Geotechnical Investigation

Proposed Islam Care Centre
312 Lisgar Street
Ottawa, Ontario

Prepared for:

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LRL File No.: 150145

September, 2015, *Revised: February 2025*

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1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by Golden Rock Architecture to perform a geotechnical investigation for the proposed Islam Care Centre building, located at 312 Lisgar Street in the City of Ottawa, Ontario.

The purpose of the investigation was to identify and characterise the subsurface conditions at the site by the completion of a limited number of boreholes. Based on the factual information obtained, this report will provide guidelines on the geotechnical engineering aspects of the design of the foundations for the proposed building including construction considerations.

This report has been prepared in consideration of the terms and conditions noted above and with the assumption that the design of the project will satisfy any applicable codes and standards. 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 DESCRIPTION

The site under investigation for the proposed Islam Care Centre building is located at 312 Lisgar Street in the City of Ottawa, Ontario (see **Appendix A**). The site is a rectangular shaped parcel of land with approximately 10.1 m of frontage on Lisgar Street and a depth of about 34.3 m. The site is currently developed with a three storey apartment building.

It is understood that the present site development plan consists of the demolition of the existing structure at the site and the construction of a seven-storey Islam Care Centre building with two level basement floor designated as Basement I Floor and Basement II Floor. The area of the proposed structure equals 343.73 m^2 in plan and will cover the entire parcel of land. The Islam Care Centre building will be serviced with electricity, municipal water, storm and sanitary sewers. It is noted that the site grading plan had not been provided at the time of submitting this report.

3 PROCEDURE

The fieldwork for this investigation was carried out on August 31st, 2015. Prior to the fieldwork, the site was cleared for the presence of any underground services and utilities. A total of two (2) boreholes, labelled BH1 and BH2, were drilled across the property, where it was possible to drill. The approximate locations of the boreholes are shown on a site plan included in **Appendix A**.

The boreholes were advanced using a truck mounted (CME 55) drill rig equipped with 200 mm diameter continuous flight hollow stem auger supplied and operated by George Downing Estate Drilling Inc. 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 mm diameter drive open conventional spoon sampler in conjunction with standard penetration testing (“N” values). BH1 was advanced to depth of approximately 9.9 m below ground surface (bgs) where boring was terminated at auger refusal, which was technically established with standard penetration test (SPT) blow counts. BH2 was augered continuously to depth of approximately 7.9 m where auger refusal was encountered on weathered shale bedrock or on a massive boulder in glacial till deposit.

BH1 was further advanced to depth of approximately 11.4 m to obtain sound bedrock, which was technically established through auger refusal by the equipment.

Upon refusal over the weather to faintly weathered shale bedrock, BH1 was advanced by core drilling techniques using a NQ-size (Ø47.7mm) double-tube wire line core barrel. The recovered core samples were visually described, and placed in core box for further identification and observation by LRL's geotechnical engineer. Upon completion, the boreholes were backfilled using overburden cuttings and compacted temporarily.

The subsurface conditions encountered at each borehole locations were classified based on visual and tactile examination of the materials recovered from the boreholes. The fieldwork was supervised throughout by a member of our engineering staff, who supervised the drilling of the boreholes, the in-situ testing, cared for the samples obtained and logged the subsurface conditions encountered at each borehole. All soil samples collected from the borehole were placed and sealed in plastic bags to prevent moisture loss. All soil samples were transported to LRL's office for further examination by LRL's geotechnical engineer.

Furthermore, the borehole locations were surveyed and located using a Garmin Etrex Legend GPS (Global Positioning System) receiver using NAD 83 datum (North American Datum). The topographic survey was conducted using a laser level. The boreholes were referenced as elevation 100.00 m to the existing ground surface where borings were conducted.

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 consist of glacial deposit comprising of till; heterogeneous mixture of material ranging from clay to large boulders, generally sandy, grade downwards to unmodified till; surface generally modified by wave of river action.

The bedrock is combination of Lindsay and Billing Formation, dark grey to black limestone, noncalcareous to slightly calcareous, pyritiferous shale with limestone laminae and grey siltstone interbeds.

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

The subsurface soil conditions encountered at each borehole location are given in the Borehole Logs presented in **Appendix B**. A greater explanation of the information presented in the boreholes 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 Pavement Structure

Boreholes BH1 and BH2 encountered a pavement structure from the surface. The pavement consisted of about 25 mm thickness of asphaltic concrete overlying about a 575 mm thickness of fill materials.

4.3 Fill

Greyish brown silty fine sand was encountered in the BH1 immediately beneath the asphaltic concrete. The fill extended to depth of about 0.6 m bgs. The fill is generally described as being a heterogeneous mixture of sand-silt-clay, with trace of black organics. It is identified as being moist.

4.4 Sand

Underneath the fill, a thick layer of medium to fine grained, brown sand of thickness about 0.6 m was encountered in the boreholes. The standard penetration test “N” value within the sand material was found to be 2 blows per 0.3 metres, indicating this layer is very loose in consistency. It is also identifies as being moist.

4.5 Clayey silt

A clayey silt deposit was encountered immediately beneath the sand and thickness of the deposit was found to be approximately 1.9 m. The clay is described as being silty with occasional silt lenses or beds, and being brownish grey near the surface and becoming grey with depth. The deposit is described as uniform, fine grained with depth, and soft to very soft in consistency.

4.6 Clay

Underlying the clayey silt, a deposit of clay with trace to some silt was encountered in the boreholes. This deposit was found extended to depth 7.9 m bgs in BH1 & BH2. It can generally be described as dark grey in colour, wet, and very soft to stiff in consistency. The moisture content of clay deposit was found varying from 11 to 65% indicating from moist to saturated or wet condition.

Standard penetration tests were carried-out in the grey clay. “N” values obtained “static weight of hammer” (WH) per 0.3 m of penetration in BH2 between the depth 3.1 and 4.3 m bgs.

In-situ field vane shear tests were carried out in cohesive soil deposit in BH1 between depth 3.1 and 4.3 m and undrained shear strength values were calculated and found to be varied between 37 and 61 kPa, suggesting that this clay deposit to be soft to stiff in consistency.

Atterberg limits and moisture content, were conducted on the clay sample collected from BH2 at depth between 3.1 m and 4.3 m bgs. Based on the test results, the clayey silt yielded liquid limit varies between 60 and 63, close to moisture content and corresponding plasticity index is found to be 34 for both samples. This value indicates the clay of extremely high plasticity, inorganic clay (CH). The laboratory reports for the physical testing can be found in **Appendix D** of this report.

Table 1: Summary of Atterberg Limits and Water Contents

Location (Sample)	Parameter					
	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Water Content (%)	USCS Group Symbol
BH1	3.1 - 3.7	63	29	34	74.42	CH
BH1	3.7 - 4.3	60	27	34	63.42	CH

4.7 Glacial Till

A glacial till deposit was encountered beneath the clay at BH1. The glacial till consists of grey sandy silt with trace clay, some gravel. The SPT “N” value was 16 blows per 0.3 m of penetration, indicating this layer to be compact in consistency.

4.8 Refusal/ Shale Bedrock

Underlying the glacial till, shale bedrock was found in BH1. The grey shale was typically highly to faintly weathered and contained limestone/siltstone interlayers. Augur refusal was encountered over inferred bedrock/massive boulder in BH1 and BH2 at depth 9.85 and 7.9 m bgs respectively.

At BH1, between the depth 9.4 and 9.85, the grey to black shale was found weathered with occasional mud seams. The bedrock was most likely faintly weathered near surface and becoming thinly bedded shaley limestone with depth. The Total Core Recovery (TCR) was 100% below a depth about 9.85 m. The discontinuities observed in the rock core are mostly horizontal, and generally associated with the bedding planes. There was only a single natural discontinuity close to 45 degree to horizontal plane. According to ASTM D6032, Rock Quality Designation (RQD) ranged between 50 and 100% is the indicative of fair to excellent quality. Based on established RQD, which was found to be 61.3%, the rock would have fair quality at coring depth between 9.85 and 11.35 m bgs. Photograph of the rock cores covered from BH1 are presented in **Appendix E**.

4.9 Groundwater Conditions

After the completion of drilling operation, both boreholes were carefully observed and no evidence of groundwater or perched-water was noticed.

Due to time constraints, the seasonal trend of groundwater could not be established. It should be noted that groundwater levels could fluctuate with seasonal weather conditions, (i.e.: rainfall, droughts, spring thawing) and due to construction activities at or in the vicinity of the site.

5 GEOTECHNICAL DESIGN CONSIDERATIONS

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

It is our understanding that the project Islam Care Centre is proposed to construct a seven (7) storey with two level basement mainly for community services and places of worship. The elevations of the proposed 01 Basement Floor (Basement I) and 00 Basement floor slab (Basement II) will be at -3.05 and -7.90 m bgs respectively as per Golden Rock Architecture Inc.’s Drawing A401. An access (entry and exit) from Lisgar Street is proposed through the north property line with no driveway/ramp, parking facilities are included as part of this proposed development.

This section will detail the specific requirements and limitations with regard to allowable foundation bearing pressure and depth, grade raise and size of the footings. The primary concern for this project is the strength of glacial till deposit which was found to be in compact state with regards to consistency and will potentially limit the allowable bearing pressure of conventional strip and pad footing.

5.1 Foundations

Based on localised borings, the subsurface found on this site consisted of approximately 1.2 m fill with medium to fine grained sand deposit overlaying approximately 6.7 m soft to stiff clay silt followed by clay, overlaying about 1.5 m compact glacial till. Based on the subsurface soil conditions established at this site, it is recommended that the footings for the proposed Islam Care Centre building be founded over sound bedrock at depth approximately 9.5 m below the exiting grade.

5.1.1 Option I – Strip and Spread footings

With the exception of the pavement structure, fill and clay deposit, the subsurface conditions encountered at the boreholes location advanced during the field investigation are suitable for the support of proposed Islam Care Centre building on conventional spread footing foundations. The proposed structure with two level basement can be supported by conventional strip and spread footings resting on the existing glacial till deposit. However, it is anticipated that the design loads for the proposed building will be considerably high for conventional footing placed over glacial till at depth 7.9 m bgs. As an option, the excavation shall be carried out over the entire building footprint down to the shale bedrock. The footings set over sound bedrock may be designed using a maximum allowable bearing pressure of **600 kPa** for serviceability limit state (**SLS**) and **900 kPa** for ultimate limit state (**ULS**) factored bearing resistance. The allowable bearing capacity is based on a footing width up to 1.8 m for strip footings and 3.0 m for pad footings on any sides. Because of time constraint, no long term monitoring well was installed. However, it is recommended that the underside of the footing be set at 0.3m above the established groundwater table. There is no allowable grade raise restriction for founding footing over the bedrock formation. Furthermore, the footing for a specific building must rest entirely over bedrock and not two different founding strata (bedrock and glacial deposit) in order to limit differential settlements.

The above bearing pressure was assumed considering all weathered shale/fractured bedrock and mud seam, if any, are completely removed from the footing founding surface. Based on subsoil profile in BH1, the thickness of weathered to faintly weathered shale is expected to be 300 to 400 mm underneath the glacial till deposit. Therefore, it is suggested after removing the weathered shale, a 50 mm diameter probe holes to be advanced at selected location to depth about 1.5 m below the proposed founding level to verify the bedrock condition. If and when there are any thick soil seam or zone of fractured bedrock encountered near the footing founding level, it may be necessary to sub-excavate and remove the soil seam/soft or fractured rock. The probe holes shall be conducted and inspected by a geotechnical engineer or a qualified representative of a geotechnical engineer.

If the sub-excavation through the overburden exceeds more than 3.0 m, consideration could be given to include an additional basement level as opposed to sub-excavating to about 3.0 m and raising the grade below the proposed building grade.

Based on our experience, a portion of bedrock could contain shale, which may become fissile upon drying or exposed for prolonged periods. It is therefore recommended that a minimum 50 mm mud slab using non-shrinkable concrete of strength 7.0 MPa be placed as soon as possible (within 24-hours following initial exposure) over any exposed bedrock surface.

5.1.2 Option II – Caisson Foundation

Consideration could also be given on supporting the proposed Islam Care Centre on drilled pier/reinforced concrete caissons set below any highly weathered shale bedrock overlying relatively sound bedrock. The caisson would need to be set on the relatively sound bedrock and can be designed using an allowable net safe bearing pressure at tip **1000 kPa**. It should be noted that the caissons are assumed to have a depth/diameter ratio of equal or greater than 3. Socketed piers/drilled caisson should have a socket length to diameter ratio at least 1.5. If the caissons are installed through the clay or glacial till deposit, temporary liners will be required installed to seal and to help silty soil from caving and thus minimize the possible formation of voids below the floor slab and to help control water seepage into the caissons. During auguring, there is possibility to encounter cobbles and boulders and allowances should be made to break the boulder where necessary. The above bearing capacity assumes that the bottom of the excavation is properly cleaned. If the bottom of the excavation is not properly cleaned, the above bearing capacity may not be mobilized before large settlement occurs owing to the compression of any debris remaining in the bottom of the excavation. The minimum caisson size to allow access for cleaning and inspection is 760 mm.

If any bedrock surface supporting foundations slopes greater than 10° off the horizontal, rock anchors may considered or adequate dowels be installed to resist sliding. A staff member from the LRL office must review the conditions of the field. Alternatively, the contractor could level the footing bearing surface using a hydraulic hammer.

5.3 Rock Anchors

The capacity of rock anchors is dependent on the bond between the rock and grout. The method of installation will also affect the capacity of the bond between the rock and the grout. It is recommended that the capacity be calculated based on the bond length in the limestone sound bedrock. An experienced structural engineer should review all typical failure modes. Experienced rock anchor supplier may be contacted to discuss/recommend type, size and anchor material with qualified personnel. An invert cone angle of 90º may be used in the design of the anchors. Pull out testing should be carried out on the anchors to verify installations and to design load capacities. If bedrock is removed through mechanical hydraulic hammers (i.e. for levelling and installation of anchors), is not expected to affect the contribution of the upper level of rock in the calculation of anchors capacity.

It is suggested that the anchors be provided with a bonded length at the base of the anchor, which will provide the anchor capacity, as well as an unbonded length between the rock surface and the start of the bonded length. Based on the rock quality established at this site, it is recommended the first 0.5 m of the bedrock (from soil-rock interface) not be included as part of the bond length of the anchor. Permanent anchors should be provided with corrosion protection. At minimum, the entire drill hole should be filled with cementious grout, where the unbounded length is provided by using a polyethylene sleeve or approved equivalent, which acts as a bond break.

The average compressive strength measured of the rock core was **75.2 MPa**. Considering that limestone can produce considerable higher tensile strengths than that of shale, the recommended factored tensile grout to rock bond resistance value used in the design of the rock anchor should be **580 kPa**. The minimum recommended grout strength should be 40 MPa.

Bar tendons rather than cable tendons should be used for rock anchors. This will reduce the required depth of the anchors, making installation easier and will likely reduce costs. The geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system.

5.4 Settlement

The footings founded below any weathered portion of the bedrock and on relatively sound bedrock, the estimated total settlement of the foundations founded over bedrock designed using the recommended serviceability limit state capacity value is less than 15 mm. The differential settlement between adjacent column footings is anticipated to be 0.75 of the maximum settlement value given or less.

5.5 Seismic

The subsoil and groundwater information at this site have been examined, and based on the limited geotechnical investigation and in accordance with the Ontario Building Code 2012 (table 4.1.8.4.A.) and Canadian Foundation Engineering Manual (4th edition), a site class “C” is recommended for the design of the structure placed directly over sound bedrock.

The above classification was recommended based on conventional method exercised for Site Classification for Seismic Site Response and in accordance with the generally accepted geotechnical engineering practice.

It is noted that a greater seismic site response class may be obtained by conducting a seismic velocity testing using a multichannel analysis of surface waves (MASW).

5.6 Potential for Soil Liquefaction

Based on the subsurface soil conditions established at this site, the foundations will set over bedrock. As such, soil liquefaction is not considered to be a concern for this site.

5.7 Frost Protection

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

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

5.8 Foundation Drainage

Permanent perimeter drainage is required to be installed surrounding the basements, elevator pits and places whenever any open spaces located below the finish ground are being considered. A 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 crushed stone wrapped in a geotextile 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.

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

5.9 Foundation Walls Backfill

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

The foundation wall backfill should be compacted to minimum 95% of its Standard Proctor Maximum Dry Density (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.

Where foundation walls are located adjacent to an exposed rock face, the exterior face of the wall should be entirely covered with a deformable material (i.e. 50 mm extruded polystyrene) to limit lateral pressure on the wall created by the backfill material during placement and compaction. Increased earth pressures due to compaction equipment should be considered in the structural design of the walls, as recommended in Figure 24.9, of the Canadian Foundation Engineering Manual, 2006 or updated version (if any).

5.10 Basement Slab-on-grade Construction

Slab-on-grade construction will be acceptable over the approved structural fill only. Therefore, all soft, incompetent or otherwise deleterious material shall be removed from the proposed building’s footprint. The exposed bedrock subgrade surface should then be inspected and approved by geotechnical personnel.

Any underfloor fill needed to raise the bedrock subgrade to underside of concrete floor slab level should consist of a minimum of 200 mm thickness of crushed stone meeting OPSS 1010 Granular A immediately beneath the concrete floor slab followed by sand and gravel meeting the OPSS Granular B Type I material or crushed stone meeting the OPSS Granular B Type II. The fill material shall be compacted in maximum 300 mm thick lifts to at least 98% of its SPMDD. The final lift shall be compacted to 100% of its SPMDD. The base material of floor slab, the OPSS Granular A shall compacted to 100% of its SPMDD.

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage of the concrete. The saw cut depth should be about one quarter of the thickness of the slab.

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

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

5.11 Temporary Shoring

It is understood that the proposed excavation may be supported by temporary shoring consisting of timber lagging and soldier piles. A caisson wall may be required to support the adjacent high-rise structure at eastside of the property. The details of caisson walls will be provided on later once the structural detail is available to our office.

The caisson wall filler piles must be drilled through weathered rock and be taken at least 0.6 m into the sound bedrock to cut-off the possible groundwater seepage. Leakage of groundwater also may occur at a depth of approximately 8.5 m within the glacial till zone and hence extra precautions must be taken to water proof the anchor zone.

The shoring system must be designed in accordance with the Fourth Edition of the Canadian Foundation Engineering Manual. The soil parameters estimated to applicable for the design as follows:

a) Earth Pressure Coefficients

Where movement must be minimal, K = 0.56

Where minor movement (0.02H) can be tolerated, K = 0.37

Passive earth pressure on soldier piles, k_p = 2.70 for the soft to stiff clay below Elevation about 96.9 m

b) For stability check

ϕ = 22 degree for soils above Elevation 96.9m and ϕ = 27 degree for soils below about Elevation 96.9 m.

γ = 18 kN/m^3 (Surcharge to be determined by shoring contractor)

c) For earth anchors

Bond value = 30 kPa for soils above Elevation 96.9 m;

Bond value = 55 kPa for soils below Elevation 96.9 m.

d) For rock anchor

A bond stress of 580 kPa can be used in sound bedrock for the design of anchor.

These values depend on anchor installation methods and grouting procedures. Gravity poured concrete can result in low bond values while pressure grouted anchor will achieve higher values and produce a more effective anchor.

Safe net bearing value for soldier pile caissons base assuming on clean dry hole, q = 1000 kPa

The soldier piles should be installed in pre-augered holes taken below the deepest excavation level and half bag mix above the base excavation. The concrete strength must be specified by the shoring designer. Temporary liners may be required to prevent caving from sand/clayey silt at surface and clayey soil below elevation 96.9 m. Positive measure may be required to prevent the loss of soil through the spaces between the lagging boards. This could be prevented by placing well graded sand and gravel behind the lagging boards or by installing a geotextile filter cloth.

Bedrock may need to be supported with pattern bolting and welded wire mesh. In zones where poor rock or fractured rock due to rock excavation has occurred; a rock shotcrete may be required.

Soil anchor will be required to support the shoring. The minimum unbonded length of 3.0 m for bar tendons and 4.5 m for strand tendons should be adopted. These minimum unbonded lengths are required to avoid unacceptable load reduction resulting from seating losses during load transfer and the overall stability of the system must be checked at each anchor level.

The top anchor must not be placed lower than 3.0 m below the top of ground surface level. Anchors will require casing when penetrating through wet silty soil layers. The bond values of 25 to 55 kPa suggested but these values are arbitrary since the contractor installation procedures will determine the actual soil to concrete value. Hence, the contractor must decide on a capacity. All anchors must be tested as indicated in the Canadian Foundation Engineering Manual (4th Edition).

Adhesion on the buried caisson or behind the shoring system must be neglected when designing this shoring system. The recommended design parameters should be confirmed by load testing on at least six anchors (1-anchor on upper level and 1-anchor at lower level at each of three sides) to 200% of design load in accordance to Canadian Foundation Engineering Manual. All remaining anchors shall then be installed and proof test should be taken to the maximum test load of 1.33 times of the service load.

5.12 Shoring Monitoring

Movement of the shoring system is expected. Vertical movements will result from the vertical loads on the soldier piles resulting from the inclined tieback and inward horizontal movement results earth and anticipated water pressures. The lateral movements of the temporary support system decreases as the shear strength of the soil increases. Movements are generally small if horizontal support are installed as soon as the support level is reached and can be expected to be in the range of 0.1 to 0.3 percent of the excavation depth in overburden. Vertical movements increase the horizontal movements because of the reduce stress in the inclined anchors and must be kept well below this value.

To ensure that movements of the shoring are within the acceptable range, monitoring must be carried out. Vertical and horizontal targets on the soldier piles must be located and surveyed before excavation begins. Weekly readings during excavation should show that the movements will be within the range those predicted; if not, the monitoring results will be enable to direct and to improve the shoring system.

5.13 Lateral Earth pressure on Basement Wall

The following equation should be used to estimate the intensity of the lateral earth pressure against any earth retaining structure/basement 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 prevent the build-up of any hydrostatic pressure behind the basement wall.

5.14 Retaining Walls and Shoring

The following **Table 2** below provides the suggested soil parameters for the design of retaining wall and/or shoring systems. For excavations near existing services and structures, the coefficient of earth pressure at rest (K_o) should be used.

Table 2: Materials Properties for Shoring and Permanent Wall Design (Static)

Type of Material	Bulk Density (kN/m^3)	Friction Angle	Pressure Coefficient			Combined static and seismic active earth pressure coefficient (K_{AE})
			Active (Ka)	Passive (Kp)	At Rest (Ko)	
Granular A	23.0	35	0.27	3.70	0.43	0.35
Granular B Type I	20.0	30	0.33	3.00	0.50	0.41
Granular B Type II	23.0	32	0.31	3.22	0.47	0.39
Sand	20.0	30	0.33	3.00	0.50	0.41
Clay	18.0	27	0.37	2.70	0.56	0.45
Glacial till	23.0	35	0.27	3.70	0.43	0.35

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

Retaining walls should also be designed to resist the earth pressures produces under seismic conditions. The Canadian Building Code recommends the used of combined coefficients of static and seismic earth pressure, referred to as K_{AE} for active conditions and K_{PE} for passive conditions for routine design purposes.

The total active and passive loads under seismic conditions can be calculated using the following two equations;

$$P_{AE} = ½ K_{AE} \gamma H^2 (1-k_V)$$

$$P_{PE} = ½ K_{PE} \gamma H^2 (1-k_V)$$

Where;

K_{AE} = Combined Static and Seismic Active Earth Pressure Coefficient

K_{PE} = Combined Static and Seismic Passive Earth Pressure Coefficient

H = Total Height of the Wall (m)

K_h = horizontal acceleration coefficient

K_v = vertical acceleration coefficient

γ = bulk density (kg/m^3)

These equations are based on a horizontal slope behind the wall and a vertical back of the retaining wall and zero wall friction. For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values.

A = Zonal acceleration ratio = 0.2

K_h = Horizontal acceleration coefficient = 0.1

K_V = Vertical acceleration coefficient = 0.067

The above value of K_h corresponds to ½ of the A value and the value K_V of corresponds to 0.67 of the K_h value. The angle of friction between the soil and the wall has been set at 0 degree to provide a conservative estimate.

6 POTENTIAL OF CORROSIVE ENVIRONMENT

6.1 Sulphate Attack on Buried Concrete

A total of Two (2) representative soil samples were submitted for a sulphate analysis. The laboratory analysis was performed by Paracel Laboratories Ltd., an accredited chemical testing laboratory. The laboratory Certificates of Analysis are presented in **Appendix D.** The results are summarized in **Table 3** below.

Table 3: Summary of Sulphate Content

Parameter	Borehole/Sample type	
	BH1(soil sample)	BH1 (bedrock sample)
Depth (m)	3.1 – 3.7	10.5
Sulphate Content (µg/g)	83	150
Sulphate Content (%)	0.0083	0.015

Based on the CAN/CSA-A23.1 standards (Concrete Materials and Methods of Concrete Construction), a sulphate concentration of 0.1% (1000 µg/g) or less in soil falls within the negligible category for sulphate attack on buried concrete. 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.

6.2 Corrosion on Buried Steel

Furthermore, as part of the requirements for this project, two representative samples (one from soil and one from bedrock) collected from BH1 at depth 3.1-3.7 m and 10.5 m respectively were submitted to Paracel Laboratories Ltd. for additional chemical analysis, which included pH, Resistivity, Chloride and Redox Potential. The purpose of this testing was to assess the potential for a corrosive environment on any buried steel. The pH and chloride concentration in the soil sample was found to be 8.2 and 757 µg/g (0.8 mg/kg) respectively. The laboratory Certificates of Analysis are presented in **Appendix D.**

The potential for an aggressive corrosive soil environment was established in reviewing the above measured parameters and according to standard provided by the American Water Works Association (AWWA) C-105/A21.5-10. Based on the noted standard, corrosion protection for buried steel is only required where a corrosivity index of 10 or greater is encountered. Based on the test results, the calculated corrosivity index was found to be all below 5 except for resistivity which is found to at or above the index point 10. As such, with the exception of cast iron pipe, any buried steel as part of this project would not require any special or specific corrosion protection measures. Ductile cast iron pipe can be used with polyethylene encasement which is found very effective for corrosion protection.

7 EXCAVATION AND BACKFILLING REQUIREMENTS

7.1 Excavation

It is anticipated that the depth of excavation for the building and underground services will be ranging from 9.0 to 10.0 m bgs. The overburden soil encountered at this site consists primarily of fill, sand and clayey silt to clay, which is underlain by compact glacial till. The overburden, which is soft, very soft or loose in consistency can be classified as Type 4 soil and compact glacial till can be classified as Type 3 soil. Therefore, shallow, temporary excavations in overburden soil classified as Type 3 can be cut at 1 horizontal to 1 vertical and Type 4 soil can cut at 3 horizontal to 1 vertical for a fully drained excavation. All excavations should be carried out in accordance with the most recent Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 213/91 and its amendments. Part III of OHSA O. Reg. 213/91 deals with excavation. In the event that the specified slopes are not possible to achieve due to space restrictions, the excavation shall be shored according to OHSA O. Reg. 213/91 and its amendments. The shoring specialist shall design the shoring and establish the shoring depth under the excavation profile. Refer to the parameters provided in **Section *5.11* & *5.14*** for the design of any shoring structures.

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

7.2 Shale Bedrock Excavation

Due to the nature of bedrock at this site, it is considered that some minor bedrock removal is required and removal of bedrock is possible by using heavy excavating equipment, most of the bedrock removal could be facilitated by means of hoe ramming operation. Both horizontal and vertical overbreak of the bedrock excavation face/bottom can be expected due to hoe ramming operation. If control of potential bedrock overbreak is required, line drilling at the proposed excavation face could be carried out. The smaller the distance between the drill holes the less overbreak can be expected. It is considered that the drilling at 150 mm horizontal spacing to the full depth of the excavation should control overbreak to an acceptable level.

In view of the proximity of the existing adjacent structures and potential for vibration during excavating and removal of the bedrock, it is considered that monitoring of the hoe ramming should be carried out throughout the operation to ensure that the limiting vibration criteria established by the specialist engineer is met. A pre-excavation condition survey of nearby structures and existing utilities should be carried out.

7.3 Groundwater Control

Based on subsurface condition and nature of the soil encountered, it is anticipated that groundwater seepage or infiltration from the native soils into the excavations during construction should be minor in nature. Ground water seepage, and infiltration entering into the excavation may increase when the excavation extended below clay deposit in glacial till. It should be noted greater groundwater control may require prior and during excavation in greater depth and this should be mitigated by pumping from sump installed near the excavation.

Surface water runoff into the excavation should be minimized and diverted away from the excavation. Any groundwater seepage or surface run-off entering the excavation should be removed from the excavation with positive drainage to sump and by pumping from sumps.

Should deeper excavations within the native overburden be anticipated as part of this development, it is recommended that a more detailed investigation be carried out with regards to potential groundwater constraints, pumping and permit requirements. A permit to take water (PTTW) is required from Ministry of Environment (MOE), if more than 50,000 litres per day groundwater to be pumped during construction period and processing and issuance of permit by MOE is normally takes 4 to 6 months.

7.4 Pipe Bedding Requirements

It is suggested that the service pipe bedding material consist of at least 150 mm of granular material meeting OPSS gradation requirements for Granular A. A provisional allowance should, however, be made for sub-excavation of any loose or disturbed material encountered at subgrade level. Granular material meeting OPSS specification for Granular B Type II could be used as a sub-bedding material. The use of clear crushed stone as a bedding or sub-bedding materials should not be permitted.

Cover material, from pipe spring level to at least 300 mm above the top of the pipe, should consist of granular material, such as OPSS Granular A.

The sub-bedding, bedding, and cover materials should be compacted in maximum 200 mm thick lifts to at least 98% of its SPMDD using suitable vibratory compaction equipment.

7.5 Trench Backfill

The general backfilling procedures should be carried out in a manner that is compatible with the future use of the area above the service trenches. 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. Any boulders larger than 150 mm in size should not be used as trench backfill. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming at minimum to OPSS Granular B Type I.

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.

8 REUSE OF ON-SITE SOILS

The existing surficial overburden soils consists of fill resting over sand followed by clayey silt. These overburden soil, other than OPSS Granular A, B and C, is considered to be frost susceptible and should not be used as backfill material directly against foundation walls or underneath unheated concrete slabs. However, these materials can still 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 that any waste and debris is removed.

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

9 PAVEMENT DESIGN

It is anticipated that no underground or surface parking is included in the proposed Islam Care Centre building project. The construction of access lanes, if any, will be acceptable over the structural fill material over undisturbed clayey silt once that all debris, organic material, objectionable fill or otherwise deleterious material are removed from the subgrade area. The pavement subgrade material must be inspected and approved by a geotechnical engineer prior to placing any granular subbase/base material.

The following are the recommended pavement structures for light duty access roads/areas proposed as part of this project.

For light vehicle access lanes, the pavement should consist of:

50 mm of hot mix asphaltic concrete (HL3) over

150 mm of OPSS Granular A base over

350 mm of OPSS Granular B Type II subbase

The base and subbase granular materials shall conform to OPSS Form 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 100% SPMDD. Asphaltic concrete shall conform to OPSS Form 1150 and be placed and compacted to at least 97% 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

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 using a fully loaded 18-tonne double axle water truck or similar heavy equipment. In areas where excessive “pumping” of the subgrade is encountered, partial removal of soils in these areas and re-compaction and/or replacement with approved granular soil will be required.

Any materials used as select subgrade should be approved by the geotechnical engineer before placement within the roadway. These materials should be placed in maximum 300 mm thick loose lifts and be compacted to at least 95% of its SPMDD using suitable compaction equipment. Following approval of the preparation of the subgrade, the pavement structure may be placed.

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 months.

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

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

If footings are to be constructed during winter months, 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. 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 locations only. Boundaries between zones presented on the Boreholes 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 report recommendations are applicable only to the project described in the report. Any changes to the project will require a review by LRL, 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.

Brad Johnson, P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
B. W. JOHNSON
100510537
2025.02.13
PROVINCE OF ONTARIO

APPENDIX A

Site and Borehole Location Plan

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

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED NEW ISLAM CARE CENTRE
312 LISGAR STREET
OTTAWA, ONTARIO

DRAWING TITLE

SITE LOCATION

CLIENT
ISLAM CARE CENTRE

DATE
SEPTEMBER 4, 2015

PROJECT
150145

FIGURE 1

ENGINEERING | INGÉNIERIE

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

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED NEW ISLAM CARE CENTRE
312 LISGAR STREET
OTTAWA, ONTARIO

DRAWING TITLE

BOREHOLE LOCATION

CLIENT

ISLAM CARE CENTRE

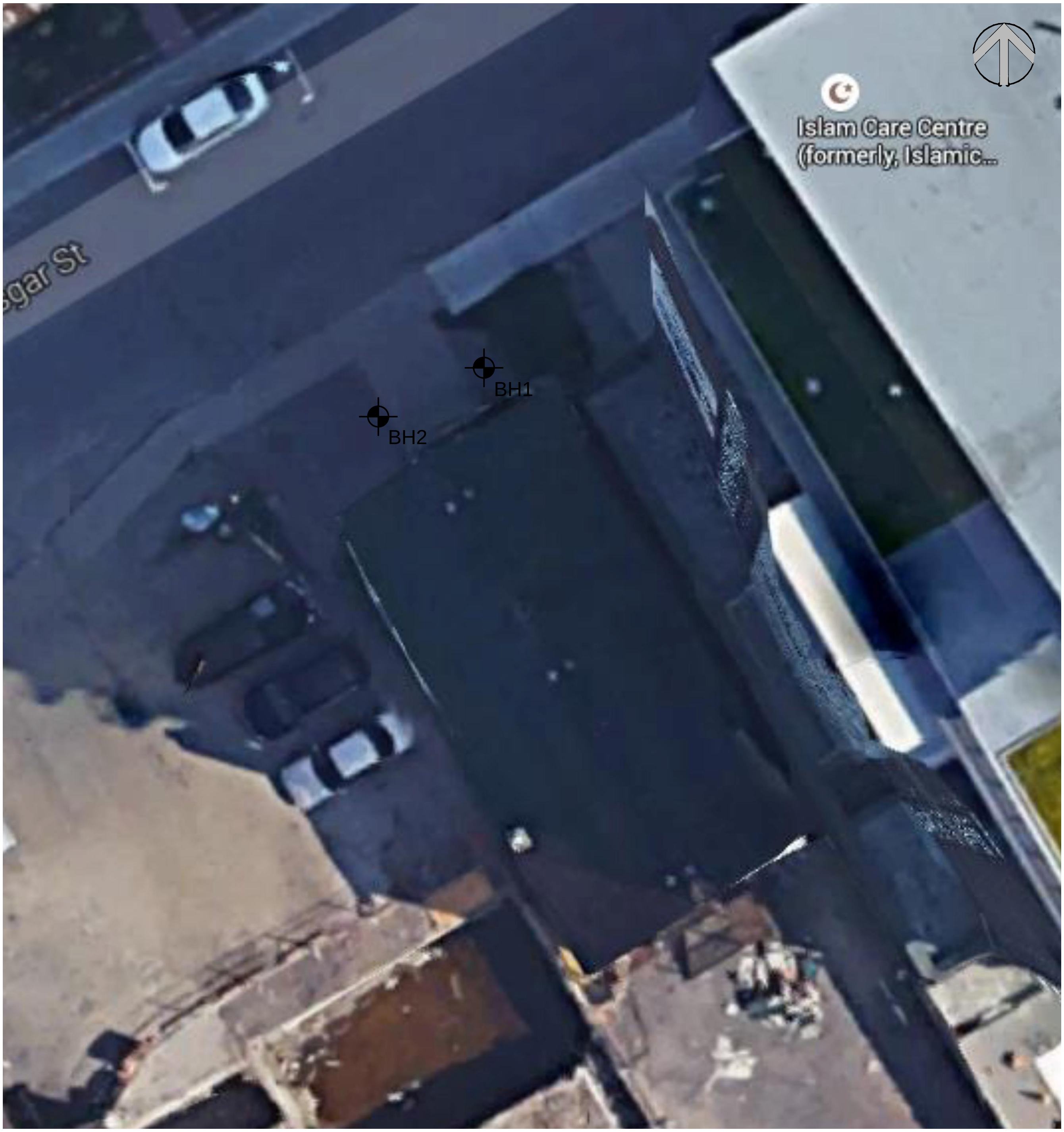
DATE

SEPTEMBER 4, 2015

PROJECT

150145

FIGURE 2



APPENDIX B

Borehole Logs

Borehole Log: BH1

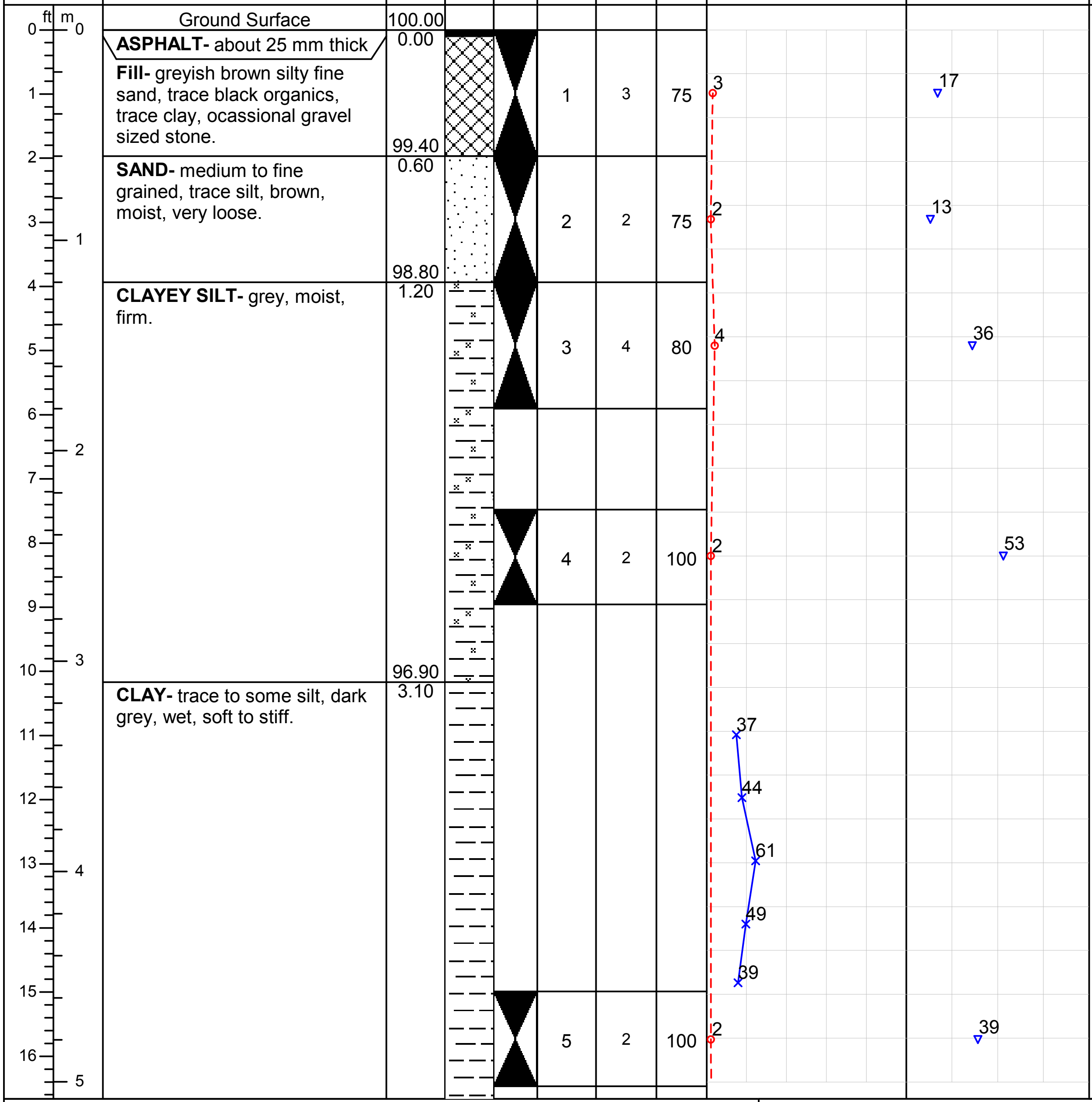
Project No.: 150145
Project: Proposed Highrise
Client: Islam Care Centre
Location: 312 Lisgar Street, Ottawa ON
Date: September 4, 2015
Field Personnel: BJ
Driller: George Downing Estate Drilling
Drilling Equipment: Truck Mount CME 55
Drilling Method: HSA

SUBSURFACE PROFILE

Soil Description	Elev./Depth(m)
Ground Surface	100.00
ASPHALT- about 25 mm thick	0.00
Fill- greyish brown silty fine sand, trace black organics, trace clay, ocassional gravel sized stone.	99.40
SAND- medium to fine grained, trace silt, brown, moist, very loose.	0.60 98.80
CLAYEY SILT- grey, moist, firm.	1.20 96.90
CLAY- trace to some silt, dark grey, wet, soft to stiff.	3.10

SAMPLE DATA

Sample Number	N or RQD	Recovery (%)
1	3	75
2	2	75
3	4	80
4	2	100
5	2	100



Easting: 445480
Northing: 5029501
Site Datum:
Groundsurface Elevation: 0.0
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

Borehole Log: BH1

Project No.: 150145

Project: Proposed Highrise

Client: Islam Care Centre

Location: 312 Lisgar Street, Ottawa ON

Date: September 4, 2015

Field Personnel: BJ

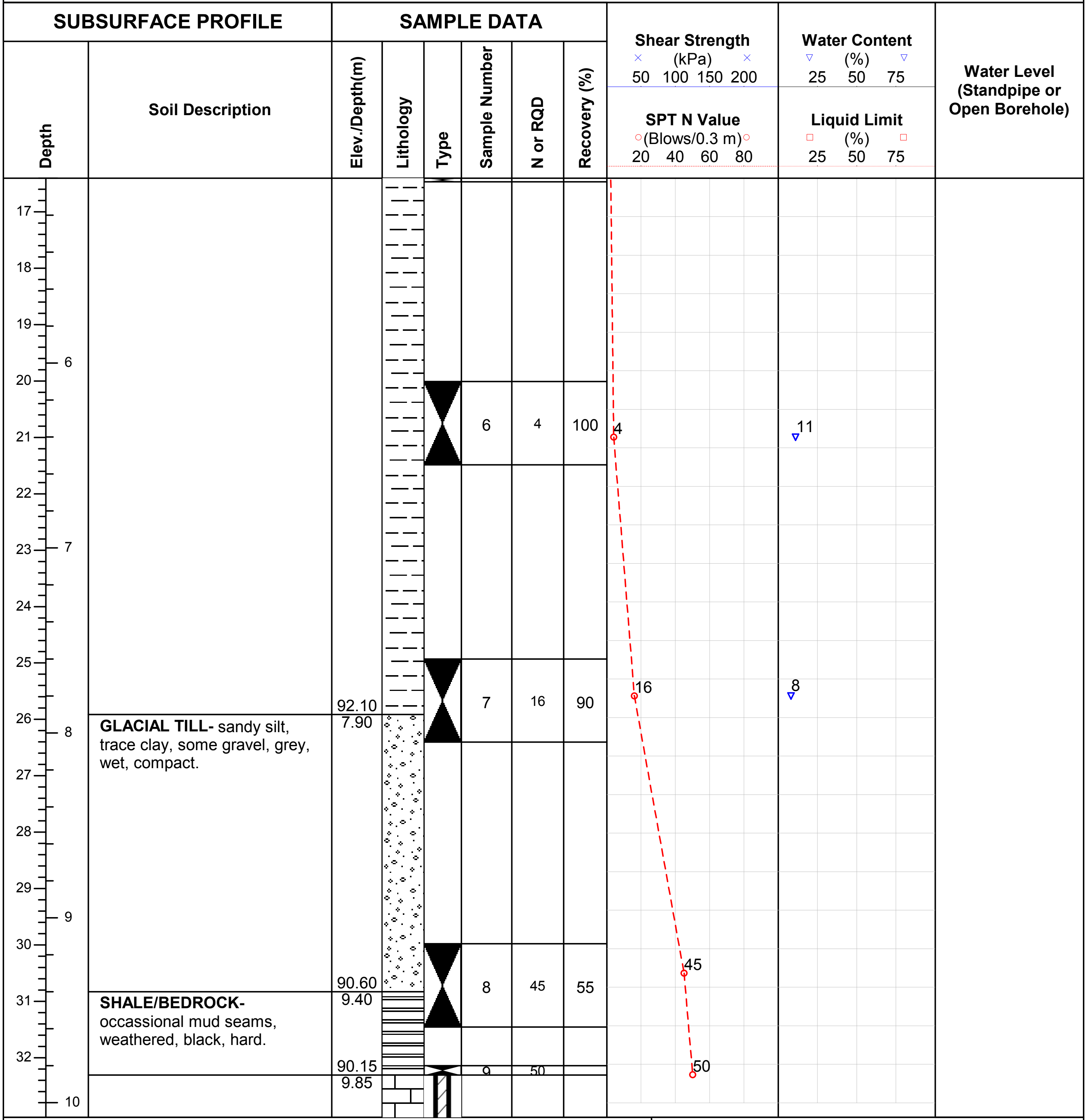
Driller: George Downing Estate Drilling

Drilling Equipment: Truck Mount CME 55

Drilling Method: HSA

SUBSURFACE PROFILE / SAMPLE DATA

Depth	Soil Description	Elev./Depth(m)	Sample Number	N or RQD	Recovery (%)	Water Content (%)
17–25 ft			6	4	100	11
25–26 ft		92.10 7.90	7	16	90	8
26–31 ft	**GLACIAL TILL-** sandy silt, trace clay, some gravel, grey, wet, compact.	90.60 9.40	8	45	55	
31–32 ft	**SHALE/BEDROCK-** occassional mud seams, weathered, black, hard.	90.15 9.85	9	50		



Easting: 445480

Northing: 5029501

Site Datum:

Groundsurface Elevation: 0.0

Top of Riser Elev.: N/A

Hole Diameter: 200 mm

Borehole Log: BH1

Project No.: 150145
Project: Proposed Highrise
Client: Islam Care Centre
Location: 312 Lisgar Street, Ottawa ON
Date: September 4, 2015
Field Personnel: BJ
Driller: George Downing Estate Drilling
Drilling Equipment: Truck Mount CME 55
Drilling Method: HSA

SUBSURFACE PROFILE		SAMPLE DATA						Shear Strength ×(kPa)× 50 100 150 200 SPT N Value ○(Blows/0.3 m)○ 20 40 60 80	Water Content ▽(%)▽ 25 50 75 Liquid Limit □(%)□ 25 50 75	Water Level (Standpipe or Open Borehole)
Depth	**Soil Description**	**Elev./Depth(m)**	**Lithology**	**Type**	**Sample Number**	**N or RQD**	**Recovery (%)**			
33 – 37 ft	**BEDROCK-** limestone fine grained, faintly weathered to fresh. Run 1: 18 Discontinuities. 3 Mechanical breaks.	88.65 11.35			RC-1	61				
	End of Borehole									

Depth scale (ft): 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49
Depth scale (m): 11, 12, 13, 14, 15

Easting: 445480
Northing: 5029501
Site Datum:
Groundsurface Elevation: 0.0
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

NOTES: End of borehole at 11.35 m.
50/4*
*50 blows for 4 cm of sample penetration at first 15 cm.
No water encountered after completion of drilling.

Borehole Log: BH2

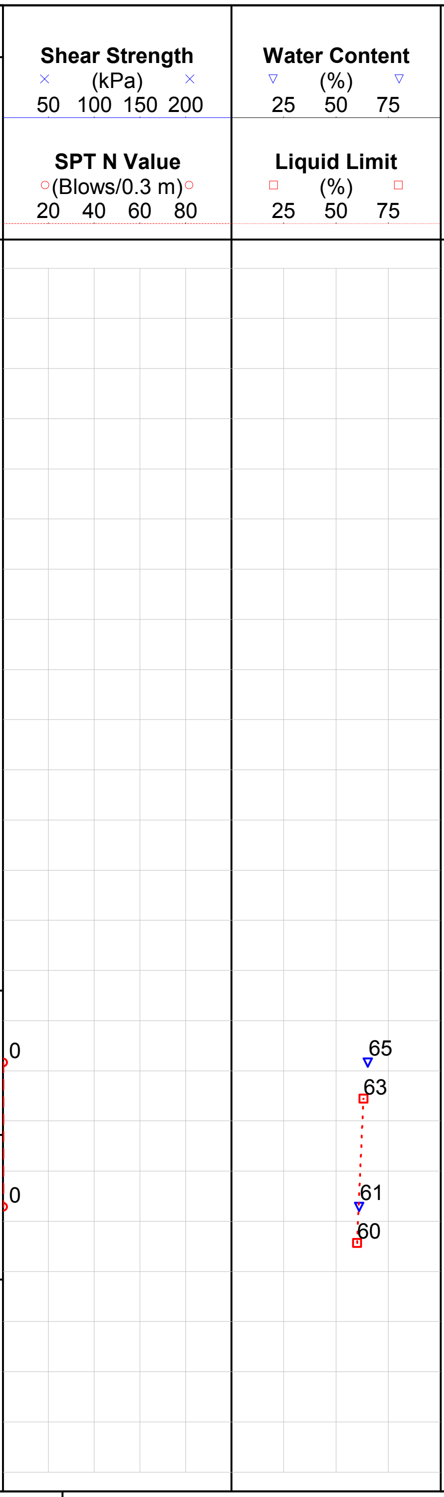
Project No.: 150145
Project: Proposed Highrise
Client: Islam Care Centre
Location: 312 Lisgar Street, Ottawa ON
Date: September 4, 2015
Field Personnel: BJ
Driller: George Downing Estate Drilling
Drilling Equipment: Truck Mount CME 55
Drilling Method: HSA

SUBSURFACE PROFILE

Soil Description	Elev./Depth(m)
Ground Surface	100.00
ASPHALT- about 25 mm thick	0.00
Fill- greyish brown silty fine sand, trace black organics, trace clay, ocassional gravel sized stone.	99.40
SAND- medium to fine grained, trace silt, brown, moist, very loose.	0.60 98.80
CLAYEY SILT- grey, moist, firm.	1.20 96.90
CLAY- trace to some silt, dark grey, wet, soft.	3.10

SAMPLE DATA

Type	Sample Number	N or RQD	Recovery (%)
	1	WH	100
	2	WH	100



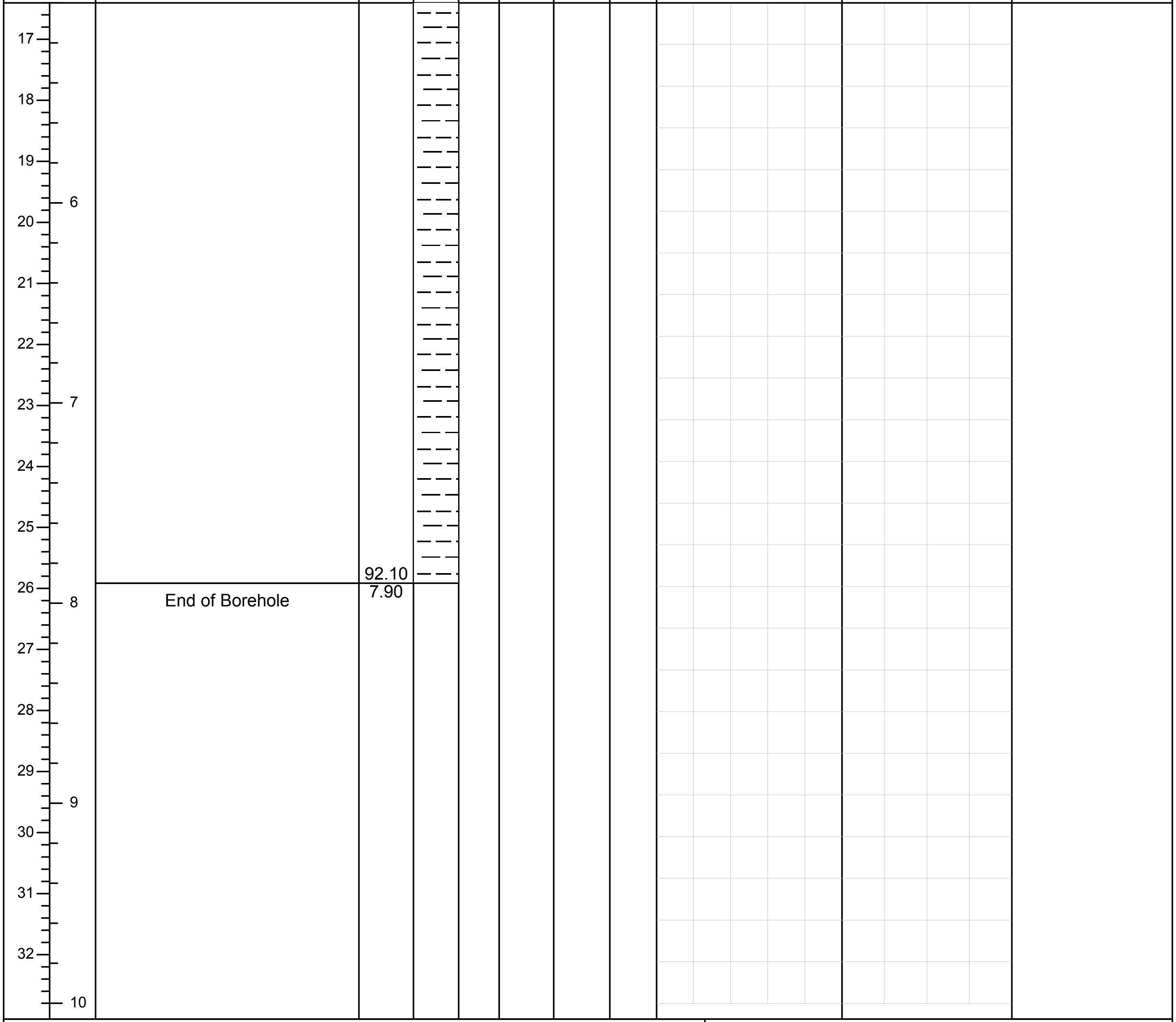
Water Level (Standpipe or Open Borehole)

Easting: 445477
Northing: 5029500
Site Datum:
Groundsurface Elevation: 100.00
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

Borehole Log: BH2

Project No.: 150145
Project: Proposed Highrise
Client: Islam Care Centre
Location: 312 Lisgar Street, Ottawa ON
Date: September 4, 2015
Field Personnel: BJ
Driller: George Downing Estate Drilling
Drilling Equipment: Truck Mount CME 55
Drilling Method: HSA

SUBSURFACE PROFILE		SAMPLE DATA								
Depth	Soil Description	Elev./Depth(m)	Lithology	Type	Sample Number	N or RQD	Recovery (%)	Shear Strength × (kPa) × 50 100 150 200 SPT N Value ○(Blows/0.3 m)○ 20 40 60 80	Water Content ▽ (%) ▽ 25 50 75 Liquid Limit □ (%) □ 25 50 75	Water Level (Standpipe or Open Borehole)
		92.10 7.90								
	End of Borehole									



Easting: 445477
Northing: 5029500
Site Datum:
Groundsurface Elevation: 100.00
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

NOTES: End of borehole at 7.9 m.
Auger refusal at 7.9 m bgs at possible shale bedrock/massive boulder.
Samples have been verified from auger cuttings.
No water encountered after completion of drilling.

APPENDIX C

Symbols and Terms Used in Borehole Logs

Symbols and Terms Used on Borehole and Test Pit Logs

The following explains the data presented in the 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 Test. See Section 2c for more details. 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"
Very loose	0 – 4
Loose	4 – 10
Compact or medium	10 - 30
Dense	30 - 50
Very dense	over - 50

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

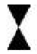
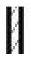
Consistency Cohesive Soils	Undrained Shear Strength (Cu) (kPa)
Very soft	under 10
Soft	10 - 25
Medium or firm	25 - 50
Stiff	50 - 100
Very stiff	100 - 200
Hard	over - 200

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. Blows (N) or RQD

This column indicates the Standard Penetration Number (N) as per ASTM D-1586. This is used to determine the state of compactness of the soil sampled. It corresponds to the number of blows required to drive 300 mm of the split spoon sampler using a 622 kg*m/s^2 hammer falling freely from a height of 760 mm. For a 600 mm long split spoon, the blow counts are recorded for every 150 mm. The "N" index is obtained by adding the number of blows from the 2nd and 3rd count. Technical refusal indicates a number of blows greater than 50.

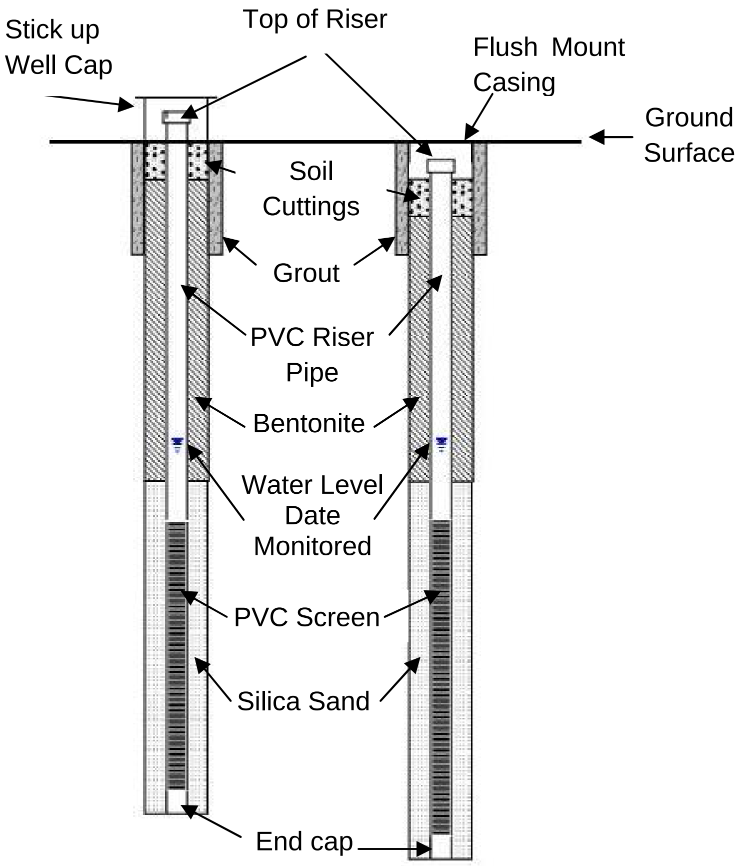
In the case of rock, this column presents the Rock Quality Designation (RQD). The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 10 cm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given below.

Rock Quality Designation (RQD) (%)	Description of Rock Quality
0 –25	very poor
25 – 50	poor
50 – 75	fair
75 – 90	good
90 – 100	excellent

e. Recovery (%)

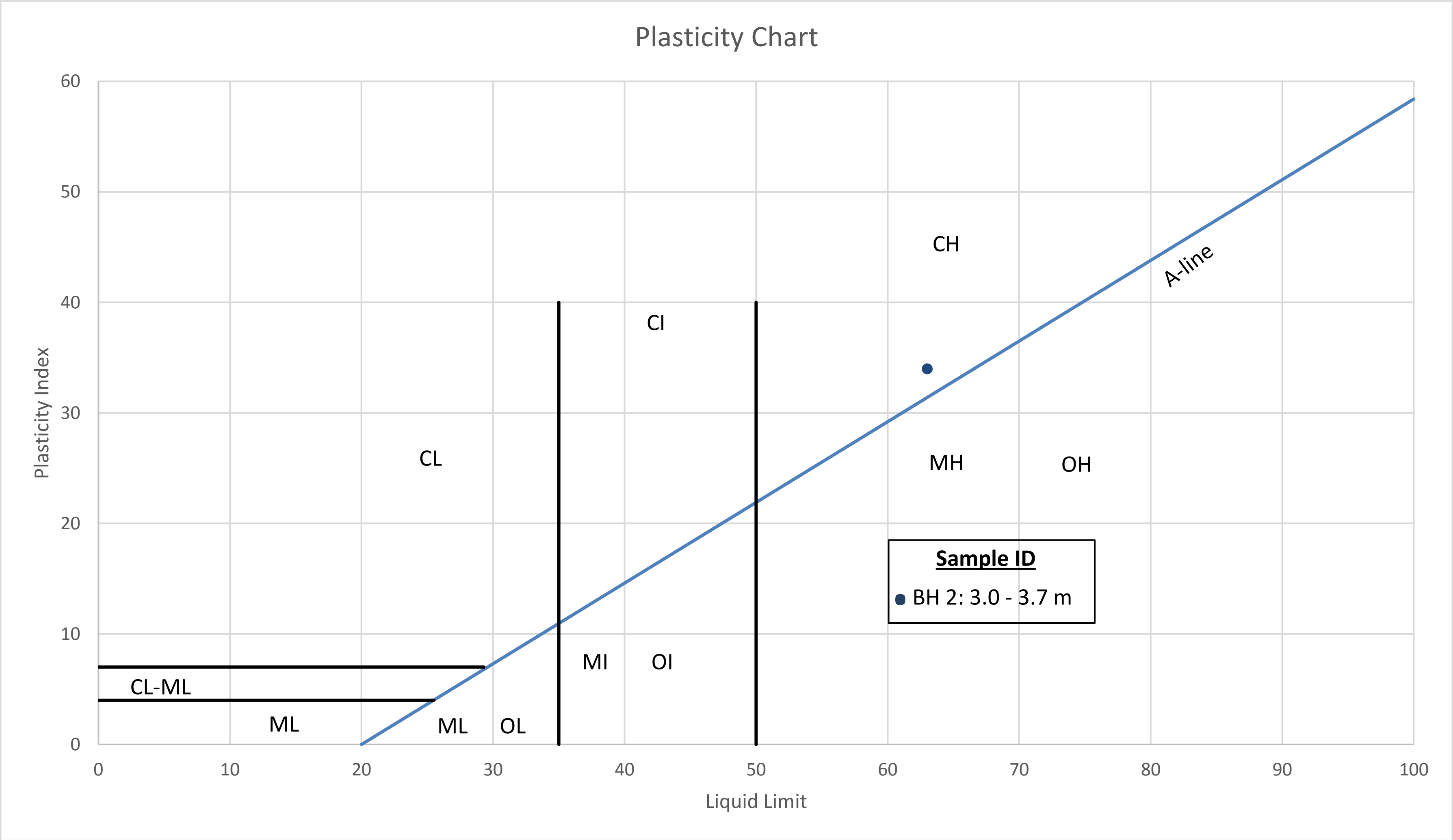
For soil samples this is the percentage of the recovered sample obtained versus the length sampled. In the case of rock, the percentage is the length of rock core recovered compared to the length of the drill run.

3. General Monitoring Well Data



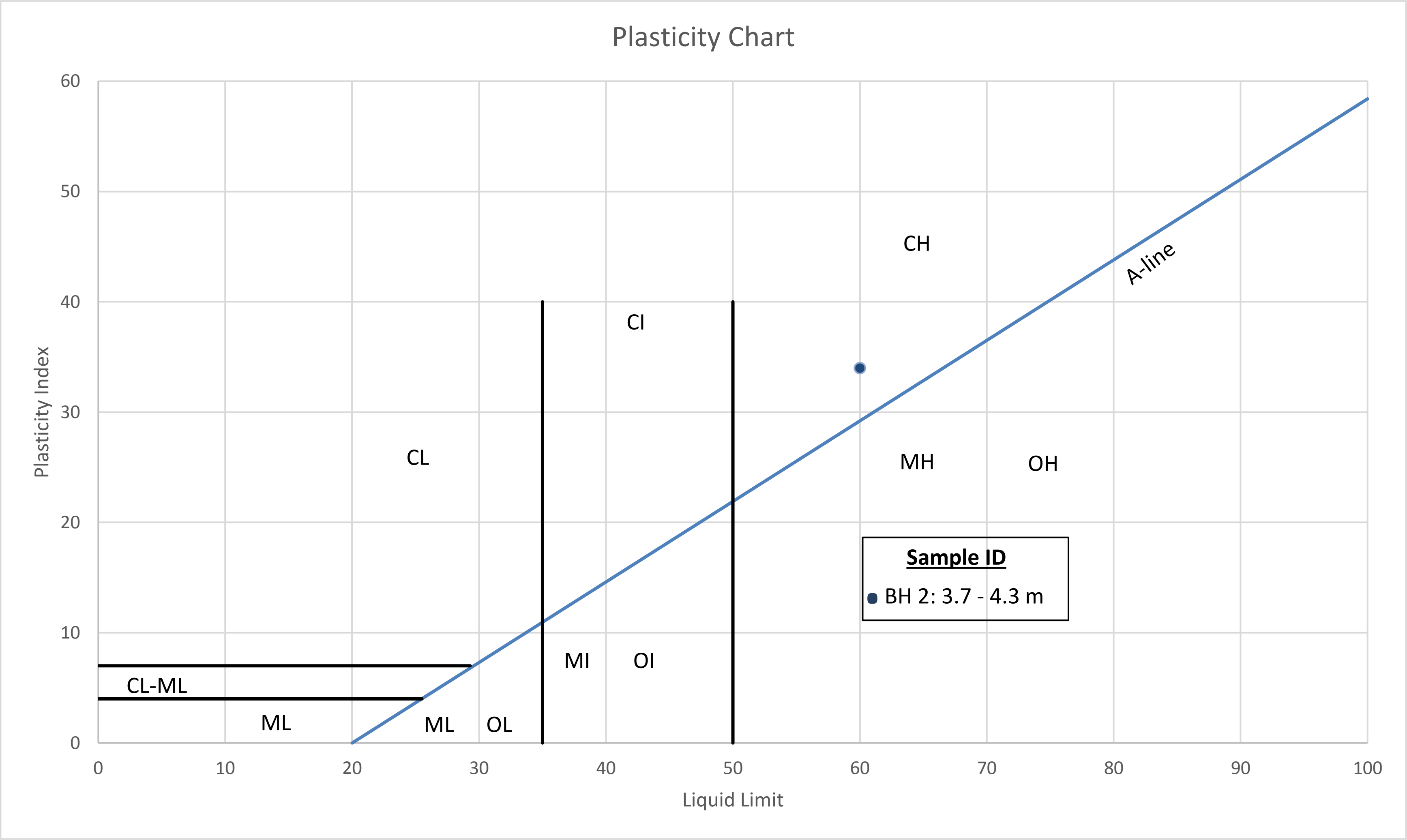
APPENDIX D

Laboratory Test Results and Certificate of Analyses



PLASTICITY CHART

Client: Islam Care Centre
LRL File Number: 150145
Date: September 9, 2015



PLASTICITY CHART

Client: Islam Care Centre

LRL File Number: 150145

Date: September 9, 2015

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www.paracellabs.com

Subcontracted Analysis

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

Tel: (613) 842-3434
Fax: (613) 446-1427

Paracel Report No **1536350**
Client Project(s): **150145**
Client PO:
Reference:
CoC Number: **105401**

Order Date: 04-Sep-15
Report Date: 8-Sep-15

Sample(s) from this project were subcontracted for the listed parameters. A copy of the subcontractor’s report is attached

Paracel ID	**Client ID**	**Analysis**
1536350-01	BH1 3.1m-3.7m (Clay Sample)	Redox potential, soil

PARACEL LABORATORIES LTD
ATTN: Dale Robertson
300-2319 St. Laurent Blvd.
Ottawa ON K1G 4J8

Date Received: 04-SEP-15
Report Date: 08-SEP-15 07:09 (MT)
Version: FINAL

Client Phone: 613-731-9577

Certificate of Analysis

Lab Work Order #: L1668716
Project P.O. #: NOT SUBMITTED
Job Reference: 1536350
C of C Numbers:
Legal Site Desc:

Austin Paterson

Austin Paterson
Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters		Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L1668716-1	BH1 3.1M-3.7M (CLAY SAMPLE)							
Sampled By:	DANIELLE CHARLEBOIS on 31-AUG-15							
Matrix:	SOIL							
Redox Potential		158		-1000	mV		05-SEP-15	R3260622

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
REDOX-POTENTIAL-WT	Soil	Redox Potential	APHA 2580

This analysis is carried out in accordance with the procedure described in the "APHA" method 2580 "Oxidation-Reduction Potential" 2012. Samples are extracted at a fixed ratio with DI water. Results are reported as observed oxidation-reduction potential of the platinum metal-reference electrode employed, in mV.

** ALS test methods may incorporate modifications from specified reference methods to improve performance.

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location

Chain of Custody Numbers:

GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.
mg/kg - milligrams per kilogram based on dry weight of sample
mg/kg wwt - milligrams per kilogram based on wet weight of sample
mg/kg lwt - milligrams per kilogram based on lipid-adjusted weight
mg/L - unit of concentration based on volume, parts per million.
< - Less than.
D.L. - The reporting limit.
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

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Subcontract Order

SENDING LABORATORY:

Paracel Laboratories Ltd.
300-2319 St. Laurent Blvd.
Ottawa, ON K1G 4J8
Phone: 613-731-9577

Fax: 613-731-9064

RECEIVING LABORATORY:

ALS Laboratory Group (Ottawa)
7-190 Colonnade Rd
Ottawa, ON K2E7J5
Phone: (613) 225-8279

Fax: (613) 225-2801

INVOICE TO:

Paracel Laboratories Ltd.
300-2319 St. Laurent Blvd.
Ottawa, ON K1G 4J8
Phone: 613-731-9577

Fax: 613-731-9064

Date Requested: **04-Sep-15**
Project Number: **1536350**
Submitted By: **Danielle Charlebois**
Email: **dcharlebois@paracellabs.com**

Required Regulation	—
Turnaround Time	RUSH

Due: Sept. 8th. (1 day)

Sample ID	**Matrix**	**Analyses Requested:**	**Sampled**	**Comments**
BH1 3.1m-3.7m (Clay Sample)	Soil	Redox potential, soil	31-Aug-15	

L1668716
MM 5/15 11:18 12am
Sep-05A

L1668716-COFC

Please email all results to mfoto@paracellabs.com, dbloom@paracellabs.com, drobertson@paracellabs.com

D.Charlebois Sept 4/15 11:33
Released By Date / Time

Andrew Cameron Sept/4/15
Received By Date

Temperature prior to Shipping: 7.2

6.5

MM Sep 8/15 10:15 8.1

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Certificate of Analysis

LRL Associates Ltd.

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

Client PO:
Project: 150145
Custody: 105401

Report Date: 8-Sep-2015
Order Date: 4-Sep-2015

Order #: 1536350

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

Paracel ID	Client ID
1536350-01	BH1 3.1m-3.7m (Clay Sample)

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis
Client: **LRL Associates Ltd.**
Client PO:

Report Date: 08-Sep-2015
Order Date:4-Sep-2015
Project Description: 150145

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	8-Sep-15	8-Sep-15
Conductivity	MOE E3138 - probe @25 °C, water ext	8-Sep-15	8-Sep-15
pH	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	8-Sep-15	8-Sep-15
Resistivity	EPA 120.1 - probe, water extraction	8-Sep-15	8-Sep-15
Solids, %	Gravimetric, calculation	8-Sep-15	8-Sep-15

Certificate of Analysis

Report Date: 08-Sep-2015

Client: **LRL Associates Ltd.**

Order Date:4-Sep-2015

Client PO:

Project Description: 150145

Client ID:		BH1 3.1m-3.7m (Clay Sample)	-	-	-
Sample Date:		31-Aug-15	-	-	-
Sample ID:		1536350-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	62.2	-	-	-
General Inorganics					
Conductivity	5 uS/cm	1420	-	-	-
pH	0.05 pH Units	8.20	-	-	-
Resistivity	0.10 Ohm.m	7.03	-	-	-
Anions					
Chloride	5 ug/g dry	757	-	-	-
Sulphate	5 ug/g dry	83	-	-	-

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									
Conductivity	ND	5	uS/cm						
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	18.6	5	ug/g dry	18.4			1.3	20	
Sulphate	139	5	ug/g dry	150			7.8	20	
General Inorganics									
Conductivity	1450	5	uS/cm	1420			2.0	6.2	
pH	7.66	0.05	pH Units	7.62			0.5	10	
Resistivity	6.89	0.10	Ohm.m	7.03			2.0	20	
Physical Characteristics									
% Solids	93.8	0.1	% by Wt.	92.5			1.4	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	10.3		mg/L	1.8	84.8	78-113			
Sulphate	23.8		mg/L	15.0	87.7	78-111			

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.

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.

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Certificate of Analysis

LRL Associates Ltd.

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

Client PO:
Project: 150145
Custody: 105401

Report Date: 8-Sep-2015
Order Date: 4-Sep-2015

Order #: 1536351

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

Paracel ID	Client ID
1536351-01	BH1 10.5 (Rock Sample)

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis
Client: **LRL Associates Ltd.**
Client PO:

Report Date: 08-Sep-2015
Order Date:4-Sep-2015
Project Description: 150145

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	8-Sep-15	8-Sep-15
Conductivity	MOE E3138 - probe @25 °C, water ext	8-Sep-15	8-Sep-15
Solids, %	Gravimetric, calculation	4-Sep-15	4-Sep-15

Certificate of Analysis

Report Date: 08-Sep-2015

Client: **LRL Associates Ltd.**

Order Date:4-Sep-2015

Client PO:

Project Description: 150145

	Client ID:	BH1 10.5 (Rock Sample)	-	-	-
	Sample Date:	31-Aug-15	-	-	-
	Sample ID:	1536351-01	-	-	-
	MDL/Units	Other	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	100	-	-	-
General Inorganics					
Conductivity	5 uS/cm	259	-	-	-
Anions					
Sulphate	5 ug/g dry	150	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Sulphate	ND	5	ug/g						
General Inorganics									
Conductivity	ND	5	uS/cm						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Sulphate	139	5	ug/g dry	150			7.8	20	
General Inorganics									
Conductivity	1450	5	uS/cm	1420			2.0	6.2	
Physical Characteristics									
% Solids	88.4	0.1	% by Wt.	88.7			0.3	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Sulphate	23.8		mg/L	15.0	87.7	78-111			

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

Please note that results are based on a surface wash of the submitted rock sample.

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.

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

Photograph of Core Sample

CORE PHOTOGRAPH

Our File Ref.: 150145
Client: Islam Care Centre
Project: Proposed Highrise
Site Location: 312 Lisgar Street, Ottawa ON

<table>
<tr><td>Photograph No. 1</td><td rowspan="3">
</td></tr>
<tr><td>Date: 9/4/2015</td></tr>
<tr><td>BH - 1
Depth: 9.85 to 11.35m</td></tr>
</table>