Geotechnical Investigation

Proposed Dog Kennel
5969 Ottawa Street
Richmond, Ontario

Prepared for:

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LRL File No.: 210341

October, 2021

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1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by Al Roberts to perform a geotechnical investigation for the proposed new dog kennel to be located at 5969 Ottawa Street, in Ottawa, Ontario.

The purpose of the investigation was to identify the subsurface conditions across the site by the completion of a 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, as well as a slope stability analysis for the slope located onsite.

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 located at 5969 Ottawa Street, in Richmond, Ontario. Currently, the site was vacant, with no structures present. The site is triangular shaped, with a stream running through the site in the north-south direction. The eastern portion of the site is covered with manicured grasses, sparse trees, and a gravel access road from Ottawa Street. The western portion of the site is heavily treed. With the exception of the banks sloping downward towards the creek, the site is considered to be relatively flat. The site is approximately 800 m^2 in size. The location is presented in Figure 1 included in **Appendix A**.

It is understood a new two (2) storey prefabricated building is intended to be built for this site. The first storey will be used as a dog kennel, and the second storey will be for living quarters.

3 PROCEDURE

The fieldwork for this investigation was carried out on August 23, 2021. Prior to the fieldwork, the site was cleared for the presence of any underground services and utilities. A total of four (4) boreholes were drilled within the proposed building footprint, and near the top of the slope. The approximate locations of the boreholes are shown in Figure 2 included in **Appendix A**.

The boreholes were advanced using a truck mounted CME 75 drill rig equipped with 200 mm diameter continuous flight hollow stem auger supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. 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 SPTs 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 the “N” value.

Boreholes were advanced to depths of 2.64 and 6.71 m below ground surface (bgs). 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 were transported back to our office for further evaluation. 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 located using a Garmin Etrex Legend GPS (Global Positioning System) receiver using NAD 83 datum (North American Datum). The existing grade elevations at the borehole locations were determined from interpolation from the Grading Plan developed by LRL. Ground surface elevations of the boring locations are shown on their respective borehole 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 this site is located at a transition zone between two (2) different deposits, glacial till, and silt/silty clay.

The subsurface conditions encountered in the boreholes were classified based on visual and tactile examination of the materials recovered from the boreholes. 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 are given in their respective borehole 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 of about 600 mm was encountered in BH1 through BH3.

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 Fill

At the surface of BH4, a layer of fill material was encountered and extended to a depth of 0.30 m bgs. This material consisted of grey granular material.

4.4 Silt

Underlying the topsoil in BH1 through BH3, and the fill in BH4, a layer of silt was encountered and extended to depths of 1.45 and 2.21 m bgs. Generally, the material can be classified as a silt material with some clay, trace sand, moist, and brown. The SPT "N" values were found ranging between 2 and 6 indicating the material is loose. The natural moisture contents were found to be 33 and 45%.

4.5 Glacial Till

Underlying the silt in all boreholes, a layer of glacial till was encountered and extended to depths 2.64 and 6.71 m bgs. This material was found to be a mixture of silt-sand, some gravel sized stone, trace clay, brown, and moist. The SPT "N" values were found ranging between 12 and 63, indicating the material is compact to very dense. The natural moisture contents were found to range between 9 and 14%.

4.6 Refusal

Practical auger refusal over large boulders within the glacial till material or possible bedrock was encountered in BH4 at a depth of 2.64 m bgs.

4.7 Laboratory Analysis

Two (2) soil samples were collected for laboratory gradation analyses. The gradation analyses comprised of sieve and hydrometer were conducted following the procedure **ASTM D422.** Details of laboratory analyses are reflected in **Table 1**.

Table 1: Gradation Analysis Summary

Sample Location	Depth (m)	Percent for Each Soil Gradation						
		Gravel		Sand			Silt (%)	Clay (%)
		Coarse (%)	Fine (%)	Coarse (%)	Medium (%)	Fine (%)		
BH1	0.8 – 1.4	0.0	0.0	0.0	0.0	1.2	88.6	10.2
BH2	1.5 – 2.1	8.5	0.0	4.9	9.3	26.5	36.1	5.8

A soil sample from BH1 was collected for laboratory sieve analyses. The results are summarized below in **Table 2.**

Table 2: Sieve Analysis Summary

Sample Location	Depth (m)	Percent for Each Soil Gradation					
		Gravel		Sand			Fines
		Coarse (%)	Fine (%)	Coarse (%)	Medium (%)	Fine (%)	Silt & Clay (%)
BH1	3.1 – 3.7	10.0	17.1	5.3	10.0	25.4	32.2

Atterberg limits and moisture contents were conducted on the spoon soil sample collected between depths 0.76 and 1.37 m in BH3. A summary of these values are provided below in **Table 3**.

Table 3: Summary of Atterberg Limits and Water Contents

Sample Location	Parameter					
	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Water Content (%)	USCS Group Symbol
BH3	0.76 – 1.37	30	25	5	37	CL

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

4.8 Groundwater Conditions

Groundwater was carefully monitored and measured during the field investigation. Immediately upon completion of drilling, groundwater was measured in all boreholes and was found to be dry.

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 near the vicinity of the site.

5 GEOTECHNICAL CONSIDERATIONS

This section of the report provides general geotechnical recommendations for the design aspect of the proposed development based on our interpretation of the information gathered from the borehole data performed at this site and from the project details.

5.1 Foundations

Based on the subsurface soil conditions established at this site, it is anticipated that the footings for the proposed building will be founded below the frost penetration depth on the native, undisturbed glacial till material. Therefore, any organic and any other deleterious material shall be stripped from the building footprint.

5.2 Shallow Foundation

Conventional strip and column footings founded over the undisturbed native soil may be designed using a maximum allowable bearing pressure of **125 kPa** for serviceability limit state **(SLS)** and **210 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, and allows for a strip footing maximum width of 2.0 m, and a pad footing maximum width of 4.0 m on any side.

In-situ field testing is required to check the strength and stability of the footing subgrade prior to any placement of concrete. Any incompetent subgrade areas as identified from in-situ testing must be sub-excavated and backfilled with approved structural fill consisting of OPSS Granular B Type II. Similarly, any soft 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.

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, consisting of OPSS Granular B Type II, 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 1.2 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 Lateral Earth Pressure

The following equation should be used to estimate the intensity of the lateral earth pressure against any earth retaining structure/foundation walls.

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

Where;

P = Earth pressure at depth h;

K = Appropriate coefficient of earth pressure;

γ = Unit weight of compacted backfill, adjacent to the wall;

h = Depth (below adjacent to the highest grade) at which P is calculated;

q = Intensity of any surcharge distributed uniformly over the backfill surface (usually surcharge from traffic, equipment or soil stockpiled and typically considered 10 kPa).

The coefficient of earth pressure at rest (K_0) should be used in the calculation of the earth pressure on the storm water manhole/basement walls, which are expected to be rather rigid and not to deflect.

The above expression assumes that perimeter drainage system prevents the build-up of any hydrostatic pressure behind the foundation wall.

5.5 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.

5.6 Seismic

Based on the results of this geotechnical investigation and in accordance with the Ontario Building Code 2012 (table 4.1.8.4.A.) and Canadian Foundation Engineering Manual (4th edition), the site can be classified as Class "D" as per the Site Classification for Seismic Site Response. It should be noted that a greater seismic site response class may be obtained by conducting seismic velocity testing using a multichannel analysis of surface waves (MASW).

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.

5.7 Liquefaction

Liquefaction is not considered to be a concern for foundations set on a compact to very dense glacial till material.

5.8 Frost Protection

All exterior footings for any heated structure exposed to frost conditions should have a minimum of 1.5 m of earth cover. Footings for any unheated structures, signage, lighting etc. and where snow will be cleared, 1.8 m of earth cover is required. 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 foundation 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.

5.9 Foundation Drainage

Permanent perimeter foundation drainage is recommended if any open spaces are present below the finished floor, or if the building has a basement. The foundation drainage shall consist of a conventional, perforated corrugated polyethylene drainage pipe (100 mm minimum), pre-wrapped with geotextile knitted sock conforming to **OPSS 1840,** 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.

5.10 Foundation Walls Backfill

To prevent possible lateral loading, the backfill material against the foundation walls should consist of free draining, non-frost susceptible material such as sand or sand and gravel meeting OPSS Granular B Type I, II or Select Subgrade Material (SSM).

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.

5.11 Slab on Grade Construction

Concrete slab-on-grade should rest directly over a minimum 150 mm thick layer of OPSS Granular A, compacted to 98% of its SPMDD. Prior to the placement of Granular A, all organic or otherwise deleterious material shall be removed from the proposed building's footprint down to the native subgrade surface. The subgrade should then be inspected and approved by qualified geotechnical personnel prior to placement of Granular A.

It is also recommended that the area of extensive exterior slab-on-grade (sidewalks, ramp etc.) shall be constructed using Granular A base of thickness 150 mm. The modulus of subgrade reaction (ks) for the design of the slabs is **18 MPa/m.**

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.

5.12 Sulphate Attack and Corrosivity Analysis on Buried Concrete

A soil sample was submitted to Paracel Laboratories Ltd. for chemical testing. The following **Table 4** below summarizes the results.

Table 4: Results of Chemical Analysis

Sample Location	Depth (m)	pH	Sulphate (µg/g)	Chloride (µg/g)	Resistivity (Ohm.cm)
BH2	1.5 – 2.1	7.64	<5	<5	1,160

The above results revealed a measured sulphate concentration of <5 µg/g in the sample. Based on the CAN/CSA-A23.1 standards (Concrete Materials and Methods of Concrete Construction), a sulphate concentration of less than 1000 µg/g falls within the negligible category for sulphate attack on buried concrete. The test results from soil samples were below the noted threshold. As such, buried concrete for footings and foundations walls will not require any special additive to resist sulphate attack and the use of normal Portland cement is acceptable.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil resistivity was measured to be 1,1600 ohm.cm, which falls within “corrosive” range.

6 EXCAVATION AND BACKFILLING REQUIREMENTS

6.1 Excavation

It is anticipated that the maximum depth of excavation for the building will not extend below 1.5 – 1.8 m. Excavation must be carried-out in accordance with the 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 for fully drained excavations. Therefore, shallow temporary excavations in the overburden soil can be cut at 1 horizontal to 1 vertical, for a fully drained excavation starting from the base of the excavation and as per requirements of the OHSA regulations.

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, groundwater seepage or infiltration into the temporary excavations during construction is expected to be minor in nature, if any. If encountered, this will be able to be controlled by pumping with sump pumps. Surface water runoff into the excavation should be minimized and diverted away from the excavation.

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 water takings range between 50,000 and 400,000 litres per day.

The actual amount of groundwater inflow into open excavations will depend on several factors such as the contractor's schedule, rate of excavation, the size of excavation, depth below the groundwater level, and at the time of year which the excavation is executed. Pumping rates will be less than 50,000 litres per day. As such, EASR registration is not required for the construction at this site.

6.3 Pipe Bedding Requirements

It is anticipated that any underground services required as part of this project will be founded over properly prepared and approved granular material. 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 I, or an 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, storm and sewer pipes should conform to the manufacturer's design requirements and to the detailed installations outlined in the Ontario Provincial Standard Specifications (OPSS) or any other applicable standards.

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 are 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 to the west of the proposed development. Overall, the slope has a relatively constant slope profile of about 8H:1V, having a total height of approximately 2 m. The slope profile was determined using a measuring tape and a magnifying eye level.

The slope was heavily vegetated with trees and shrubs. No visible signs of erosions or past slope failures were present within the slope and its surroundings.

7.2 Slope Stability Results

The slope modelling program, Slide 5.0 (Rocscience), was used to implement the Bishop simplified method of slices. One (1) slope profile was chosen which was considered to be the worst case scenario. The proposed loading for the building was included in the models. The slope was analyzed under both the undrained (short term failure) and drained (long term failure) conditions.

The seismic analysis was performed by incorporating the seismic coefficient (k_h) into the modelling. The peak ground acceleration (PGA) for this area is equal to 0.26 for the 2% in 50 year probability of exceedance as per the NBC 2015. The value for k_h was taken as 50% of the PGA, which equates to 0.13. The minimum factor of safety (FoS) with regards to seismic condition is 1.10.

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.

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)			
Silt	5	34	18.0
Glacial Till	2	40	20.0
Undrained and Seismic Parameters (Short Term)			
Silt	55	-	18.0
Glacial Till	2	40	20.0

The designed load for the building was not provided (design bearing pressure at serviceability limit state) during our field investigation. However, a typical value of 75 kPa for residential construction was assumed and included within the model.

The FoS against slope failure for the selected slope profile was determined to be 4.44, 9.52, and 4.32 for the drained, undrained, and seismic conditions respectively. A FoS of 1.50 or greater is considered to be safe with regards to slope stability.

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, and during a seismic event.

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

7.3 Conclusion

The following recommendations should be adhered to during the construction and post construction to ensure the long-term stability of the slope.

- The existing vegetation cover near and within the existing slope should not be disturbed any more than is absolutely necessary to construct the building and parking area, as it promotes stability and erosion control to the slope.
- Any site drainage should be diverted away from the slope. Drainage outlets, if any, shall be protected with riprap over approved geotextile to eliminate erosion in the slope.
- The slope profiles should not be modified in any way as part of the proposed construction. If modifications to the current slope profile are proposed, LRL should be consulted to ensure that the results of this report are still valid.

8 RECOMMENDED PAVEMENT STRUCTURE

For predictable performance of the pavement areas, any organic, soft, and/or deleterious materials should be removed from the proposed pavement areas to expose native undisturbed subgrade soil. The exposed subgrade should be inspected and approved by geotechnical personnel and any evidently loose and unstable areas should be sub-excavated and replaced with suitable earth borrow approved by the geotechnical engineer. Following approval of the preparation of the subgrade, the granular subbase may be placed.

The recommended pavement structures for the proposed light duty access roads and parking areas are provided below:

50 mm of hot mix asphaltic concrete (HL3/SP12.5) over;

150 mm of OPSS Granular A base over;

350 mm of OPSS Granular B Type II subbase.

The base and subbase granular materials should conform to **OPSS 1010** material specifications. Prior to importing any granular material onto the site, it should be tested and approved by a geotechnical engineer prior to delivery to the site and should be compacted to 98% SPMDD. Compaction of the granular pavement materials should be carried out in maximum 300 mm thick loose lifts.

Asphaltic concrete should conform to **OPSS 1150** and be placed and compacted to at least 95% of the Marshall Density. The mix and its constituents should be reviewed, tested and approved by a geotechnical engineer prior to delivery to the site.

8.1 Subgrade Preparation

Following the backfilling and satisfactory compaction of any underground service trenches up to the subgrade level, the subgrade should be shaped, crowned and proof-rolled using heavy roller with any resulting soft areas sub-excavated down to an adequate bearing layer and replaced with approved backfill. Following approval of the preparation of the subgrade, the pavement structure may be placed.

If the roadway subgrade is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or incorporate a non-woven geotextile separator between the roadway subgrade surface and the granular subbase material.

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 structure 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 any proposed the curb/edge of pavement line but be extended beyond the curb.

For areas of the site that require the subgrade to be raised, the material should consist of OPSS Granular B Type I, II, or approved equivalent. Any materials proposed for this use should be approved by the geotechnical engineer before placement. Materials used for raising the subgrade to the proposed roadway subgrade level should be placed in maximum 300 mm thick loose lifts and be compacted to at least 95% of the SPMDD using suitable compaction equipment.

The preparation of subgrade should be scheduled and carried out in such a manner that a protective cover of overlying granular material is placed as quickly as possible in order to avoid unnecessary circulation by heavy equipment over the subgrade. Frost protection of the surface should be implemented (i.e. insulated tarps, etc.), if works are carried out during the winter months.

Transitions should be constructed between new and existing pavement structures where new access lanes will meet with existing road. In areas where the new pavement structure will abut existing pavement structure, the depths of granular materials should be tapered up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement..

9 REUSE OF ON-SITE SOILS

The existing surficial overburden material for this site that is expected to be excavated 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.

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, and approved for reuse by a geotechnical engineer.

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 buildings 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 required gradation and compaction specifications.

If the 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 works 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 test pit locations only. Boundaries between zones presented on the test pit logs 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 ensure 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
B. W. JOHNSON
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Nov 10, 2021
PROVINCE OF ONTARIO

Brad Johnson, P. Eng.
Geotechnical Engineer

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APPENDIX A

Site and Borehole Location Plan

LRL

ENGINEERING | INGÉNIERIE

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

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED DOG KENNEL
5969 OTTAWA STREET
RICHMOND, ONTARIO

DRAWING TITLE

SITE LOCATION
SOURCE: GEO-OTTAWA

CLIENT

AL ROBERTS

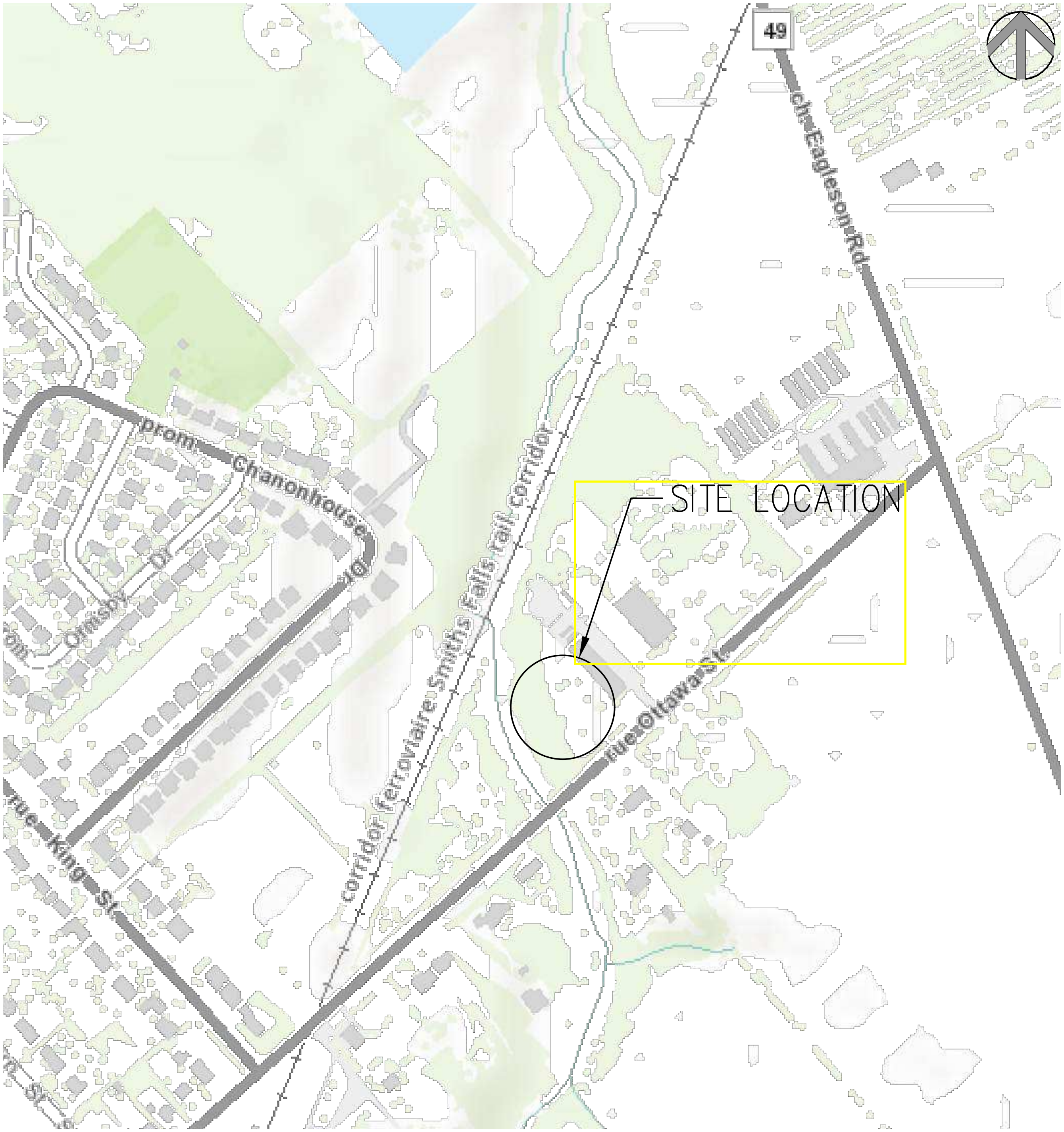
DATE

AUGUST 2021

PROJECT

210341

FIGURE 1



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PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED DOG KENNEL
5969 OTTAWA STREET
RICHMOND, ONTARIO

DRAWING TITLE

BOREHOLE LOCATION
SOURCE: Imagery 2021 Google, Digital Globe Map Data

CLIENT
AL ROBERTS

DATE
AUGUST 2021

PROJECT
210341

FIGURE 2

APPENDIX B

Borehole Logs

Borehole Log: BH1

Project No.: 210341

Project: Geotechnical Investigation - Site Redevelopment

Client: Al Roberts

Location: 5969 Ottawa Street, Richmond ON

Date: August 23, 2021

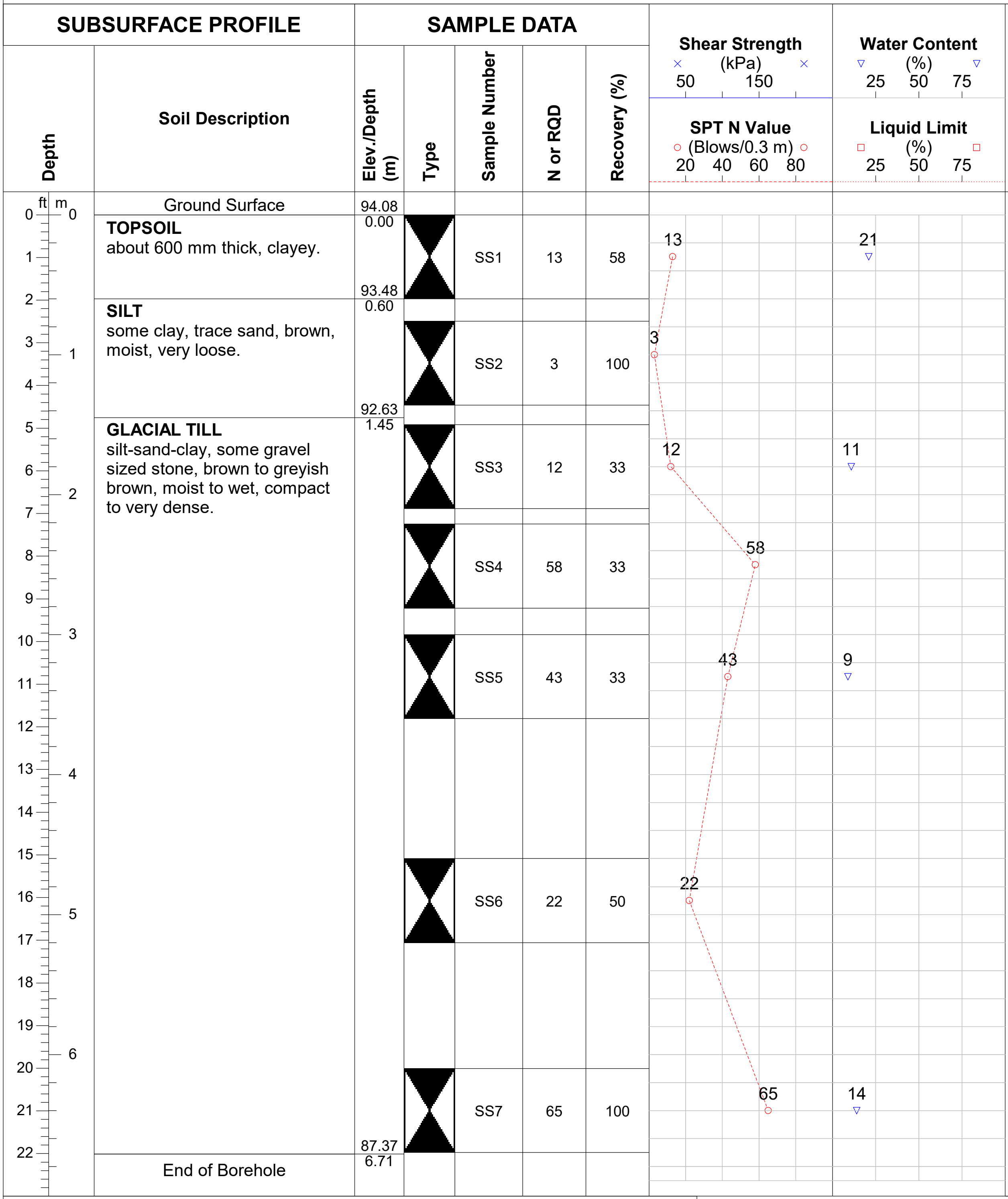
Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Truck Mount CME 55

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA				
Depth	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)
	Ground Surface	94.08				
	TOPSOIL about 600 mm thick, clayey.	0.00	⧗	SS1	13	58
	SILT some clay, trace sand, brown, moist, very loose.	93.48 0.60	⧗	SS2	3	100
	GLACIAL TILL silt-sand-clay, some gravel sized stone, brown to greyish brown, moist to wet, compact to very dense.	92.63 1.45	⧗	SS3	12	33
			⧗	SS4	58	33
			⧗	SS5	43	33
			⧗	SS6	22	50
			⧗	SS7	65	100
	End of Borehole	87.37 6.71				



Monitoring Well Details

Easting: 435621 m

Northing: 5004465 m

Site Datum: Assumed

Groundsurface Elevation: 94.08 m

Top of Riser Elev.: NA

Hole Diameter: 200 mm

Monitoring Well Diameter: N/A

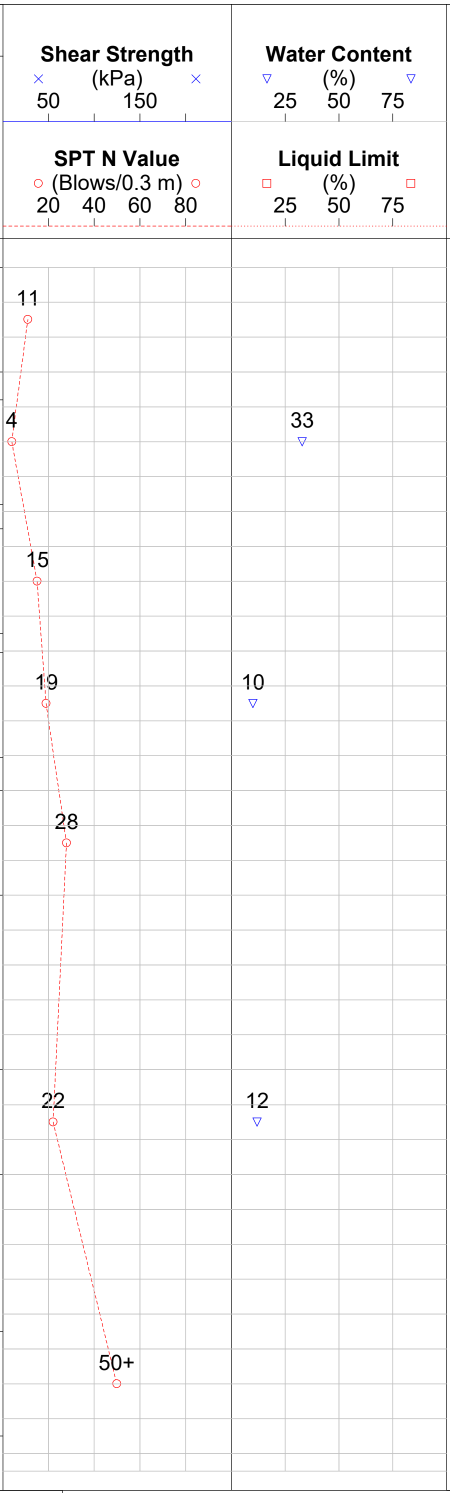
NOTES:

LRL

Borehole Log: BH2

Project No.: 210341
Project: Geotechnical Investigation - Site Redevelopment
Client: Al Roberts
Location: 5969 Ottawa Street, Richmond ON
Date: August 23, 2021
Field Personnel: SV
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Truck Mount CME 55
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA						
Depth (ft / m)	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	Water Content (%)	Monitoring Well Details
0 / 0	Ground Surface	94.04 0.00						
	TOPSOIL about 600 mm thick, clayey.	93.44 0.60	⧗	SS1	11	50		
	SILT some clay, trace sand, brown, moist, very loose.	92.59 1.45	⧗	SS2	4	100	33	
	GLACIAL TILL silt-sand, some gravel sized stone, trace clay brown to greyish brown, moist to wet, compact to very dense.		⧗	SS3	15	84		
			⧗	SS4	19	25	10	
			⧗	SS5	28	50		
			⧗	SS6	22	33	12	
		87.33 6.71	⧗	SS7	50+	58		
	End of Borehole							



Easting: 435632 m
Northing: 5004454 m
Site Datum: Assumed
Groundsurface Elevation: 94.04 m
Top of Riser Elev.: NA
Hole Diameter: 200 mm
Monitoring Well Diameter: N/A

NOTES:

Borehole Log: BH3

Project No.: 210341

Project: Geotechnical Investigation - Site Redevelopment

Client: Al Roberts

Location: 5969 Ottawa Street, Richmond ON

Date: August 23, 2021

Field Personnel: SV

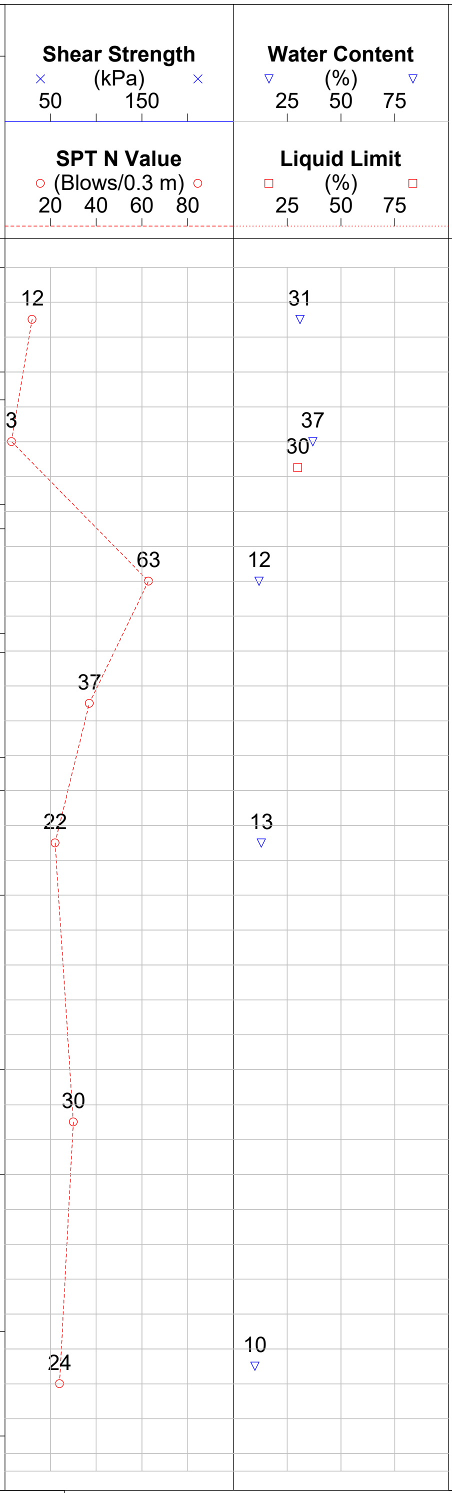
Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Truck Mount CME 55

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE / SAMPLE DATA

Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)
Ground Surface	93.90				
TOPSOIL about 600 mm thick, clayey.	0.00	⧗	SS1	12	58
SILT some clay, trace sand, greyish brown, moist, very loose.	93.30 0.60	⧗	SS2	3	100
GLACIAL TILL silt-sand, some gravel sized stone, trace clay brown to greyish brown, moist to wet, compact to very dense.	92.45 1.45	⧗	SS3	63	33
		⧗	SS4	37	17
		⧗	SS5	22	75
		⧗	SS6	30	58
		⧗	SS7	24	50
End of Borehole	87.19 6.71				



Monitoring Well Details

Easting: 435625 m

Northing: 5004453 m

Site Datum: Assumed

Groundsurface Elevation: 93.90 m

Top of Riser Elev.: NA

Hole Diameter: 200 mm

Monitoring Well Diameter: N/A

NOTES:

Borehole Log: BH4

Project No.: 210341

Project: Geotechnical Investigation - Site Redevelopment

Client: Al Roberts

Location: 5969 Ottawa Street, Richmond ON

Date: August 23, 2021

Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

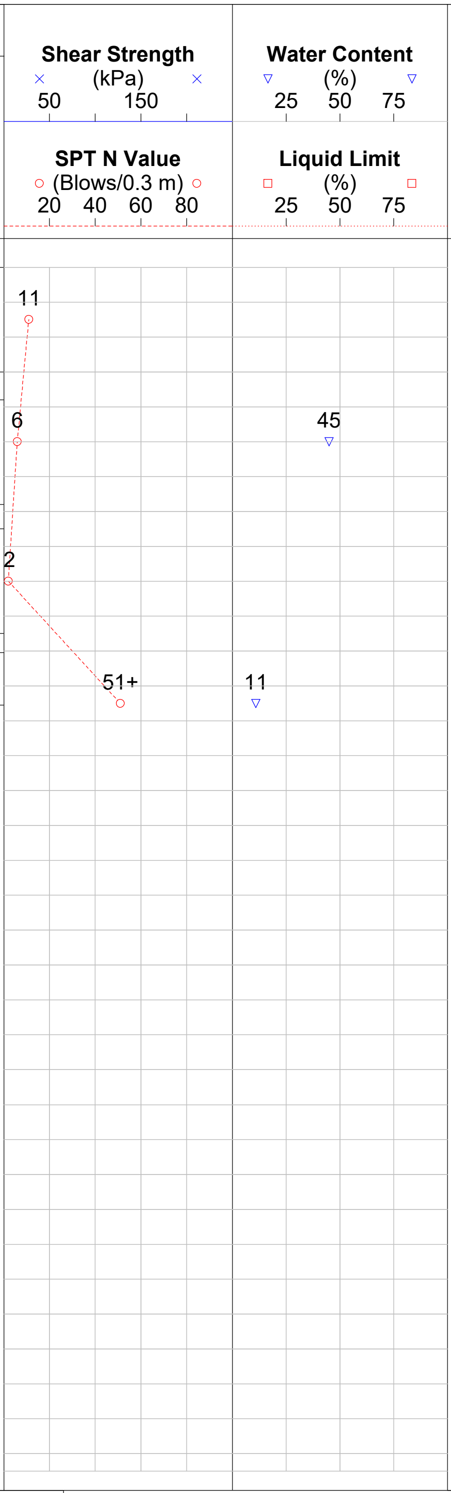
Drilling Equipment: Truck Mount CME 55

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE / SAMPLE DATA

Depth (ft / m)	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)
0 / 0	Ground Surface	93.85				
	FILL Granular material, grey, dry, loose.	0.00		SS1	11	21
	SILT some clay, trace sand, brown, moist, loose.	93.55 0.30				
				SS2	6	100
				SS3	2	92
	GLACIAL TILL silt-sand, some gravel sized stone, trace clay, brown, moist, very dense.	91.64 2.21		SS4	51+	50
	End of Borehole Borehole terminated after practical auger refusal.	91.21 2.64				

Depth scale — ft: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22; m: 0 1 2 3 4 5 6



Monitoring Well Details

Easting: 435611 m

Northing: 5004441 m

Site Datum: Assumed

Groundsurface Elevation: 93.85 m

Top of Riser Elev.: NA

Hole Diameter: 200 mm

Monitoring Well Diameter: N/A

NOTES:

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

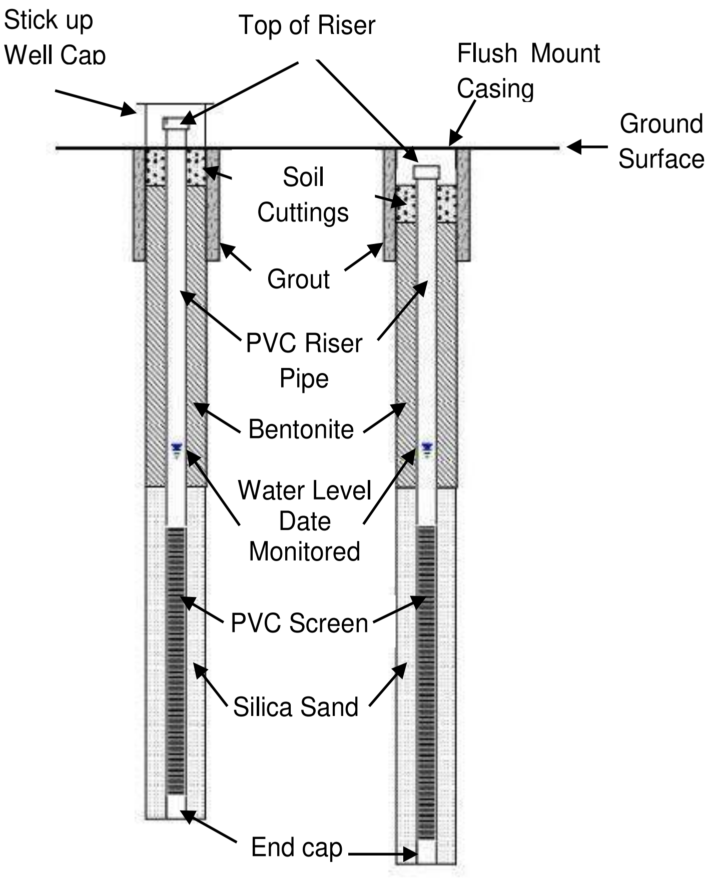
Rock Quality Designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mas. The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 100 mm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given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

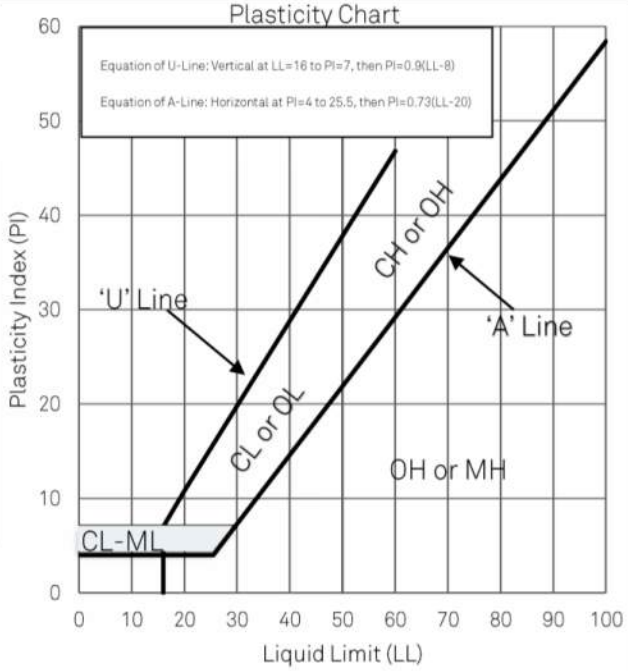
Strength Classification	Range of Unconfined Compressive Strength (MPa)
Extremely weak	< 1
Very weak	1 – 5
Weak	5 – 25
Medium strong	25 – 50
Strong	50 – 100
Very strong	100 – 250
Extremely strong	> 250

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

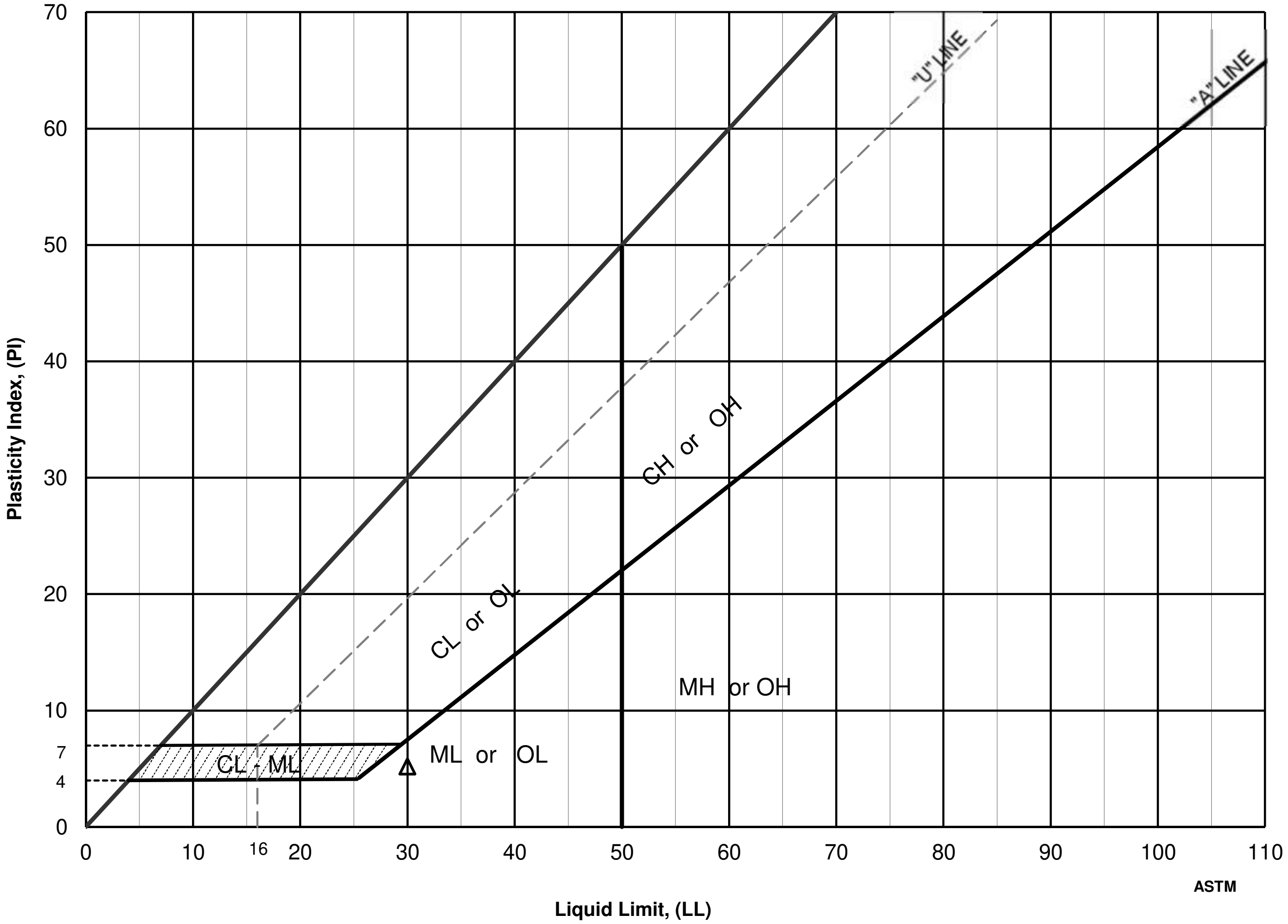
LRL Associates Ltd.

PLASTICITY INDEX

ASTM D 4318 / LS-703/704

Client: Al Roberts
File No.: 210341
Project: Geotechnical Investigation
Report No.: 1
Location: 5969 Ottawa Street, Ottawa, ON
Date: August 23, 2021

Plasticity Chart



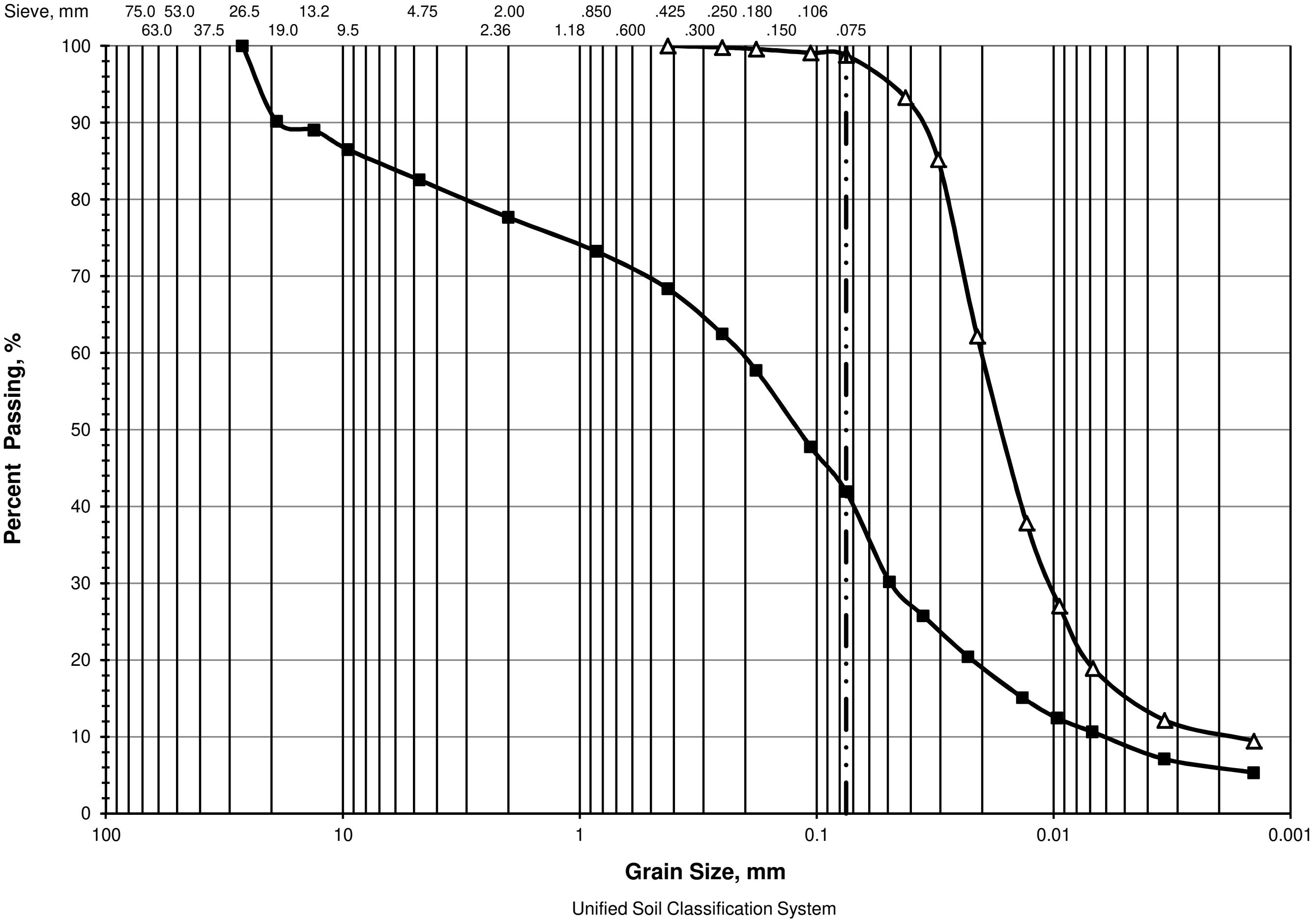
	Location	Sample	Depth, m	Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Activity Number	USCS
△	BH 3	2	0.76 - 1.37	37	30	25	5	2.29	n/d	ML

LRL Associates Ltd.

PARTICLE SIZE ANALYSIS

ASTM D 422 / LS-702

Client: Al Roberts
Project: Hydrogeological Assessment
Location: 5969 Ottawa Street, Ottawa, ON
File No.: 210341
Report No.: 2
Date: August 23, 2021



	> 75 mm	% GRAVEL		% SAND			% FINES	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
△	0.0	0.0	0.0	0.0	0.0	1.2	88.6	10.2
■	0.0	8.5	8.9	4.9	9.3	26.5	36.1	5.8

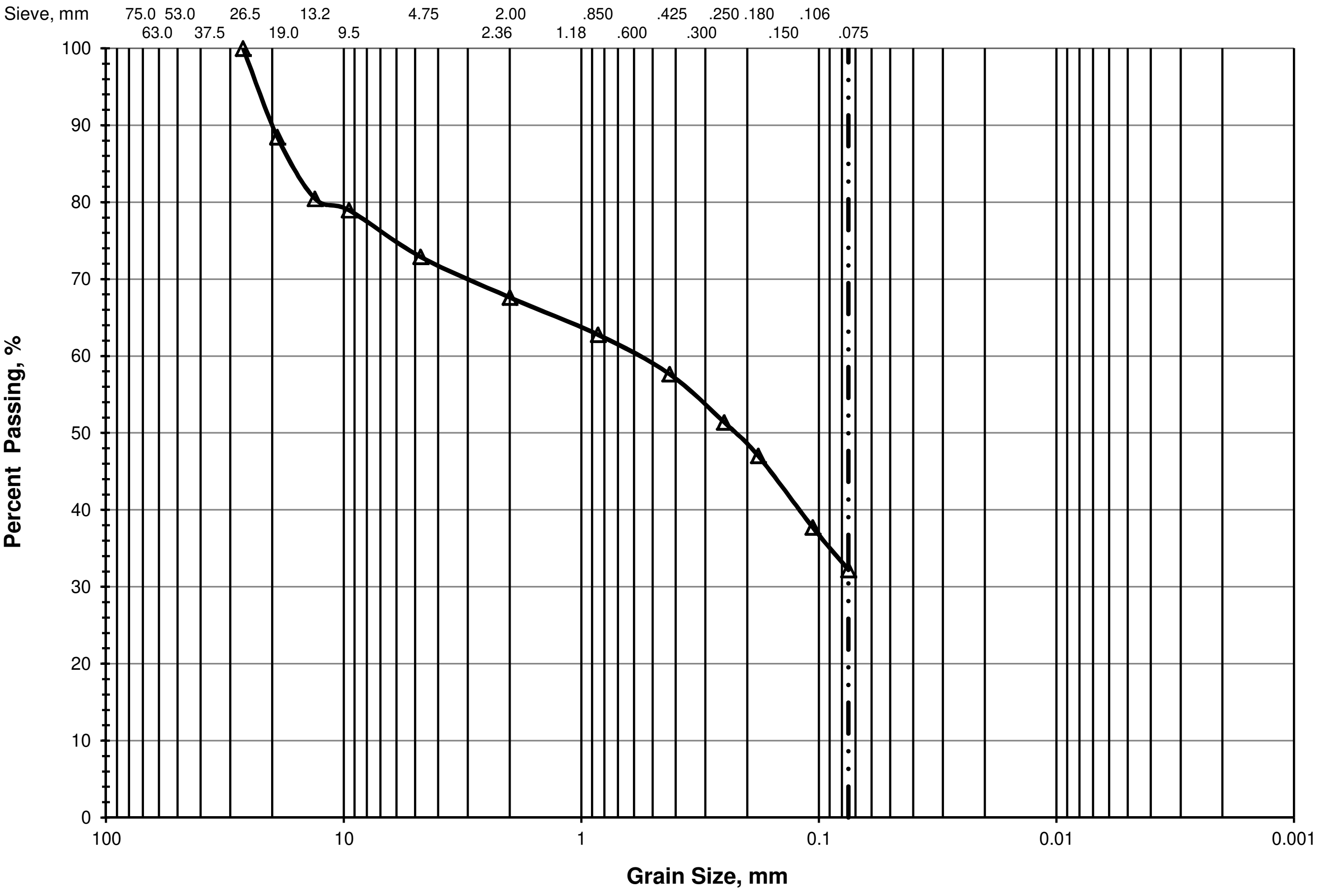
	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH 1	2	0.76 - 1.37	0.0203	0.0170	0.0104	0.0048	0.0018	3.0	11.3
■	BH 2	3	1.52 - 2.13	0.2136	0.1226	0.0486	0.0134	0.0062	1.8	34.5

LRL Associates Ltd.

PARTICLE SIZE ANALYSIS

ASTM D 422 / LS-702

Client: Al Roberts
Project: Geotechnical Investigation
Location: 5969 Ottawa Street, Ottawa, ON
File No.: 210341
Report No.: 2a
Date: August 23, 2021



Unified Soil Classification System

	> 75 mm	% GRAVEL		% SAND			% FINES
		Coarse	Fine	Coarse	Medium	Fine	Silt & Clay
△	0.0	10.0	17.1	5.3	10.0	25.4	32.2

	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH 1	5	3.05 - 3.66	0.6196	0.2279					

300 - 2319 St. Laurent Blvd
Ottawa, ON, K1G 4J8
1-800-749-1947
www.paracellabs.com

Certificate of Analysis

LRL Associates Ltd.

5430 Canotek Road
Ottawa, ON K1J 9G2
Attn: Brad Johnson

Client PO:
Project: 210341
Custody: 60813

Report Date: 1-Sep-2021
Order Date: 26-Aug-2021

Order #: 2135540

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
2135540-01	BH2 5-7'

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	31-Aug-21	31-Aug-21
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	26-Aug-21	27-Aug-21
Resistivity	EPA 120.1 - probe, water extraction	31-Aug-21	1-Sep-21
Solids, %	Gravimetric, calculation	26-Aug-21	27-Aug-21

Certificate of Analysis

Client: **LRL Associates Ltd.**

Client PO:

Report Date: 01-Sep-2021

Order Date: 26-Aug-2021

Project Description: 210341

	Client ID:	BH2 5-7'	-	-	-
	Sample Date:	23-Aug-21 09:00	-	-	-
	Sample ID:	2135540-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	88.6	-	-	-
General Inorganics					
pH	0.05 pH Units	7.64	-	-	-
Resistivity	0.10 Ohm.m	116	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	<5	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g dry	ND			NC	20	
Sulphate	ND	5	ug/g dry	ND			NC	20	
General Inorganics									
pH	7.77	0.05	pH Units	7.78			0.1	2.3	
Resistivity	113	0.10	Ohm.m	116			2.3	20	
Physical Characteristics									
% Solids	78.0	0.1	% by Wt.	79.0			1.3	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	91.3	5	ug/g	ND	91.3	82-118			
Sulphate	90.3	5	ug/g	ND	90.3	80-120			

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

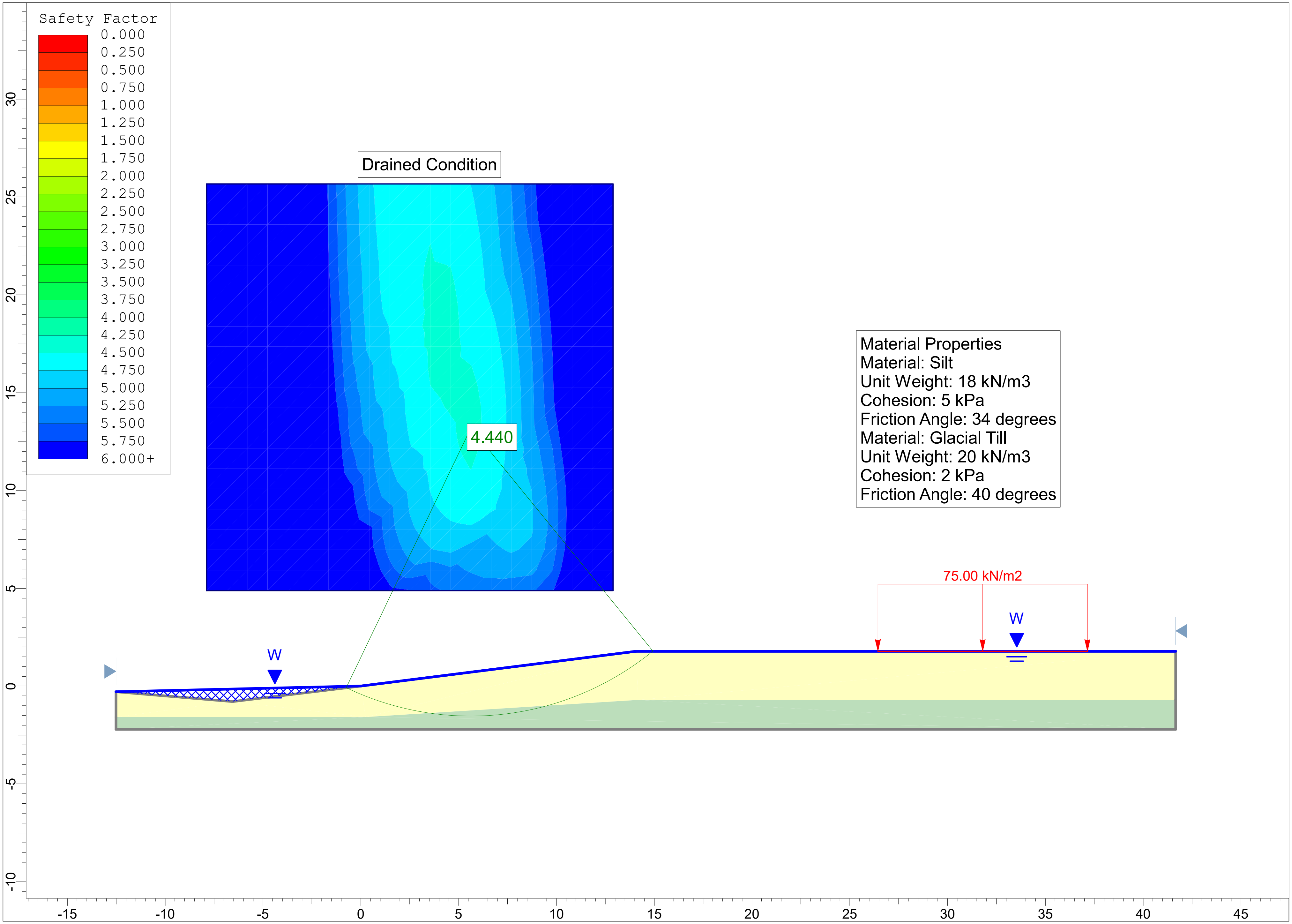
Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.
NC: Not Calculated

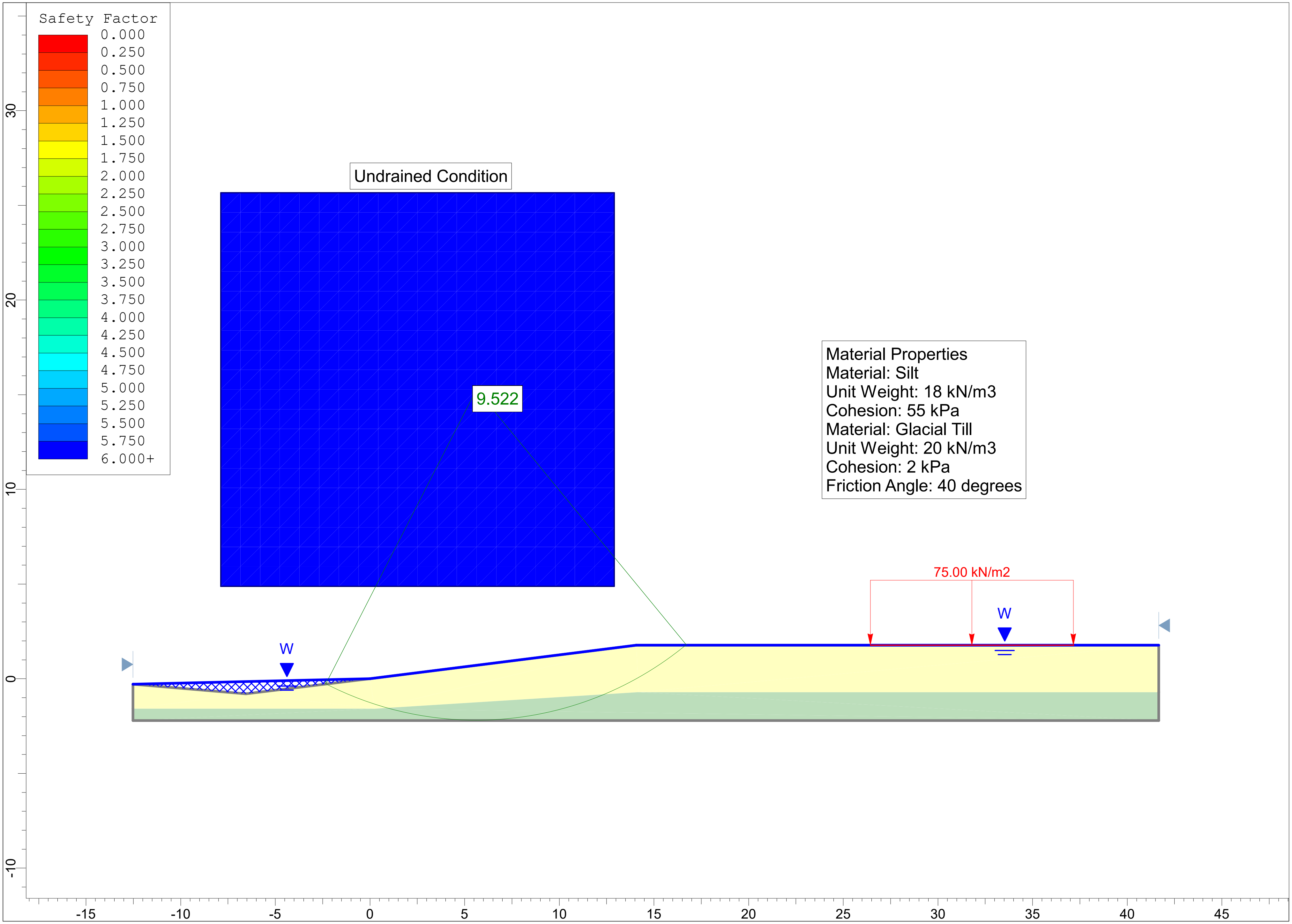
Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

APPENDIX E

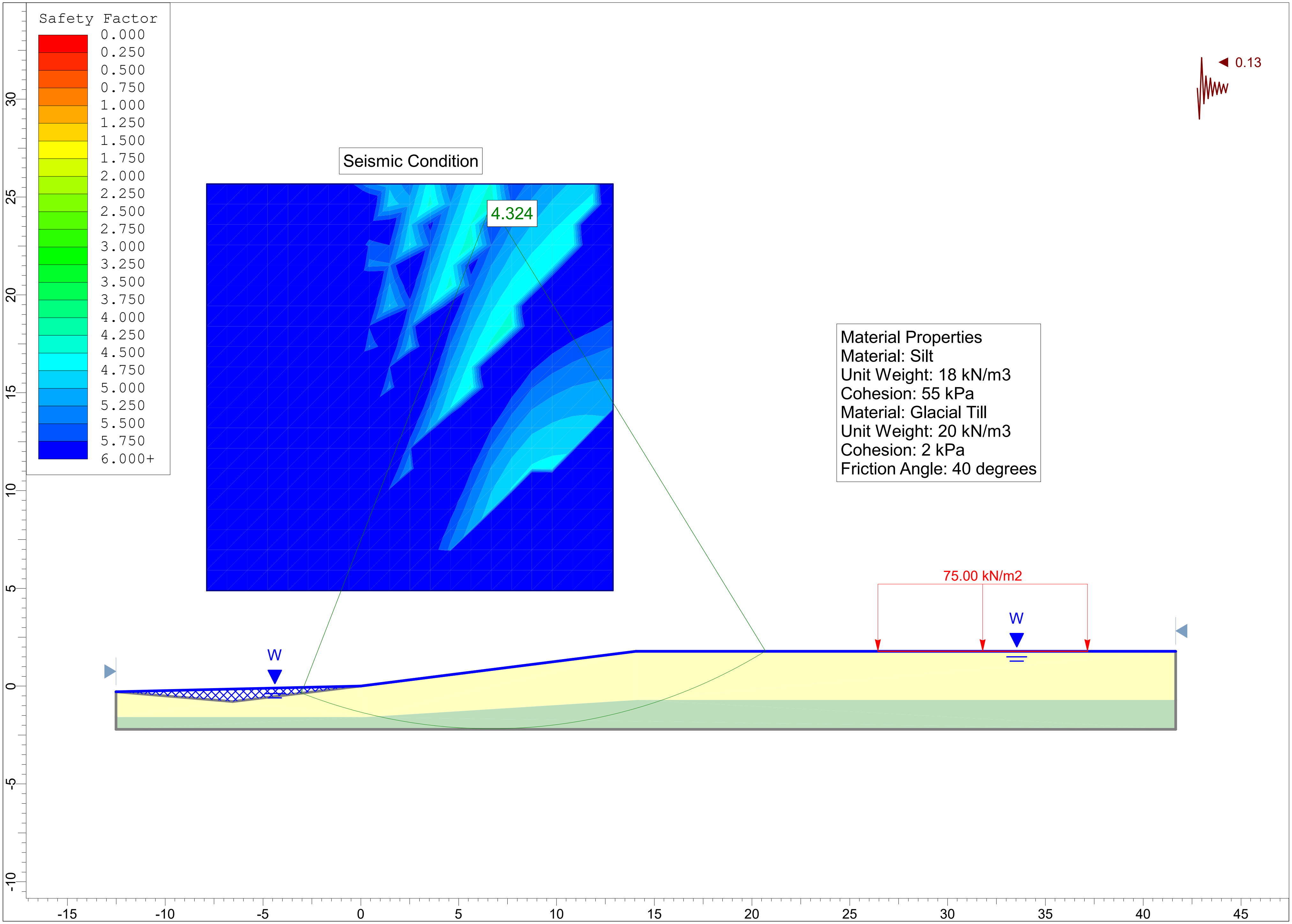
Slope Stability Analysis Results



Material Properties
Material: Silt
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 34 degrees
Material: Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 2 kPa
Friction Angle: 40 degrees



Material Properties
Material: Silt
Unit Weight: 18 kN/m3
Cohesion: 55 kPa
Material: Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 2 kPa
Friction Angle: 40 degrees



Seismic Condition

Material Properties
Material: Silt
Unit Weight: 18 kN/m3
Cohesion: 55 kPa
Material: Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 2 kPa
Friction Angle: 40 degrees