Geotechnical Investigation

Proposed Residential Development
2009-2013 Prince of Wales Drive
Ottawa, Ontario
Revision 2

Prepared for:

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1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by Alex Sivasambu to perform a geotechnical investigation for a proposed Residential Development, located at 2009-2013 Prince of Wales Drive, 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.

In addition, a section of the report will also include a section pertaining to the stability of the slope, located adjacent to the Rideau River.

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 2009 and 2013 Prince of Wales Drive, in Ottawa ON. Currently, there is a single-family residential dwelling located at each of the civic addresses mentioned above. The site is bound by Rideau River to the East, the Via Rail corridor to the South, Prince of Wales Drive to the West, and 2005 Prince of Wales Drive to the North. This site is vegetated with manicured grasses and some mature trees. The general topography of the site is considered to be relatively flat, with the exception of the river banks adjacent to Rideau River. The location is presented in Figure 1 included in **Appendix A**.

At the time of generating this report, it is understood the site will be developed into seven (7) residential lots, and serviced with City of Ottawa infrastructure. A road will also be constructed intersecting Prince of Wales Drive in order to provide access to the new lots.

3 PROCEDURE

The fieldwork for this investigation was carried out on November 28, 2022. Prior to the fieldwork, the site was cleared for the presence of any underground services and utilities. A total of five (5) boreholes were drilled onsite to get a general representation of the site's underlying soil conditions, and labelled BH1 through BH5. 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.

The boreholes were advanced to depths of 6.70 and 8.23 m below ground surface (bgs). Upon completion, the boreholes were backfilled and compacted using the overburden cuttings.

A piezometer was installed in BH3 to measure the long term groundwater level. The piezometers consisted of 19 mm diameter PVC pipe with slotted bottoms to allow for groundwater infiltration.

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). An elevation survey was carried out onsite to determine the borehole locations' elevation. A Temporary Benchmark (TBM) was assigned using the bolts on the flange of the fire hydrant in front of the site, and given an elevation of 100.00 m. Ground surface elevations of the boring locations are shown on their respective borehole logs, attached in **Appendix B**.

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 is made up of "Abandoned River Channel Deposits", consisting of silt and 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

At the surface of all boring locations, with the exception of BH2, a layer of topsoil was encountered. This was found to be about 600 mm thick.

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 Material

Underlying the topsoil in BH1 and at the surface of BH2, a layer of fill material was encountered and extended to a depth of 1.45 m bgs. This material can generally be described as a mixture of sand-silt-clay, some crushed stone, grey, and moist. The SPT "N" values were found ranging between 11 and 27, indicating the material is compact. The natural moisture content was found to be 10%.

4.4 Silty Clay

Underlying the fill material in BH1, a layer of silty clay was encountered and extended to a depth of 2.21 m bgs. This material had some sand seams, brownish grey in colour, and moist. The SPT "N" value was found to be 7, indicating the material is firm. The natural moisture content was found to be 37%.

4.5 Silt and Clay

Underlying the topsoil in BH3, a layer silt and clay was encountered, and extended to a depth of 8.23 m bgs. (end of exploration). The material had some sand, greyish brown to grey, and moist. The SPT "N" values were found ranging between 14 and 1, indicating the material is stiff to very soft with increased depth. The natural moisture contents were found to range between 26 and 39%.

4.6 Sandy Clay to Clayey Sand

Underlying the silty clay in BH1, the fill material in BH2, and the topsoil in BH4 and BH5, a layer of sandy clay to clayey sand was encountered and extended to a depth of 6.70 m bgs. (end of exploration). The material had some silt, greyish brown, and moist. The SPT "N" values were found ranging between 19 and 0, indicating the material is very stiff to very soft with increased depth. The natural moisture contents were found to range between 24 and 45%.

4.7 Sand and Silt

Underlying the clayey sand to sandy clay in BH5, a layer of sand and silt was encountered and extended to a depth of 6.7 m bgs. (end of exploration). This material had some clay, grey in colour, and moist. The SPT "N" values were found to be 1 and 0, indicating the material is very soft. The natural moisture content was found to be 29%.

4.8 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							Estimated Hydraulic Conductivity K (m/s)
		Gravel		Sand			Silt (%)	Clay (%)	
		Coarse (%)	Fine (%)	Coarse (%)	Medium (%)	Fine (%)			
BH3	1.5-.21	0.0	0.0	0.0	0.0	16.8	37.5	45.7	1 x 10^{-7}
BH5	6.1-6.7	0.0	0.0	0.0	0.0	47.2	38.2	14.6	1 x 10^{-6}

Atterberg limits and moisture contents were conducted on a split spoon soil sample. Based on the test result, the values indicate that the subsoils contain inorganic clays of high 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
BH1	1.5-2.1	60	25	35	37	CH

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

4.9 Groundwater Conditions

A piezometer was installed to measure the long-term ground water level within BH3. The piezometers consisted of 19 mm diameter slotted PVC pipe, backfilled with silica sand, and sealed with bentonite. The piezometer was installed at a depth of 3.0 m bgs.

The piezometer was measured on December 6, 2022. The water was found to be at 4.8 m bgs. The ground water level is shown on its respective borehole log.

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 requirements.

5.1 Foundations

Based on the subsurface soil conditions established at this site, it is recommended that the footings for the any proposed residential dwelling be founded on the native silt and clay and/or clayey sand to sandy clay. Therefore, all topsoil, organic and any other deleterious material shall be stripped from the dwellings' footprint.

5.2

5.2 Shallow Foundation

Conventional strip and column footings founded over the undisturbed native material 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, and allows for a strip footing maximum width of 1.8 m, and a pad footing maximum width of 3.6 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 on a lot-by-lot basis. 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 “E” as per the Site Classification for Seismic Site Response.

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.

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).

5.7 Liquefaction Potential

As recommended in Canadian Foundation Engineering Manual 4th 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.

Based on the above criteria, liquefaction is not a concern for this site.

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 or lighting, 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 Walls Backfill

To prevent possible 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 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.

5.10 Basement Construction

Basement floor slabs can rest either on undisturbed native material or approved structural fill. For bedding, a minimum 200 mm thick layer of 19 mm clear stone meeting the **OPSS 1004** gradation requirements should be placed.

A moisture barrier with vapour retarder shall be placed directly underlying the concrete slab, and overlying the clear stone bedding.

5.11 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.

5.12 Corrosion Potential and Cement Type

A soil sample was submitted to Paracel Laboratories Ltd. for chemical testing. The following **Table 3** below summarizes the results.

Table 3: Results of Chemical Analysis

Sample Location	Depth (m)	pH	Sulphate (µg/g)	Chloride (µg/g)	Resistivity (Ohm.cm)
BH5	2.3-2.9	7.32	42	23	5,540

The above results revealed a measured sulphate concentration of 42 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 5,540 ohm.cm, which falls in the “moderate” corrosive range.

5.13 Tree Planting Guidelines

It shall be noted that the cohesive soils encountered onsite may be sensitive to water depletion by trees of high water demand during periods of dry weather. When trees draw water the underlying soils may undergo shrinkage which can result in settlement of adjacent structures.

Small (7.5 m mature tree height) to medium (7.5 – 14.0 m mature tree height) size trees are permitted to be planted provided they are set back a minimum of 4.5 m from the foundation if the following conditions are met:

- The USF is 2.1 m or greater below the lowest finished grade.
- A small tree must have a minimum of 25 m^3 of available soil volume, and a medium tree must be provided with a minimum of 30 m^3 of available soil volume as determined by a landscape architect.
- Foundation walls are reinforced, at minimum, with two (2) upper and two (2) lower 15M rebar.
- Grading surrounding the tree must promote draining to the tree root zone.

5.14 Swimming Pools

In-ground and above-ground swimming pools can be constructed on the Lots; provided the following precautions are respected.

5.14.1 In-ground Swimming Pools

The installation of an in-ground swimming pool will result in a negligible net gain of any increased loading to the site’s underlying soil conditions.

Any site re-grading due to the pool construction shall respect the grade raise restrictions outlined in **Section 5.2.** It is not recommended to stockpile any excavated material onsite.

5.14.2 Above-ground Pools

The addition of an above-ground pool will result in a net gain of loading imposed on the site’s underlying soils due to the weight of water above ground surface. The site’s underlying soil is able to withstand an above-ground pool depth of 2.1 m (7’).

It is recommended to install above-ground pools a minimum of 2.5 m from the foundation of the dwelling in order to avoid any lateral loading on the foundation from the pool.

6 EXCAVATION AND BACKFILLING REQUIREMENTS

6.1 Excavation

It is anticipated that the maximum depth of excavation for this development will be 2.1 m bgs. 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, very minor groundwater seepage or infiltration into the temporary excavations during construction is expected to be 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. It is anticipated that pumping rates will be less than 50,000 litres per day. As such, EASR registration is not required for the construction at this site. However, this requirement could be confirmed by undertaking a hydrogeological study to determine the maximum volume of ground water inflow that will required to be pumped.

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 structural fill. 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

The slope under review is located at the eastern portion of the site, adjacent to the Rideau River. The slope has a relatively constant slope profile throughout the site, and was found to have a profile of about 0.8 Horizontal to 1.0 Vertical 0.8H:1V. The slope profile was determined using a combination of a magnifying eye level, and a measuring tape.

The slope onsite was sparsely vegetated with some mature trees.

After a visual inspection of the slope, no erosion nor past slope failure was observed within the slope or its surroundings.

7.1 Slope Stability Results

The slope modelling program, Slide 5.0 (Rocscience), was used to implement the Bishop simplified method of slices. A slope profile, considered to be the steepest onsite (worst case scenario) was selected 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 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 0.28 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 0.14. The minimum factor of safety (FoS) with regards to seismic condition is 1.10.

The field measurements from the boreholes 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 4: 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 and Clay	5	36	18.0
Undrained Parameters (Short Term)			
Silt and Clay	75	-	18.0

The below **Table 5** is a summary of the factor of safety (FoS) values.

Table 5: FOS Values for Slope Stability Modelling

	Drained Condition	Undrained Condition	Seismic
Factor of Safety	0.42	2.59	1.81
Min. Required	**1.50**	**1.50**	**1.10**

These results indicate that the slope is unstable in the drained (long-term) condition. A setback for any permanent structure(s) (dwellings, decks, sheds, gazebos, pools, etc.) from the top of the slope is required to ensure that in the event of a slope failure, the structure(s) will be unharmed. The model was filtered to illustrate the failure surface with a FOS below 1.50.

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

7.2 Setback Requirements

As outlined in the Ministry of Natural Resources (MNR) Guidelines, The Limit of Hazard Land consists of three components as follows:

Limit of Hazard land = Stable Slope Allowance + Toe Erosion Allowance + Erosion Access Allowance.

The Stable Slope Allowance is the area where a factor of safety is less than 1.50 against overall rotational failure. As indicated in the attached model, **a 19.0 m setback is required for Stable Slope Allowance**.

Based on our field observations made onsite, some minor toe erosion was observed. Therefore, **a Toe Erosion Allowance of 5.0 m is recommended**.

An Erosion Access Allowance is intended to provide a corridor of sufficient width that allows equipment to access the site to undertake a repair for any future unforeseen slope failure. **A 6.0 m allowance is recommended for Erosion Access Allowance on this site**.

In summary, the following Limit of Hazard Lands can be taken as:

Limit of Hazard land = Stable Slope Allowance + Toe Erosion Allowance + Erosion Access Allowance.

= 19 m + 5 m + 6 m

= 30 m (measured from top of the slope)

7.3 Conclusions/Recommendations

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

- The existing vegetation cover near and within the existing slope should not be disturbed any more than is absolutely necessary for any proposed construction, as it promotes stability and erosion control to the slope.
- If it is decided that significant grade raises are needed, LRL must be contacted to ensure that the results of this report are still applicable.
- Where possible, 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.
- No backfill or excavated material shall be placed within the setback.
- 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 REUSE OF ON-SITE SOILS

The existing surficial overburden materials consists of silt and clay to sandy clay and clayey sand. These material are 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.

9 RECOMMENDED PAVEMENT STRUCTURE

It is anticipated that the subgrade soils for the roadway will consist of silt and clay to sandy clay and clayey sand. The construction of the road will be acceptable over the native subgrade once all organic, or otherwise deleterious materials are removed from the subgrade area.

The following **Table 6** presents the recommended pavement structure to be constructed over a stable subgrade.

Table 6: Recommended Pavement Structure

Course	Material	Residential Roadway (thickness, mm)
Surface	HL3/SP12.5	40
Binder	HL8/SP19.0	50
Base course	Granular A	150
Sub-base	Granular B Type II	450
Total:		690

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 95% 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 roadway shall be stripped of organics/vegetation, debris and other obvious objectionable fill 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 loaded Tandem axle, dual wheel dump truck or approved equivalent 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 subgrade shall be scheduled and carried out in 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 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
100510537
2023.11.01
PROVINCE OF ONTARIO

Brad Johnson, P. Eng.
Geotechnical Engineer

W:\FILES 2022\220528\05 Geotechnical\01 Investigation\05 Reports\ 2023-11-01_Geotechnical Investigation_2009-2013 Prince of Wales_Residential Development_LRL220528_R2.docx

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 RESIDENTIAL DEVELOPMENT
2009 & 2013 PRINCE OF WALES DR.
OTTAWA, ONTARIO

DRAWING TITLE

SITE LOCATION
SOURCE: GEOOTTAWA

CLIENT

JANE THOMPSON ARCHITECT

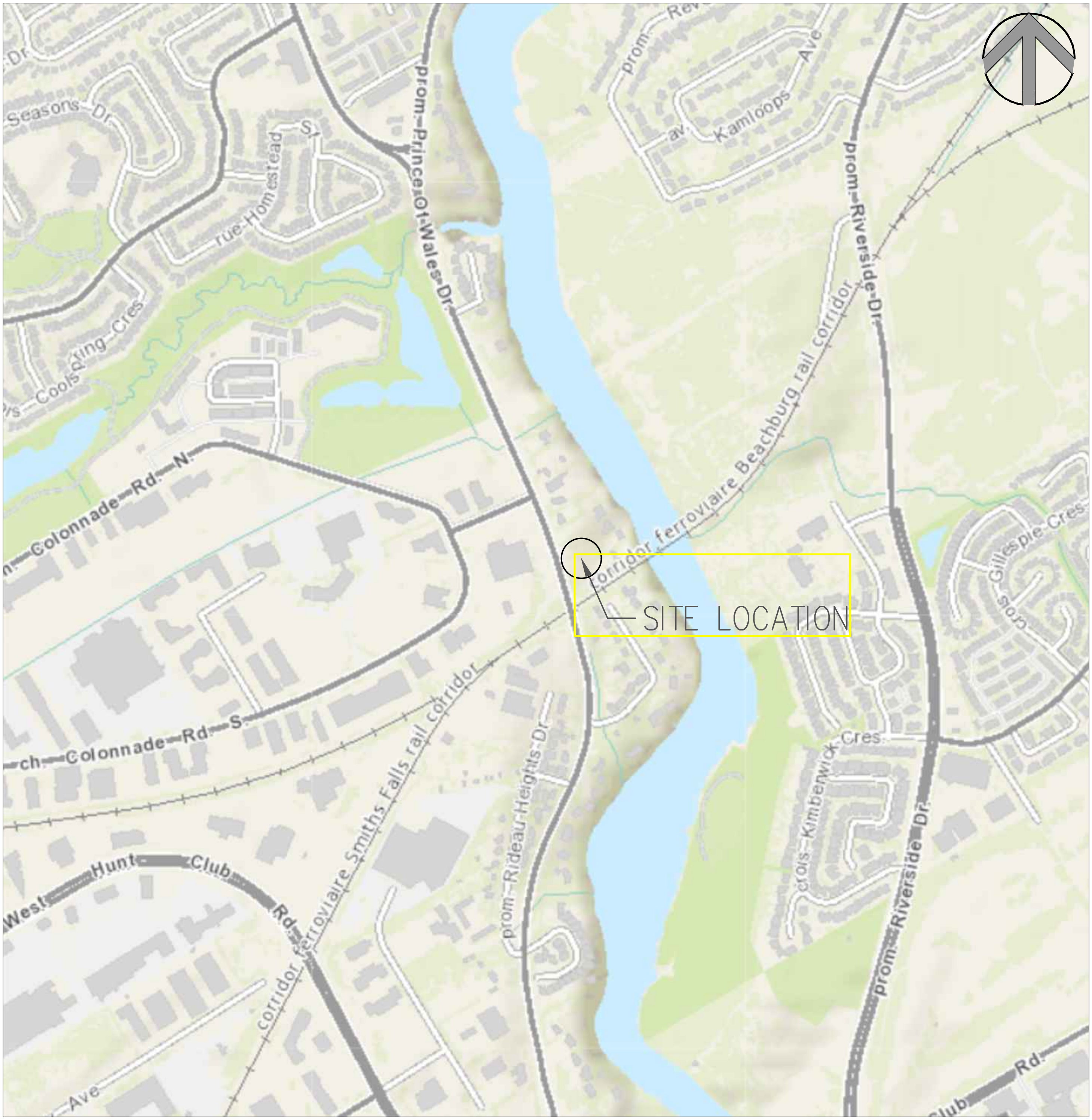
DATE

DECEMBER 2022

PROJECT

220528

FIGURE 1



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

PROJECT
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
2009 & 2013 PRINCE OF WALES DR.
OTTAWA, ONTARIO

DRAWING TITLE
BOREHOLE LOCATION
SOURCE: GOOGLE AERIAL VIEW

CLIENT	DATE	PROJECT	
JANE THOMPSON ARCHITECT	DECEMBER 2022	220528	FIGURE 2

APPENDIX B

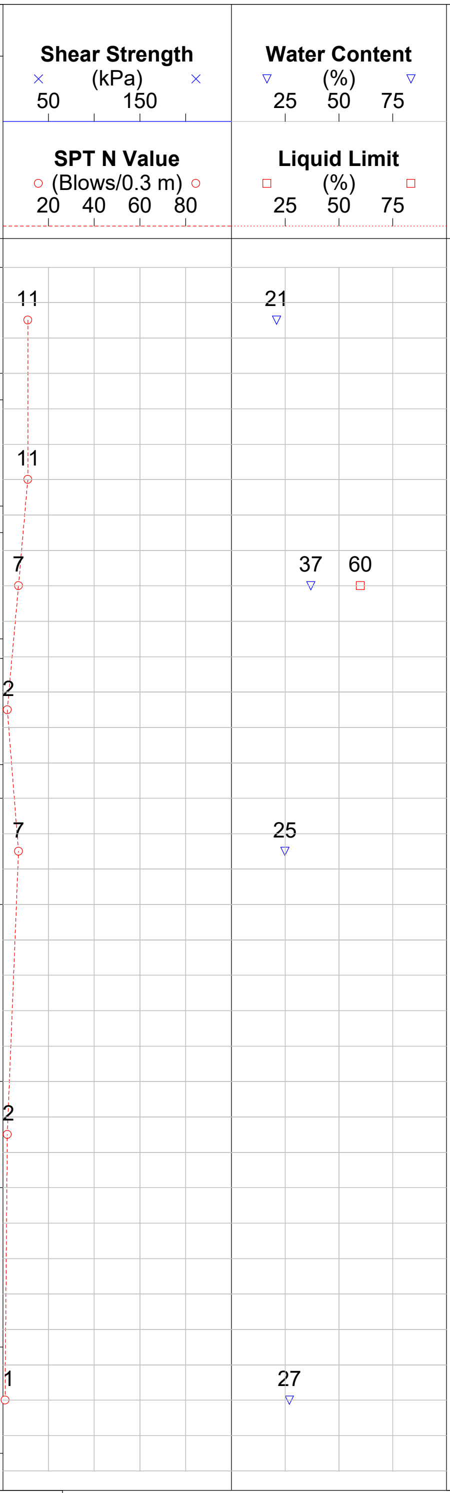
Borehole Logs

Borehole Log: BH1

Project No.: 220528
Client: Jane Thompson Architect
Date: November 28, 2022
Project: Geotechnical Investigation - Residential Development
Location: 2009-2013 Prince of Wales Dr., Ottawa ON
Field Personnel: SV
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Truck Mount CME 75
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE / SAMPLE DATA

Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)	Water Content (%)	Liquid Limit (%)	Monitoring Well Details
Ground Surface	99.03								
TOPSOIL clayey, approximately 600 mm thick	0.00	⧗	SS1	11	42	11	21		
FILL MATERIAL sand-silt-clay, brick debris, crushed stone, grey, compact, moist.	98.43 0.60	⧗	SS2	11	33	11			
SILTY CLAY sand seams, brownish grey, moist, firm.	97.58 1.45	⧗	SS3	7	83	7	37	60	
SANDY CLAY some silt, greyish brown, moist, firm to very soft.	96.82 2.21	⧗	SS4	2	100	2			
		⧗	SS5	7	100	7	25		
		⧗	SS6	2	100	2			
		⧗	SS7	1	100	1	27		
End of Borehole	92.33 6.70								



Easting: 445177 m
Northing: 5021522 m
Site Datum: TBM - Bolts on Flange of FH in front of Site (100.00 m)
Groundsurface Elevation: 99.029 m
Top of Riser Elev.: NA
Hole Diameter: 200 mm
Monitoring Well Diameter: N/A

NOTES:

Borehole Log: BH2

Project No.: 220528

Project: Geotechnical Investigation - Residential Development

Client: Jane Thompson Architect

Location: 2009-2013 Prince of Wales Dr., Ottawa ON

Date: November 28, 2022

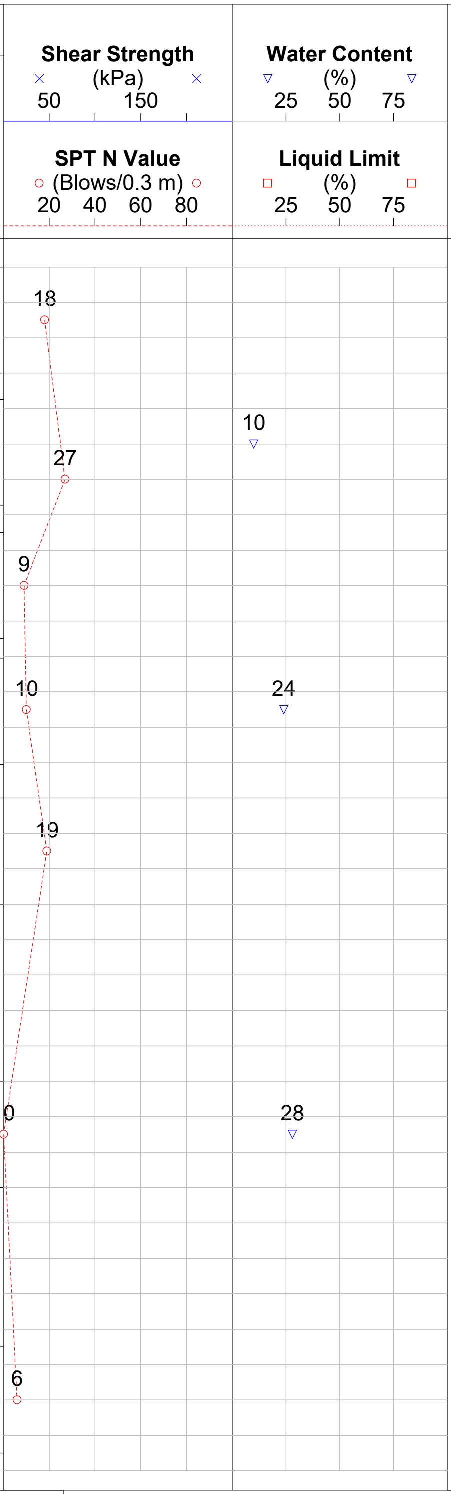
Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Truck Mount CME 75

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA					Monitoring Well Details
Depth	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	
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	Ground Surface	99.12 0.00					
	FILL MATERIAL sand-silt-clay, crushed stone, grey, compact, moist.		⧗	SS1	18	50	
			⧗	SS2	27	33	
	CLAYEY SAND to SANDY CLAY some silt, greyish brown, moist, very stiff to very soft.	97.67 1.45	⧗	SS3	9	100	
			⧗	SS4	10	100	
			⧗	SS5	19	0	
			⧗	SS6	0	100	
			⧗	SS7	6	5	
	End of Borehole	92.42 6.70					



Easting: 445203 m

Northing: 5021560 m

Site Datum: TBM - Bolts on Flange of FH in front of Site (100.00 m)

Groundsurface Elevation: 99.116 m

Top of Riser Elev.: NA

Hole Diameter: 200 mm

Monitoring Well Diameter: N/A

NOTES:

Borehole Log: BH3

Project No.: 220528

Project: Geotechnical Investigation - Residential Development

Client: Jane Thompson Architect

Location: 2009-2013 Prince of Wales Dr., Ottawa ON

Date: November 28, 2022

Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Truck Mount CME 75

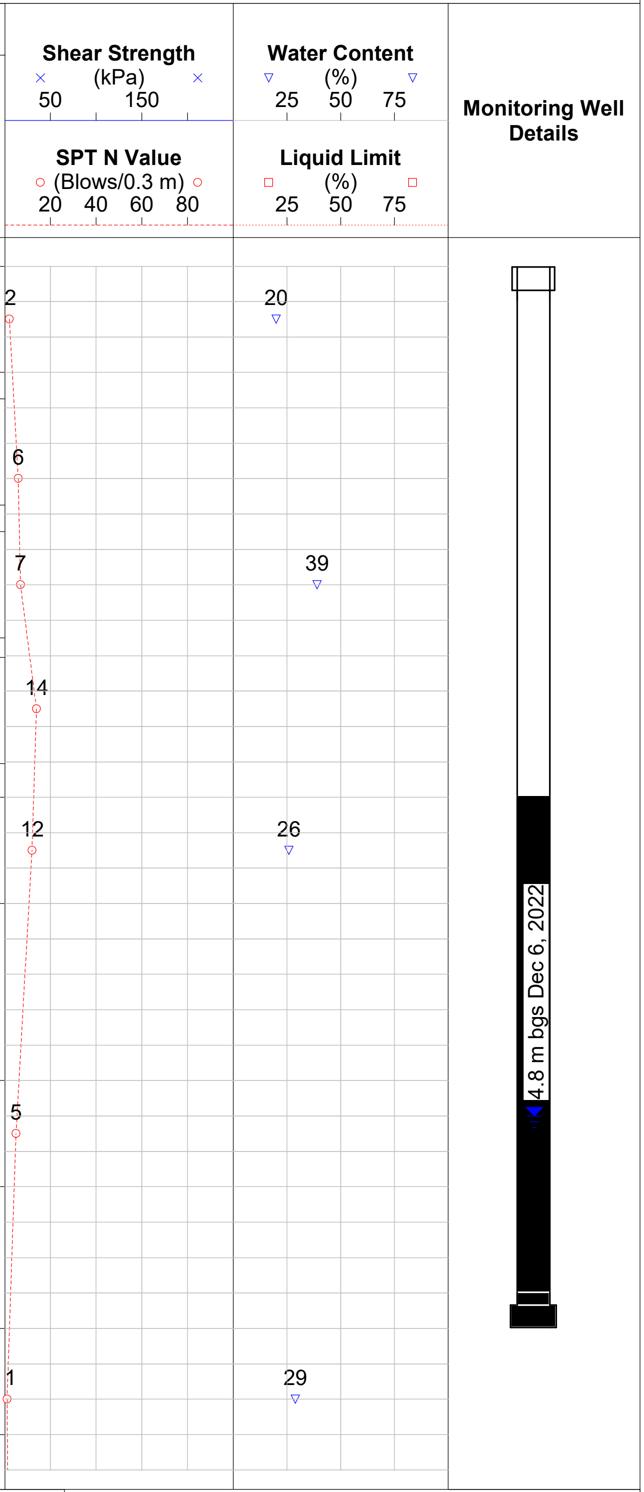
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE

Depth (m)	Soil Description	Elev./Depth (m)
0	Ground Surface	100.67
0.00	**TOPSOIL** clayey, approximately 600 mm thick	100.67 0.00
0.60	**SILT and CLAY** some sand, greyish brown, becoming grey with depth, moist, stiff to soft.	100.07 0.60

SAMPLE DATA

Type	Sample Number	N or RQD	Recovery (%)
⧗	SS1	2	50
⧗	SS2	6	50
⧗	SS3	7	100
⧗	SS4	14	100
⧗	SS5	12	100
⧗	SS6	5	100
⧗	SS7	1	100



Depth scale: ft 0–22; m 0–6

Easting: 445263 m

Northing: 5021584 m

Site Datum: TBM - Bolts on Flange of FH in front of Site (100.00 m)

Groundsurface Elevation: 100.671 m

Top of Riser Elev.: NA

Hole Diameter: 200 mm

Monitoring Well Diameter: 19 mm

NOTES:

Borehole Log (continued): BH3

Project No.: 220528

Project: Geotechnical Investigation - Residential Development

Client: Jane Thompson Architect

Location: 2009-2013 Prince of Wales Dr., Ottawa ON

Date: November 28, 2022

Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

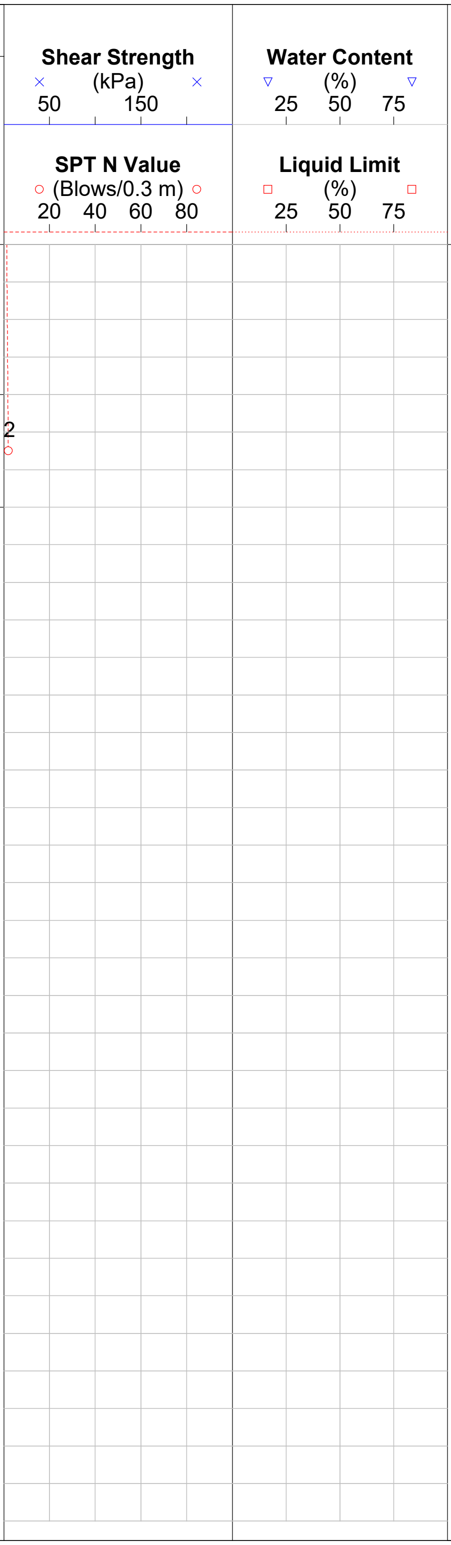
Drilling Equipment: Truck Mount CME 75

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA				
Depth	**Soil Description**	**Elev./Depth (m)**	**Type**	**Sample Number**	**N or RQD**	**Recovery (%)**
25–27 ft / 8 m			⧗	SS8	2	100
27 ft	End of Borehole	92.44 8.23				

Depth scale: 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44 (ft); 8, 9, 10, 11, 12, 13 (m)

Monitoring Well Details



NOTES

Borehole Log: BH4

Project No.: 220528

Project: Geotechnical Investigation - Residential Development

Client: Jane Thompson Architect

Location: 2009-2013 Prince of Wales Dr., Ottawa ON

Date: November 28, 2022

Field Personnel: SV

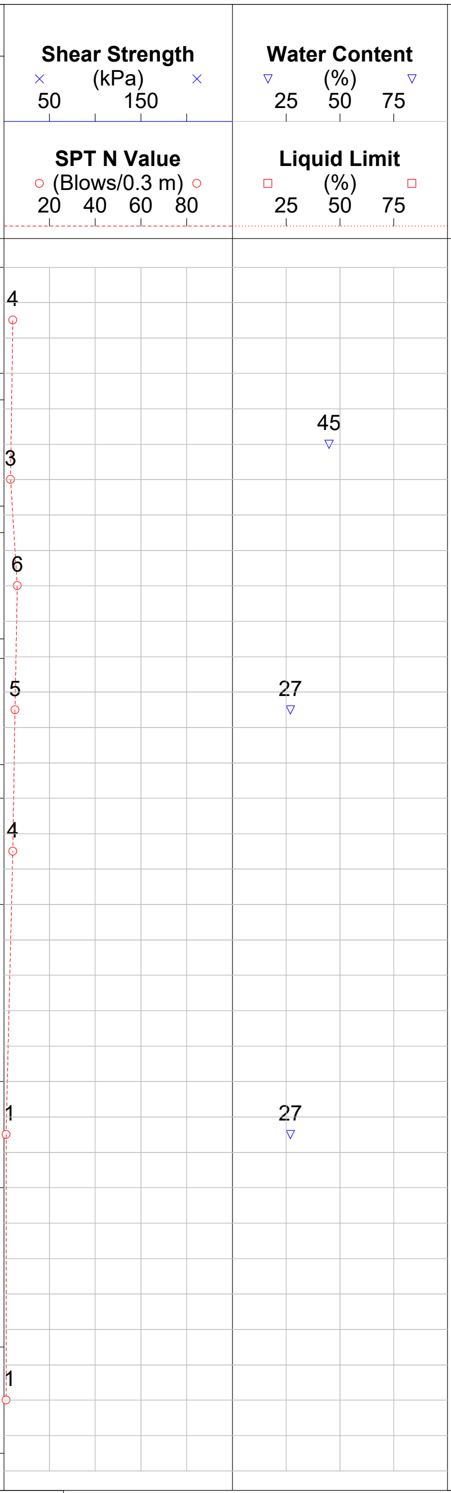
Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Truck Mount CME 75

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	99.21				
	TOPSOIL clayey, about 600 mm thick.	0.00 98.61	⧗	SS1	4	50
	CLAYEY SAND to SANDY CLAY some silt, greyish brown, moist, firm to very soft.	0.60	⧗	SS2	3	100
			⧗	SS3	6	100
			⧗	SS4	5	100
			⧗	SS5	4	100
			⧗	SS6	1	100
		92.51	⧗	SS7	1	100
	End of Borehole	6.70				



Monitoring Well Details

Easting: 445217 m

Northing: 5021592 m

Site Datum: TBM - Bolts on Flange of FH in front of Site (100.00 m)

Groundsurface Elevation: 99.214 m

Top of Riser Elev.: NA

Hole Diameter: 200 mm

Monitoring Well Diameter: N/A

NOTES:

Borehole Log: BH5

Project No.: 220528

Project: Geotechnical Investigation - Residential Development

Client: Jane Thompson Architect

Location: 2009-2013 Prince of Wales Dr., Ottawa ON

Date: November 28, 2022

Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Truck Mount CME 75

Drilling Method: Hollow Stew Auger

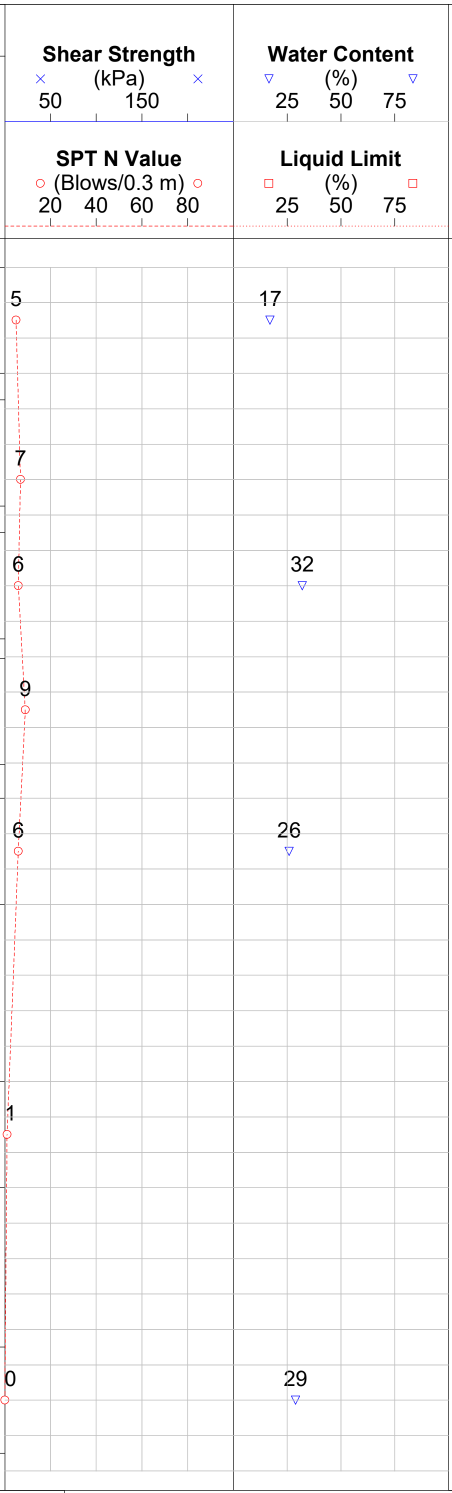
SUBSURFACE PROFILE

SAMPLE DATA

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

Depth m: 0 1 2 3 4 5 6

Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)
Ground Surface	98.96				
TOPSOIL clayey, about 600 mm thick.	0.00		SS1	5	50
	98.36				
CLAYEY SAND to SANDY CLAY some silt, greyish brown, moist, stiff to soft.	0.60		SS2	7	100
			SS3	6	100
			SS4	9	100
			SS5	6	100
	94.84				
SAND and SILT some clay, grey, moist, very soft.	4.12		SS6	1	100
			SS7	0	100
	92.26				
End of Borehole	6.70				



Monitoring Well Details

Easting: 445181 m

Northing: 5021573 m

Site Datum: TBM - Bolts on Flange of FH in front of Site (100.00 m)

Groundsurface Elevation: 98.961 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

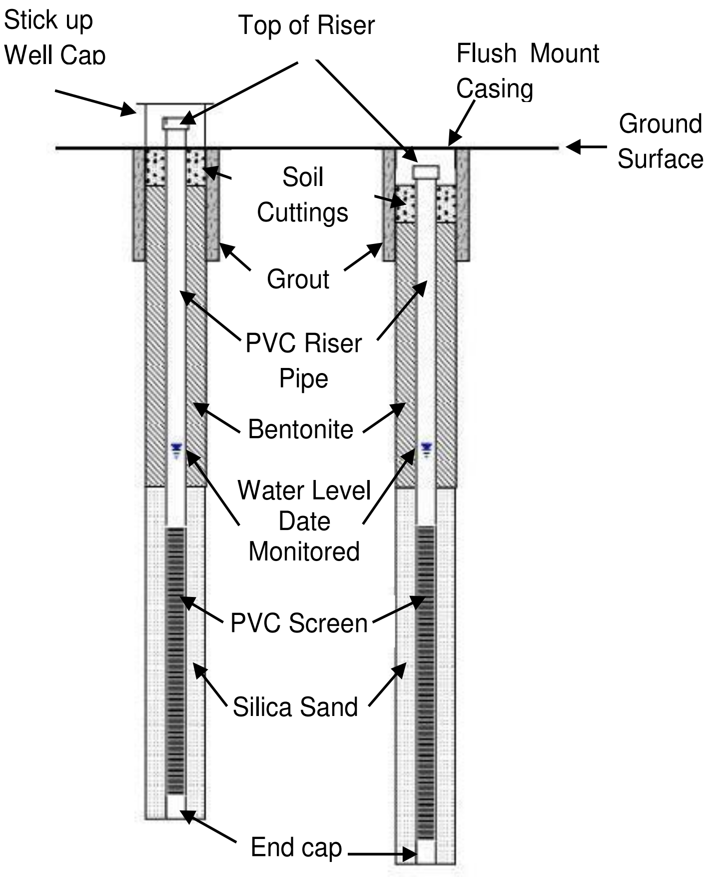
Rock Quality Designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mas. The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 100 mm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given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

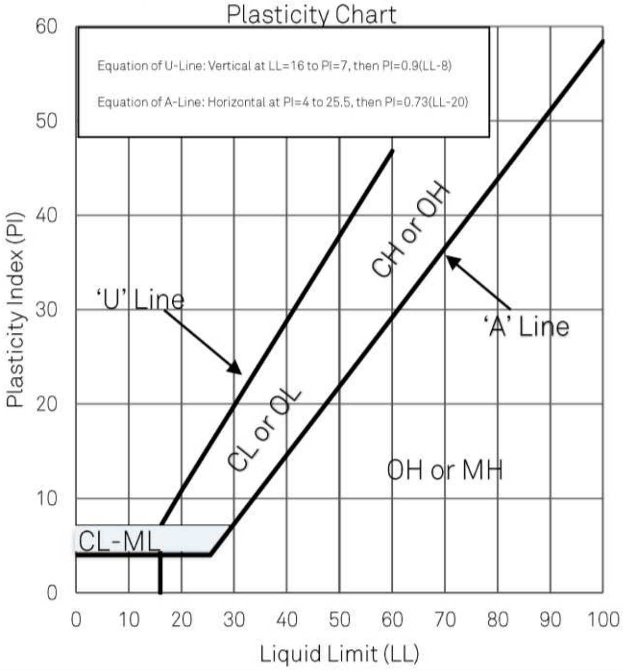
Strength classification of rock is presented below.

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 C_u or C_c 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 C_u or $C_c$criteria 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: Jane Thompson Architect

Project: Geotechnical Investigation

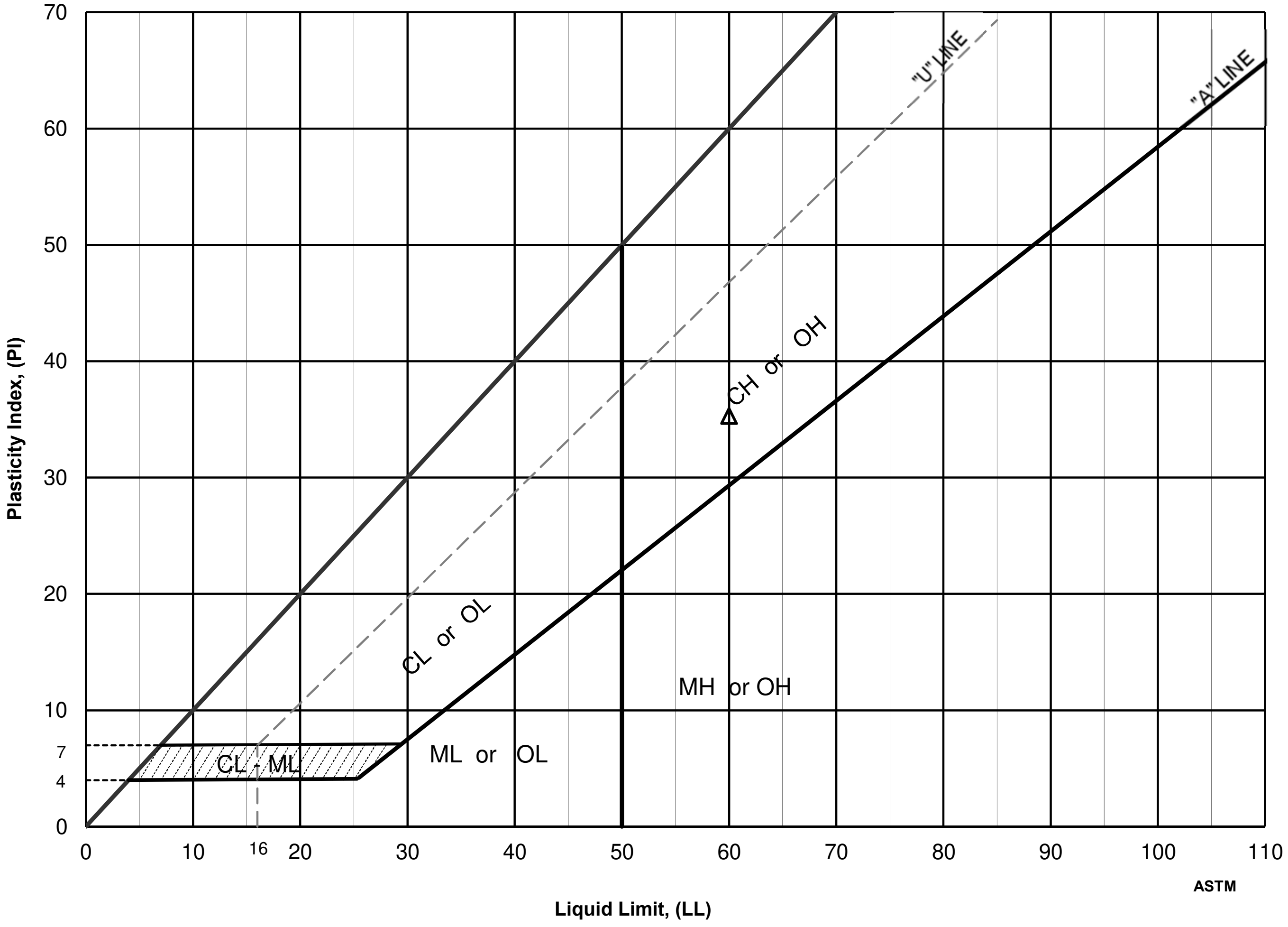
Location: 2009 & 2013 Prince of Wales Drive, Ottawa, ON.

File No.: 220528

Report No.: 1

Date: November 28, 2022

Plasticity Chart



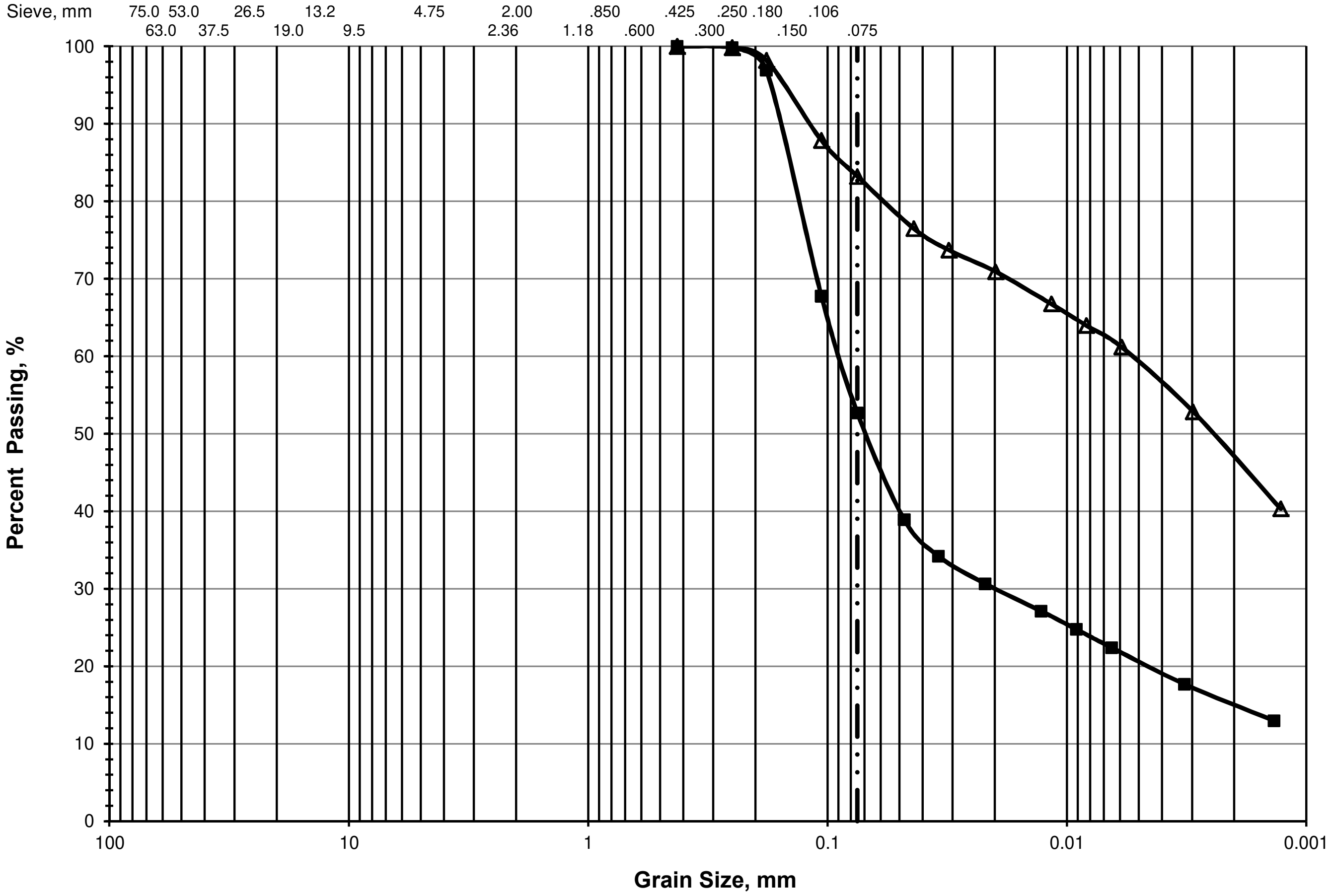
	Location	Sample	Depth, m	Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Activity Number	USCS
△	BH 1	SS-3	1.52 - 2.13	37	60	25	35	0.36	n/d	CH

LRL Associates Ltd.

PARTICLE SIZE ANALYSIS

ASTM D 422 / LS-702

Client:	Jane Thompson Architect	**File No.:**	220528
Project:	Geotechnical Investigation	**Report No.:**	2
Location:	2009 & 2013 Prince of Wales Drive, Ottawa, ON.	**Date:**	November 28, 2022



Unified Soil Classification System

	> 75 mm	% GRAVEL		% SAND			% FINES	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
△	0.0	0.0	0.0	0.0	0.0	16.8	37.5	45.7
■	0.0	0.0	0.0	0.0	0.0	47.2	38.2	14.6

	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH 3	SS-3	1.52 - 2.13	0.0055						
■	BH 5	SS-7	6.10 - 6.71	0.0900	0.0697	0.0203	0.0022			

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Certificate of Analysis

LRL Associates Ltd.
5430 Canotek Road
Ottawa, ON K1J 9G2
Attn: Brad Johnson

Client PO:
Project: 220528
Custody: 141039

Report Date: 6-Dec-2022
Order Date: 30-Nov-2022

Order #: 2249225

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

Paracel ID	**Client ID**
2249225-01	BH 5 - 5-7'

Approved By:

Milan Ralitsch, PhD

Senior Technical Manager

Report Date: 06-Dec-2022

Client: **LRL Associates Ltd.**

Order Date: 30-Nov-2022

Client PO:

Project Description: 220528

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	2-Dec-22	2-Dec-22
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	1-Dec-22	2-Dec-22
Resistivity	EPA 120.1 - probe, water extraction	5-Dec-22	5-Dec-22
Solids, %	CWS Tier 1 - Gravimetric	1-Dec-22	2-Dec-22

Summary of Criteria Exceedances

(If this page is blank then there are no exceedances)

Only those criteria that a sample exceeds will be highlighted in red

Regulatory Comparison:

Paracel Laboratories has provided regulatory guidelines on this report for informational purposes only and makes no representations or warranties that the data is accurate or reflects the current regulatory values. The user is advised to consult with the appropriate official regulations to evaluate compliance. Sample results that are highlighted have exceeded the selected regulatory limit. Calculated uncertainty estimations have not been applied for determining regulatory exceedances.

Sample	Analyte	MDL / Units	Result	-	-

	Client ID:	BH 5 - 5-7'	-	-	-		
	Sample Date:	28-Nov-22 12:00	-	-	-	-	-
	Sample ID:	2249225-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	76.9	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.32	-	-	-	-	-
Resistivity	0.1 Ohm.m	55.4	-	-	-	-	-
Anions							
Chloride	5 ug/g	23	-	-	-	-	-
Sulphate	5 ug/g	42	-	-	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%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	18.5	5	ug/g	18.1			2.4	20	
Sulphate	10.5	5	ug/g	9.28			12.3	20	
General Inorganics									
pH	8.02	0.05	pH Units	7.91			1.4	10	
Resistivity	21.4	0.10	Ohm.m	21.3			0.4	20	
Physical Characteristics									
% Solids	82.6	0.1	% by Wt.	82.4			0.2	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	126	5	ug/g	18.1	108	82-118			
Sulphate	121	5	ug/g	9.28	112	80-120			

Qualifier Notes:

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.

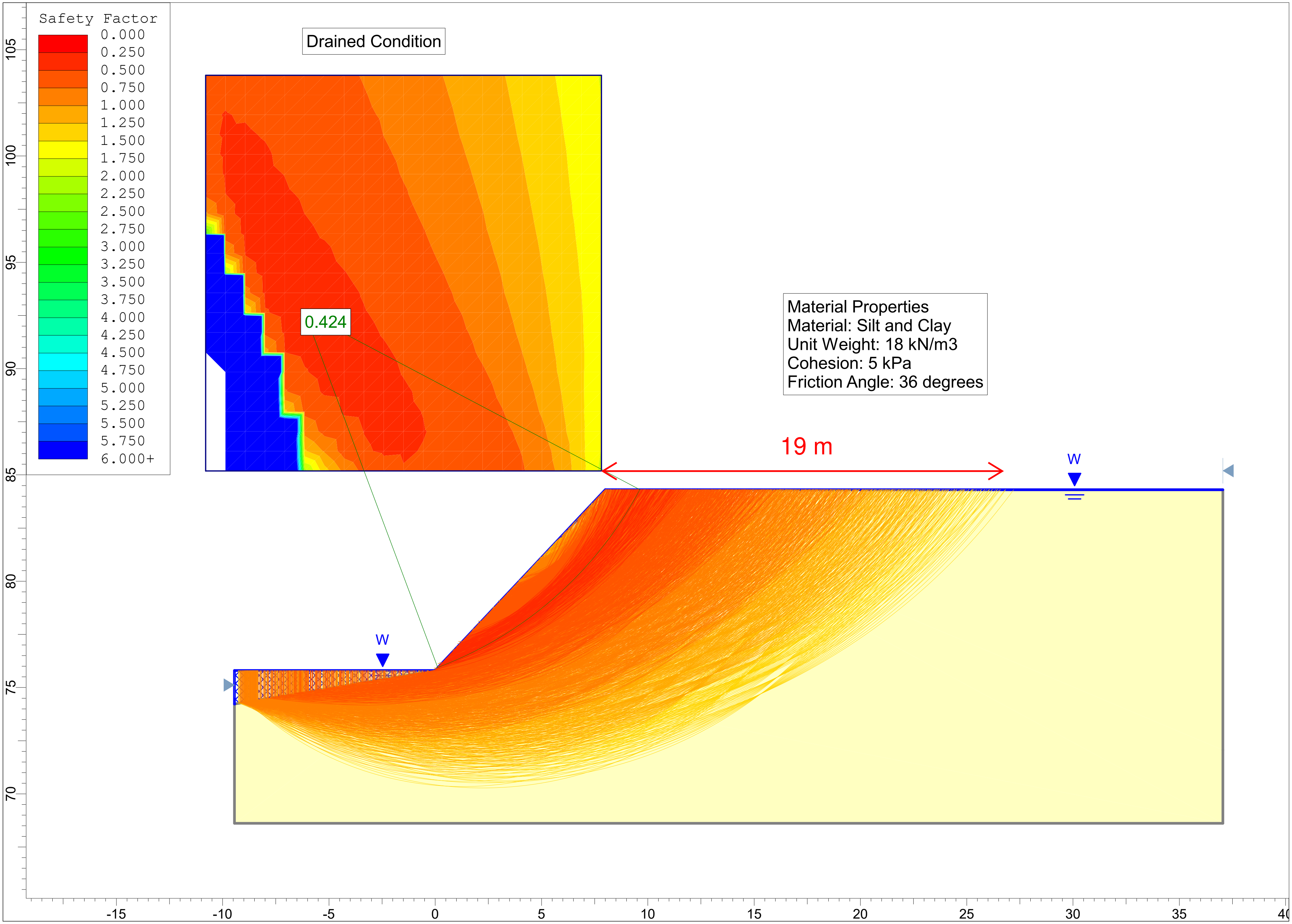
NC: Not Calculated

Soil results are reported on a dry weight basis unlesss otherwise noted.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

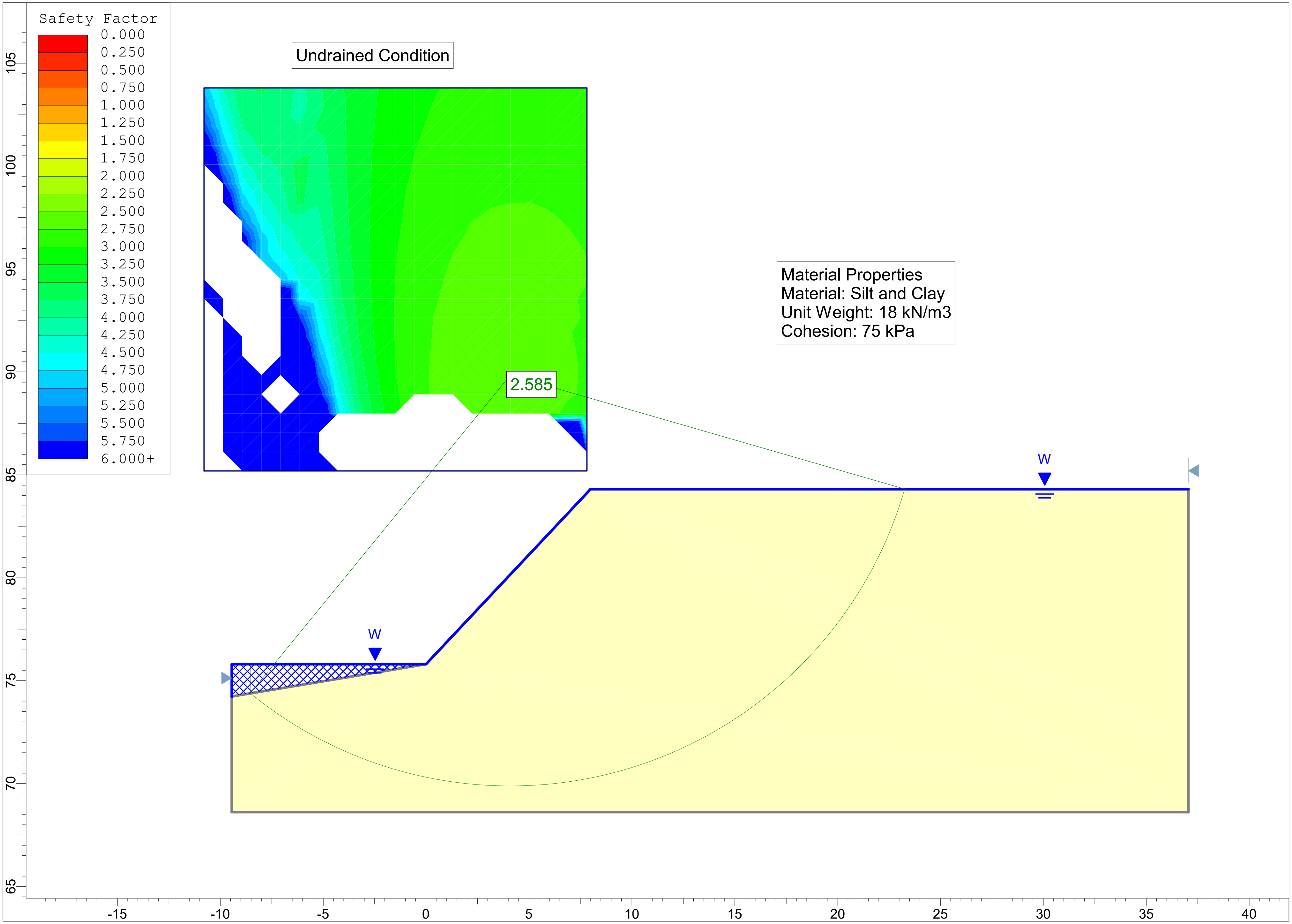
APPENDIX E

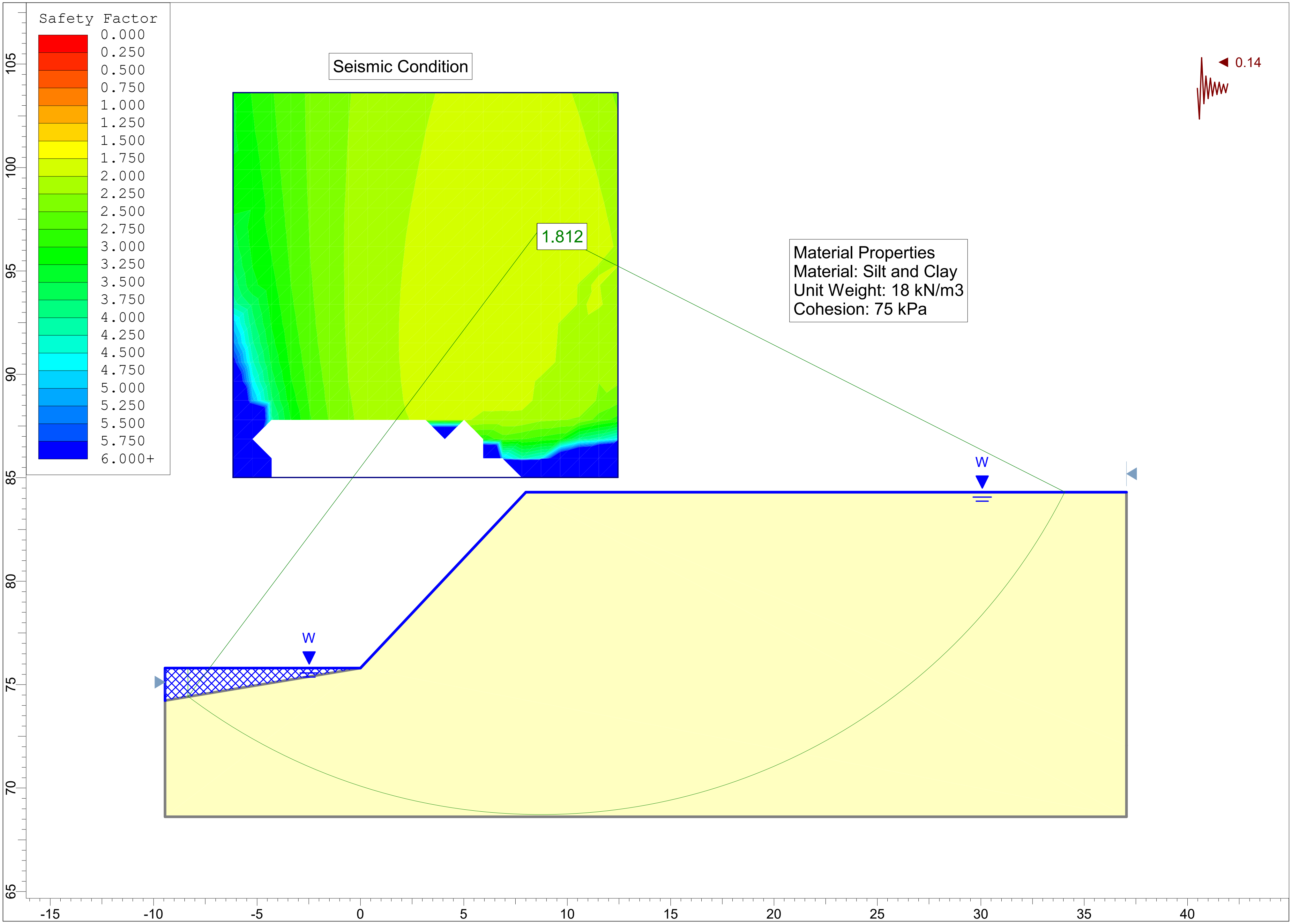
Slope Stability Modelling Results



Drained Condition

Material Properties
Material: Silt and Clay
Unit Weight: 18 kN/m3
Cohesion: 5 kPa
Friction Angle: 36 degrees


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+
Undrained Condition
2.585
Material Properties
Material: Silt and Clay
Unit Weight: 18 kN/m3
Cohesion: 75 kPa
W
W
105
100
95
90
85
80
75
70
65
-15
-10
-5
0
5
10
15
20
25
30
35
40


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+
Seismic Condition
1.812
Material Properties
Material: Silt and Clay
Unit Weight: 18 kN/m3
Cohesion: 75 kPa
0.14
W
W
105
100
95
90
85
80
75
70
65
-15
-10
-5
0
5
10
15
20
25
30
35
40