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Proposed Multi-Storey Buildings
South Keys Redevelopment - Phase 1
2210 Bank Street
Ottawa, Ontario

Prepared For

Calloway REIT (South Keys) Inc./
Canadian Property Holdings (South Keys) Inc.

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Report PG5242-1
Revision 2

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Calloway REIT (South Keys) Inc./ Canadian Property Holdings (South Keys) Inc. to conduct a geotechnical investigation for the proposed multi-storey buildings to be constructed as part of the South Keys Redevelopment - Phase 1 at 2210 Bank Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- ❏ Determine the subsoil and groundwater conditions at this site by means of boreholes.
- ❏ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Development

Based on the current concept drawings, it is understood that Phase 1 of the proposed development will consist of 2 high-rise buildings located at the north and south portions of the Phase 1 site. The high-rise buildings are to be Iniked by 4 and 6 storey parking and residential podium structures throughout the central and eastern portions of the Phase 1 site, respectively.

The proposed development is understood to have 1 common below-grade level with a lowest level slab at approximate geodetic elevation 85.7 m. The ground floor is anticipated to consists of commercial and residential amenity spaces, whereas the upper levels are anticipated to consist of residential units. At-grade paved access lanes, parking areas and landscaped areas are also anticipated as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on March 27 and March 30, 2020. At that time, 3 boreholes (BH 1-20 through BH 3-20) were advanced to a maximum depth of 15.8 m below the existing ground surface. A previous geotechnical investigation by others also included 4 boreholes (BH 94-33, BH 94-35, BH 94-36 and BH 94-37) advanced at, or in the vicinity of, the subject site to a maximum depth of 27.4 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the test holes are shown on Drawing PG5242-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a truck-mounted auger drill rig operated by a two-person crew. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted at each borehole in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at the current borehole locations. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Groundwater monitoring wells were installed in BH 1-20 and BH 2-20 to permit monitoring of the groundwater levels and to perform hydraulic conductivity testing subsequent to the completion of the sampling program. A flexible polyethylene standpipe was installed within BH 3-20 to measure the stabilized groundwater levels subsequent to completion of the sampling program.

Hydraulic Conductivity Testing

Hydraulic conductivity testing had not yet been completed at the time of issuing this report, however will be updated once the testing has been complete. Preliminary hydraulic conductivities based on our experience and available published values for the soils encountered at the subject site are are discussed further in Section 4.3.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5242-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and are discussed in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The Phase 1 site is partially occupied by an existing commercial plaza consisting of several one storey commercial units. The existing buildings within the footprint of the subject site are connected to a cinema building beyond southern boundary of the subject site. The remainder of the site is generally occupied by asphalt paved access lanes and parking areas with landscaped margins.

The site is bordered by a Transitway to the west, a cinema structure followed by parking areas to the south, and asphalt paved parking areas to the east and an existing commercial grocery store to the north. The existing ground surface across the site slopes downward gradually from south to north from approximate geodetic elevation of 90 to 87 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of asphalt underlain by fill extending to an approximate depth of 1.5 to 1.8 m below the existing ground surface. The fill was generally observed to consist of a compact brown silty sand with crushed stone.

A silty clay deposit was encountered underlying the fill. This deposit was observed to consist of a very stiff to stiff, brown silty clay, becoming a firm to stiff, grey silty clay to clayey silt below depths of 3.5 to 4.5 m below the existing ground surface.

Underlying the silty clay deposit below approximate depths of 9 to 12 m, interbedded layers of compact to dense sandy silt, silty sand, sand and/or firm to stiff silty clay were encountered. Within BH 2-20, a glacial till deposit was encountered at an approximate depth of 15 m, consisting of a clayey silt to silty clay with sand, some gravel, and occasional cobbles.

Practical refusal of the DCPTs were encountered at depths ranging from 25.1 to 29.4 m below the existing ground surface.

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of limestone with interbedded shale of the Verulam formation with a drift thickness of 25 to 50 m.

4.3 Groundwater

Groundwater levels measured in the standpipes are summarized in Table 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-20	88.84	2.94	85.90	April 17, 2020
BH 2-20	87.05	1.28	85.77	April 17, 2020
BH 3-20	89.60	Blocked	-	April 17, 2020

Note:
- The ground surface elevations at the borehole locations are referenced to a TBM, consisting of the top of grate of the existing catch basin located in the parking lot of the site with geodetic elevation = 89.65 m.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 3 to 4 m below ground surface within the low permeability silty clay layer. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Hydraulic Conductivity Testing

Following the completion of the slug testing, the test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent with zero-storage assumption, and a screen length significantly greater than the monitoring well diameter. The assumption regarding aquifer storage is considered to be appropriate for groundwater flow through the overburden aquifer. The assumption regarding screen length and well diameter is considered to be met based on a screen length of 3 m and a diameter of 0.03 m.

While the idealized assumptions regarding aquifer extent, homogeneity, and isotropy are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site.

Hvorslev analysis is based on the line of best fit through the field data (hydraulic head recovery vs. time), plotted on a semi-logarithmic scale. In cases where the initial hydraulic head displacement is known with relative certainty, such as in this case where a physical slug has been introduced, the line of best fit is considered to pass through the origin.

Based on the above test methods, the monitoring wells screened in the silty sand to sandy silt displayed hydraulic conductivity values ranging between **1.92×10^{-4} and 3.82×10^{-5} m/sec**. The values measured within the monitoring wells are consistent with similar material Paterson has encountered on other sites and typical published values for silty sand to sandy silt. These values typically range from 1×10^{-4} to 1×10^{-6} m/sec. The range in hydraulic conductivity values is due to the variability of the sand encountered. The results of the hydraulic conductivity testing are presented in Appendix 1.

It should be noted that testing was not completed in the monitoring wells screened in the grey silty clay given the low hydraulic conductivity of the material and the time constraint of the testing. However, based on our experience and available published values, the hydraulic conductivity for the grey silty clay is anticipated to range between 1×10^{-9} to 1×10^{-12} m/s.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. Based on the subsurface conditions encountered in the test holes and the anticipated building loads, it is recommended that foundation support for the proposed high-rise buildings consist of a deep foundation, such as end-bearing piles, which extends to the bedrock surface. It is also recommended that the proposed mid-rise podium structures be founded on conventional spread footings or a raft foundation bearing on an undisturbed, stiff silty clay bearing surface.

It is further recommended that a construction joint be provided to allow for differential settlement between the proposed high-rise and mid-rise structures, which are anticipated to be supported on deep and shallow foundations, respectively. If differential settlements of up to 25 mm are not tolerable between the high-rise and mid-rise structures, the proposed mid-rise podium structures should also be supported on piles.

Due to the presence of the silty clay deposit, a permissible grade raise restriction will be required for the proposed grading.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

The existing fill, where free of organics and deleterious materials, may be considered to be left in place as subgrade for floor slab and pavement construction only. If considered suitable by the geotechnical consultant at the time of construction, the fill layer should be proof-rolled by a suitably sized vibratory roller making several passes and approved by Paterson personnel, as noted in Section 5.5. Areas with poor performing fill should be removed and reinstated with a compacted engineered fill as detailed below.

Protection of Subgrade (Raft Foundation)

Should the proposed podium structures be supported on raft foundations, the subgrade material will most likely consist of a silty clay deposit. In this case, it is recommended that a minimum 100 mm thick lean concrete mud slab be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation. The main purpose of the mudslab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay to potential disturbance due to drying.æ

Compacted Granular Fill Working Platform (Pile Foundation)

For the proposed high-rise buildings to be supported on a driven pile foundation, the use of heavy equipment would be required to install the piles (i.e. pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance.

A typical working platform could consist of 600 mm of OPSS Granular B, Type II crushed stone which is placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in lifts not exceeding 300 mm in thickness.

Once the piles have been driven and cut off, the working platform can be re-graded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and re-compacted to act as the substrate for further fill placement for the basement slab.

Fill Placement

Fill used for grading beneath the proposed buildings should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Spread Footing Foundations - Mid-Rise Podium Structures

Foundations for the proposed mid-rise parking and residential podium structures, and other light-loaded ancillary structures, may consist of strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, stiff silty clay bearing surface using bearing resistance values at Serviceability Limit State (SLS) of **120 kPa** and factored bearing resistance values at Ultimate Limit States (ULS) of **180 kPa**.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS. The bearing resistance values at SLS for conventional style footings will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a silty clay or silty sand bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as the soil.

Raft Foundations - Mid-Rise Podium Structures

Based on the expected loads from the proposed mid-rise parking and residential podium structures, a raft foundation bearing on the undisturbed, stiff silty clay may be required to provide foundation support for these buildings. For one below-grade level, it is anticipated that the underside of raft would be located at approximate geodetic elevation 83 to 84 m.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The contact pressure provided considers the stress relief associated with the soil removal required for the proposed basement level.

For one below-grade level, a bearing resistance value at SLS (contact pressure) of **135 kPa** will be considered acceptable for a raft supported on the undisturbed, stiff silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **200 kPa**. For this case, the modulus of subgrade reaction was calculated to be **6 MPa/m** for a contact pressure of **140 kPa.**

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Based on the following assumptions for the raft foundation, the the proposed mid-rise parking and residential podium structures can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

End Bearing Pile Foundation - High-Rise Buildings

A deep foundation system driven to refusal in the bedrock is recommended for foundation support of the proposed high-rise buildings. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance values at SLS and ULS are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance	Final Set (blows/12 mm)	Transferred Hammer Energy (kJ)
		Factored at ULS (kN)		
245	9	1090	10	28.5
245	11	1260	10	34.2
245	13	1500	10	40.7

Re-striking of all piles, at least once, will also be required after at least 48 hours have elapsed since initial driving. A full-time field review program should be conducted during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical).

The minimum recommended centre-to-centre pile spacing is 2.5 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.Å

Foundation Lateral Load Resistance

Lateral loads on the foundations can be resisted using passive resistance on the sides of the foundations. For Limit States Design, the resistance factor to be applied to the ultimate lateral resistance, including passive pressure, is 0.50. The total lateral resistance will be comprised of the individual contributions from up to several material layers, as follows.

Geotechnical parameters for the native silty clay and for typical backfill materials, compacted to 98% of SPMDD in 300 mm lift thicknesses, are provided in Table 3,along with the associated earth pressure coefficients for horizontal resistance calculated for footings under lateral loads or deadman anchors. Friction factors between concrete and the various subgrade materials are also provided in Table 3, where normal loads allow them to be used.

Where granular soils and/or granular backfill materials are present, the passive pressure can be calculated using a triangular distribution equal to $K_P \cdot \gamma \cdot H$ where:

K_P = factored passive earth pressure coefficient of the applicable retained soil
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the equivalent wall or footing side (m)

Note that for cases where the depth to the top of the structure pushing against the soil does not exceed 50% of the depth to the base of the structure, the effective value of H in the above noted relationship will be the overall depth to the base of the structure. There will also be “edge effects” where the effective width of soil providing the resistance can be increased by 50% of the effective depth on each side of the pushing structural component.

Note that where the foundation extends below the groundwater level, the effective unit weight should be utilized for the saturated portion of the soil or fill.

Should additional passive resistance be require, the horizontal component of the axial resistance of battered piles (up to 1H:3V inclination), or anchors can be used in the building foundation design.

Foundation Uplift Resistance

Uplift forces on the proposed foundations can be resisted using the dead weight of the concrete foundations, the weight of the materials overlying the foundations, and the submerged weight of the piles. Unit weights of materials are provided in Table 3.

For soil above the groundwater level, calculate using the “drained” unit weight and below groundwater level use the “effective” unit weight. Backfilled excavations in low permeability soils can be expected to fill with water and the use of the effective unit weights would be prudent if drainage of the anchor footings is not provided.

As noted, the piles will generally be located below the groundwater level, so the submerged, or effective, weight of the pile or caisson will be available to contribute to the uplift resistance, if required. Considering that this is a reliable uplift resistance, and is really counteracting a dead load, it is our opinion that a resistance factor of 0.9 is applicable for the ULS weight component.

Should the pile uplift resistance capacities be insufficient for the foundation uplift loads, rock anchors should be utilized. These are discussed further in Section 5.3.

A sieve analysis and standard Proctor test should be completed on each of the fill materials proposed to obtain an accurate soil density to be expected, so the applicable unit weights can be estimated.

Table 3 - Geotechnical Parameters for Lateral Resistance Design

Material Description	Unit Weight (kN/m^3)		Internal Friction Angle (°) φ'	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
OPSS Granular A (Crushed Stone)	22.0	13.7	38	0.60	0.22	0.36	8.8
OPSS Granular B, Type II (Well-Graded Sand-Gravel)	21.5	13.4	36	0.55	0.26	0.41	7.5
In Situ Silty Sand	19.0	11.0	34	0.45	0.28	0.43	3.5
In Situ Silty Clay	17.0	10.0	33	0.40	0.30	0.45	3.4

Notes:
❏ Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
❏ The earth pressure coefficients provided are for horizontal backfill profile.
❏ Passive pressure coefficients incorporate wall friction of 0.5 φ'.

Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **1.5 m** is recommended for grading at the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Rock Anchor Design

Overview of Anchor Features

Where the foundation uplift resistance, as discussed above, is insufficient for the proposed buildings, rock anchors can be utilized to provide additional foundation uplift resistance.

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each individual anchor.

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

Centre-to-centre spacing between anchors should be at least four times the anchor hole diameter and greater than 1/5 of the total anchor length (minimum of 1.2 m) to lower the group influence effects. Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout does not flow from one hole to an adjacent empty one.

The anchor 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 top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic.

Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed buildings, the rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength(UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of limestone and shale ranges between about 50 and 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.821 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 4 below.

Table 4 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone or Shale Hoek and Brown parameters	65 m=0.821 and s=0.00293
Unconfined compressive strength - Limestone	50 MPa
Unit weight - Submerged Bedrock	15.2 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 5. The factored tensile resistance values given in Table 5 are based on a single anchor with no group influence effects. A detailed analysis, including potential group influence effects, could be provided once loading for the proposed building is determined.

Table 5 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2	0.8	2.8	450
	2.6	1	3.6	600
	3.2	1.3	4.5	750
	4.5	2	6.5	1000
125	1.6	1	2.6	600
	2	1.2	3.2	750
	2.6	1.4	4	1000
	3.2	1.8	5	1250

Other considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout.

5.5 Design for Earthquakes

The site class for seismic site response can be taken as **Class D**. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.6 Basement Slab and Slab-on-Grade Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the native soil or existing suitable fill subgrade approved by the geotechnical consultant at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for floor slab construction.

Where the existing fill is encountered at the subgrade level for the proposed floor slabs, it is recommended that the slab subgrade surface be proof-rolled **under dry conditions and above freezing temperatures** by an adequately sized vibratory roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by Paterson at the time of construction. In poor performing areas, the existing fill should be removed and replaced with an approved engineered fill. Care should be taken not to disturb adequate bearing soils below the subgrade level during site preparation activities.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab for this purpose.

For basement slabs that consist of storage or other non-parking uses, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. Parking areas throughout the lowest basement levels should be designed as per the recommended pavement structure noted in Section 5.8.

For slab-on-grade construction, it is recommended that the upper 300 mm of sub-slab fill consists of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

A sub-slab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided under the lowest level floor slab. The spacing of the sub-slab drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels, if any. This is discussed further in Section 6.1.

5.7 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.8 Pavement Structure

Car only parking areas, heavy truck parking areas and access lanes are anticipated at this site. The proposed flexible pavement structures are presented in Tables 6 and 7.

Table 6 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill.	

Table 7 - Recommended Pavement Structure
Access Lanes, Ramp and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Rigid Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 8 below.

Table 8 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on maintaining the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed structures. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structures. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or building sump systems (if applicable).

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a composite drainage blanket, such as Miradrain G100N or Delta Drain 6000.

Sub-Slab Drainage

Sub-slab drainage is recommended to control water infiltration below the lowest level basement floor slab, such as the P1 parking level. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximately 6 m centres along the perimeter of the basement foundation wall and at the footing to foundation wall interface. The pipe should be surrounded with a geosock and a minimum of 150 mm of 19 mm clear crushed stone on all of its sides.

The underfloor drainage layout should be detailed by the geotechnical consultant once the structures basement layout has been completed by the architect and structural engineer. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or an equivalent combination of soil cover and foundation insulation, should be provided.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion due to rain and surface water runoff if shoring is not anticipated to be implemented. This can be accomplished by covering the surface of the slope with tarps secured at the top and bottom of the excavation and approved by Paterson personnel at the time of construction. It recommended to secure tarps with pins and/or stakes embedded a minimum of 450 mm below the ground surface at the top and bottom of the slope. Tarps are not recommended to be secured by objects such as cobbles, construction debris or other material that may be placed upon the ground surface.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations, where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary system could consist of soldier piles and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated with the following parameters.

Table 9 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight (γ'), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

The pipe bedding for sewer and water pipes placed on a relatively dry, undisturbed subgrade surface should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the firm grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

Generally, it should be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay material will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

To reduce long-term lowering of the groundwater at this site, clay seals should be provided within the service trenches excavated through the silty clay deposit. The seals should be at least 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. The seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches excavated through the silty clay deposit.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay material encountered throughout the subject site and inferred depths of the proposed foundations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of the Environment, Conservation, and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum of 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Long-Term Groundwater Control

Our recommendations for the proposed building's foundation drainage system are presented in Subsection 6.1. Based on our review, the proposed buildings will be founded above the long-term groundwater table. It is therefore expected that infiltration will be very low to negligible (i.e.- less than 50,000 L/day) with peak periods noted after rain and snow-melt events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed.

Impacts on Neighbouring Properties

Since the proposed development will be founded above the long term groundwater level, no significant groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. Therefore, long-term dewatering of the site is not anticipated and should have no adverse effects to the surrounding buildings or structures. The short term dewatering of surface water during the excavation program will be managed by the excavation contractor, as discussed above.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

The subsoil conditions at this site mostly consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters, tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a very agressive corrosive environment.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

☐ A review of the site grading plan(s) from a geotechnical perspective, once available.

☐ A review of architectural and structural drawings to ensure adequate frost protection is provided to the subsoil.

☐ Review the Contractor's design of the temporary shoring system, if applicble.

☐ Review of waterproofing details for elevator shafts and building sump pits.

☐ Review and inspection of all foundation drainage systems.

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Sampling and testing of the concrete and fill materials used.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Calloway REIT (South Keys) Inc./ Canadian Property Holdings (South Keys) Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Sept.13, 2021
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

Scott Dennis

Scott S. Dennis, P.Eng.

Report Distribution

☐ Calloway REIT (South Keys) Inc./Canadian Property Holdings (South Keys) Inc. (e-mail copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY ANALYSIS

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Development - 2210 Bank St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

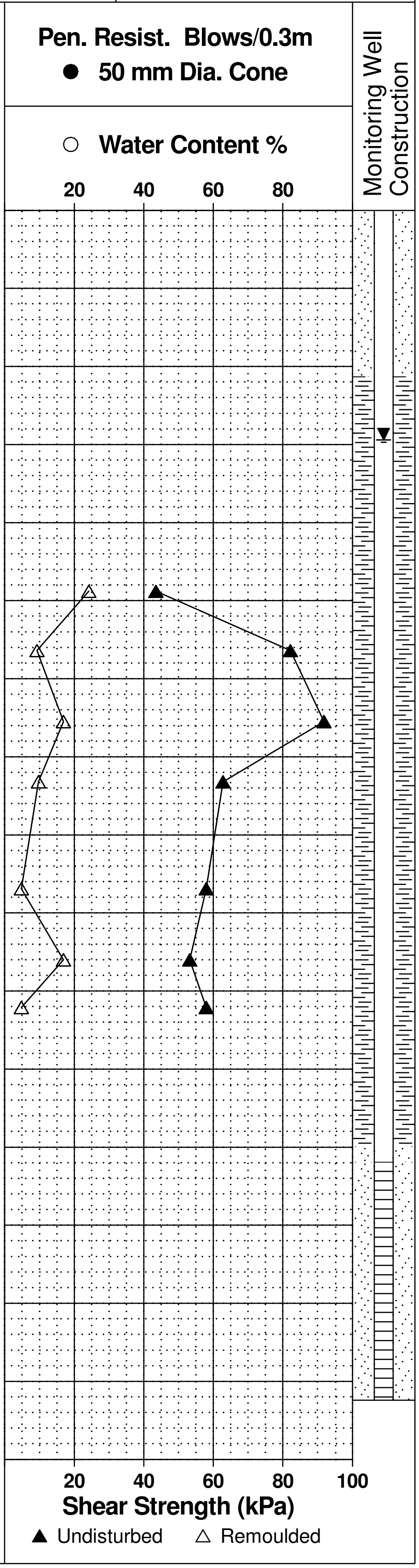
BORINGS BY CME-55 Low Clearance Drill

DATE March 27, 2020

FILE NO. PG5242

HOLE NO. BH 1-20

SOIL DESCRIPTION	STRATA PLOT	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	88.84
Asphaltic concrete 0.08		AU	1				
FILL: Brown silty sand with crushed stone 0.51							
FILL: Brown silty sand 1.45		SS	2	21	8	1	87.84
FILL: Brown silty clay, some sand, trace organics 1.83		SS	3	50	6	2	86.84
Brown **SILTY CLAY,** some sand		SS	4	67	5		
3.66		SS	5	12	4	3	85.84
Firm to stiff, grey **SILTY CLAY**		SS	6	100	2	4	84.84
						5	83.84
						6	82.84
						7	81.84
		SS	7	100	W	8	80.84
						9	79.84
						10	78.84
10.67							
Compact, grey **SANDY SILT to SILTY SAND,** some clay, trace gravel 11.58		SS	8	38	17	11	77.84
Compact, grey **SAND** with gravel		SS	9	4	28	12	76.84
						13	75.84
14.78		SS	10	33	25	14	74.84
Dense, grey **SILTY SAND**						15	73.84
15.85		SS	11		40	16	72.84



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Development - 2210 Bank St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

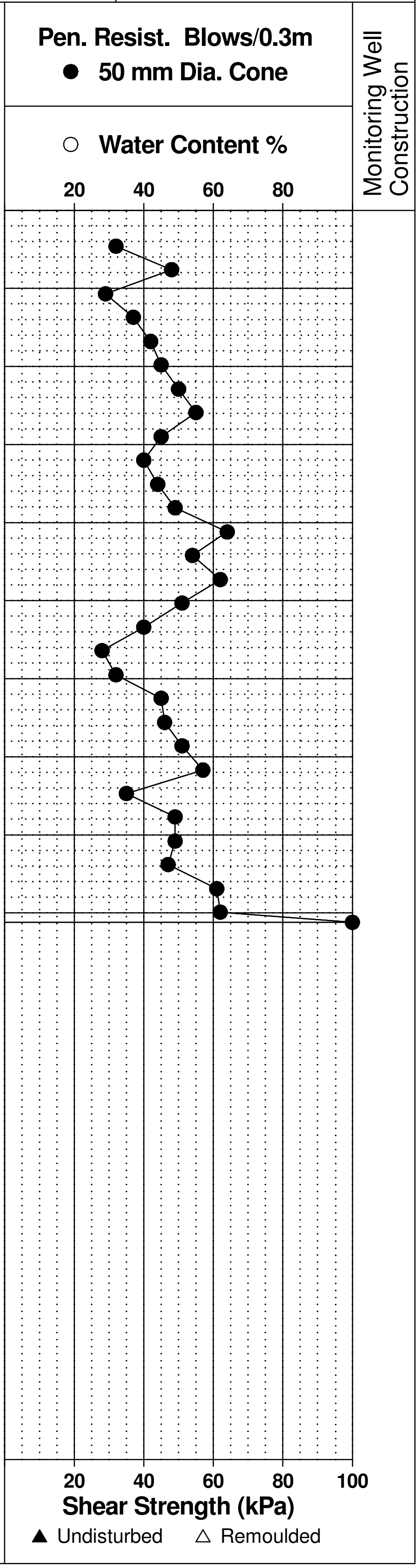
BORINGS BY CME-55 Low Clearance Drill

DATE March 27, 2020

FILE NO. PG5242

HOLE NO. BH 1-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
Dynamic Cone Penetration Test commenced at 15.85m depth.						16	72.84
						17	71.84
						18	70.84
						19	69.84
						20	68.84
						21	67.84
						22	66.84
						23	65.84
						24	64.84
25.12						25	63.84
End of Borehole Practical DCPT refusal at 25.12m depth. (GWL @ 2.94m - April 17, 2020)							

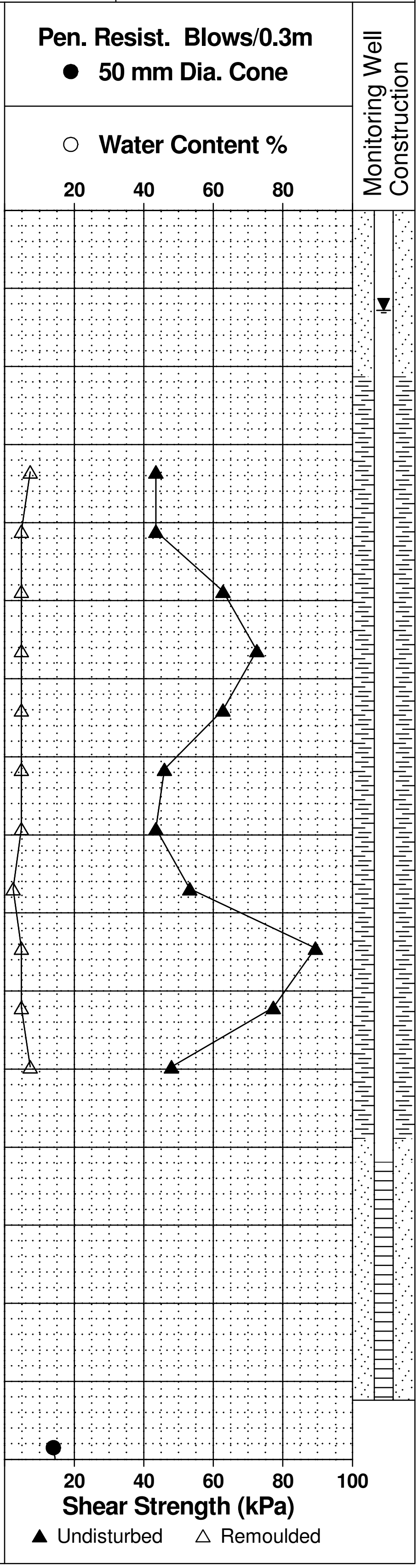


SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Development - 2210 Bank St.
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE March 30, 2020
FILE NO. PG5242
HOLE NO. BH 2-20

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	87.05
Asphaltic concrete 0.10	AU	1				
FILL: Brown silty sand with crushed stone, some clay 1.52	SS	2	54	14	1	86.05
	SS	3	92	4	2	85.05
	SS	4	100	2		
Firm to stiff, brown **SILTY CLAY**					3	84.05
- grey by 3.8m depth					4	83.05
					5	82.05
					6	81.05
					7	80.05
					8	79.05
					9	78.05
					10	77.05
- some sand by 10.7m depth 11.58					11	76.05
					12	75.05
	SS	5	25	26		
Comapct, grey **SANDY SILT to SILTY SAND,** some gravel, trace clay	SS	6	4	21	13	74.05
	SS	7	17	12	14	73.05
15.09	SS	8	0	17	15	72.05
GLACIAL TILL: Grey clayey silt to silty clay with sand, some gravel, 15.85	SS	9	38	18	16	71.05



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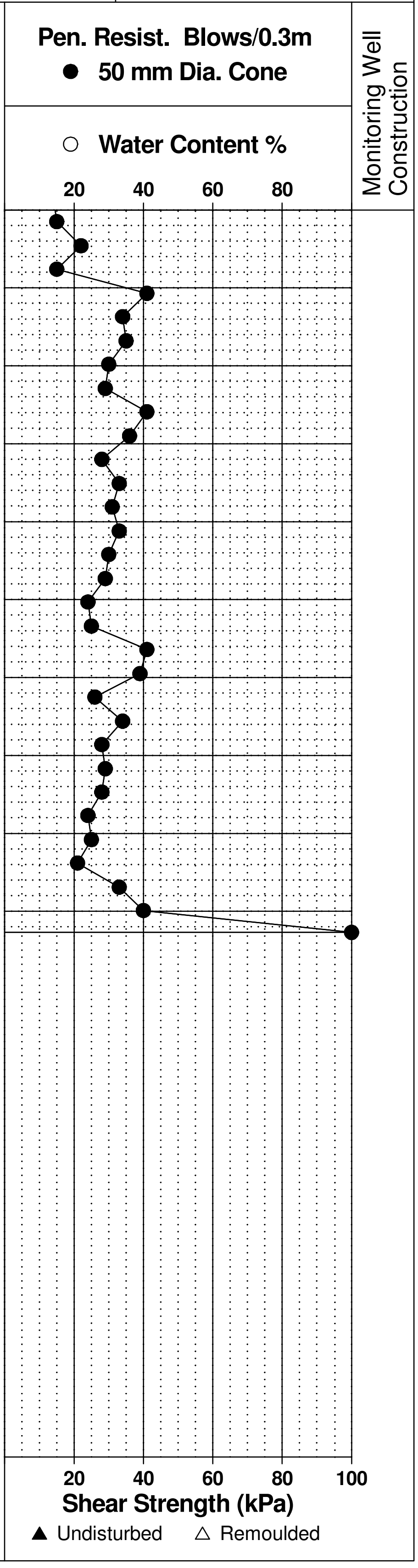
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Development - 2210 Bank St.
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE March 30, 2020
FILE NO. PG5242
HOLE NO. BH 2-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
occasional cobbles						16	71.05
Dynamic Cone Penetration Test commenced at 15.85m depth.						17	70.05
						18	69.05
						19	68.05
						20	67.05
						21	66.05
						22	65.05
						23	64.05
						24	63.05
25.27						25	62.05
End of Borehole Practical DCPT refusal at 25.27m depth. (GWL @ 1.28m - April 17, 2020)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Development - 2210 Bank St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE March 27, 2020

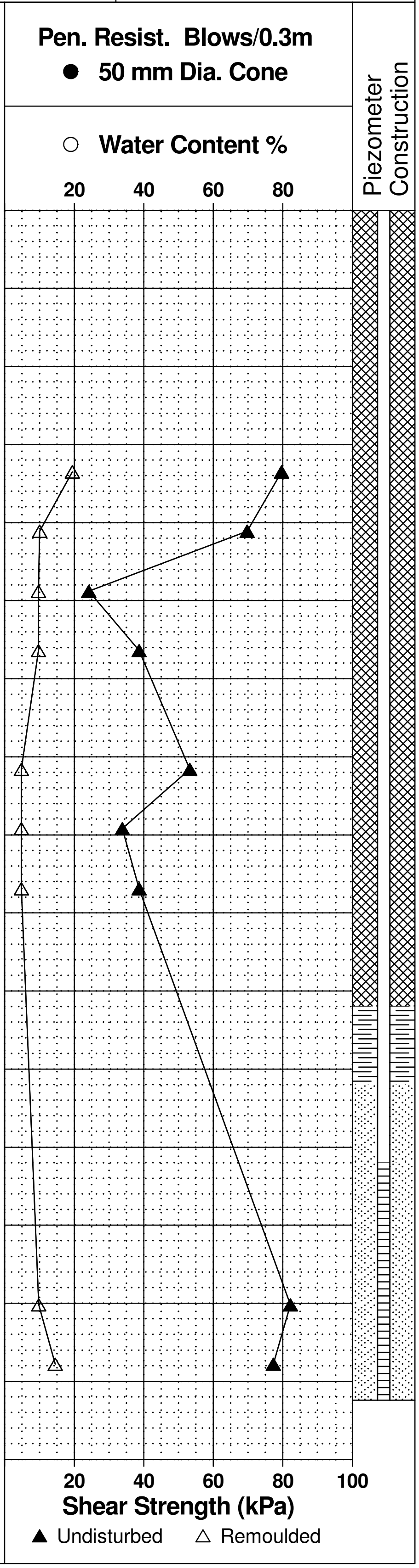
FILE NO. PG5242

HOLE NO. BH 3-20

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone - crushed stone decreasing with depth	1.52
Very stiff to stiff, brown **SILTY CLAY** - firm to stiff and grey by 4.6m depth	9.14
Grey **SANDY SILT to SILTY SAND,** some clay	9.90
Firm, grey **CLAYEY SILT**	11.28
Stiff, grey **SILTY CLAY**	15.24
Inferred compact, grey **SILTY SAND** with gravel	15.85

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	23
SS	3	79	13
SS	4	62	8
SS	5	100	W
SS	6	92	W
SS	7	100	2
SS	8	54	4
SS	9	29	3
SS	10	100	2
SS	11	0	17

DEPTH (m)	ELEV. (m)
0	89.60
1	88.60
2	87.60
3	86.60
4	85.60
5	84.60
6	83.60
7	82.60
8	81.60
9	80.60
10	79.60
11	78.60
12	77.60
13	76.60
14	75.60
15	74.60
16	73.60



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Development - 2210 Bank St.
Ottawa, Ontario

DATUM Geodetic

REMARKS

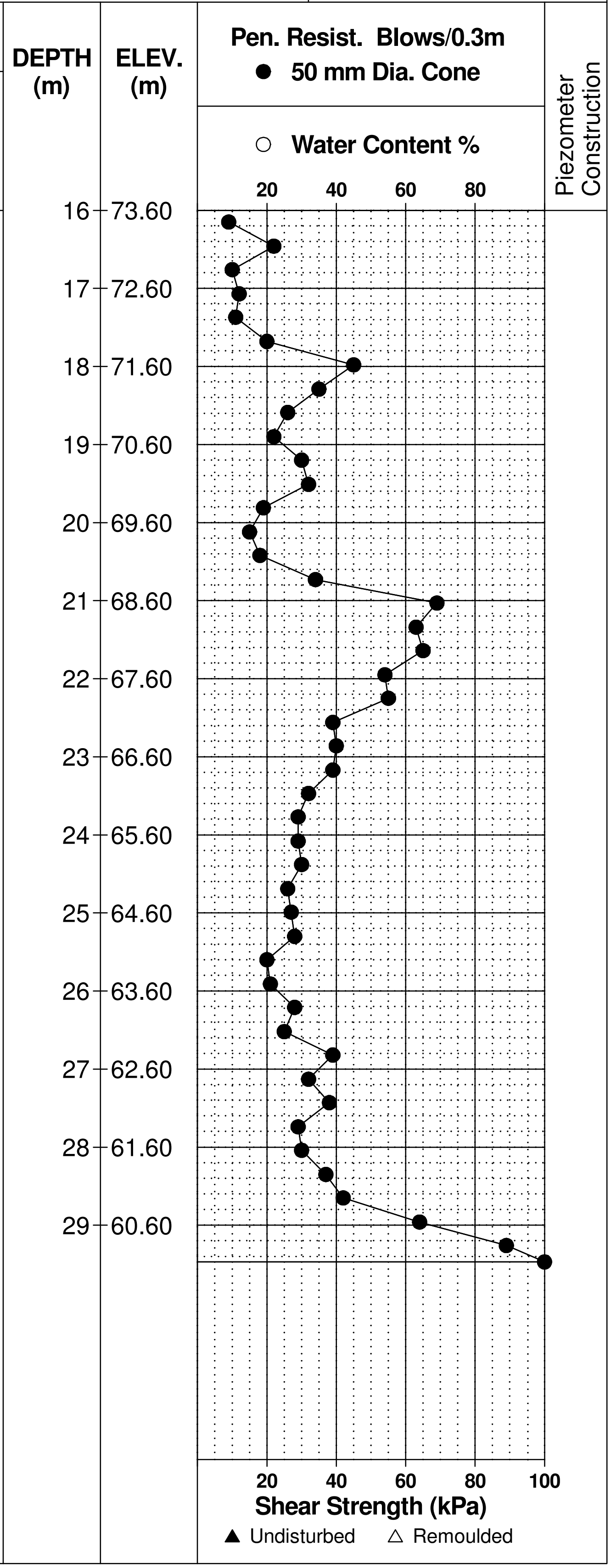
BORINGS BY CME-55 Low Clearance Drill

DATE March 27, 2020

FILE NO. PG5242

HOLE NO. BH 3-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Dynamic Cone Penetration Test commenced at 15.85m depth.					
29.47					
End of Borehole Practical DCPT refusal at 29.47m depth. (Piezometer blocked - April 17, 2020)					



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

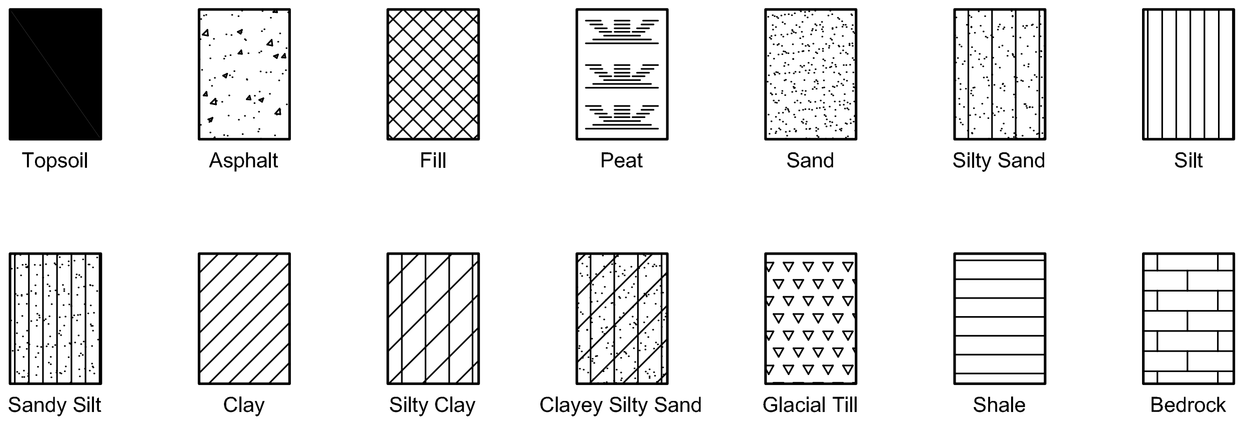
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

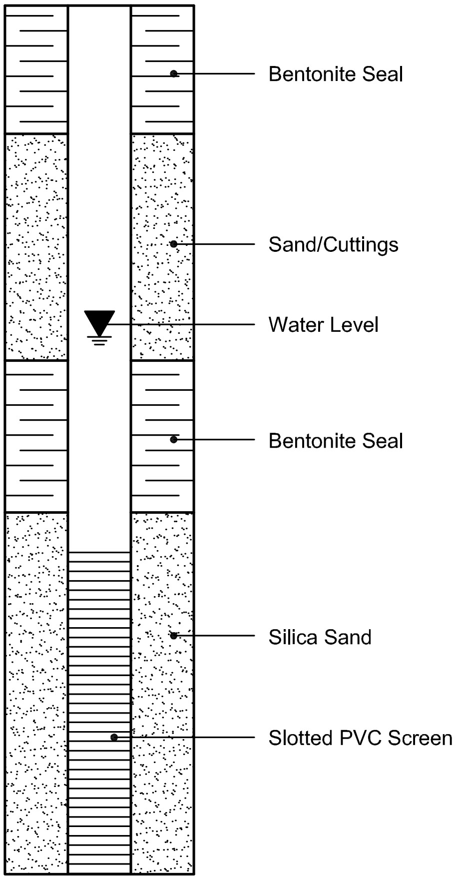
SYMBOLS AND TERMS (continued)

STRATA PLOT

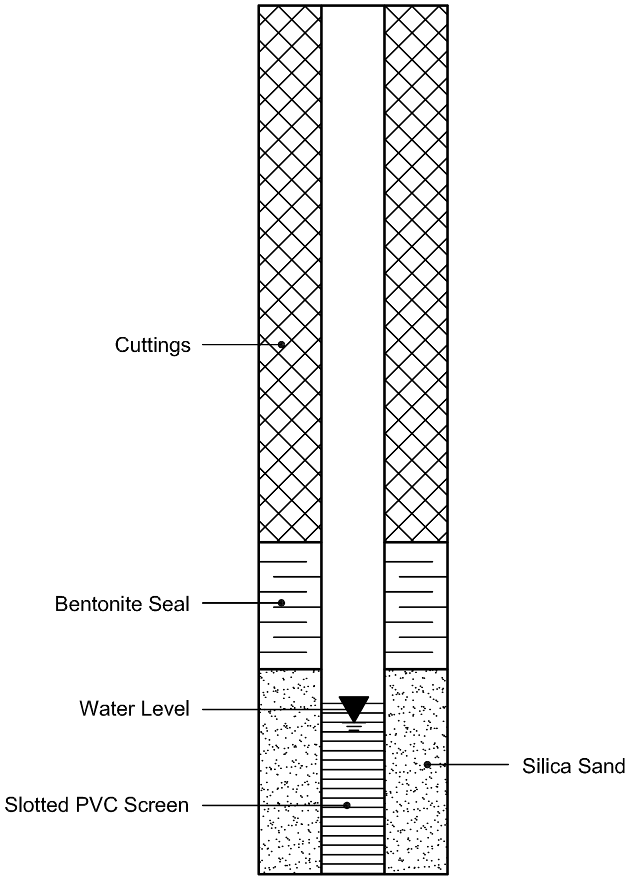


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



JACQUES, WHITFORD LIMITED

BOREHOLE RECORD

94-33

CLIENT The Future Group

LOCATION South Keys Commercial Development, Ottawa, Ontario

DATES: BORING 94-07-26

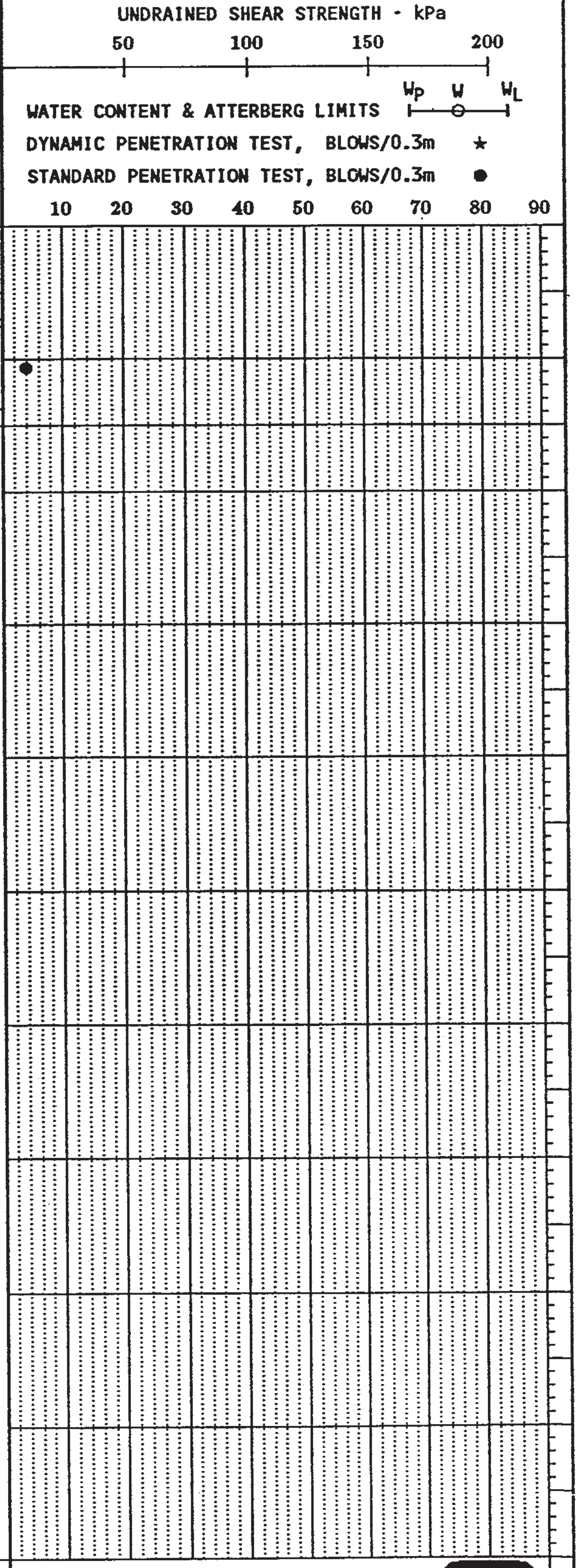
WATER LEVEL

BOREHOLE No. 94-33

PROJECT No. 10519

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR RQD
	86.08							
0	86.0	50 mm Brown to grey silty clayey TOPSOIL	▨					
					BS	1		
1		Stiff to firm, brown to grey SILTY CLAY	▨		SS	2	580	4
	84.6	End of Borehole						



∇ ▼ Proposed Pipe Invert

■ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

BOREHOLE RECORD

94-35

CLIENT The Future Group

LOCATION South Keys Commercial Development, Ottawa, Ontario

DATES: BORING 94-07-20

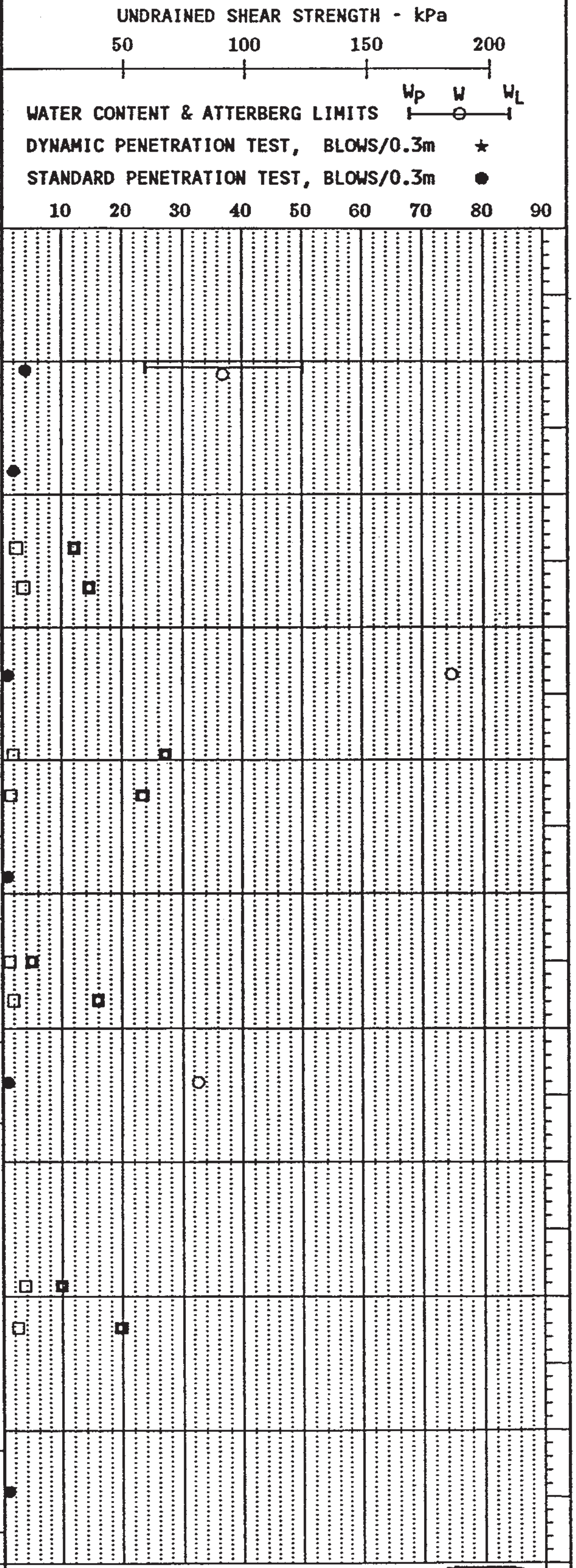
WATER LEVEL 94-08-05

BOREHOLE No. 94-35

PROJECT No. 10519

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY mm	N-VALUE OR RQD
0	86.89 86.8	50 mm Brownish grey silty clayey TOPSOIL			BS	1		
1		Stiff to firm, brown to grey SILTY CLAY, occasional sand seam		▽	SS	2	270	4
					SS	3	490	2
2	84.8	Firm, grey SILTY CLAY, occasional sand seam						
3					SS	4	610	1
5					SS	5	600	1
6					SS	6	610	1
9					SS	7	610	1
10								



▽ ▼ Proposed Pipe Invert

Continued Next Page

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

JACQUES, WHITFORD
LIMITED

BOREHOLE RECORD

94-35

CLIENT **The Future Group**

LOCATION **South Keys Commercial Development, Ottawa, Ontario**

DATES: BORING **94-07-20** WATER LEVEL **94-08-05**

BOREHOLE No. **94-35**

PROJECT No. **10519**

DATUM **Geodetic**

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR RQD
10	76.3	Firm to soft, grey SILTY CLAY, occasional sand seam	▨					
11		Compact, brown to grey medium SAND, trace silt						
12	74.8							
13		Inferred brown to grey SAND						
14								
15								
16								
17								
18								
19								
20								

UNDRAINED SHEAR STRENGTH - kPa

50 100 150 200

WATER CONTENT & ATTERBERG LIMITS Wp W WL

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

▽
▼

◀ Proposed Pipe Invert

Continued Next Page

◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

JACQUES, WHITFORD
LIMITED

BOREHOLE RECORD

94-35

CLIENT The Future Group

LOCATION South Keys Commercial Development, Ottawa, Ontario

DATES: BORING 94-07-20 WATER LEVEL 94-08-05

BOREHOLE No. 94-35

PROJECT No. 10519

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR RQD
20		Inferred brown to grey SAND						
21								
22								
23								
24								
25								
26								
27	59.4							
28		Auger refusal at 27.4 m End of Borehole Standpipe installed						
29								
30								

UNDRAINED SHEAR STRENGTH - kPa
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

∇

▼

◄ Proposed Pipe Invert

▣ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

JACQUES, WHITFORD LIMITED

BOREHOLE RECORD

94-36

CLIENT The Future Group

LOCATION South Keys Commercial Development, Ottawa, Ontario

DATES: BORING 94-07-26

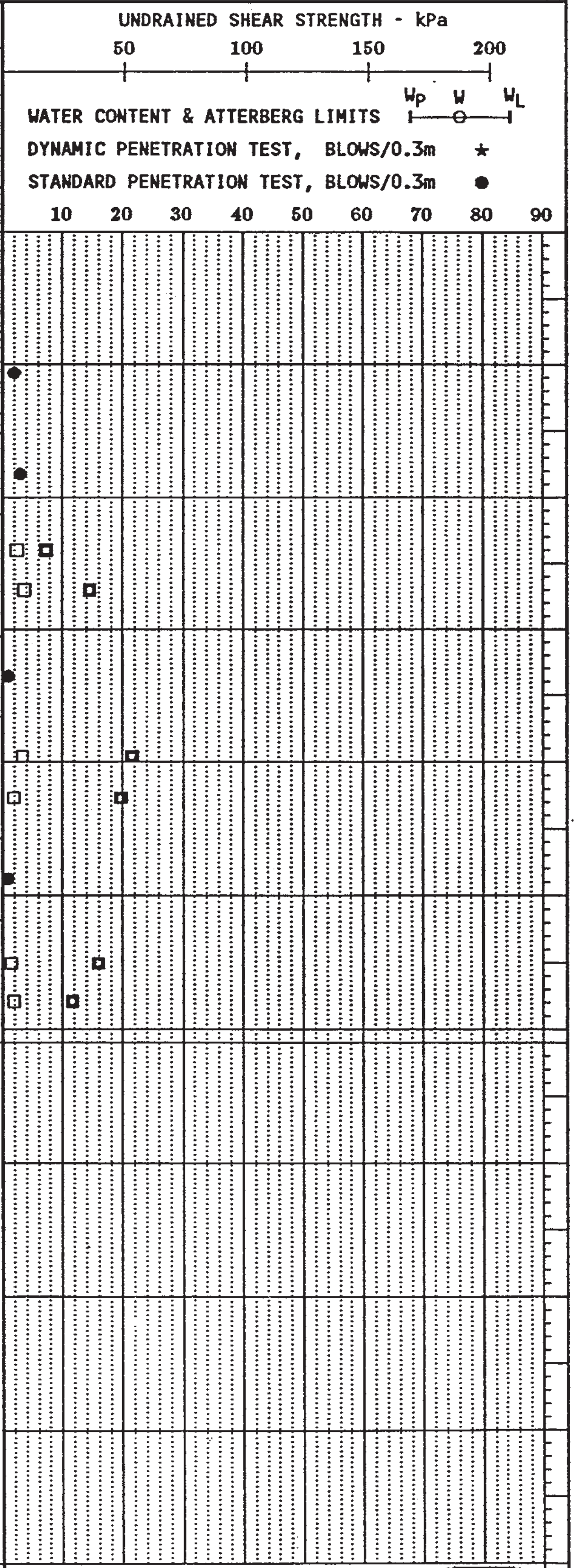
WATER LEVEL

BOREHOLE No. 94-36

PROJECT No. 10519

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY mm	SAMPLES N-VALUE OR RQD
0	86.52							
	86.3	TOPSOIL						
		Stiff to firm, brown to grey SILTY CLAY, trace organics						
1					SS	1	350	2
					SS	2	320	3
2	84.4							
		Firm, grey SILTY CLAY, occasional sand seam						
3					SS	3	610	1
4								
5					SS	4	610	1
6	80.4							
		End of Borehole						
7								
8								
9								
10								



▽
▼
◀ Proposed Pipe Invert

◘ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

JACQUES, WHITFORD
LIMITED

BOREHOLE RECORD

94-37

CLIENT The Future Group

LOCATION South Keys Commercial Development, Ottawa, Ontario

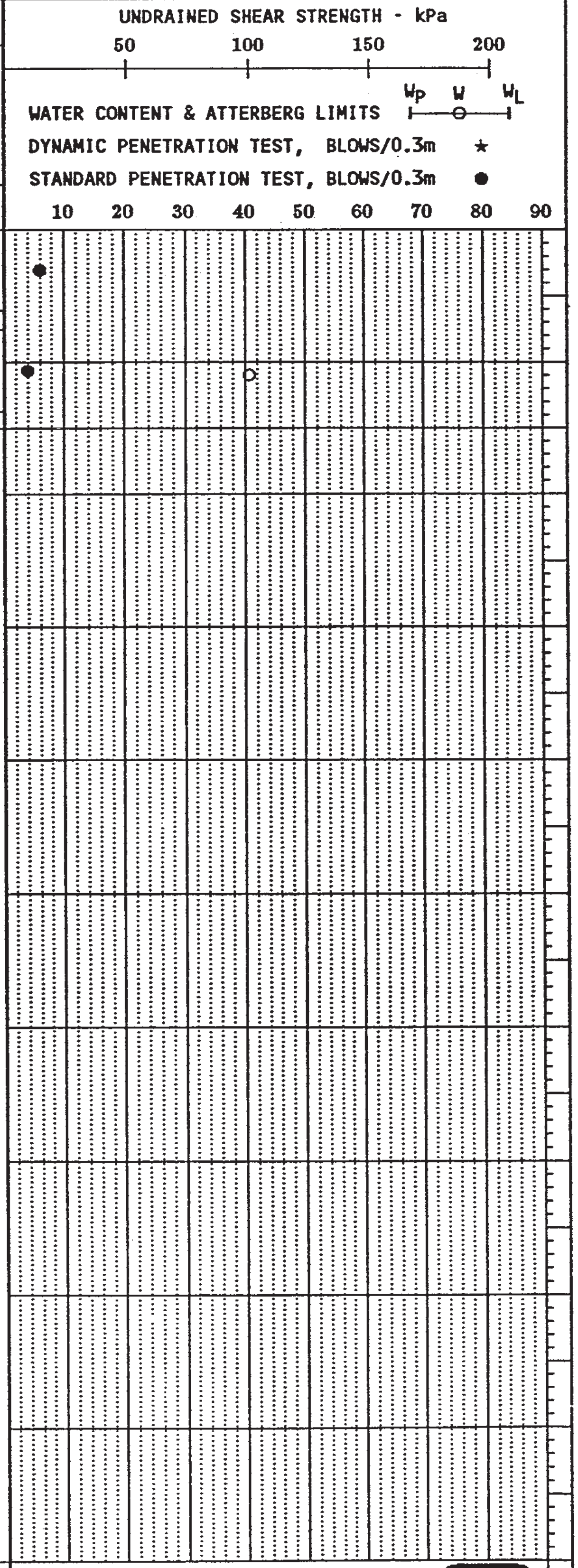
DATES: BORING 94-07-21 WATER LEVEL

BOREHOLE No. 94-37

PROJECT No. 10519

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES			
					TYPE	NUMBER	RECOVERY	N-VALUE OR RQD
	86.91						mm	
0	86.8	80 mm Sandy TOPSOIL						
		Firm, brown to grey SILTY CLAY, occasional sand seam			SS	1	200	6
1					SS	2	160	4
	85.4							
		End of Borehole						
2								
3								
4								
5								
6								
7								
8								
9								
10								



∇ ▼ Proposed Pipe Invert

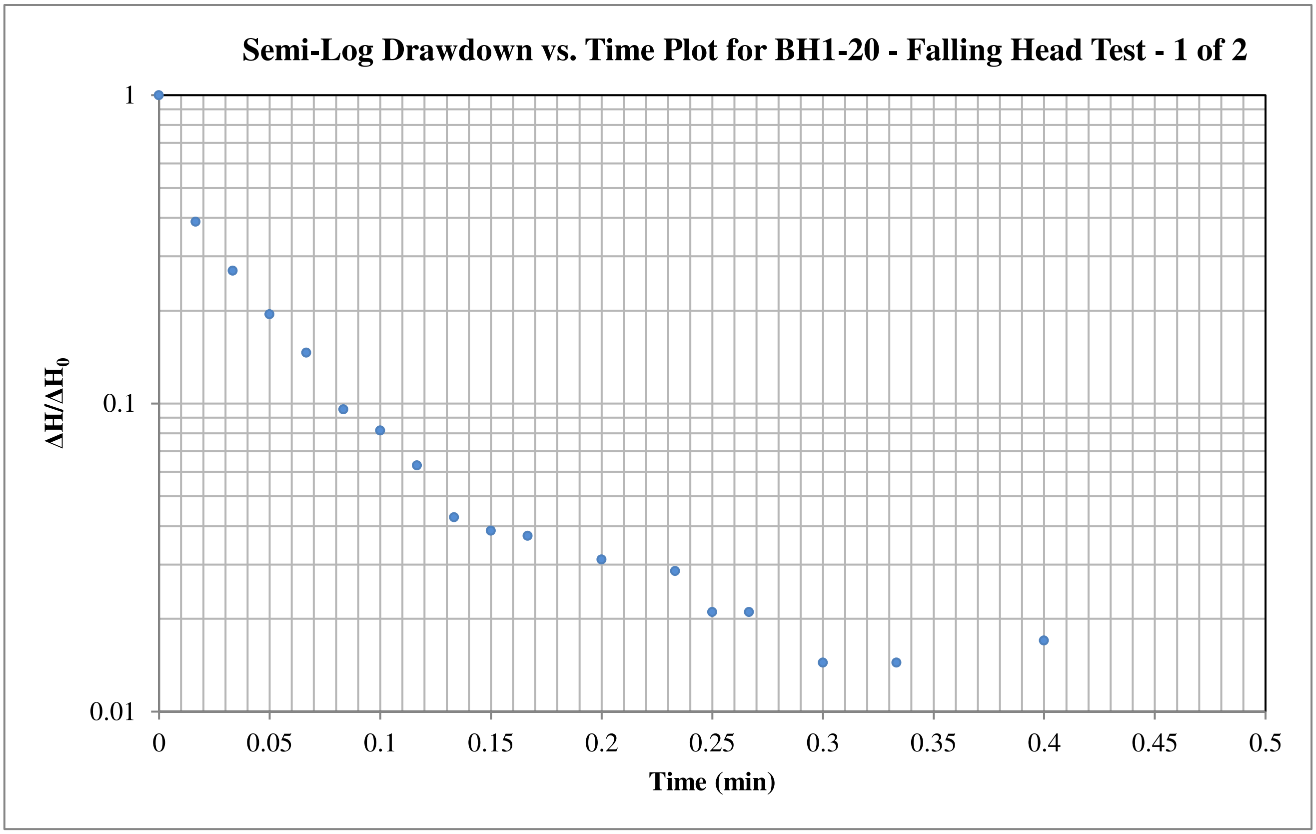
◘ Field Vane Test, kPa

☐ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Hvorslev Hydraulic Conductivity Analysis

Project: Calloway REIT (South Keys) Inc. - 2210 Bank Street
Test Location: BH1-20
Test: Falling Head - 1 of 2
Date: May 13, 2020



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^2}{F} \frac{1}{t*} \ln\left(\frac{\Delta H *}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.60152

Well Parameters:

L	3 m	Saturated length of screen or open hole
D	0.032 m	Diameter of well
r_c	0.016 m	Radius of well

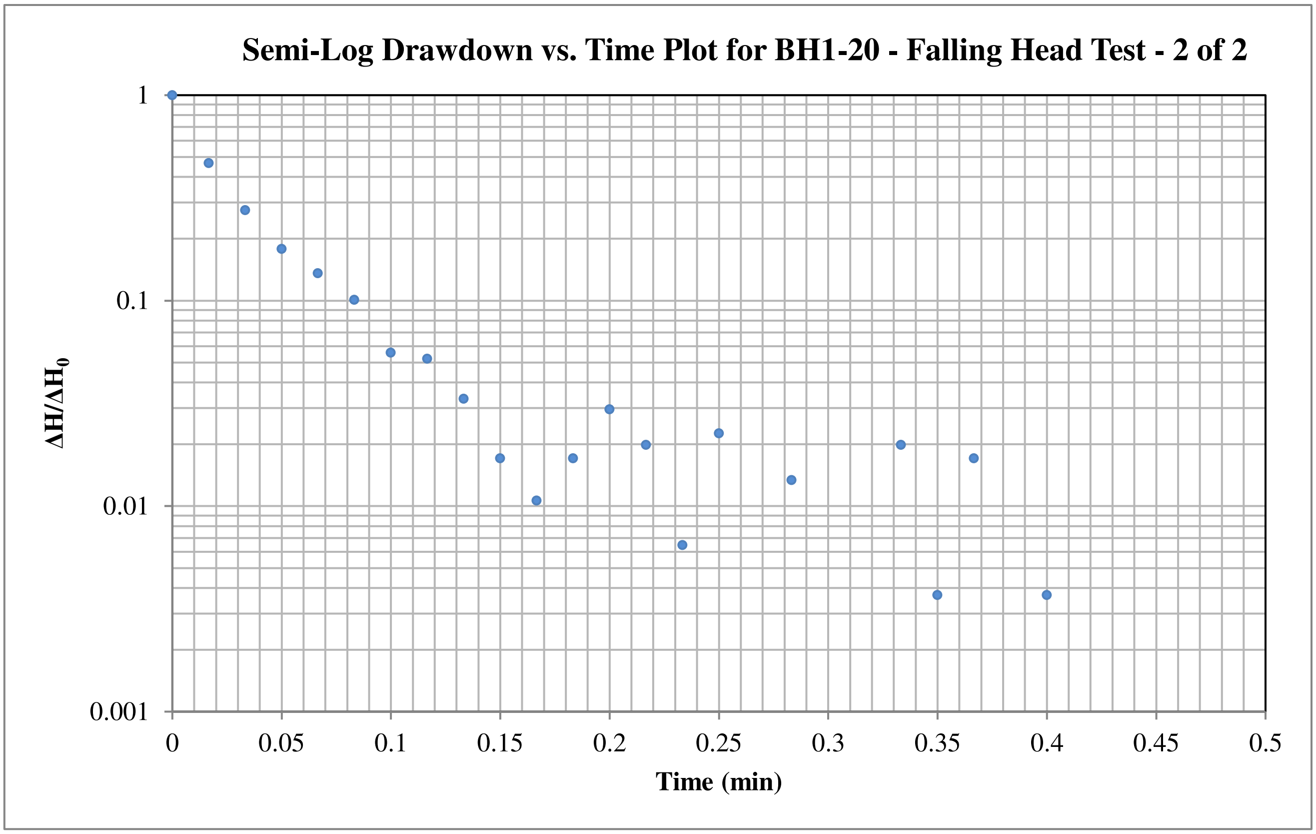
Data Points (from plot):

t*: 0.019 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.92E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Calloway REIT (South Keys) Inc. - 2210 Bank Street
Test Location: BH1-20
Test: Falling Head - 2 of 2
Date: May 13, 2020



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.60152

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.032	m	Diameter of well
r_c	0.016	m	Radius of well

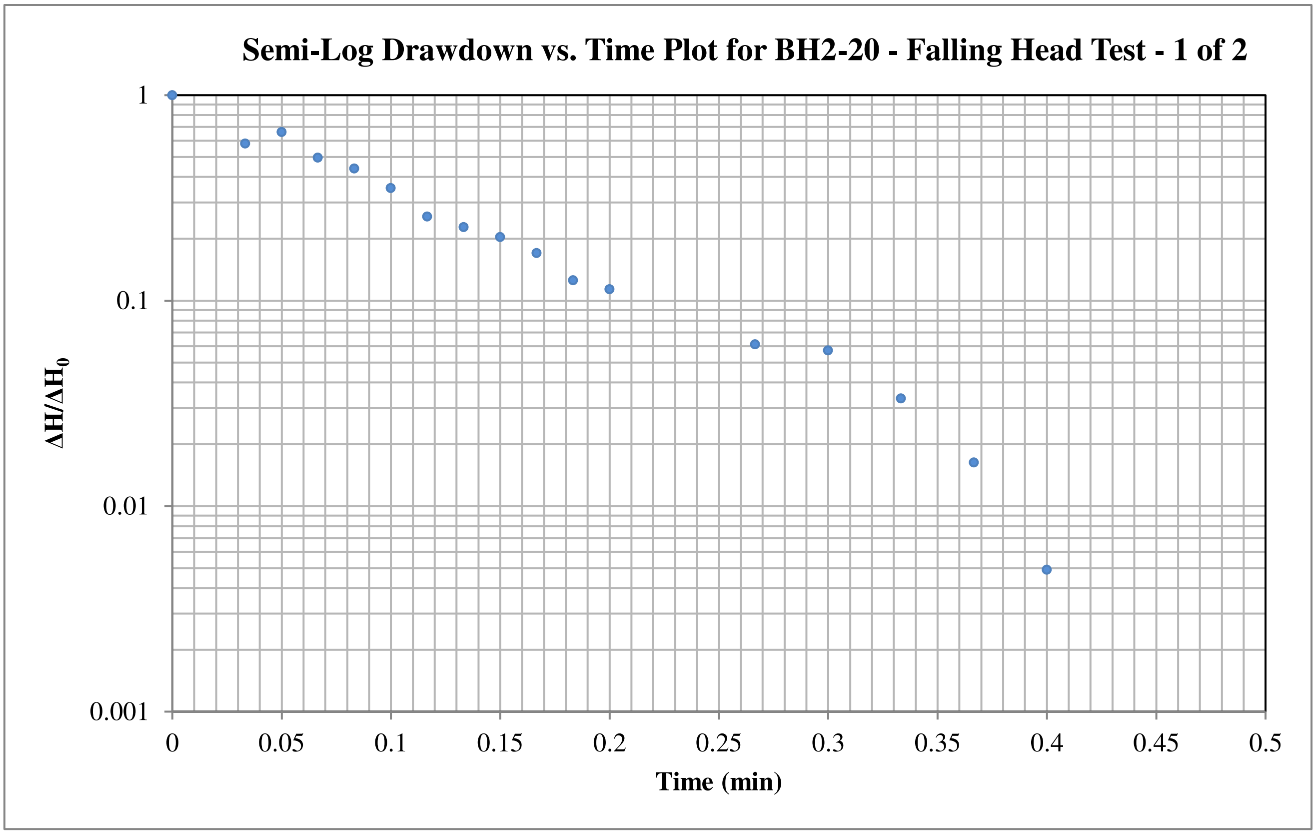
Data Points (from plot):

t*: 0.025 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.47E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Calloway REIT (South Keys) Inc. - 2210 Bank Street
Test Location: BH2-20
Test: Falling Head - 1 of 2
Date: May 13, 2020



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.60152

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.032	m	Diameter of well
r_c	0.016	m	Radius of well

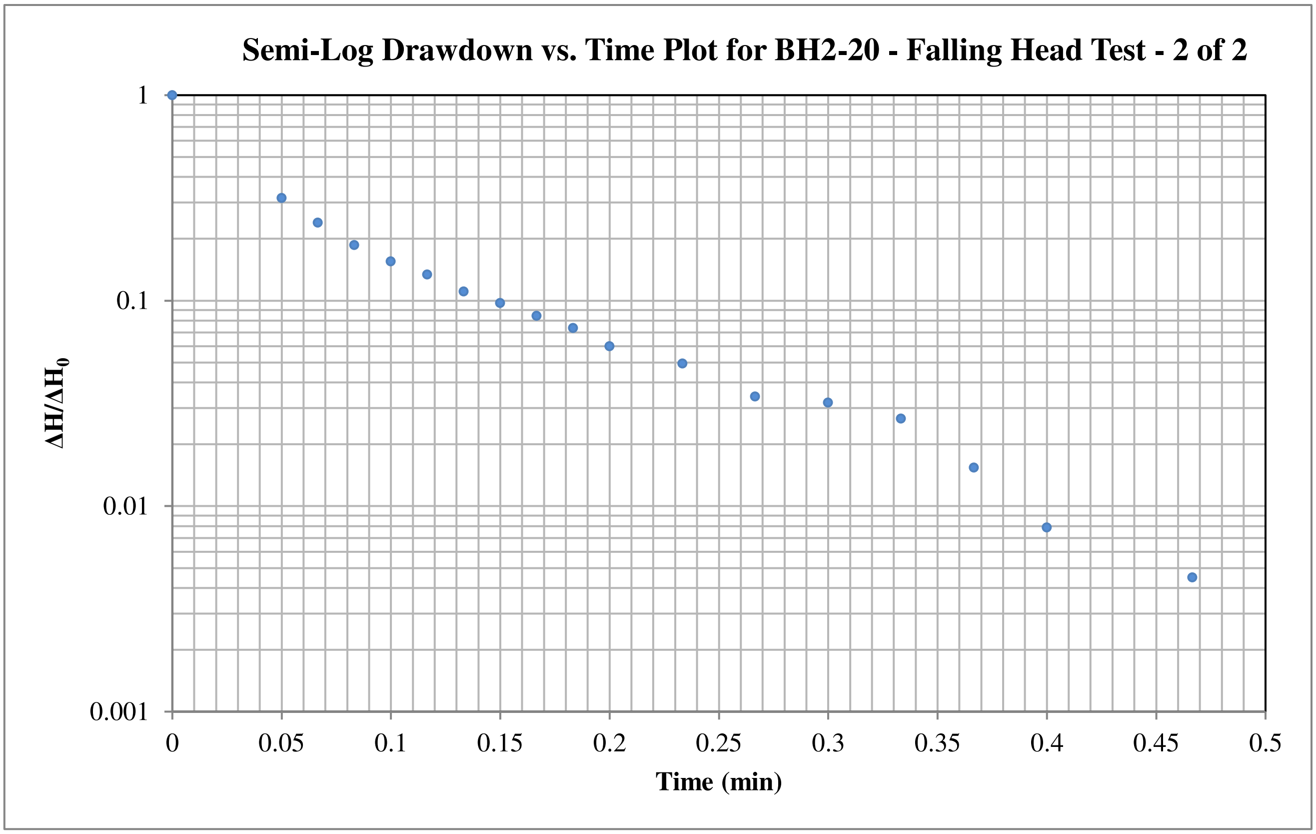
Data Points (from plot):

t*: 0.097 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.82E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Calloway REIT (South Keys) Inc. - 2210 Bank Street
Test Location: BH2-20
Test: Falling Head - 2 of 2
Date: May 13, 2020



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t*} \ln\left(\frac{\Delta H *}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.60152

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.032	m	Diameter of well
r_c	0.016	m	Radius of well

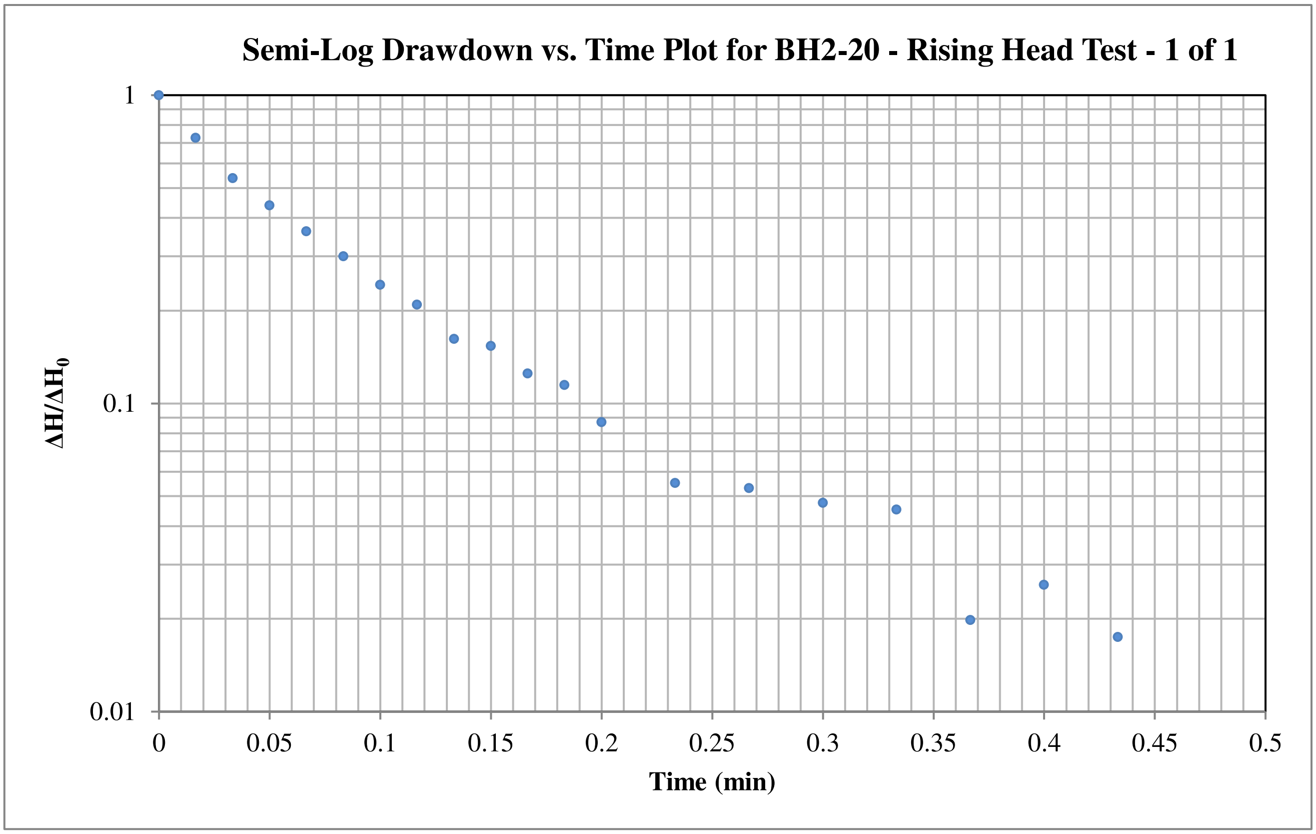
Data Points (from plot):

t*: 0.046 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 8.04E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Calloway REIT (South Keys) Inc. - 2210 Bank Street
Test Location: BH2-20
Test: Rising Head - 1 of 1
Date: May 13, 2020



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^2}{F} \frac{1}{t*} \ln\left(\frac{\Delta H *}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.60152

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.032	m	Diameter of well
r_c	0.016	m	Radius of well

Data Points (from plot):

t*: 0.065 minutes

$\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 5.71E-05 m/sec

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 2957

Report Date: 06-Apr-2020

Order Date: 31-Mar-2020

Project Description: PG5242

Client ID:		BH2-SS3	-	-	-
Sample Date:		30-Mar-20 13:00	-	-	-
Sample ID:		2014134-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	66.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.40	-	-	-
Resistivity	0.10 Ohm.m	5.13	-	-	-
Anions					
Chloride	5 ug/g dry	1040	-	-	-
Sulphate	5 ug/g dry	730	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5242-1 - TEST HOLE LOCATION PLAN

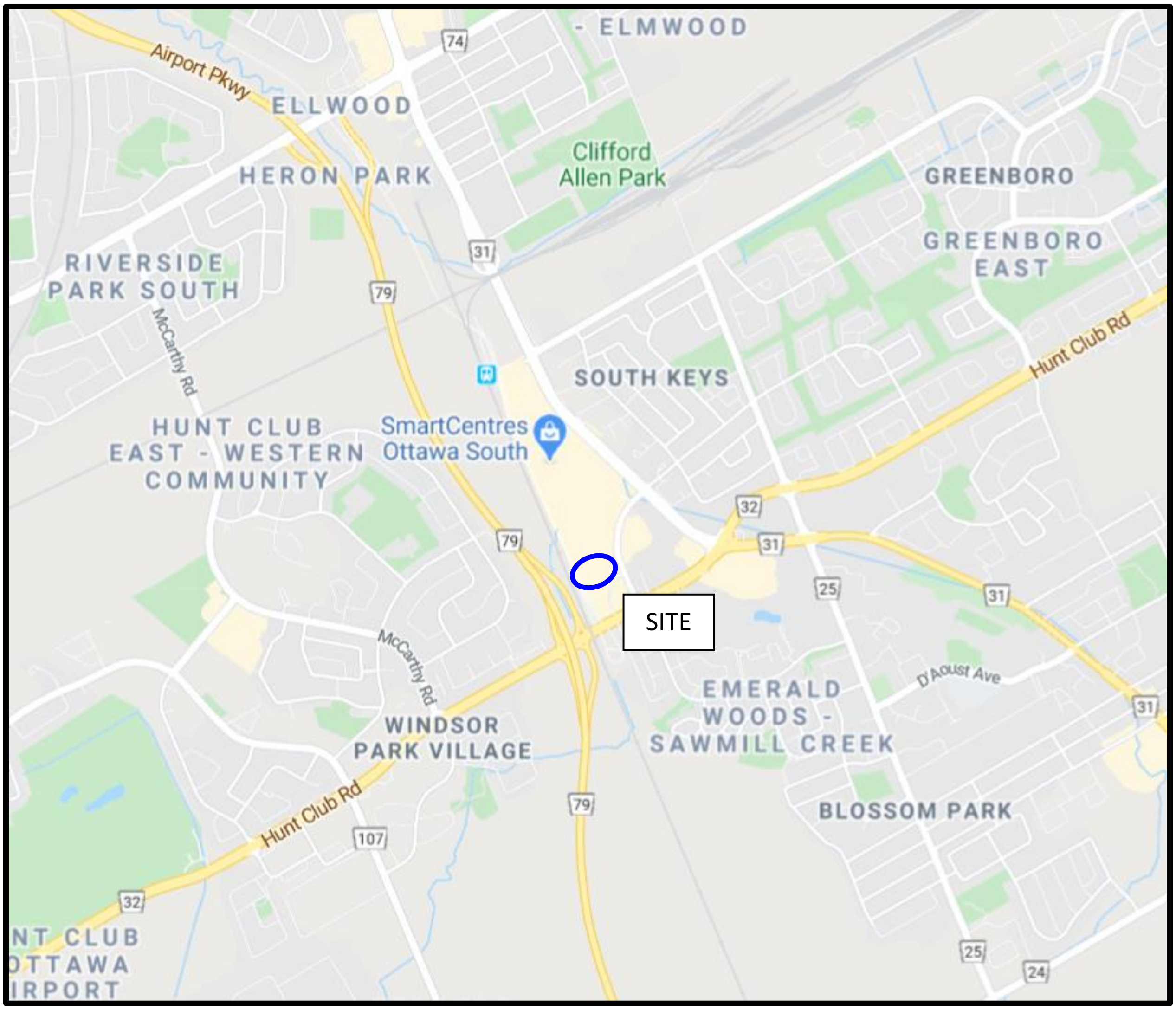
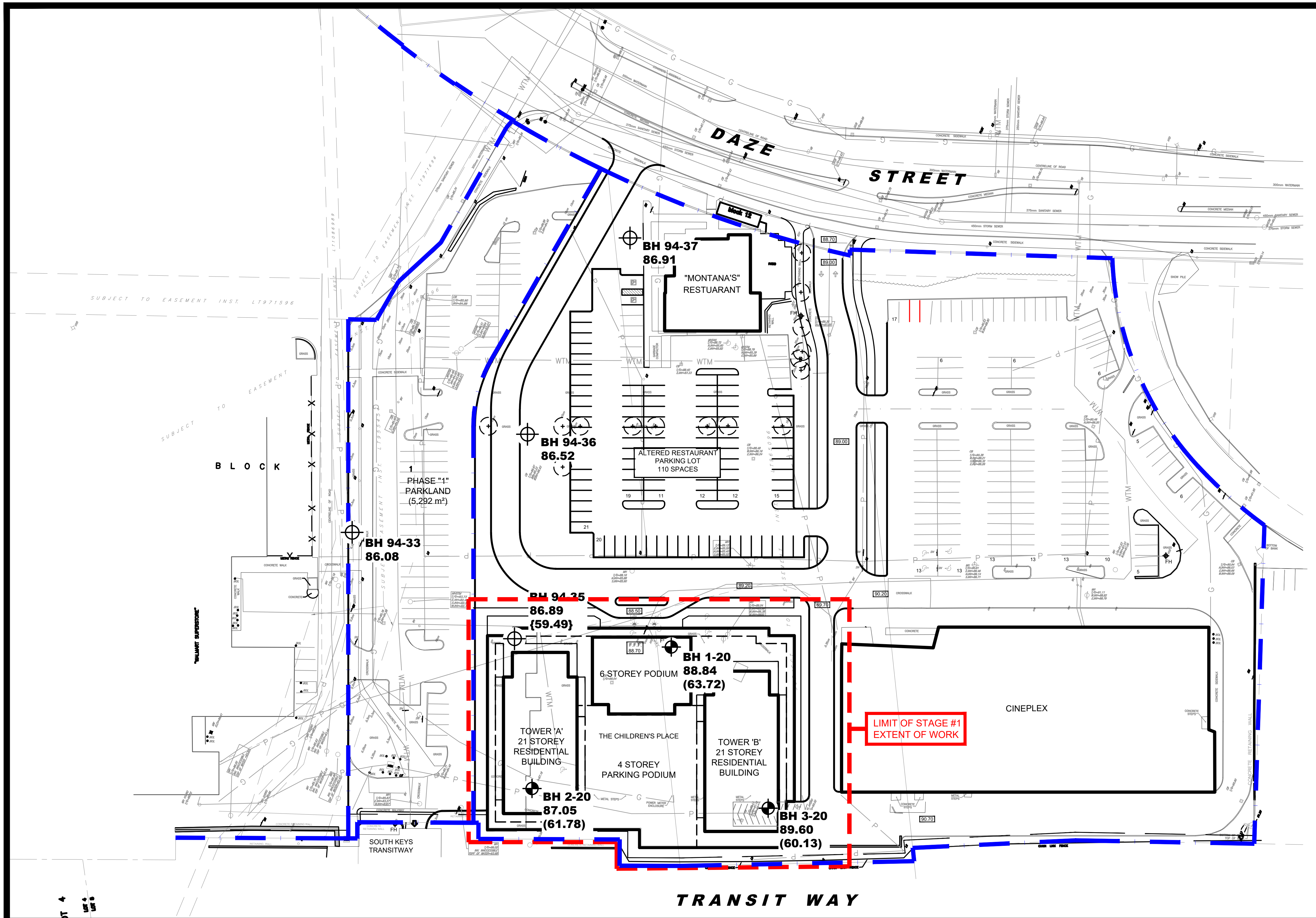


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

⊕	BOREHOLE LOCATION, 2020
⊕	BOREHOLE BY OTHERS LOCATION (JACQUES WHITFORD - 1994)
99.65	GROUND SURFACE ELEVATION (m)
(61.78)	PRACTICAL REFUSAL TO DCPT ELEV.(m)
{59.49}	PRACTICAL REFUSAL TO AUGERING ELEV.(m)

CONCEPTUAL PLAN PROVIDED BY RLA ARCHITECTS

SCALE: 1:1000

0 10 20 30 40 50m 75m

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO NEW CONCEPTUAL PLAN	13/09/2021	DP

CALLOWAY REIT (SOUTH KEYS) INC. / CANADIAN PROPERTY HOLDINGS (SOUTH KEYS) INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MULT-STOREY BUILDING - PHASE 1
2210 BANK STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:1000
Date:	04/2020
Drawn by:	RCG
Report No.:	PG5242-1
Checked by:	RG
Dwg. No.:	PG5242-1
Approved by:	DJG
Revision No.:	1

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