the pavement areas and underground services should be inspected and approved by geotechnical personnel. In-situ density testing should be carried out on the pavement granular materials, pipe bedding and backfill to ensure the materials meet the specifications for required compaction.

If footings are to be constructed during winter season, the footing subgrade should be protected from freezing temperatures using suitable construction techniques.

11 REPORT CONDITIONS AND LIMITATIONS

It is stressed that the information presented in this report is provided for the guidance of the designers and is intended for this project only. The use of this report as a construction document or its use by a third party beyond the client specifically listed in the report is neither intended nor authorized by LRL Associates Ltd. Contractors bidding on or undertaking the work should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible contamination resulting from previous uses or activities at this site or adjacent properties, and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report.

The recommendations provided in this report are based on subsurface data obtained at the specific boring locations only. Boundaries between zones presented on the borehole are often not distinct but transitional and were interpreted. Experience indicates that the subsurface soil and groundwater conditions can vary significantly between and beyond the test locations. For this reason, the recommendations given in this report are subject to a field verification of the subsurface soil conditions at the time of construction.

The recommendations are applicable only to the project described in this report. Any changes to the project will require a review by LRL Associates Ltd., to insure compatibility with the recommendations contained in this project.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report or if we may be of further services to you, please do not hesitate to contact the undersigned.

Yours truly,
LRL Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
Feb 22, 2019
B. W. JOHNSON
100510537
PROVINCE OF ONTARIO

Brad Johnson, P.Eng.
Geotechnical Engineer

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APPENDIX A

Site and Borehole Location Plan

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED 12-UNIT RESIDENTIAL DEVELOPMENT
1164 - 1166 HIGHCROFT DR.
MANOTICK, ON

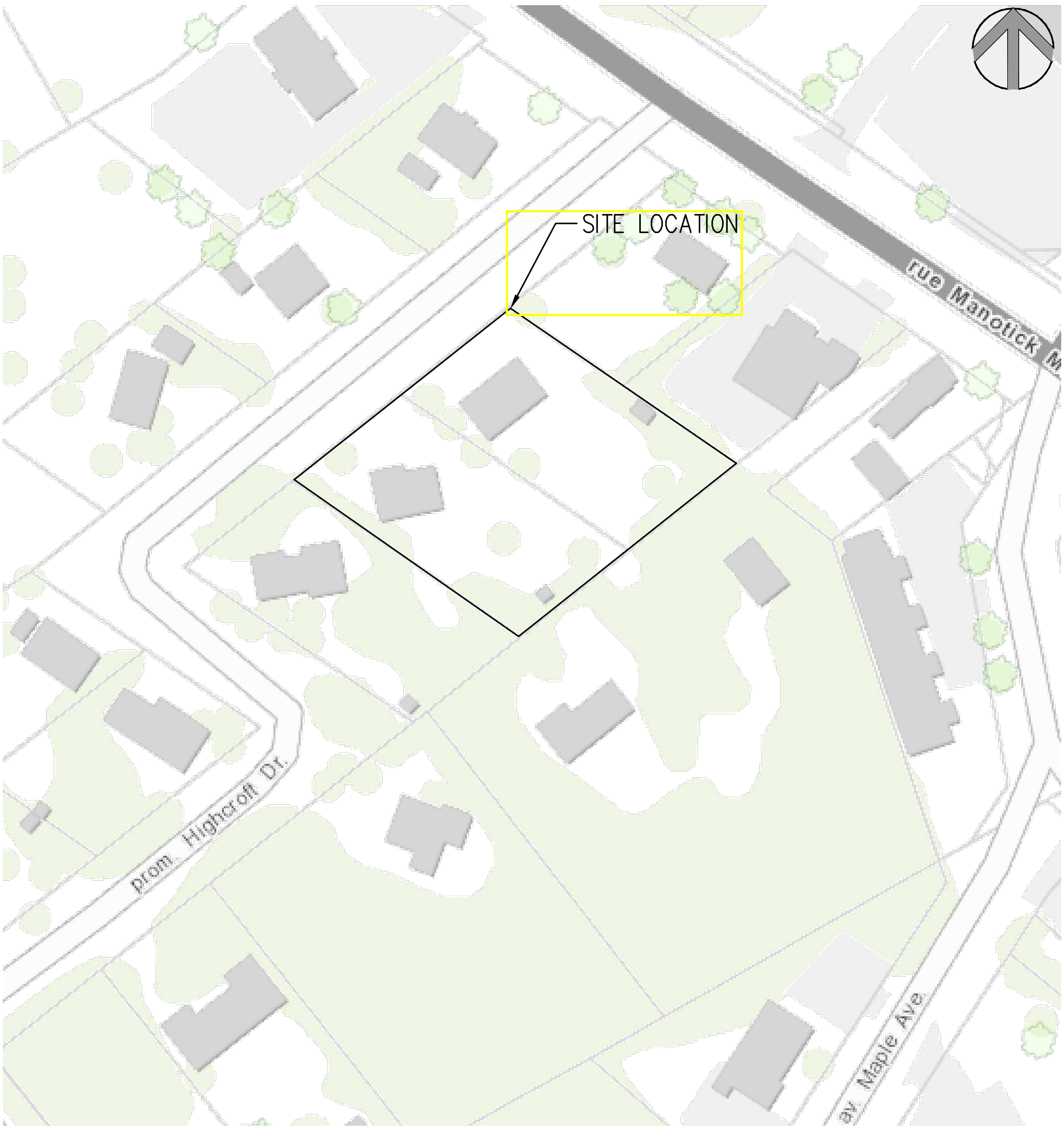
LRL
ENGINEERING | INGÉNIERIE

5430 Canotek Road | Ottawa, ON, K1J 9G2
www.lrl.ca | (613) 842-3434

DRAWING TITLE

SITE LOCATION
SOURCE: geoOttawa

CLIENT	DATE	PROJECT	
ARK CONSTRUCTION LTD.	FEBRUARY, 2019	180783	FIGURE 1



PROJECT

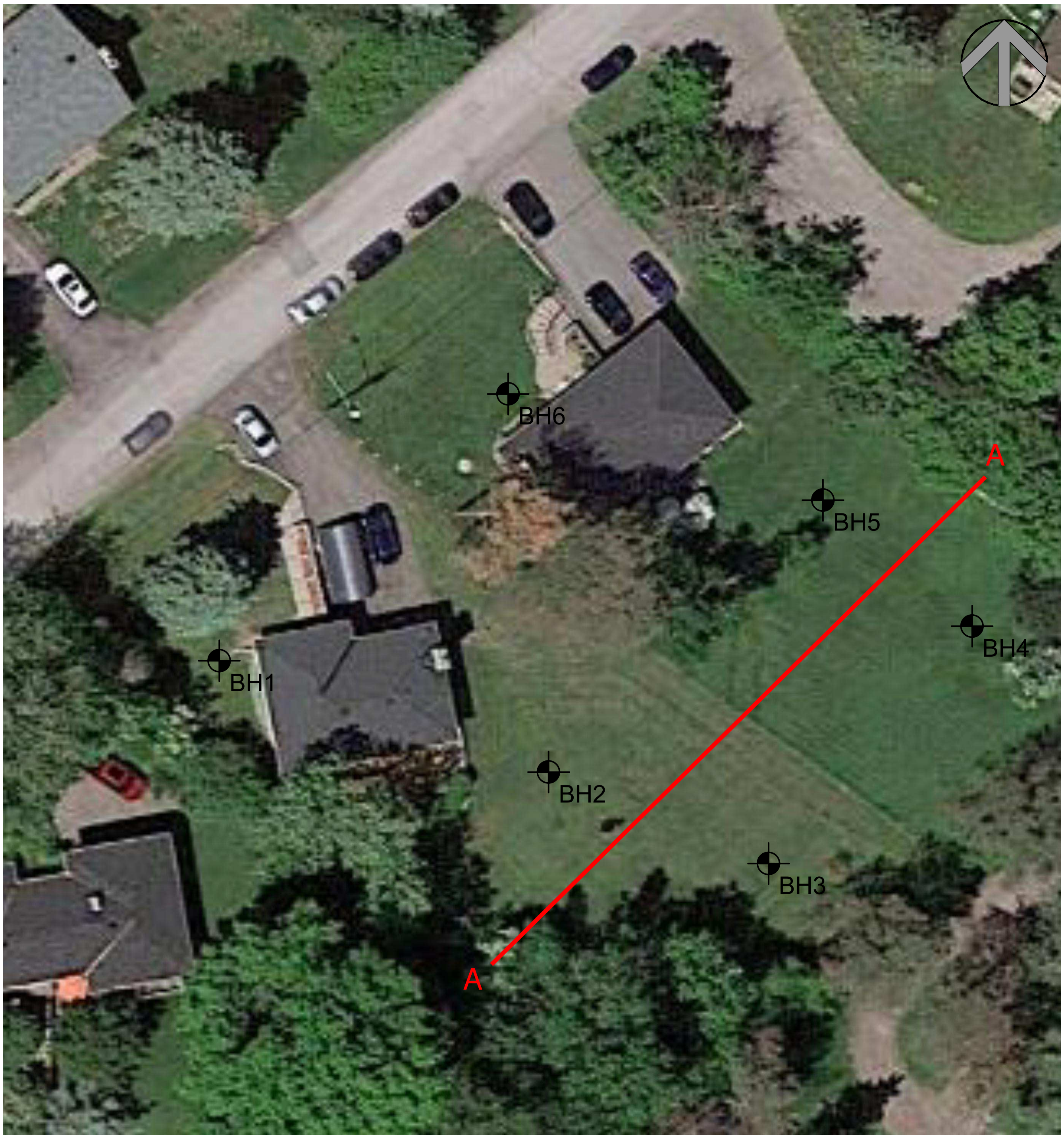
GEOTECHNICAL INVESTIGATION
PROPOSED 12-UNIT RESIDENTIAL DEVELOPMENT
1164 - 1166 HIGHCROFT DR.
MANOTICK, ON

LRL
ENGINEERING | INGÉNIERIE
5430 Canotek Road | Ottawa, ON, K1J 9G2
www.lrl.ca | (613) 842-3434

DRAWING TITLE

BOREHOLE LOCATION
SOURCE: Imagery 2018 Google, DigitalGlobe Map Data

CLIENT	DATE	PROJECT	
ARK CONSTRUCTION LTD.	FEBRUARY, 2019	180783	FIGURE 2



APPENDIX B

Borehole Logs

Borehole Log: BH1

Project No.: 180783

Project: Proposed 12 Unit Residential Development

Client: ARK Construction Ltd.

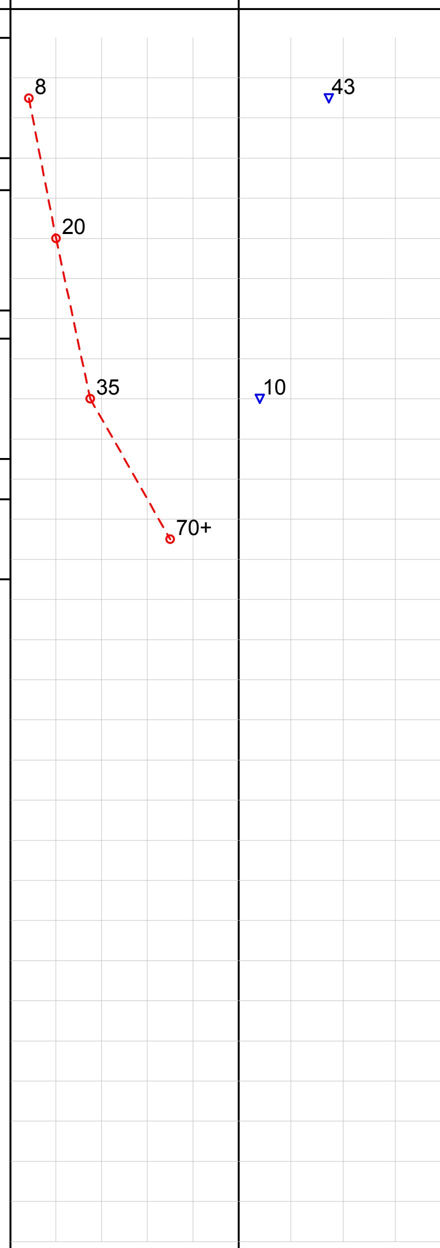
Location: 1164-1166 Highcroft Drive, Manotick ON

Date: January 29, 2019

Field Personnel: BJ

Driller: George Downing Estate Drilling Ltd. **Drilling Equipment:** Track Mount CME 45 **Drilling Method:** HSA

SUBSURFACE PROFILE		SAMPLE DATA						Shear Strength (kPa) 50 100 150 200 / SPT N Value (Blows/0.3 m) 20 40 60 80	Water Content (%) 25 50 75 / Liquid Limit (%) 25 50 75	Water Level (Standpipe or Open Borehole)
Depth (ft / m)	Soil Description	Elev./Depth(m)	Lithology	Type	Sample Number	N or RQD	Recovery (%)			
0	Ground Surface	98.79								
	TOPSOIL- sandy, mixed with black organics.	0.00			SS1-1	8	85	8	43	
		98.19								
	SILT and SAND TILL- trace clay, some gravel sized stone, greyish brown, moist, compact to very dense.	0.60			SS1-2	20	75	20		
					SS1-3	35	60	35	10	
		96.30								
	End of Borehole	2.49			SS1-4	70+	50	70+		



Easting: 445968 **Northing:** 5008314

Site Datum: Top of Public Utility Pedestal Box Located at NW of 1166 Highcroft Dr.

Groundsurface Elevation: 99.785 m **Top of Riser Elev.:** N/A

Hole Diameter: 200 mm

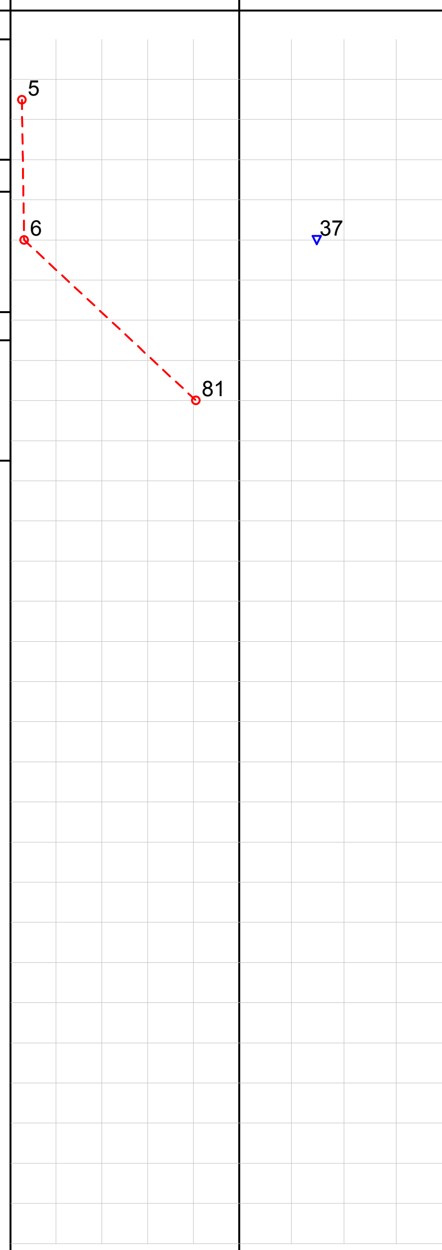
NOTES: Practical auger refusal encountered over large boulder or possible bedrock.

Borehole Log: BH2

Project No.: 180783
Project: Proposed 12 Unit Residential Development
Client: ARK Construction Ltd.
Location: 1164-1166 Highcroft Drive, Manotick ON
Date: January 29, 2019
Field Personnel: BJ
Driller: George Downing Estate Drilling Ltd.
Drilling Equipment: Track Mount CME 45
Drilling Method: HSA

SUBSURFACE PROFILE		SAMPLE DATA					
Depth (m)	Soil Description	Elev./Depth(m)	Lithology	Type	Sample Number	N or RQD	Recovery (%)
0	Ground Surface	96.69					
0	**TOPSOIL-** sandy, mixed with black organics.	0.00					
0.25	**SILT and SAND TILL-** trace clay, some gravel sized stone, greyish brown, moist, loose to very dense.	96.44 0.25		SS	SS2-1	5	75
				SS	SS2-2	6	50
				SS	SS2-3	81	85
2.06	End of Borehole	94.63 2.06					

Shear Strength (kPa): 50 100 150 200
SPT N Value (Blows/0.3 m): 20 40 60 80
Water Content (%): 25 50 75
Liquid Limit (%): 25 50 75
Water Level (Standpipe or Open Borehole)



Easting: 445994
Northing: 5008297
Site Datum: Top of Public Utility Pedestal Box Located at NW of 1166 Highcroft Dr.
Groundsurface Elevation: 96.69 m
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

NOTES: Practical auger refusal encountered over large boulder or possible bedrock.

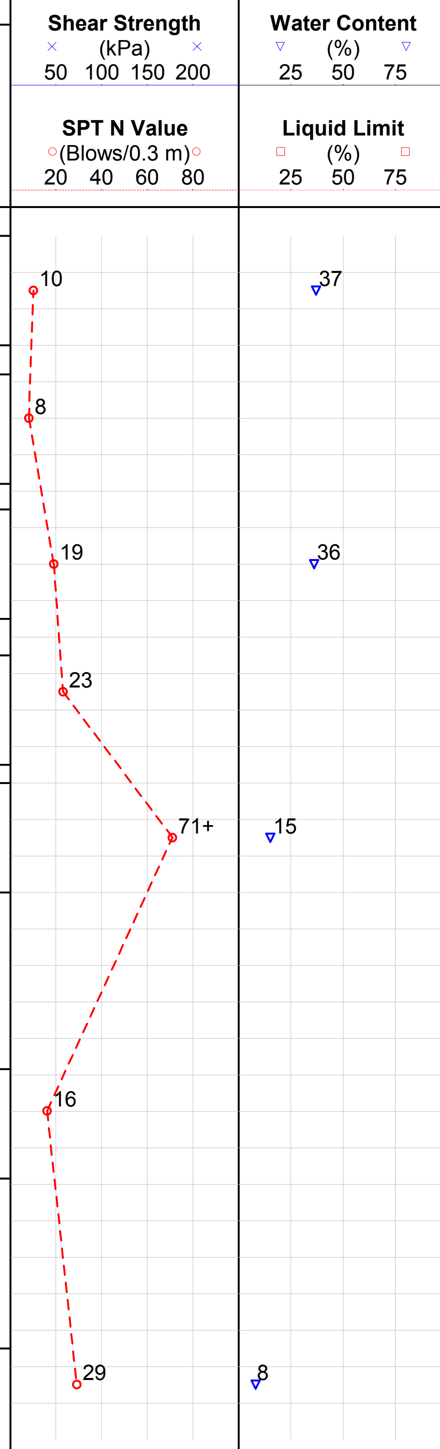
Borehole Log: BH3

Project No.: 180783
Project: Proposed 12 Unit Residential Development
Client: ARK Construction Ltd.
Location: 1164-1166 Highcroft Drive, Manotick ON
Date: January 29, 2019
Field Personnel: BJ
Driller: George Downing Estate Drilling Ltd.
Drilling Equipment: Track Mount CME 45
Drilling Method: HSA

SUBSURFACE PROFILE / SAMPLE DATA

Soil Description	Elev./Depth(m)	Sample Number	N or RQD	Recovery (%)
Ground Surface	95.13			
TOPSOIL- sandy, mixed with black organics.	0.00 / 94.68	SS3-1	10	66
SILT and CLAY- trace sand, moist, grey, firm.	0.45 / 93.68	SS3-2	8	100
SILT and SAND TILL- trace clay, trace gravel sized stone, greyish brown, moist, compact to very dense.	1.45 / 89.03	SS3-3	19	70
		SS3-4	23	85
		SS3-5	71+	50
		SS3-6	16	45
End of Borehole	6.10	SS3-7	29	80

Depth: ft 0–21; m 0–6



Water Level (Standpipe or Open Borehole)

Easting: 446016
Northing: 5008296

Site Datum: Top of Public Utility Pedestal Box Located at NW of 1166 Highcroft Dr.

Groundsurface Elevation: 95.13 m
Top of Riser Elev.: N/A

Hole Diameter: 200 mm

Borehole Log: BH4

Project No.: 180783
Project: Proposed 12 Unit Residential Development
Client: ARK Construction Ltd.
Location: 1164-1166 Highcroft Drive, Manotick ON
Date: January 29, 2019
Field Personnel: BJ
Driller: George Downing Estate Drilling Ltd.
Drilling Equipment: Track Mount CME 45
Drilling Method: HSA

SUBSURFACE PROFILE / SAMPLE DATA

Soil Description	Elev./Depth(m)
Ground Surface	92.81
TOPSOIL- sandy, mixed with black organics.	0.00
SILT and CLAY- trace sand, grey, moist, firm to soft. -occasional sand seams around 3.0 m.	92.21 / 0.60
SILT and SAND TILL- trace clay, some gravel sized stone, greyish brown, wet, compact.	87.92 / 4.89
End of Borehole	5.01

Sample Number	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)	Water Content (%)
SS4-1	11	68	11	
SS4-2	10	100	10	35
SS4-3	11	100	11	
SS4-4	6	100	6	42
SS4-5	3	100	3	
SS4-6	19	80	19	32

Shear Strength (kPa): 50 100 150 200
SPT N Value (Blows/0.3 m): 20 40 60 80
Water Content (%): 25 50 75
Liquid Limit (%): 25 50 75
Water Level (Standpipe or Open Borehole)

Depth: ft 0–19; m 0–5

Easting: 446027
Northing: 5008312
Site Datum: Top of Public Utility Pedestal Box Located at NW of 1166 Highcroft Dr.
Groundsurface Elevation: 92.805 m
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

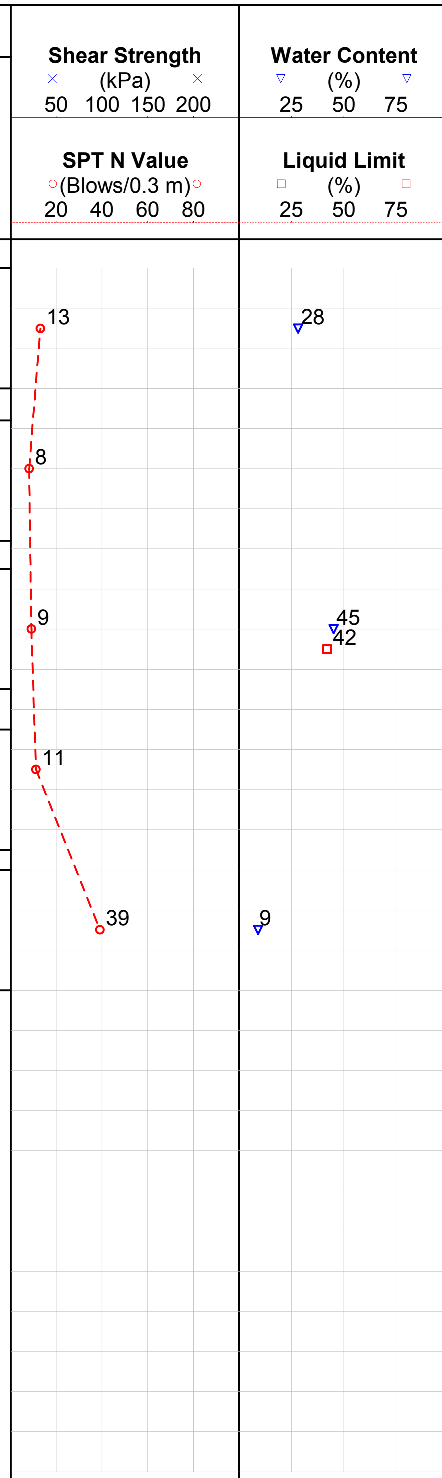
NOTES: Practical auger refusal encountered over large boulder or possible bedrock.

Borehole Log: BH5

Project No.: 180783 | **Project:** Proposed 12 Unit Residential Development
Client: ARK Construction Ltd. | **Location:** 1164-1166 Highcroft Drive, Manotick ON
Date: January 29, 2019 | **Field Personnel:** BJ

Driller: George Downing Estate Drilling Ltd. **Drilling Equipment:** Track Mount CME 45 **Drilling Method:** HSA

SUBSURFACE PROFILE		SAMPLE DATA					
Depth (ft / m)	Soil Description	Elev./Depth(m)	Lithology	Type	Sample Number	N or RQD	Recovery (%)
0 / 0	Ground Surface	92.81					
	TOPSOIL- sandy, mixed with black organics.	0.00					
	SILT and CLAY- trace sand, brownish grey, moist, stiff	0.15			SS5-1	13	60
					SS5-2	8	100
					SS5-3	9	100
					SS5-4	11	100
	SILT and SAND TILL- trace clay, some gravel sized stone, moist, brownish grey, dense.	89.84 2.97			SS5-5	39	60
	End of Borehole	88.80 4.01					



Water Level (Standpipe or Open Borehole)

Easting: 446019 **Northing:** 5008323

Site Datum: Top of Public Utility Pedestal Box Located at NW of 1166 Highcroft Dr.

Groundsurface Elevation: 92.805 m **Top of Riser Elev.:** N/A

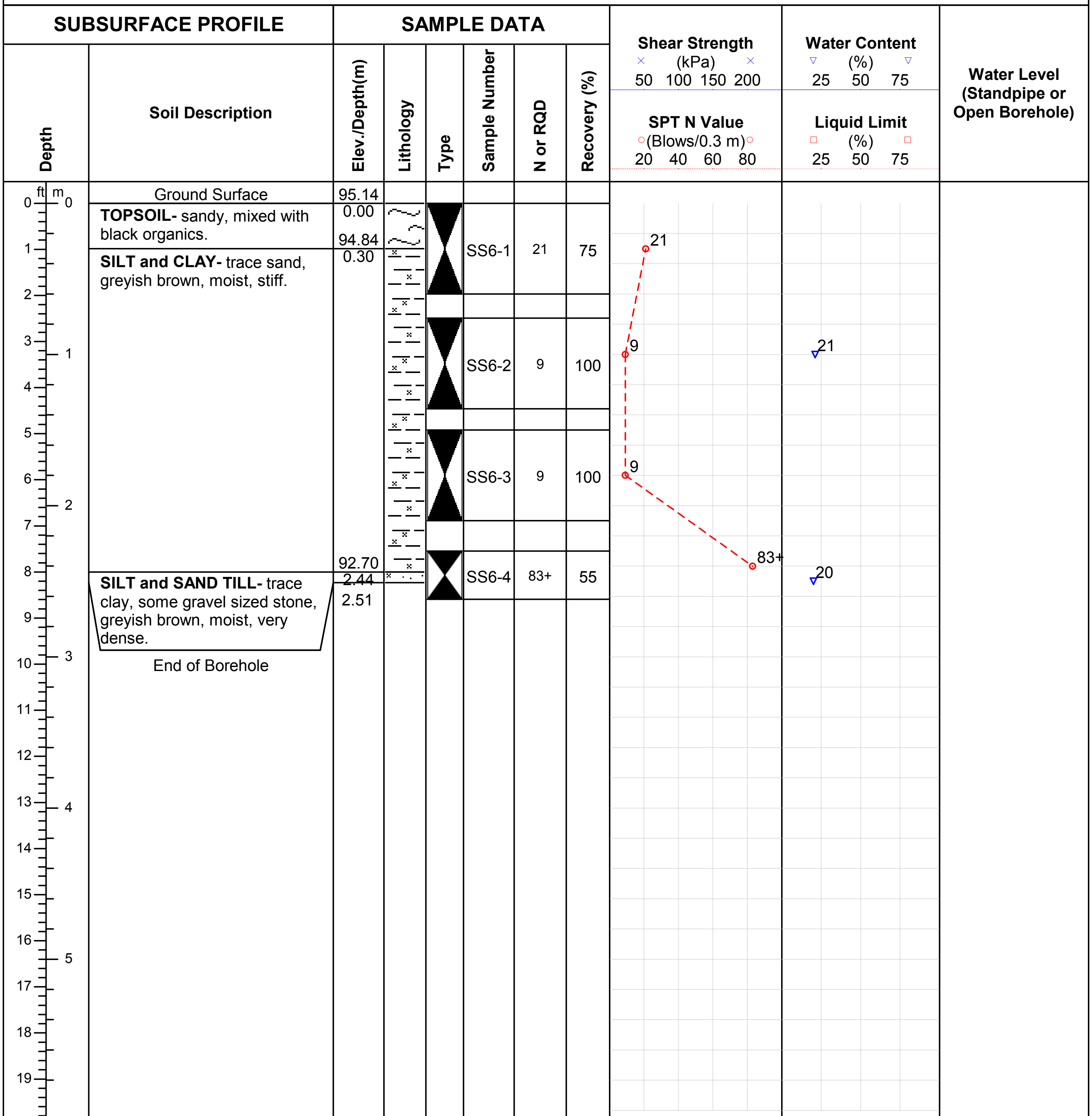
Hole Diameter: 200 mm

NOTES: Practical auger refusal encountered over large boulder or possible bedrock.

Borehole Log: BH6

Project No.: 180783
Project: Proposed 12 Unit Residential Development
Client: ARK Construction Ltd.
Location: 1164-1166 Highcroft Drive, Manotick ON
Date: January 29, 2019
Field Personnel: BJ
Driller: George Downing Estate Drilling Ltd.
Drilling Equipment: Track Mount CME 45
Drilling Method: HSA

Depth (ft/m)	Soil Description	Elev./Depth(m)	Sample Number	N or RQD	Recovery (%)	SPT N Value	Water Content (%)
0	Ground Surface	95.14					
0–0.30 m	**TOPSOIL-** sandy, mixed with black organics.	0.00	SS6-1	21	75	21	
0.30–2.44 m	**SILT and CLAY-** trace sand, greyish brown, moist, stiff.	94.84 / 0.30	SS6-2	9	100	9	21
			SS6-3	9	100	9	
2.44–2.51 m	**SILT and SAND TILL-** trace clay, some gravel sized stone, greyish brown, moist, very dense.	92.70 / 2.44	SS6-4	83+	55	83+	20
2.51 m	End of Borehole	2.51					



Easting: 445993
Northing: 5008332
Site Datum: Top of Public Utility Pedestal Box Located at NW of 1166 Highcroft Dr.
Groundsurface Elevation: 95.135 m
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

NOTES: Practical auger refusal encountered over large boulder or possible bedrock.

APPENDIX C

Symbols and Terms used in Borehole Logs

Symbols and Terms Used on Borehole and Test Pit Logs

1. Soil Description

The soil descriptions presented in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves some judgement and LRL Associates Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice. Boundaries between zones on the logs are often not distinct but transitional and were interpreted.

a. Proportion

The proportion of each constituent part, as defined by the grain size distribution, is denoted by the following terms:

Term	Proportions
“trace”	1% to 10%
“some”	10% to 20%
prefix (i.e. “sandy” silt)	20% to 35%
“and” (i.e. sand “and” gravel)	35% to 50%

b. Compactness and Consistency

The state of compactness of granular soils is defined on the basis of the Standard Penetration Number (N) as per ASTM D-1586. It corresponds to the number of blows required to drive 300 mm of the split spoon sampler using a metal drop hammer that has a weight of 62.5 kg and free fall distance of 760 mm. For a 600 mm long split spoon, the blow counts are recorded for every 150 mm. The “N” value is obtained by adding the number of blows from the 2nd and 3rd count. Technical refusal indicates a number of blows greater than 50.

The consistency of clayey or cohesive soils is based on the shear strength of the soil, as determined by field vane tests and by a visual and tactile assessment of the soil strength.

The state of compactness of granular soils is defined by the following terms:

State of Compactness Granular Soils	Standard Penetration Number “N”	Relative Density (%)
Very loose	0 – 4	<15
Loose	4 – 10	15 – 35
Compact	10 - 30	35 – 65
Dense	30 - 50	65 - 85
Very dense	> 50	> 85

The consistency of cohesive soils is defined by the following terms:

Consistency Cohesive Soils	Undrained Shear Strength (C_u) (kPa)	Standard Penetration Number “N”
Very soft	<12.5	<2
Soft	12.5 - 25	2 - 4
Firm	25 - 50	4 - 8
Stiff	50 - 100	8 - 15
Very stiff	100 - 200	15 - 30
Hard	>200	>30

c. Field Moisture Condition

Description (ASTM D2488)	Criteria
Dry	Absence of moisture, dusty, dry to touch.
Moist	Dump, but not visible water.
Wet	Visible, free water, usually soil is below water table.

2. Sample Data

a. Elevation depth

This is a reference to the geodesic elevation of the soil or to a benchmark of an arbitrary elevation at the location of the borehole or test pit. The depth of geological boundaries is measured from ground surface.

b. Type

Symbol	Type	Letter Code
	Auger	AU
	Split Spoon	SS
	Shelby Tube	ST
	Rock Core	RC

c. Sample Number

Each sample taken from the borehole is numbered in the field as shown in this column.

LETTER CODE (as above) – Sample Number.

d. Recovery (%)

For soil samples this is the percentage of the recovered sample obtained versus the length sampled. In the case of rock, the percentage is the length of rock core recovered compared to the length of the drill run.

3. Rock Description

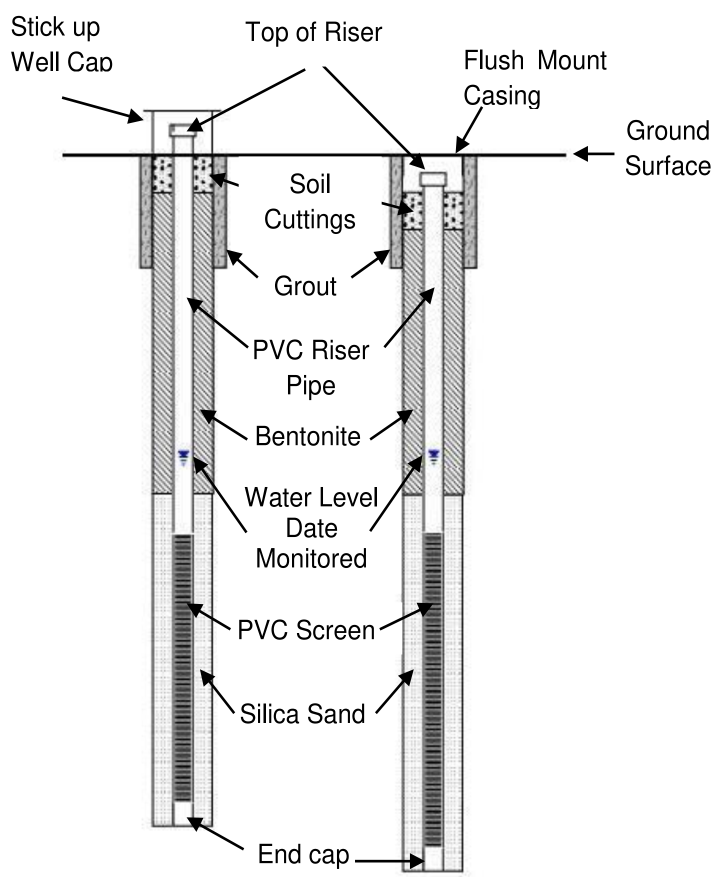
Rock Quality Designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mas. The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 100 mm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given below.

Rock Quality Designation (RQD) (%)	Description of Rock Quality
0 –25	Very poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

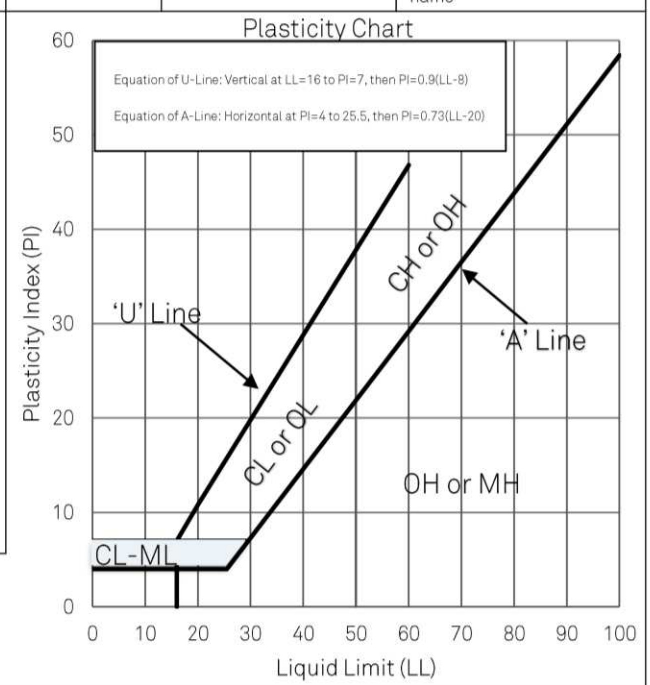
Strength classification of rock is presented below.

Strength Classification	Range of Unconfined Compressive Strength (MPa)
Extremely weak	< 1
Very weak	1 – 5
Weak	5 – 25
Medium strong	25 – 50
Strong	50 – 100
Very strong	100 – 250
Extremely strong	> 250

4. General Monitoring Well Data



5. Classification of Soils for Engineering Purposes (ASTM D2487)
(United Soil Classification System)

Major divisions			Group Symbol	Typical Names	Classification Criteria			
Coarse-grained soils More than 50% retained on No. 200 sieve* (>0.075 mm)	Gravels More than 50% of coarse fraction retained on No. 4 sieve(4.75 mm)	Clean gravels <5% fines	GW	Well-graded gravel	If 15% sand add "with sand" to group name.	Classification on basis of percentage of fines: Less than 5% pass No. 200 sieve - GW, GP, SW, SP More than 12% pass No. 200 sieve - GM, GC, SM, SC 5 to 12% pass No. 200 sieve - Borderline classifications, use of dual symbols	$C_u = \frac{D_{60}}{D_{10}} \geq 4$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravel			Not meeting either Cu or Cc criteria for GW	
		Gravels with >12% fines	GM	Silty gravel			Atterberg limits below "A" line or PI less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravel			Atterberg limits on or above "A" line and PI > 7	If fines are organic add "with orgnic fines" to group name
	Sands 50% or more of coarse fraction passes No. 4 sieve(<4.75 mm)	Clean sands <5% fines	SW	Well-graded sand	If 15% gravel add "with gravel to group name		$C_u = \frac{D_{60}}{D_{10}} \geq 6$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sand			Not meeting either Cu or C ccriteria for SW	
		Sands with >12% fines	SM	Silty sand			Atterberg limits below "A" line or PI less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sand			Atterberg limits on or above "A" line and PI > 7	If fines are organic add "with orgnic fines" to group name
Fine-grained soils50% or more passes No. 200 sieve* (<0.075 mm)	Silts and Clays Liquid Limit <50%	Inorganic	ML	Silt	If 15 to 29% coarse-grained, add "with sand" or "with gravel" as appropriate. If > 30% coarse-grained, add "sandy" or "gravelly" as appropriate. Class as organic when oven dried liquid limit is < 75% of undried liquid limit.			
			CL	Lean Clay -low plasticity				
		Organic	OL	Organic clay or silt (Clay plots above 'A' Line)				
	Silts and Clays Liquid Limit >50%	Inorganic	MH	Elastic silt				
			CH	Fat Clay -high plasticity				
		Organic	OH	Organic clay or silt (Clay plots above 'A' Line)				
	Highly Organic Soils		PT	Peat, muck and other highly organic soils				

APPENDIX D

Laboratory Results

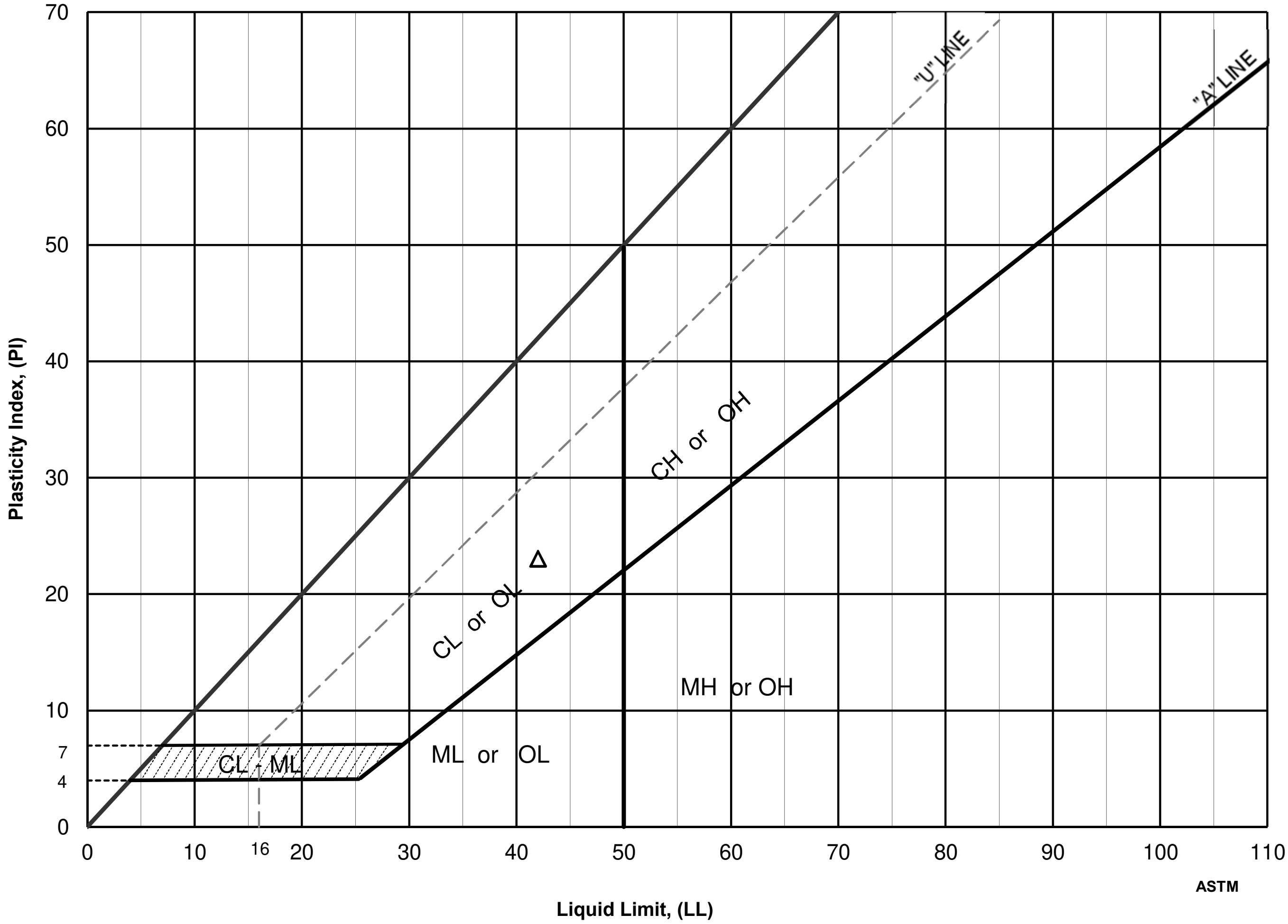
LRL Associates Ltd.

PLASTICITY INDEX

ASTM D 4318 / LS-703/704

Client:	ARK Construction Ltd.	**File No.:**	180783
Project:	Geotechnical Investigation	**Report No.:**	1
Location:	1164-1166 Highcroft Drive, Manotick, ON.	**Date:**	January 29, 2019

Plasticity Chart



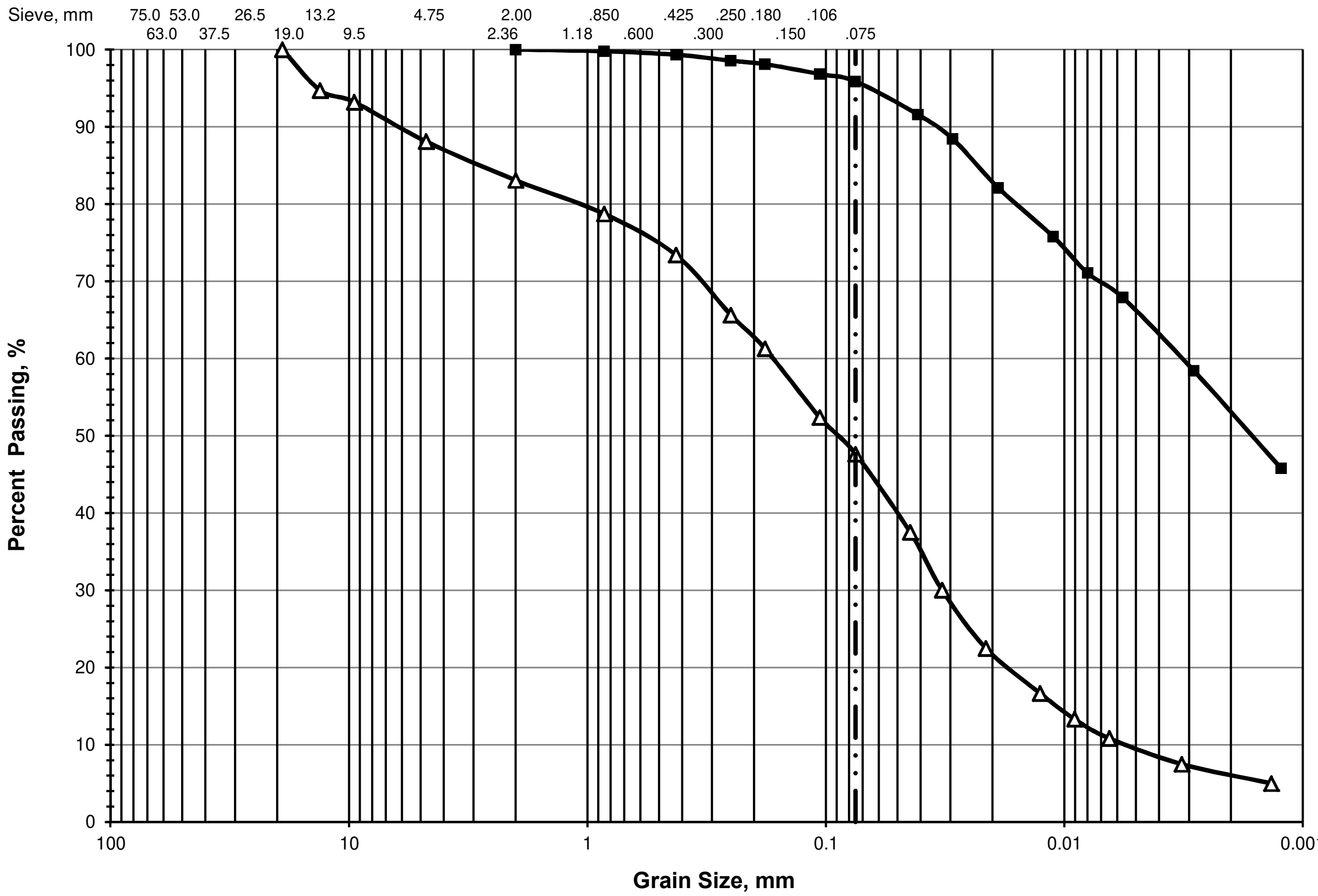
	Location	Sample	Depth, m	Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Activity Number	USCS
△	BH 5	SS-3	1.52 - 2.13	45	42	19	23	1.12	n/d	CL

LRL Associates Ltd.

PARTICLE SIZE ANALYSIS

ASTM D 422 / LS-702

Client:	ARK Construction Ltd.	**File No.:**	180783
Project:	Geotechnical Investigation	**Report No.:**	2
Location:	1164-1166 Highcroft Drive, Manotick, ON.	**Date:**	January 29, 2019



Unified Soil Classification System

	> 75 mm	% GRAVEL		% SAND			% FINES	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
△	0.0	0.0	11.9	5.0	9.7	25.8	41.8	5.9
■	0.0	0.0	0.0	0.0	0.7	3.5	44.1	51.7

	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH 2	SS-2	0.76 - 1.37	0.1693	0.0904	0.0325	0.0108	0.0057	1.1	29.7
■	BH 4	SS-2	0.76 - 1.37	0.0033	0.0018					

5430 Canotek Road | Ottawa, ON, K1J 9G2 | info@lrl.ca | www.lrl.ca | (613) 842-3434

APPENDIX E
Order of Water Demand for Common Trees

Some common trees in decreasing order of water demand:

Broad Leaved Deciduous

Poplar
Alder
Aspen
Willow
Elm
Maple
Birch
Ash
Beech
Oak

Deciduous Conifer

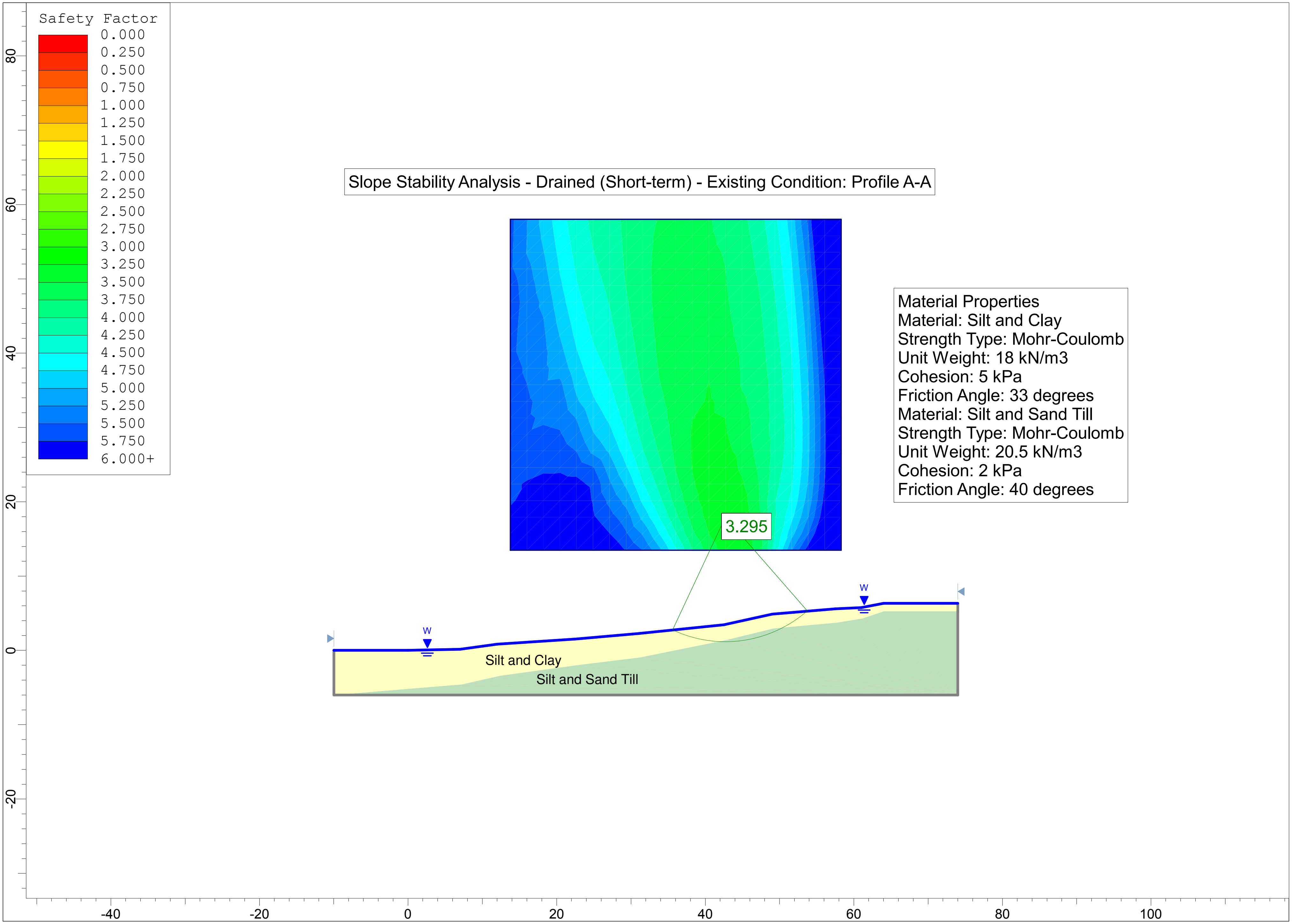
Larch

Evergreen Conifers

Spruce
Fir
Pine

APPENDIX F

Slope Stability Analysis Results


Safety Factor
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0.750
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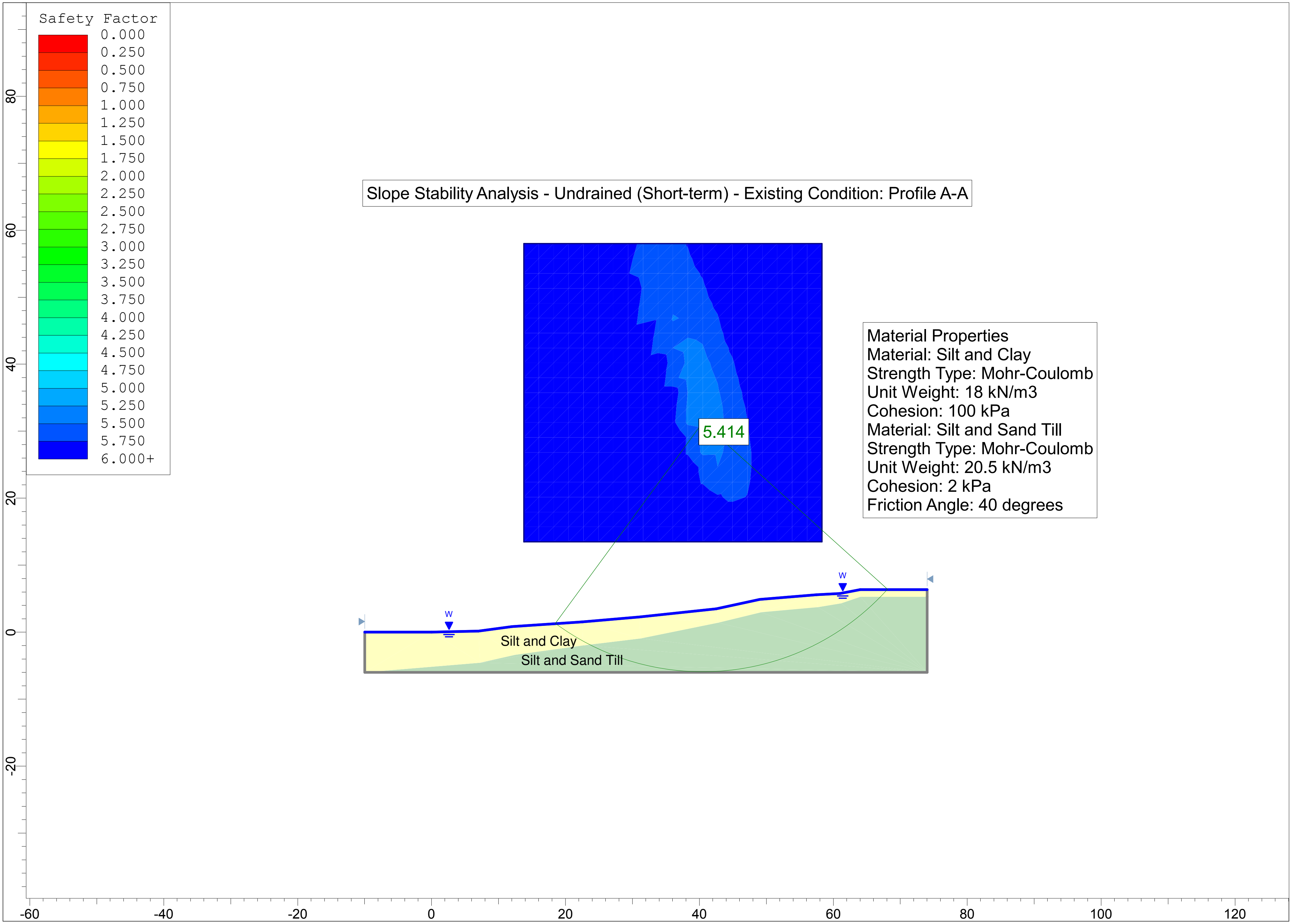
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Slope Stability Analysis - Drained (Short-term) - Existing Condition: Profile A-A
3.295
Material Properties
Material: Silt and Clay
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Material: Silt and Sand Till
Strength Type: Mohr-Coulomb
Unit Weight: 20.5 kN/m3
Cohesion: 2 kPa
Friction Angle: 40 degrees
W
W
Silt and Clay
Silt and Sand Till
80
60
40
20
0
-20
-40
-20
0
20
40
60
80
100


Safety Factor
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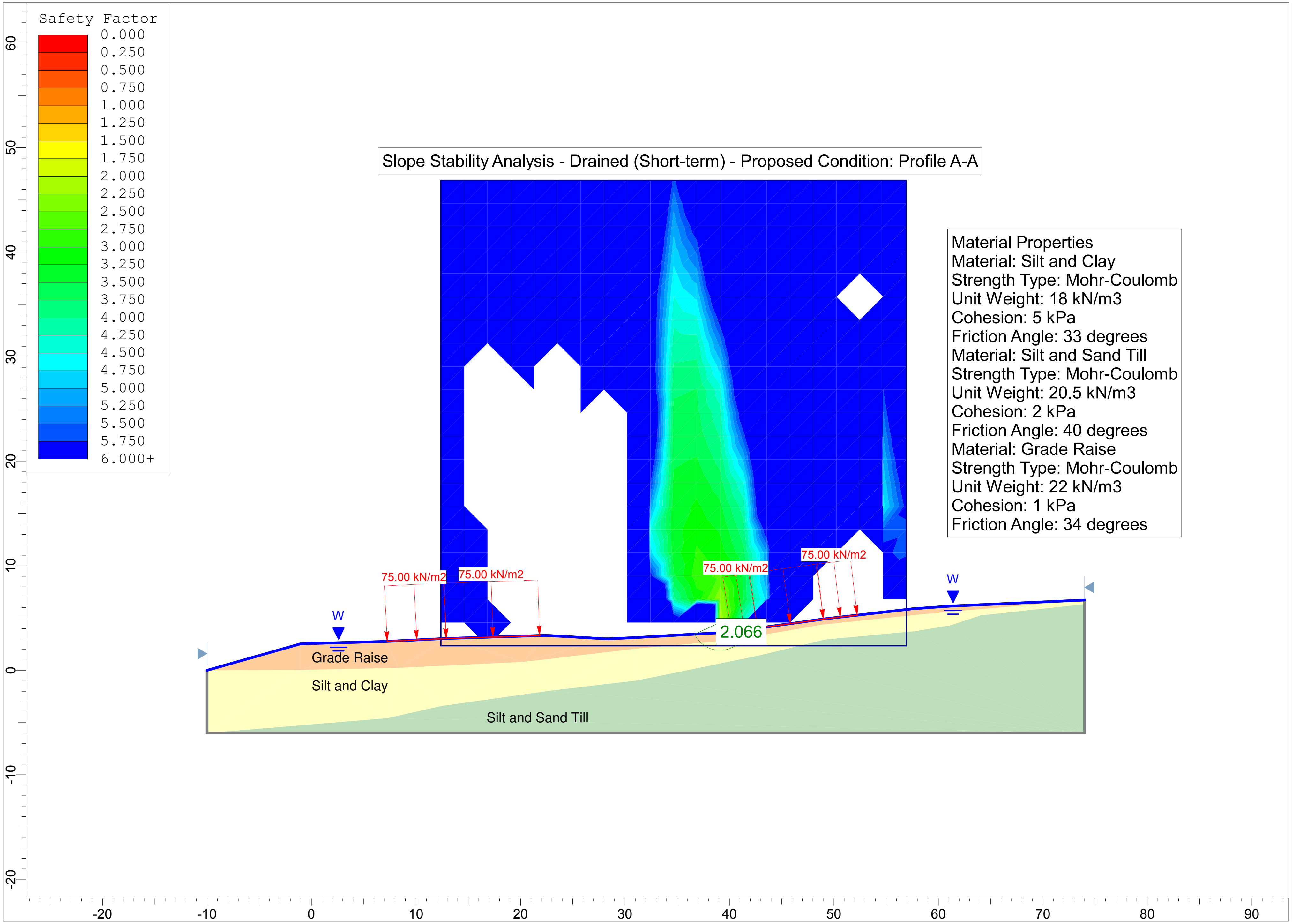
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Slope Stability Analysis - Undrained (Short-term) - Existing Condition: Profile A-A
5.414
Material Properties
Material: Silt and Clay
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 100 kPa
Material: Silt and Sand Till
Strength Type: Mohr-Coulomb
Unit Weight: 20.5 kN/m3
Cohesion: 2 kPa
Friction Angle: 40 degrees
W
W
Silt and Clay
Silt and Sand Till
80
60
40
20
0
-20
-60
-40
-20
0
20
40
60
80
100
120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0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
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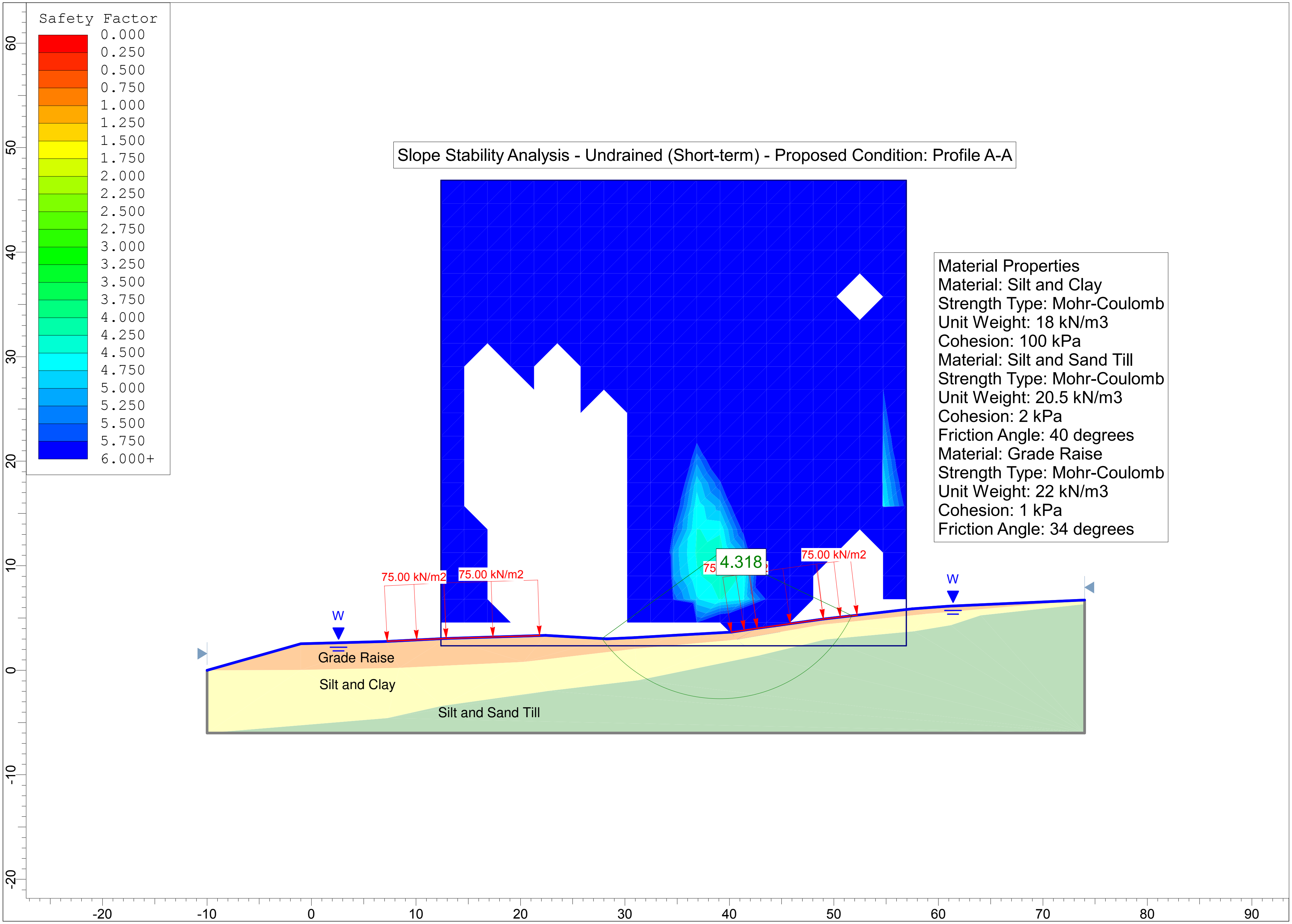
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Slope Stability Analysis - Drained (Short-term) - Proposed Condition: Profile A-A
Material Properties
Material: Silt and Clay
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Material: Silt and Sand Till
Strength Type: Mohr-Coulomb
Unit Weight: 20.5 kN/m3
Cohesion: 2 kPa
Friction Angle: 40 degrees
Material: Grade Raise
Strength Type: Mohr-Coulomb
Unit Weight: 22 kN/m3
Cohesion: 1 kPa
Friction Angle: 34 degrees
75.00 kN/m2
75.00 kN/m2
75.00 kN/m2
75.00 kN/m2
2.066
W
W
Grade Raise
Silt and Clay
Silt and Sand Till



Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Slope Stability Analysis - Undrained (Short-term) - Proposed Condition: Profile A-A
Material Properties
Material: Silt and Clay
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 100 kPa
Material: Silt and Sand Till
Strength Type: Mohr-Coulomb
Unit Weight: 20.5 kN/m3
Cohesion: 2 kPa
Friction Angle: 40 degrees
Material: Grade Raise
Strength Type: Mohr-Coulomb
Unit Weight: 22 kN/m3
Cohesion: 1 kPa
Friction Angle: 34 degrees
75.00 kN/m2
75.00 kN/m2
75.00 kN/m2
4.318
W
W
Grade Raise
Silt and Clay
Silt and Sand Till