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REPORT ON

Geotechnical Investigation Proposed Retirement Residence 5157 Innes Road and 1980 Trim Road Ottawa, Ontario

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REPORT

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for a proposed retirement residence to be located at the northwest corner of the intersection of Innes Road and Trim Road in Ottawa, Ontario.

The geotechnical investigation included an assessment of the general subsurface conditions in the area of the proposed development by means of three boreholes and laboratory testing. Based on an interpretation of the factual information obtained, a general description of the subsurface conditions is presented. These interpreted subsurface conditions and available project details were used to prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

Golder Associates has also completed Phase I and Phase II environmental site assessment (ESA) reports for the proposed development, which are provided under separate cover.

The reader is referred to the “Important Information and Limitations of This Report” which follows the text but forms an integral part of this document.

2.0 DESCRIPTION OF PROJECT AND SITE

Plans are being prepared for the construction of a retirement residence to be located at the northwest corner of the intersection of Innes Road and Trim Road in Ottawa, Ontario. The approximate location of the site is shown on the Key Map inset on the attached Site Plan (Figure 1).

The following is understood about the site and proposed development:

- The site encompasses two property parcels (5157 Innes Road and 1980 Trim Road). Together, the site is roughly ‘L’ shaped, with maximum dimensions of about 80 metres wide and 100 metres long.
- 5157 Innes Road is undeveloped and vegetated with grass.
- 1980 Trim Road is the site of an abandoned residential dwelling and is vegetated with grass and mature trees.
- The site topography is relatively flat and level with the existing roadways. The ground surface elevation ranges from about 88.4 to 88.7 metres.
- The proposed building will also be ‘L’ shaped. Each wing will be about 19 metres wide and 60 to 70 metres long.
- It is understood that that the building will be 5 to 6 storeys in height and will have one basement level for underground parking.

Based on published geological mapping, the subsurface conditions on the site are indicated to consist of a thick deposit of sensitive silty clay. The geological mapping indicates that the depth to the underlying bedrock surface ranges from 15 to 50 metres below the ground surface, deepening to the west. A fault is indicated to transect the site from west to east. To the north of the fault, the bedrock is indicated to consist of interbedded limestone and dolomite of the Gull River Formation. To the south of the fault, the bedrock is indicated to consist of interbedded limestone and shale of the Lindsay Formation.

3.0 PROCEDURE

The fieldwork for the geotechnical investigation was carried out between October 26 and 28, 2016. During that time, three boreholes (numbered 16-1, 16-2, and 16-3) were advanced using an all-terrain truck-mounted hollow-stem auger drill rig supplied and operated by CCC Geotechnical and Environmental Drilling of Ottawa, Ontario. The approximate locations of the boreholes are shown on the attached Site Plan (Figure 1).

All of the boreholes were advanced to a target depth of about 15.2 metres below the existing ground surface. Standard penetration tests were carried out within the boreholes at regular intervals of depth and soil samples were recovered using split-spoon sampling equipment. In situ vane testing was carried out, where possible, in the silty clay to determine the undrained shear strength of this soil unit. In addition, six relatively undisturbed 73 millimetre diameter thin-walled Shelby tube samples of the silty clay were obtained using a fixed piston sampler.

A dynamic cone penetration test (DCPT) was carried out in borehole 16-3 below 15.2 metres depth to determine the probable depth to the bedrock surface on the site. The DCPT in this borehole was advanced to practical refusal to advancement, which was encountered at about 34.3 metres depth.

Monitoring wells were sealed into boreholes 16-2 and 16-3 to allow for subsequent measurement of the groundwater level and environmental groundwater sampling. The groundwater levels were measured in the monitoring wells on November 10, 2016.

The fieldwork was supervised by experienced personnel from our staff who directed the drilling and in situ testing operations, logged the boreholes and samples, and took custody of the samples retrieved. On completion of the drilling operations, soil samples obtained from the boreholes were transported to our laboratory for further examination by the project engineer and for laboratory testing, including natural water content, Atterberg limits, and three oedometer consolidation tests.

One sample of groundwater from the monitoring well installed in borehole 16-2 was submitted to Paracel Laboratories for basic chemical analyses related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements.

The borehole locations were selected, marked in the field, and subsequently surveyed by Golder Associates personnel. The location and ground surface elevation at each borehole location were determined using a precision Trimble R8 GPS survey unit. The geo-reference coordinates are based on NAD 83 Coordinate system, UTM Zone 18. The elevations are referenced to Geodetic datum (CGVD 1928).

4.0 SUBSURFACE CONDITIONS

4.1 General

Information on the subsurface conditions is presented as follows:

- Record of Borehole Sheets are provided in Appendix A.
- Results of the basic chemical analyses are provided in Appendix B.
- A plot of undrained shear strength versus elevation for the grey silty clay deposit is provided on Figure 2.
- Results of the laboratory oedometer consolidation testing are provided on Figures 3 to 5.
- Results of the laboratory natural water content and Atterberg limits testing are provided on the Record of Borehole sheets.

In general, the subsurface conditions on this site consist of fill overlying a thick deposit of silty clay to clay that extends to over 30 metres depth. The following sections present a more detailed overview of the subsurface conditions encountered in the boreholes advanced during the investigation.

4.2 Fill

A surficial layer of silty clay fill exists at boreholes 16-2 and 16-3. The silty clay fill at these locations extends to depths of about 0.8 and 1.5 metres below the existing ground surface, respectively.

Standard penetration tests carried out within the silty clay fill gave SPT 'N' values ranging from 6 to 10 blows per 0.3 metres of penetration. The density or relative consistency of the fill is expected to vary greatly depending on the method of placement and the compactive effort that was applied at the time it was originally placed.

The measured water content of one sample of the silty clay fill is about 23 percent.

4.3 Silty Clay to Clay

A thick deposit of sensitive silty clay to clay (hereafter collectively referred to as silty clay) exists beneath the fill at boreholes 16-2 and 16-3 and directly below the ground surface at borehole 16-1. The silty clay deposit was not fully penetrated at boreholes 16-1 and 16-2, but was proven to about 15.2 metres below the existing round surface. Based on the DCPT advanced in borehole 16-3, the silty clay deposit likely extends to a depth of about 32.6 metres below the existing ground surface. The bottom of the deposit may contain clayey silt, silt, and/or silty sand seams/layers.

The upper portion of the silty clay deposit has been weathered to a grey brown crust, which extends to a depth of about 3.7 metres below the existing ground surface. Standard penetration tests carried out within the weathered crust gave SPT 'N' values ranging from 1 to 13 blows per 0.3 metres of penetration, indicating a very stiff to stiff consistency. The measured water content of four samples of the weathered crust ranged from about 22 to 62 percent, increasing with depth.

Beneath the depth of weathering, the silty clay is grey in colour. The results of in situ vane testing in the grey silty clay gave undrained shear strengths ranging from about 30 to 61 kilopascals, indicating a firm to stiff consistency, generally increasing below about elevation 80 metres. A plot of the undrained shear strengths versus elevation is provided on the attached Figure 2. The remolded shear strengths are low, indicating a sensitive soil. The results of Atterberg limit testing carried out on three samples of the grey silty clay gave plasticity index values ranging from 29 to 47 percent and liquid limit values ranging from about 57 to 74 percent, indicating high plasticity soil. The measured water content of six samples of the grey silty clay ranged from about 73 to 86 percent, which are generally higher than the measured liquid limits.

Laboratory oedometer consolidation testing was carried out on three thin-walled Shelby tube samples of the grey silty clay. The results of that testing are provided on Figures 3 to 5 and are summarized in the table below.

Borehole/ Sample Number	Sample Depth/ Elevation (m)	Unit Weight (kN/m^3)	σ'_p (kPa)	σ'_{vo} (kPa)	C_c	C_r	e_0	OCR
16-2 / 8	8.1 / 80.5	15.1	120	70	3.10	0.018	2.27	1.7
16-2 / 10	11.1 / 77.5	15.3	140	90	2.30	0.021	2.13	1.6
16-3 / 8	6.6 / 82.1	15.2	130	65	2.50	0.018	2.22	2.0

Notes:

σ'_p - Apparent preconsolidation pressure
C_c - Compression index
e_o - Initial void ratio
σ'_{vo} - Computed existing vertical effective stress
C_r - Recompression index
OCR - Overconsolidation ratio

4.4 Dynamic Cone Penetration Testing and Possible Bedrock

Borehole 16-3 was advanced below 15.2 metres depth by means of a dynamic cone penetration test (DCPT). Based on the measured blows per 0.3 metres of penetration, a glacial till deposit likely exists below the silty clay deposit between depths of about 32.6 and 34.3 metres below the existing ground surface. Refusal to DCPT advancement (greater than 100 blows per 0.3 metres penetration) was encountered at a depth of about 34.3 metres below the existing ground surface. DCPT refusal could indicate the bedrock surface; however, it could also represent cobbles or a boulder within a glacial till deposit.

4.5 Groundwater

The groundwater levels measured during the investigation are summarized in the following table:

Borehole	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Date of Measurement
16-2	88.60	1.98	86.62	November 10, 2016
16-3	88.73	2.09	86.64	November 10, 2016

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

5.0 DISCUSSION

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the proposed development based on our interpretation of the borehole information and project requirements. Reference should be made to the "Important Information and Limitations of This Report", which follows the text but forms part of this document.

The following guidelines are provided on the basis that the multi-storey building will be designed in accordance with Part 4 of the 2012 Ontario Building Code (OBC).

5.2 Site Grading

The subsurface conditions on this site consist of surficial fill overlying a thick deposit of sensitive silty clay that extends to over 30 metres depth.

The compressibility of the silty clay deposit negatively impacts the permissible filling of this site. The silty clay deposit has limited capacity to support the combined loading from grade raise filling, foundation loads, groundwater level lowering, floor loads, etc. Overstressing of the silty clay will lead to excessive foundation settlements. For the purposes of this assessment, it has been assumed that the proposed grading will be up to about elevation 89.0 metres to allow for a 0.3 to 0.5 metre high grade raise. This final grade will need to be maintained for the bearing resistance values given in Section 5.4 to be applicable. Additional filling above elevation 89.0 metres will require additional geotechnical analysis.

In addition to the material that will be excavated within the footprint of the building for construction of the basement, the topsoil should also be removed from beneath pavement areas. It is considered that the existing fill can remain in place beneath the pavement provided some settlement of the pavement structure can be tolerated.

The topsoil and fill containing organic matter are not suitable as engineered fill and should be removed from the site or stockpiled separately for re-use in landscaping applications only. It is important that stockpiles, if located on site, not be adjacent to excavations but rather should be located within the future landscaping areas.

5.3 Excavations

It is understood that the proposed building will include 1 basement level. For preliminary design purposes, it has been assumed that the base of the excavation would be 3.5 metres below the final grade. It has also been assumed that the final grade elevation would be 89.0 metres (allowing for a grade raise of 0.3 to 0.5 metres). With these assumptions, the base of the excavation would be at about elevation 85.5 metres, which is about 3 metres below the existing ground surface.

The excavations for the basement will be through surficial fill and into the very stiff to stiff weathered silty clay crust. Based on the borehole information, the sensitive grey silty clay is present below about elevation 85 metres. If the founding level is lowered below this elevation, the excavations will potentially extend into this sensitive layer, but the following guidelines would remain unchanged.

No unusual problems are anticipated with excavating the overburden using conventional hydraulic excavating equipment. If the excavations are carried out in the sensitive silty clay, it is suggested that the excavation equipment be fitted with a smooth bladed bucket (i.e., no teeth), to limit disturbance of the subgrade.

The existing fill and silty clay deposits in the area of the proposed building would generally be classified as Type 3 soils in accordance with the Occupational Health and Safety Act (OHSA) and therefore open cut side slopes would need to be cut back at an inclination no steeper than 1 horizontal to 1 vertical (1H:1V). For slopes which are unsupported in the longer term, and might experience freeze-thaw cycles, flatter side slope inclinations could be required.

For the 3 metre deep excavations required at this site, it is anticipated that open-cut excavations will generally be feasible. If excavation support (i.e., temporary shoring) is deemed required due to space constraints, further geotechnical guidance would be required. Assistance in this regard can be provided, if required.

Based on present groundwater levels, excavations deeper than about 2 metres will extend below the groundwater level. Groundwater inflow into the excavations should feasibly be handled by pumping from sumps within the excavations. Groundwater inflow from the weathered silty clay crust is expected to be low to moderate; however, the actual rate of groundwater inflow will depend on many factors including the contractor's schedule and rate of excavation, the size of the excavation, the number of working areas being excavated at one time, and the time of year at which the excavation is made. Also, there may be instances where significant volumes of precipitation, surface runoff and/or groundwater collects in an open excavation, and must be pumped out.

Under the new regulations, a Permit-To-Take-Water (PTTW) is required from the Ministry of the Environment and Climate Change (MOECC) if a volume of water greater than 400,000 litres per day is pumped from the excavation. If the volume of water to be pumped will be less than 400,000 litres per day, but more than 50,000 litres per day, the water taking will not require a PTTW, but will need to be registered in the Environmental Activity and Sector Registry (EASR) as a prescribed activity. Based on the groundwater information collected during the investigation, it is considered unlikely that an EASR or PTTW would be required during construction for this project. However, the requirement for registration in the EASR is possible if inflows are greater than expected. The requirement for registration (i.e., if more than 50,000 litres per day is being pumped) can be assessed at the time of construction. Registration is a quick process that will not significantly disrupt the construction schedule.

5.4 Foundations

As discussed previously, the subsurface conditions at this site consist of surficial fill underlain by a thick deposit of sensitive silty clay. The bedrock surface is at least 34 metres below the existing ground surface. A layer of glacial till likely overlies the bedrock.

Shallow spread footing foundations or a raft slab foundation could be considered provided that the bearing resistance values provided in the subsequent sections are adequate to support the loads imposed by the structure. If the loading from the structure prohibits the use of these options, deep foundations will need to be considered. The most feasible and practical deep foundation system for this building will likely be driven end-bearing steel piles.

5.4.1 Shallow Spread Footings

Shallow spread footings can be considered provided that they can be designed using the bearing resistance values provided below. For this assessment, an underside of footing elevation of 85.5 has been assumed. At this elevation, the spread footings would bear on the very stiff to stiff weathered silty clay crust.

The Serviceability Limit States (SLS) bearing resistance value for spread footing foundations is based on limiting the stress increases on the firm grey silty clay to an acceptable level, so that foundation settlements do not become excessive. Important parameters in calculating the stress increase on the grey silty clay are:

- The thickness of the weathered crust below the underside of the footings;
- The size (dimensions) of the footings;
- The amount of surcharge in the vicinity of the foundation due to landscape fill, underslab fill, floor loads, etc.;
- The amount of unloading due to the soil removed for basement construction; and,
- The effects of groundwater lowering caused by this or other construction.

The floor load for the basement floor slab has been assumed to be 12 kilopascals. A net unloading, assuming 3 metres of soil is excavated for the basement level, has also been used in this assessment.

Based on the above footing elevations and floor loadings, the SLS net bearing resistance and the factored Ultimate Limit States (ULS) bearing resistance values for spread footing foundations may be taken as follows:

Footing Type	Maximum Footing Size (m)	SLS Bearing Resistance (kPa)	Factored ULS Bearing Resistance (kPa)
Exterior Strip[1]	1.0	170	190
	2.0	100	155
	3.3	80	140
	4.0	65	130
	5.0	65	125
	6.0	65	120
Exterior Pad[1] (sides of building)	1.0	230	230
	2.0	150	230
	3.0	105	195
	4.0	90	175
	5.0	75	165
	6.0	75	155
Exterior Pad[1] (corners of building)	1.0	230	230
	2.0	135	230
	3.0	95	195
	4.0	80	175
	5.0	65	165
	6.0	65	155
Interior Pad	1.0	230	230
	2.0	190	230
	3.0	130	195
	4.0	110	175
	5.0	90	165
	6.0	90	155

Note* [1] *– *Bearing resistance values for the exterior pad footings could be increased to match the interior pad footings if extruded polystyrene (EPS) lightweight fill is used to backfill around the building exterior. The EPS would need to be placed for the full height of the basement wall and extend at least 5.4 metres beyond the outside of the wall. The exterior strip footing bearing resistance values could also be increased if EPS is used. New values for strip footings can be provided, if requested. In areas outside of the building which will not experience vehicular traffic, the lightweight fill should meet the requirements of EPS12 Geofoam in accordance with ASTM D6817. The areas in which the Geofoam will be subjected to heavily loaded vehicular traffic, the EPS should meet the requirements of EPS22. In addition, the EPS in areas that will experience heavy loaded vehicular traffic should be placed below the subbase layer of the pavement structure, to distribute the wheel loads and avoid overstressing of the EPS.*

For the above bearing resistance values to be applicable, the zone of influence of adjacent footings must not overlap within the softer portions of the silty clay deposit. If overlapping occurs, reduced bearing resistance values will apply. The zone of influence is a function of the footing size and contact pressure. Further geotechnical guidance can be provided in this regard once additional details are provided.

The post construction total and differential settlements of footings sized using the above SLS net bearing resistance values should be less than about 25 and 15 millimetres, respectively, provided that the soil at or below founding level is not disturbed during construction. Further, these bearing resistances correspond to a settlement resulting from consolidation of the silty clay. Consolidation of the silty clay is a process which takes months or longer and, as such, results from sustained loading. Therefore, the foundation loads to be used in conjunction with the SLS resistance values given above should be the full dead load plus sustained live load. The factored dead plus full factored live load should be used in conjunction with the ULS factored bearing resistance.

The silty clay subgrade is sensitive to disturbance (such as from construction traffic) and a mud slab of lean concrete, at least 50 millimetres in thickness, should be provided on the bearing surfaces after excavation to founding level following review/approval of the bearing surface by geotechnical personnel. Excavations to expose the silty clay subgrade should be carried out using smooth-edged excavator bucket (i.e., no teeth) to minimize disturbance.

5.4.2 Raft Slab

A foundation alternative for the proposed building at this site would be to use a ‘raft’ foundation. A raft foundation would need to be sufficiently rigid so that the building loads would be relatively uniformly distributed over the entire building footprint.

The available bearing resistance for support of the raft foundation depends on the founding level, since it impacts on both the bearing stratum and on the compensating effect of the weight of the excavated soil. For preliminary design purposes, the founding level has been assumed at about elevation 85.5 metres, which is within the weathered silty clay crust. The founding level should ideally be uniform across the footprint to limit differential settlements.

For the assumed founding level, it is considered that the raft foundation can be designed using an SLS gross contact stress of 75 kilopascals. This bearing resistance is based on maintaining the stress level in the clay deposit at a reasonable margin below the preconsolidation pressure of the clay deposit below founding level; i.e., such that the stress level in the clay will not approach or surpass its ‘yield’ stress.

The ULS factored bearing resistance that may be used for the design of the raft foundation is 115 kilopascals.

The post-construction total and differential settlements of the raft will depend, in part, upon the duration of time from when the excavation is made to when the building load is applied, since the clay will “rebound” (i.e., swell) following removal of the weight of the overlying soil. This rebound will be recovered as settlement once the structure loads are imposed on the raft. The post-construction settlements will be larger for corresponding longer lengths of time between excavating and re-loading. In addition, the clay will also undergo heave and subsequent settlement as a result of undrained distortion of the deposit.

If the bearing stress under the raft were to reach the SLS bearing resistance provided above (i.e., if the full structure weight were to equal the full available SLS bearing resistance), the *calculated total* settlement of the raft foundation is expected to be in the order of 25 to 50 millimetres (accounting for the recovered rebound and distortion settlement of the clay), depending in part upon that duration of unloading/construction and noting that the larger settlement estimate would correspond to a period of several months of full unloading.

For design purposes, it is recommended that a differential settlement of up to 70 percent of the total settlement be expected/accommodated. However, this differential settlement will also depend greatly on the stiffness of the raft; even for uniform ground conditions, the settlement of the edge of the raft would typically be less than that of the centre. If variations in the raft level are needed, such as to accommodate sloping parking levels or deeper foundation areas for elevator pits, there would be an increased potential for differential settlements.

It should also be noted that the localized differential settlements (i.e., raft slab deflections) within/beneath individual bays (such as directly beneath a column versus the mid-span of the bay) will depend upon the relative stiffness between the raft slab and the underlying subgrade. The deflections and the resulting forces and bending moments in the slab to be used in its structural design could be determined by structural analysis using a modulus of subgrade reaction, k_s, for the subgrade.

It should be noted, however, that the modulus of subgrade reaction is not a fundamental soil property and its value depends, in part, on the size and shape of the loaded area. For the analysis of the contact stress distribution beneath a raft foundation, its value would depend on the size of the areas over which increased/concentrated contact stresses are anticipated (analogous to equivalent footings beneath the columns); the size of these areas is in turn related to the value the modulus of subgrade reaction, i.e., they are inter-related.

Accordingly, the analysis of the raft slab should ideally involve an iterative analysis between the determination of the contact stress distribution by the structural engineer and the geotechnical determination of the modulus of subgrade reaction value, until the two are consistent with each other. For initial analyses, the modulus of subgrade reaction may be assumed to be in the range of 1 to 5 megapascals per metre. This range reflects both uncertainty in the size of the loaded area as well as variability in the properties of the subgrade soils. The structural design of the slab at any location should be determined based on whichever value causes the larger effect, since either the maximum and minimum modulus values may govern for different locations and load effects (e.g., shear force versus bending moments).

The silty clay subgrade is sensitive to disturbance (such as from construction traffic) and a mud slab of lean concrete, at least 50 millimetres in thickness, should be provided on the bearing surfaces after excavation to founding level following review/approval of the bearing surface by geotechnical personnel. Excavations to expose the silty clay subgrade should be carried out using smooth-edged excavator bucket (i.e., no teeth) to minimize disturbance.

5.4.3 Pile Foundations

A piled foundation system could be used to transfer the foundation loads through the silty clay and glacial till to more competent bearing at depth (i.e., down to the bedrock surface).

A suitable pile type would be concrete filled steel pipe piles (driven closed-ended) or H-piles, with the piles end-bearing on bedrock. For this site, the piles would be driven to practical refusal on the bedrock surface which appears to be at a depth of about 34 metres below the existing ground surface (i.e., about 31 metres from the underside of the foundations).

A minimum 0.6 metre thick granular working mat should be provided for pile driving equipment to protect the silty clay subgrade.

It should be noted that the bedrock surface was not confirmed as part of the current investigation. If piled foundations are required, it is recommended that a supplemental geotechnical investigation be carried out, which would include a deep borehole cored into the bedrock.

5.4.3.1 Axial Resistance

As one possible design example, the ULS factored *structural* resistance of a 245-millimetre diameter steel pipe pile with a wall thickness of 9 millimetres may be taken as 1,500 kilonewtons. The ULS factored *geotechnical* resistance of the pile should equal or exceed the structural resistance if the piles are driven to the bedrock, and are installed using an appropriate set criteria and using a hammer of sufficient energy. Note: The pile capacity/size to be used in the design may also be controlled by the dynamic testing program (see later discussion in this section).

For piles end-bearing on or within bedrock, SLS conditions generally do not govern the design since the stresses required to induce 25 millimetres of movement (i.e., the typical SLS criteria) exceed those at ULS. Accordingly, the post-construction settlement of structural elements which derive their support from piles bearing on bedrock should be negligible.

Pipe piles should be equipped with a base plate having a thickness of at least 20 millimetres to limit damage to the pile tip during driving.

The piles should be driven no closer than three pile widths/diameters centre to centre.

The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile, and length of pile; the criteria must therefore be established at the time of construction and after the piling equipment is known. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles will have adequate capacity, but are also not overdriven and damaged. In this regard, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

Relaxation of the piles following the initial set could result from several processes, including:

- Softening of shale bedrock, if present (bedrock geology mapping indicates that the bedrock is primarily limestone);
- The dissipation of negative excess pore water pressures in the overburden material above the bedrock surface; and,
- The driving of adjacent piles.

Provision should therefore be made for restriking all of the piles at least once to confirm the design set and/or the permanence of the set and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first restrike should receive additional restriking until the design set is met. All restriking should be performed after 48 hours of the previous set.

Some of the piles may not fully penetrate the bouldery glacial till to reach the bedrock surface; some of the piles may instead “hang up” at a shallower depth in the glacial till. In that case, pre-drilling of the glacial till could be considered, which would be costly. Alternatively, these particular piles may need to be designed for a reduced capacity. The ULS factored axial resistance of these piles will depend on the depth to which they penetrate and the set that is achieved. The capacities of these piles may have to be confirmed in the field by carrying out load testing.

Due to their smaller cross section, H-piles might have more success in penetrating the glacial till and reaching the bedrock surface. However the integrity of pipe piles following driving may be more readily inspected (by visual examination of the pile interiors) than for H-piles, and therefore damaged piles can be more easily identified. As well, H-piles are typically more expensive. The option of using H-piles could however be discussed with the piling contractor.

It is recommended that dynamic monitoring and capacity testing (known as PDA testing) be carried out (by the contractor) at an early stage in the piling operation to verify both the transferred energy from the pile driving equipment and the load carrying capacity of the piles. As a preliminary guideline, the specification should require that at least 10 percent of the piles be included in the dynamic testing program. CASE method estimates of the capacities should be provided for all piles tested. These estimates should be provided by means of a field report on the day of testing. As well, CAPWAP analyses should be carried out for at least one third of the piles tested, with the results provided no later than one week following testing. The final report should be stamped by a professional engineer licensed in the province of Ontario.

The purpose of the PDA testing will be to confirm that the contractor’s proposed set criteria is appropriate and that the required pile geotechnical capacity is being achieved. It will therefore be necessary for the pile to have sufficient structural capacity to survive that testing, which could require a stronger pile section than would otherwise be required by the design loading.

For example, for the PDA testing to be able to record/confirm a factored geotechnical resistance of 1,500 kilonewtons (per the previously indicated design example), it will be necessary to successfully proof load the tested piles to 3,000 kilonewtons during the PDA testing (per the resistance factor of 0.5 to be applied to PDA test results, as specified in Commentary K of the National Building Code of Canada). However, that proof load may exceed the actual structural capacity of the piles. If the piles fail (structurally) at a lower load, then the full geotechnical capacity cannot be confirmed (and piles will have been damaged and will need to be wasted).

The following options could therefore be considered:

1) Piles with a higher *structural* capacity could be specified (i.e., piles with a ULS factored structural resistance higher than the factored geotechnical resistance, and higher than required by the design loading), so that the piles can be successfully tested to the required loading, so that the geotechnical capacity can then be confirmed by the PDA testing. This option could significantly increase the cost of the piled foundations (due, for example, to the increased wall thickness or diameter of pile that would be used). It might be feasible to use these stronger piles only for those that will be tested, however this option would not permit random testing of the ‘production’ piles, as is typically part of a PDA testing program.

2) A reduced ULS factored geotechnical resistance could be used for the design (e.g., 1,000 kilonewtons instead of 1,500 kilonewtons), such that the piles would have sufficient structural capacity to be loaded to twice the design geotechnical resistance. This option would again increase the cost for the piled foundations, by increasing the number of piles that would be required.

3) Static load testing could be carried out, rather than PDA testing, to confirm the ULS geotechnical resistance of the piles, since the OBC/NBCC specify a resistance factor of 0.6 for static load tests (instead of 0.5). However, it may still not be feasible to prove the full geotechnical resistance.

The foundation and piling specifications should be reviewed by Golder Associates prior to tender and the contractor's submission (i.e., shop drawings, equipment, procedures, and set criteria) should be reviewed by the geotechnical consultant prior to the start of piling. That submission should include a WEAP (Wave Equation Analysis of Piles) analysis of the driveability of the pile, to the design depth, using the contractor's selected hammer.

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at nearby existing structures are maintained below tolerable levels. A maximum peak particle velocity of 50 millimetres per second is recommended for structures.

Piling operations should be inspected on a full time basis by geotechnical personnel to monitor the pile locations and plumbness, initial sets, penetrations on restrike, and to check the integrity of the piles following installation.

5.4.3.2 Resistance to Lateral Loading

Lateral loading could be resisted fully or partially by the use of battered piles.

Alternatively, the resistance to lateral loading could be derived from the soil resistance in front of the piles, and it may be assumed that this resistance will be nearly the same for vertical and inclined piles.

The SLS geotechnical response of the soil in front of the piles under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition).

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h = the constant of horizontal subgrade reaction, as given below;

z = the depth (m); and,

B = the pile diameter/width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where: s_u = the undrained shear strength of the soil (kPa); and,

B = the pile diameter/width (m).

The constant of horizontal subgrade reaction depends on the soil type and soil density/consistency around the pile shaft. For the design of resistance to lateral loads, the values indicated in the table below may be used. The values provided are unfactored geotechnical parameters.

Elevation (m)	Soil Type	n_h (kPa/m)	s_u (kPa)
Pile cap to 85.0	Weathered silty clay crust	-	100
85.0 to 83.0	Stiff grey silty clay	-	50
83.0 to 77.0	Firm grey silty clay	-	35
77.0 to 74.0	Firm grey silty clay	-	45
74.0 to 56.0	Firm to stiff grey silty clay	-	50
56.0 to 54.4	Compact glacial till	4,400	-

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
8d	1.0
6d	0.7
4d	0.4
3d	0.25

The coefficient of horizontal subgrade reaction values calculated as described above may then be used to calculate the lateral deflection of the pile (i.e., the SLS response of the pile), taking into the account the soil-structure interaction.

For establishing the ULS factored *structural* resistance, the shear force and bending moment distribution in the piles under factored loading can be established using these same procedures and parameters for evaluating the SLS response of the pile.

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure.

For individual piles in cohesive soils (i.e., silty clay) the ULS lateral resistance is assumed to vary linearly with a magnitude of $2S_u$ at the surface of the deposit (i.e., the underside of pile cap level) and a magnitude of $9S_u$ at a depth equal to three pile diameters below the underside of the pile cap (where S_u is the previously provided undrained shear strength). Below a depth equal to 3 pile diameters, and to the bottom of the deposit, the lateral resistance is assumed to be constant at $9S_u$.

The ULS lateral resistance of a pile group may be estimated as the sum of the individual resistances across the face of the group, perpendicular to the direction of the applied lateral force.

The ULS resistances obtained using the above parameters represent unfactored values; a resistance factor of 0.5 should be applied in calculating the horizontal resistance.

If uplift resistance is required, the piles would have some capacity which could be relied upon. Rock anchors could also be used, but the significant depth to the bedrock surface could make that an expensive option. Further geotechnical input on both issues can be provided, if required.

5.4.4 Frost Protection

All perimeter and exterior foundation elements or interior foundation elements in unheated areas should be provided with a minimum of 1.5 metres of earth cover for frost protection purposes. Isolated, unheated exterior footings/pile caps adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 metres of earth cover.

It is expected that these requirements will be satisfied for all of the foundation elements of the building due to the deep founding level required to accommodate the 1 level of underground parking. It is also assumed that the basement level will be heated.

5.4.5 Seismic Design

The seismic design provisions of the 2012 OBC depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or rock below founding level. The OBC permits the Site Class to be specified based solely on the stratigraphy and in situ testing data (i.e., shear strengths and standard penetration test results), rather than from direct measurements of the shear wave velocity.

Based on the in situ testing data, this site can be assigned a Site Class of D for seismic design purposes.

5.5 Basement Floor Slab

The following guidelines are provided on the basis that a ‘drained’ foundation system will be provided; i.e., that a water tight foundation is not to be provided.

In preparation for the construction of the basement floor slab, all loose, wet, and disturbed material should be removed from beneath the floor slab.

If spread footings or a piled foundation system are used, then either a slab-on-grade concrete floor slab, or an asphalt surfaced pavement can be provided.

In both cases, provision should be made for a drainage layer consisting of at least 300 millimetres of free draining granular material, such as 16 millimetre clear crushed stone, to underlie the floor slab or pavement. To prevent hydrostatic pressure build up, this granular layer should be drained. This should be achieved by installing rigid 100 millimetre diameter perforated pipes in the floor slab bedding at 6 metre centres. The perforated pipes should discharge to a positive outlet such as a sump from which the water is pumped.

If or where an asphalt surface will be provided for the basement level, at least 150 millimetres of OPSS Granular A base should be provided above the clear stone, compacted to at least 100 percent of the material’s standard Proctor maximum dry density.

If a raft foundation is provided, then the raft slab will also form the floor slab. Although that raft would not necessarily be designed to resist water pressures (since a ‘drained’ foundation system is being provided), it could presumably accommodate some level of water pressure build-up. The perimeter drainage system will permanently lower the groundwater level along the edge of the raft, but some higher water pressures could persist beneath the central portion of the raft. Groundwater levels as high as elevation 86.6 metres have been recorded in the monitoring wells. The raft should therefore be designed to accommodate that level of piezometric pressure. The weight of the raft may, in fact, be sufficient to resist that pressure. Alternatively, a drainage layer could be provided beneath the entire raft footprint. Similar to as described for the piled foundation option, the drainage layer could consist of at least 300 millimetres of free draining granular material, such as 16 millimetre clear crushed stone, with overlaps of at least 0.5 metres between rolls. Drainage pipes are not considered necessary in this case, however the drainage layer must be hydraulically continuous with the perimeter drainage system.

5.6 Foundation Wall Backfill and Lateral Earth Pressure

The backfill and drainage requirements for basement walls, as well as the lateral earth pressures, will depend on the type of excavation that is made to construct the basement levels. The following guidelines apply to any portions of the basement walls made in open cut excavations. If temporary shoring is used, additional geotechnical guidelines would be required.

The following guidelines are also provided on the basis that the structure foundations are designed to be ‘drained’.

The soils at this site are frost susceptible and should not be used as backfill against exterior, unheated, or well insulated foundation elements within the depth of potential frost penetration (1.5 metres) to avoid problems with frost adhesion and heaving. Free draining backfill materials are also required if hydrostatic water pressure against the basement walls (and potential leakage) is to be avoided. The foundation and basement walls therefore should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular B Type I.

To avoid ground settlements around the foundations, which could affect site grading and drainage, all of the backfill materials should be placed in maximum 300 millimetre thick lifts, compacted to at least 95 percent of the material’s standard Proctor maximum dry density.

The basement wall backfill (for the full height of the wall) should be drained by means of a perforated pipe subdrain in a surround of 19 millimetres clear stone, fully wrapped in a geotextile, which leads by positive drainage to a storm sewer or to a sump from which the water is pumped.

The magnitude of the lateral earth pressures will depend on the backfill materials and backfill conditions adjacent to the foundation walls. If the backfill materials for open cut excavations consist of compacted sand or sand and gravel (OPSS Granular ‘B’ Type I or II), as described above, then the lateral earth pressures may be taken as:

$$\sigma_h(z) = K_o (\gamma z + q)$$

Where: $\sigma_h(z)$ = Lateral earth pressure on the wall at depth z, kilopascals;

K_o = At-rest earth pressure coefficient, use 0.5;

γ = Unit weight of retained soil, use 20 kilonewtons per cubic metre;

z = Depth below top of wall, metres; and

q = Uniform surcharge at ground surface to account for traffic and equipment (not less than 15 kilopascals), plus any surcharge due to adjacent foundation loads.

These lateral earth pressures would increase under seismic loading conditions. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) for design may be determined as follows:

$$\sigma_h(z) = K_o \gamma z + (K_{AE} - K_o) \gamma (H\text{-}z)$$

Where: $\sigma_h(d)$ = Lateral earth pressure at depth z, kilopascals;

K_{AE} = Seismic earth pressure coefficient, use 0.8; and,

H = Total height of the wall, metres.

It should be noted that all of the lateral earth pressure equations are given in an unfactored format and will need to be factored for ULS design purposes.

It should also be noted that the above lateral earth pressure equations assume that the foundation walls will be drained. If the walls are design to be water-tight, the walls will have to be designed to resist the additional hydro-static pressure.

It has been assumed that the underground parking level will be maintained at minimum temperatures but will not be permitted to freeze. If these areas are to be unheated, additional guidelines for the design of the basements walls will need to be provided.

In areas where pavement or other hard surfacing will about the building, differential frost heaving could occur between the granular fill immediately adjacent to the building and the more frost susceptible materials beyond the wall backfill. To reduce the severity of this differential heaving, the backfill adjacent to the wall should be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 metres below finished exterior grade level at a slope of 3H:1V, or flatter, away from the wall. The granular fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.7 Basement Entrance Ramp - Retaining Walls

The foundation design guidelines provided in Section 5.4 of this report are applicable to the design of the basement entrance ramp retaining walls.

The soils at this site are considered to be frost susceptible. The frost penetration depth for design should be taken as 1.8 metres. Insulating the bearing surface with high density extruded polystyrene (foam) insulation could also be considered to provide frost protection where 1.8 metres of earth cover cannot be provided. Further details on the insulation can be provided, if and when required.

The retaining walls should be backfilled with free draining non-frost susceptible sand or sand and gravel meeting the requirements for OPSS Granular B Type I. The granular fill should be placed within the wedge-shaped zone defined by a line drawn at 1 horizontal to 1 vertical extending up and back from the rear face of the wall footing. Where that backfill will underlie paved surfacing above/behind the wall, the backfill within the depth of frost penetration (1.8 metres) should be provided with a frost taper which slopes up to the pavement subgrade level at an inclination of 3 horizontal to 1 vertical. The frost taper will help limit the severity of the differential heaving between the pavement surface above the non-frost susceptible backfill and areas underlain with more frost susceptible subgrade soils.

The backfill should be compacted to 95 percent of the materials standard Proctor maximum dry density. Small vibratory compaction equipment should be used within about 0.5 metres of the wall to minimize compaction induced stresses.

Longitudinal drains should be installed at footing level to provide positive drainage of the granular backfill by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in geotextile, which leads by gravity drainage to a positive outlet. Weep holes through the base of the walls, as an additional drainage measure, would also be appropriate.

Retaining walls backfilled with granular material and effectively drained (as described above) should be designed to resist lateral earth pressures calculated using a triangular distribution of the stress with a magnitude of:

$$\sigma_H(z) = K_a (\gamma z + q)$$

Where: $\sigma_h(z)$ = Lateral earth pressure on the wall at depth z, kilopascals;

K_a = Active earth pressure coefficient, 0.33;

γ = Unit weight of retained soil, 22 kilonewtons per cubic metre;

z = Depth below top of wall, metres; and,

q = Uniform surcharge at ground surface behind the wall to account for traffic, equipment, or stockpiled soil (use 15 kilopascals).

These lateral earth pressures would increase under seismic loading conditions. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The combined pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(z) = K_a \gamma z + (K_{AE} – K_a) \gamma (H-z)$$

Where: K_{AE} = The seismic earth pressure coefficient, use 0.5; and,

H = The total depth to the bottom of the foundation wall (metres).

All of the lateral earth pressure equations are given in an unfactored format and will need to be factored for Limit States Design purposes.

The above seismic design parameters are consistent with the wall being an unrestrained structure. For a retaining wall to be considered as an unrestrained structure under seismic conditions, the wall should be capable of displacing 100 millimetres outward under seismic conditions.

5.8 Site Servicing

At least 150 millimetres of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface occurs during construction, it may be necessary to place a sub-bedding layer consisting of 300 millimetres of compacted OPSS Granular B Type II beneath the Granular

A. The bedding material should, in all cases, extend to the spring line of the pipe and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials and native soils could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from the spring line of the pipe to at least 300 millimetres above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 millimetres. The cover material should be compacted to at least 95 percent of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the existing inorganic fill and weathered silty clay as trench backfill. Where the trench will be covered with hard surfaced areas, the type of material placed in the frost zone (between subgrade level and 1.8 metres depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.9 Pavement Design

In preparation for pavement construction, all topsoil and any unsuitable fill (i.e., fill containing organic matter) should be excavated from the pavement areas for predictable pavement performance.

Those portions of the fill not containing organic matter may be left in place provided that some long term settlement of the pavement surface can be tolerated. However, the surface of the fill material at subgrade level should be proof rolled with a heavy smooth drum roller under the supervision of qualified geotechnical personnel to compact the surface of the existing fill and to identify soft areas requiring sub-excavation and replacement with more suitable fill.

Areas requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material. The existing inorganic fill on site may be suitable for this purpose, but would need to be confirmed by the geotechnical engineer at the time of construction. Grade raise fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable compaction equipment.

The surface of the subgrade or fill should be crowned to promote drainage of the pavement granular structure. Perforated pipe subdrains should be provided at subgrade level extending from the catch basins for a distance of at least 3 metres in four orthogonal directions, or longitudinally where parallel to a curb.

The pavement structure for car parking areas should consist of:

Pavement Component	Thickness (mm)
Asphaltic Concrete	50
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	300

The pavement structure for access roadways and truck traffic areas should consist of:

Pavement Component	Thickness (mm)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	450

The granular base and subbase materials should be uniformly compacted to at least 100 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in accordance with OPSS 310.

The composition of the asphaltic concrete pavement in car parking areas should be as follows:

- Superpave 12.5 Surface Course – 50 millimetres

The composition of the asphaltic concrete pavement in access roadways and truck traffic areas should be as follows:

- Superpave 12.5 Surface Course – 40 millimetres
- Superpave 19.0 Binder Course – 50 millimetres

The pavement design should be based on a Traffic Category of Level B. The asphalt cement used on this project should be made with PG 58-34 asphalt cement on all lifts.

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill and grade raise fill have been adequately compacted to the required densities and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.

5.10 Corrosion and Cement Type

One sample of groundwater from the monitoring well installed in borehole 16-2 was submitted to Paracel Laboratories for basic chemical analyses related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements. The results of this testing are provided in Appendix B.

The results indicate that concrete made with Type GU Portland cement should be acceptable for substructures. The results also indicate a potential for corrosion of exposed ferrous metal, which should be considered in the design of substructures.

5.11 Trees

The silty clay on this site is potentially sensitive to water depletion by trees of high water demand during periods of dry weather. When trees draw water from the clayey soil, the clay undergoes shrinkage which can result in settlement of adjacent structures. The radial zone of influence of a tree is conventionally considered to be approximately equal to the height of the tree. Therefore, some restrictions will therefore need to be imposed on the planting of trees of higher water demand in close proximity to the foundations on this site. Trees which have a high water demand should not be planted closer to structures than the ultimate height of the tree. This restriction could potentially be relaxed if it can be relaxed if it can be shown that the soils have a low shrinkage potential. However, additional testing is required before this decision could be made. Table 1 provides a list of the common trees in decreasing order of water demand and, accordingly, decreasing risk of potential effects on structures.

6.0 ADDITIONAL CONSIDERATIONS

The soils at this site are sensitive to disturbance from ponded water, construction traffic, and frost.

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that soil having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction view point.

Ontario Regulation 903 would ultimately require abandonment of the monitoring wells installed for this investigation. However, these devices may be useful during construction. It is therefore proposed that decommissioning of these devices be made part of the construction contract.

At the time of the writing of this report, only conceptual details for the proposed development were available. Golder Associates should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

7.0 CLOSURE

We trust that this report meets your current needs. If you have any questions, or if we may be of further assistance, please do not hesitate to contact the undersigned.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
S. W. DUNLOP
100151238
Oct.23/17
PROVINCE OF ONTARIO

Stephen Dunlop, P.Eng.
Geotechnical Engineer

Troy Skinner, P.Eng.
Associate, Geotechnical Engineer

KM/SD/TMS/mvrd
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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, **Claridge Homes Corporation.** The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

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The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

TABLE 1

SOME COMMON TREES IN DECREASING ORDER OF WATER DEMAND

BROAD LEAVED DECIDUOUS

Poplar

Alder

Aspen

Willow

Elm

Maple

Birch

Ash

Beech

Oak

DECIDUOUS CONIFER

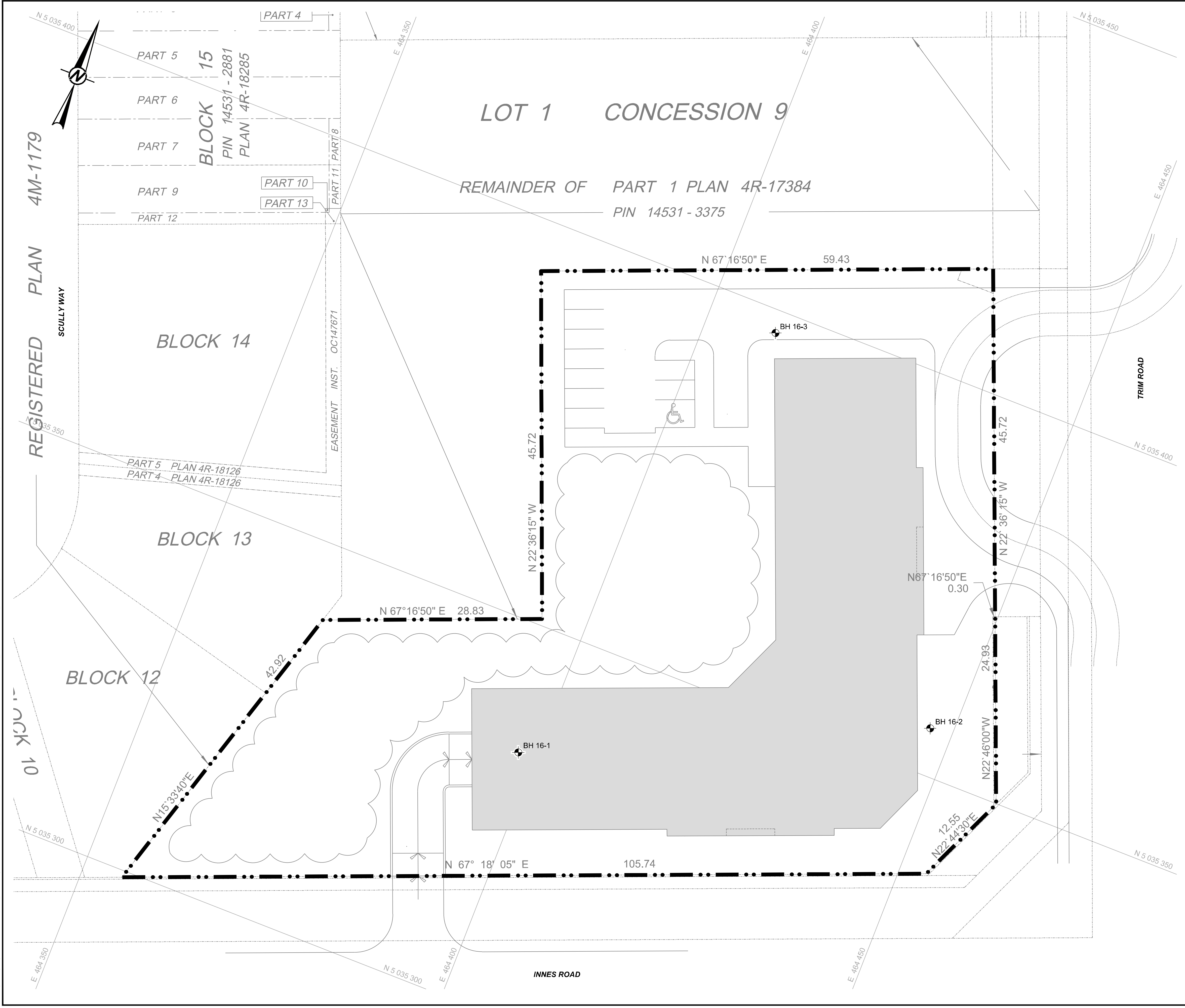
Larch

EVERGREEN CONIFERS

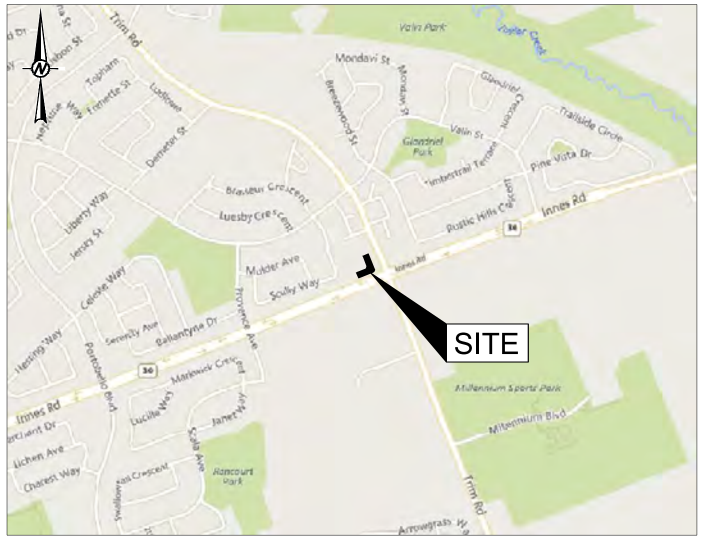
Spruce

Fir

Pine



KEY MAP



SCALE 1:25,000

LEGEND

◈	APPROXIMATE BOREHOLE LOCATION
▬ • • ▬	SITE BOUNDARY

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 1664873

REFERENCE(S)

1. BASE MAP SUPPLIED IN ELECTRONIC FORMAT BY CLARIDGE HOMES CORPORATION
2. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83, COORDINATE SYSTEM: UTM ZONE 18, VERTICAL DATUM: CGVD28

0 10 20
1:500 METRES

CLIENT
CLARIDGE HOMES CORPORATION

PROJECT
GEOTECHNICAL INVESTIGATION
PROPOSED RETIREMENT RESIDENCE
INNES ROAD AND TRIM ROAD, OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT
Golder Associates

YYYY-MM-DD	2016-11-21
DESIGNED	----
PREPARED	JM
REVIEWED	SD
APPROVED	TMS

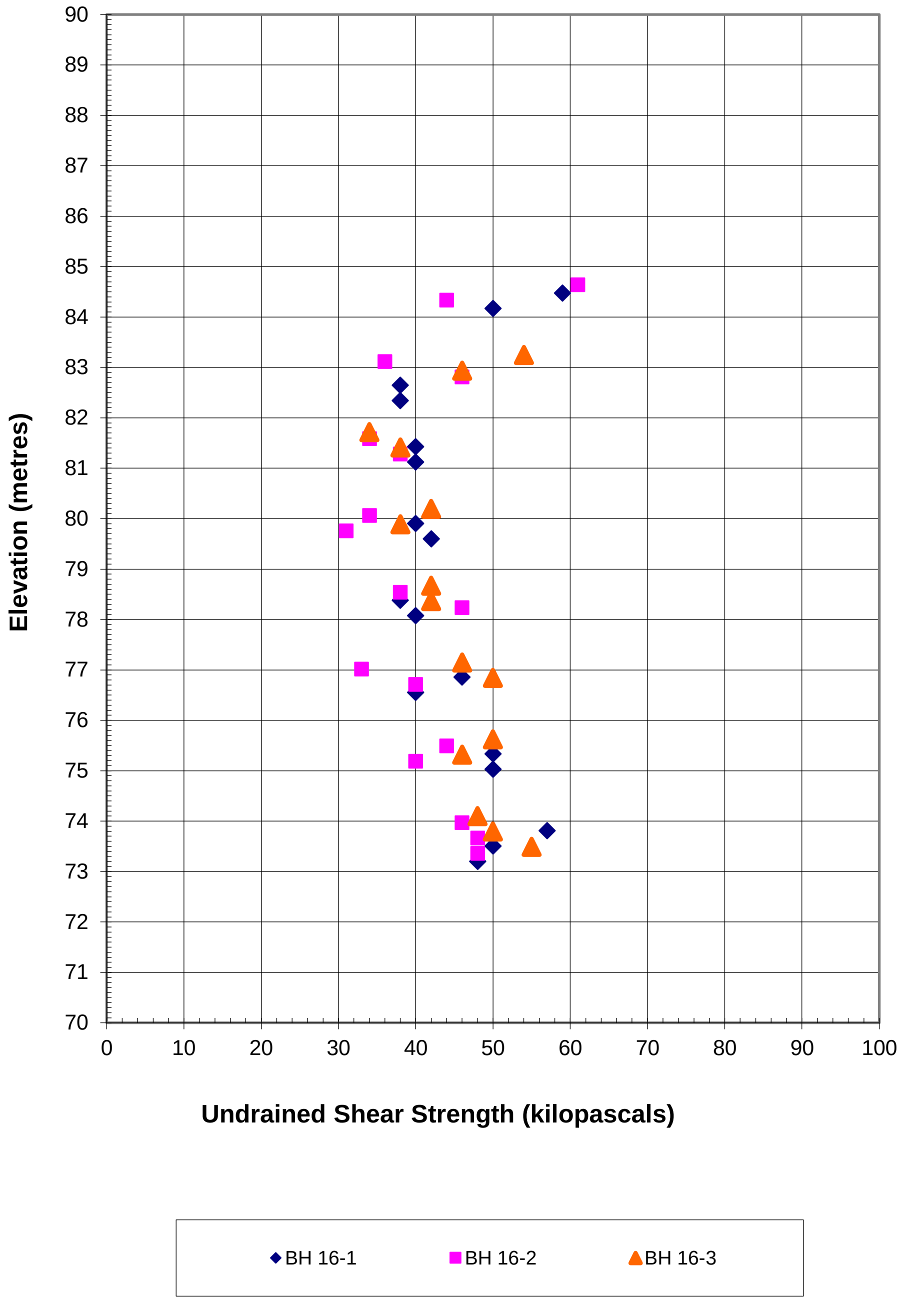
PROJECT NO.	PHASE	REV.	FIGURE
1664873	1000	A	1

25 mm

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

SUMMARY OF UNDRAINED SHEAR STRENGTHS VERSUS ELEVATION

FIGURE 2



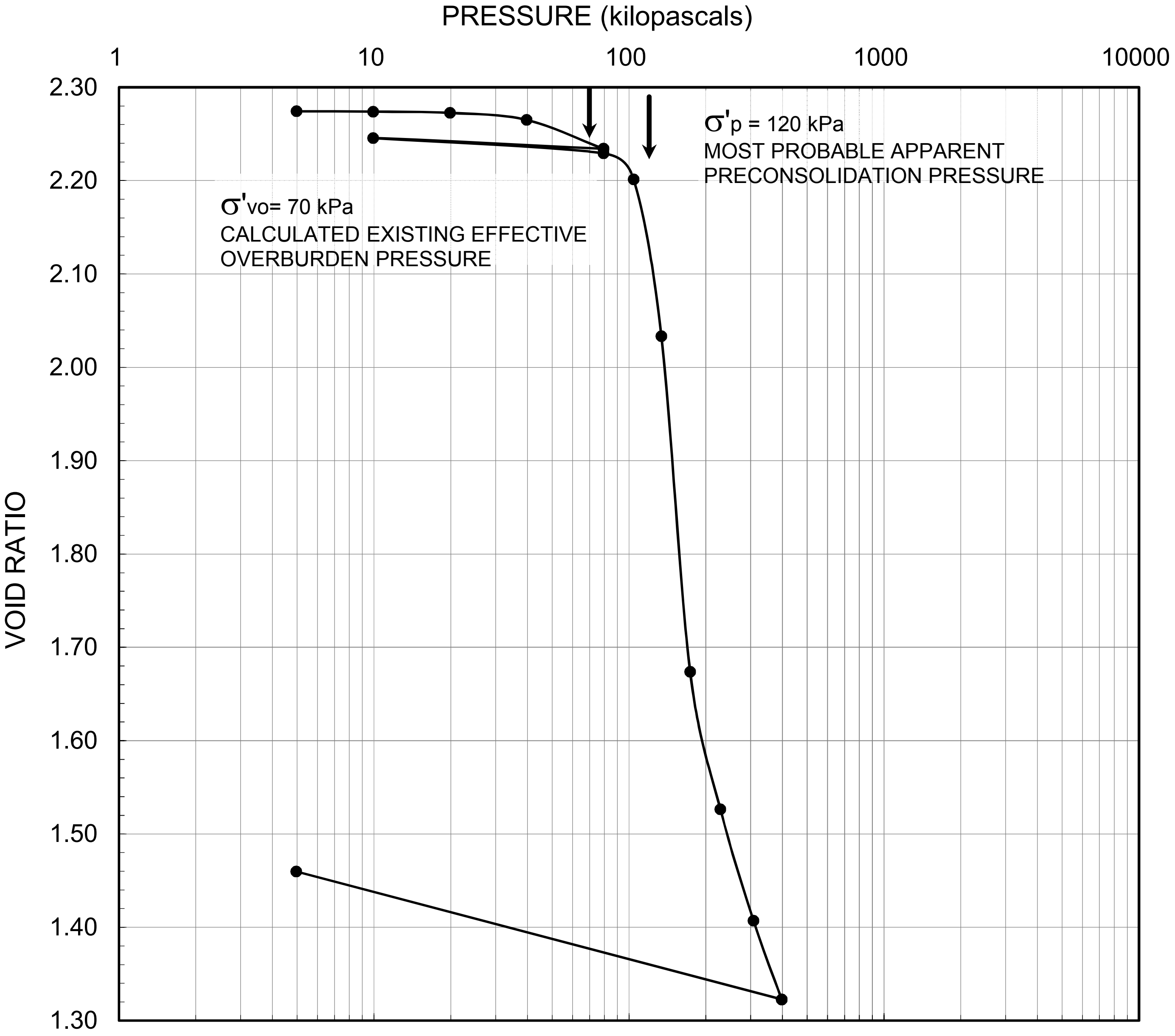
Date December 2016

Project 1664873

Golder Associates

Drawn SM

Chkd SD



σ'p = 120 kPa
MOST PROBABLE APPARENT
PRECONSOLIDATION PRESSURE

σ'vo= 70 kPa
CALCULATED EXISTING EFFECTIVE
OVERBURDEN PRESSURE

LEGEND

Borehole:	16-2	w_i = 81%	S_o = 99%	γ = 15.1 kN/m³
Sample:	8	w_f = 52%	e_o = 2.27	G_s = 2.78
Depth (m):	8.1	w_l = 67%	C_c = 3.10	
Elevation (m):	80.5	w_p = 26%	C_r = 0.018	

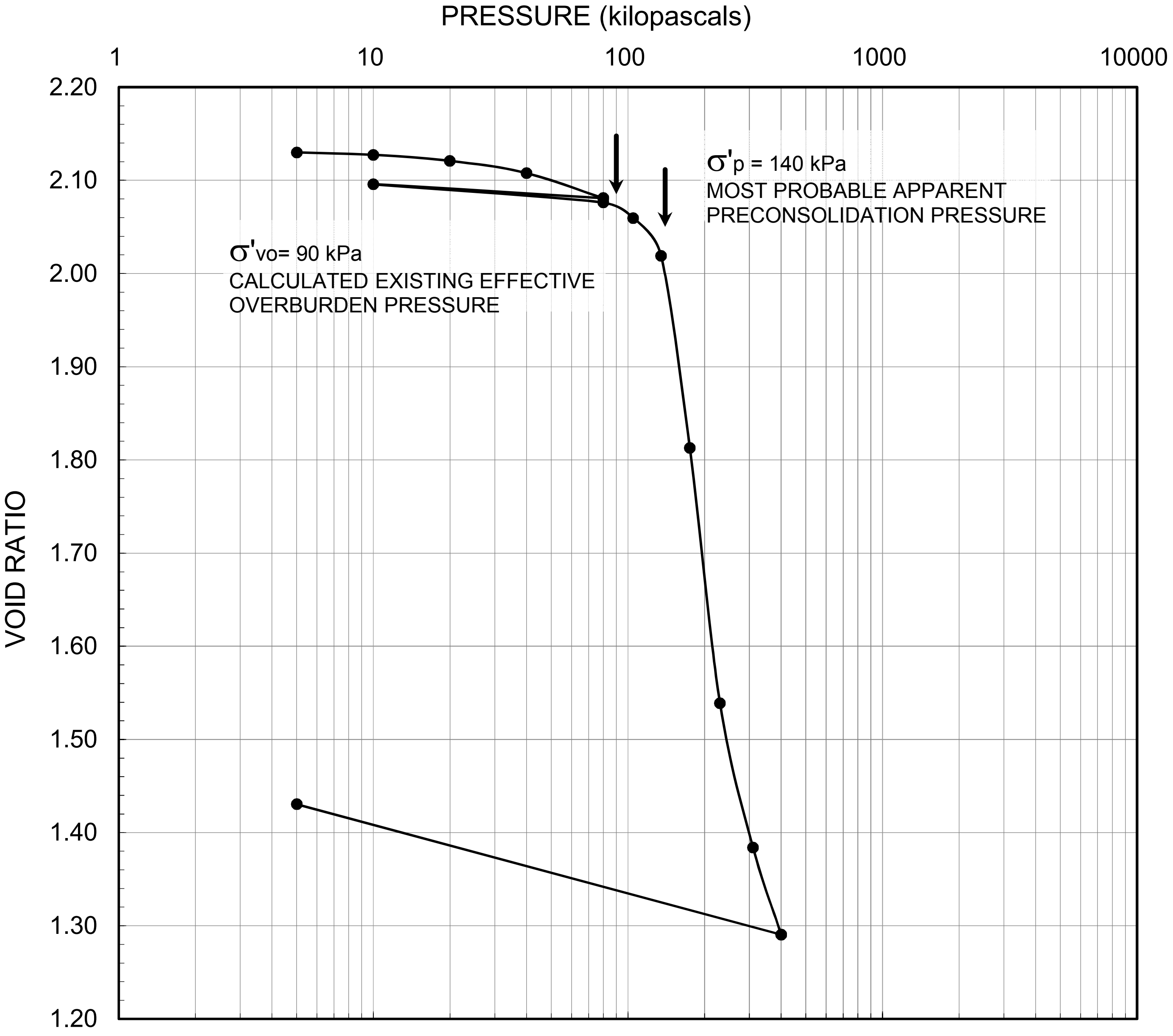
Golder Associates

SCALE AS SHOWN
DATE 12/14/16
CADD N/A
ENTERED MI
CHECK CNM
REVIEW SD

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary
PROJECT No. 1664873 REV. 1
FIGURE 3



LEGEND

Borehole:	16-2	w_i	= 77%	S_o	= 100%	γ	= 15.3 kN/m³
Sample:	10	w_f	= 53%	e_o	= 2.13	G_s	= 2.77
Depth (m):	11.1	w_l	= 57%	C_c	= 2.30		
Elevation (m):	77.5	w_p	= 28%	C_r	= 0.021		

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SCALE AS SHOWN
DATE 12/14/16
CADD N/A
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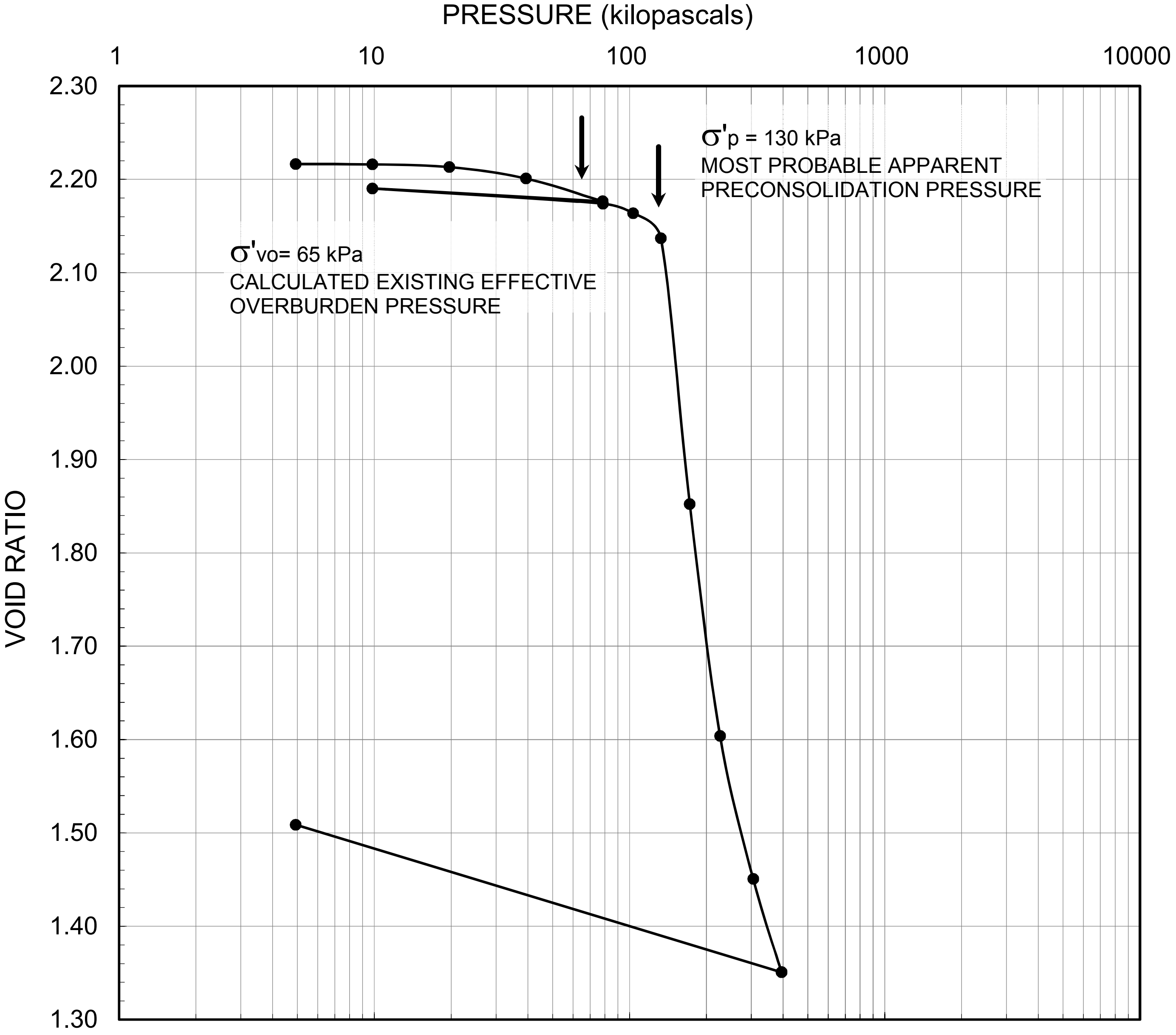
REVIEW SD
FILE No. Consolidation summary
PROJECT No. 1664873
REV. 1

TITLE

CONSOLIDATION TEST RESULTS

FIGURE 4



LEGEND

Borehole:	16-3	w_i = 80%	S_o = 100%	γ = 15.2 kN/m³
Sample:	8	w_f = 55%	e_o = 2.22	G_s = 2.78
Depth (m):	6.6	w_l = 74%	C_c = 2.50	
Elevation (m):	82.1	w_p = 27%	C_r = 0.018	

Golder Associates

SCALE AS SHOWN
DATE 12/14/16
CADD N/A
ENTERED MI
CHECK CNM
REVIEW SD

FILE No. Consolidation summary
PROJECT No. 1664873 REV. 1

TITLE

CONSOLIDATION TEST RESULTS

FIGURE 5

APPENDIX A

List of Abbreviations and Symbols
Record of Borehole Sheets

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL, SAND and CLAY)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.).

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size
TP	Thin-walled, piston – note size
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 - 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects.
2. Definition of compactness descriptions based on SPT 'N' ranges from Terzaghi and Peck (1967) and correspond to typical average N_{60} values.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan\delta$
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1 $\tau = c' + \sigma' \tan\phi'$

2 shear strength = (compressive strength)/2

PROJECT: 1664873

RECORD OF BOREHOLE: 16-1

SHEET 1 OF 2

LOCATION: See Site Plan

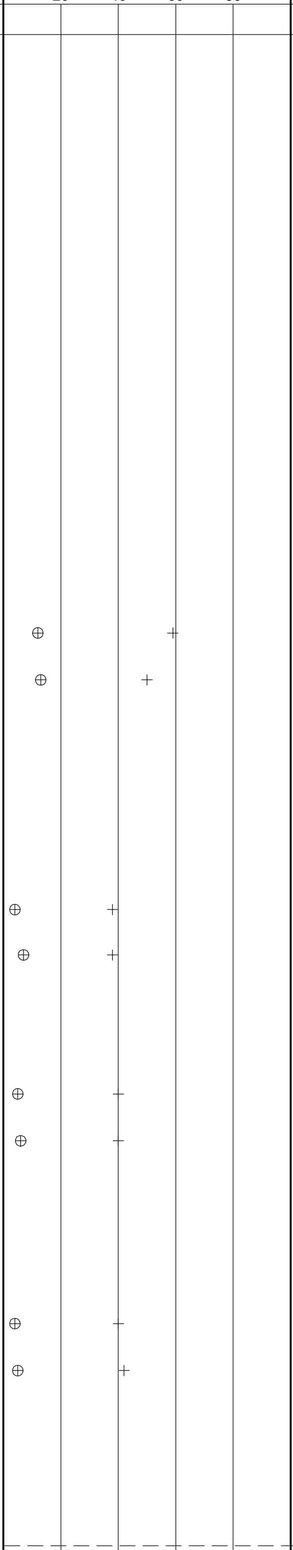
BORING DATE: October 26, 2016

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		88.44					
0	Power Auger 200 mm Diam. (Hollow Stem)	(CL/CI) SILTY CLAY; grey brown, contains roots (WEATHERED CRUST); cohesive, w~PL, very stiff	▨	0.00	1	SS	6		
1					2	SS	8		
2					3	SS	8		
					4	SS	6		
3					5	SS	2		
		(CI/CH) SILTY CLAY to CLAY; grey with black mottling; cohesive, w>PL, firm to stiff	▨	84.78 3.66					
5					6	SS	WH		
					7	SS	WH		
6					8	SS	WH		
8					9	SS	WH		
9					10	SS	WH		
10		*CONTINUED NEXT PAGE*							



HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl; 20 40 60 80

DEPTH SCALE

1 : 50

LOGGED: DG

CHECKED: KPH

PROJECT: 1664873

RECORD OF BOREHOLE: 16-1

SHEET 2 OF 2

LOCATION: See Site Plan

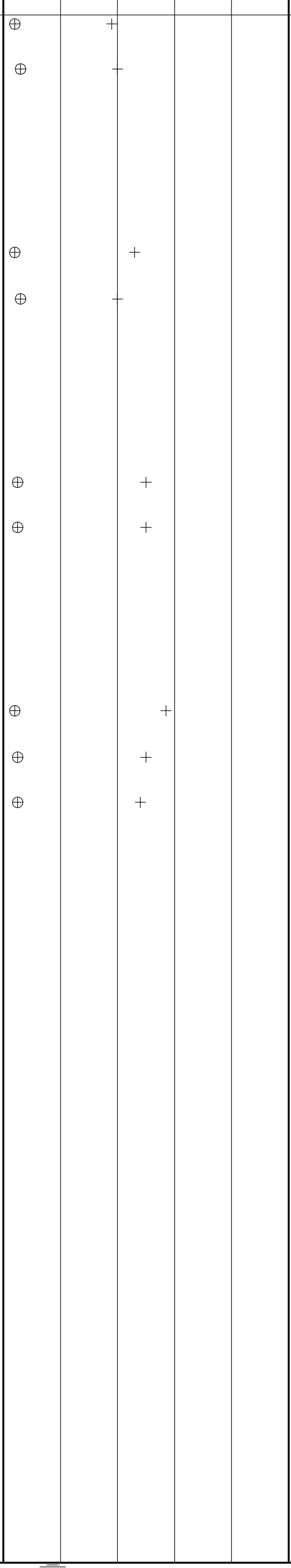
BORING DATE: October 26, 2016

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Power Auger 200 mm Diam. (Hollow Stem)	(CI/CH) SILTY CLAY to CLAY; grey with black mottling; cohesive, w>PL, firm to stiff	▨								
11					11	SS	WH				
12					12	SS	WH				
13											
14					13	SS	WH				
15				73.20 15.24							
		End of Borehole									
16											
17											
18											
19											
20											



MIS-BHS 001 1664873.GPJ GAL-MIS.GDT 12/20/16 JM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: DG

CHECKED: KPH

PROJECT: 1664873

RECORD OF BOREHOLE: 16-2

SHEET 1 OF 2

LOCATION: See Site Plan

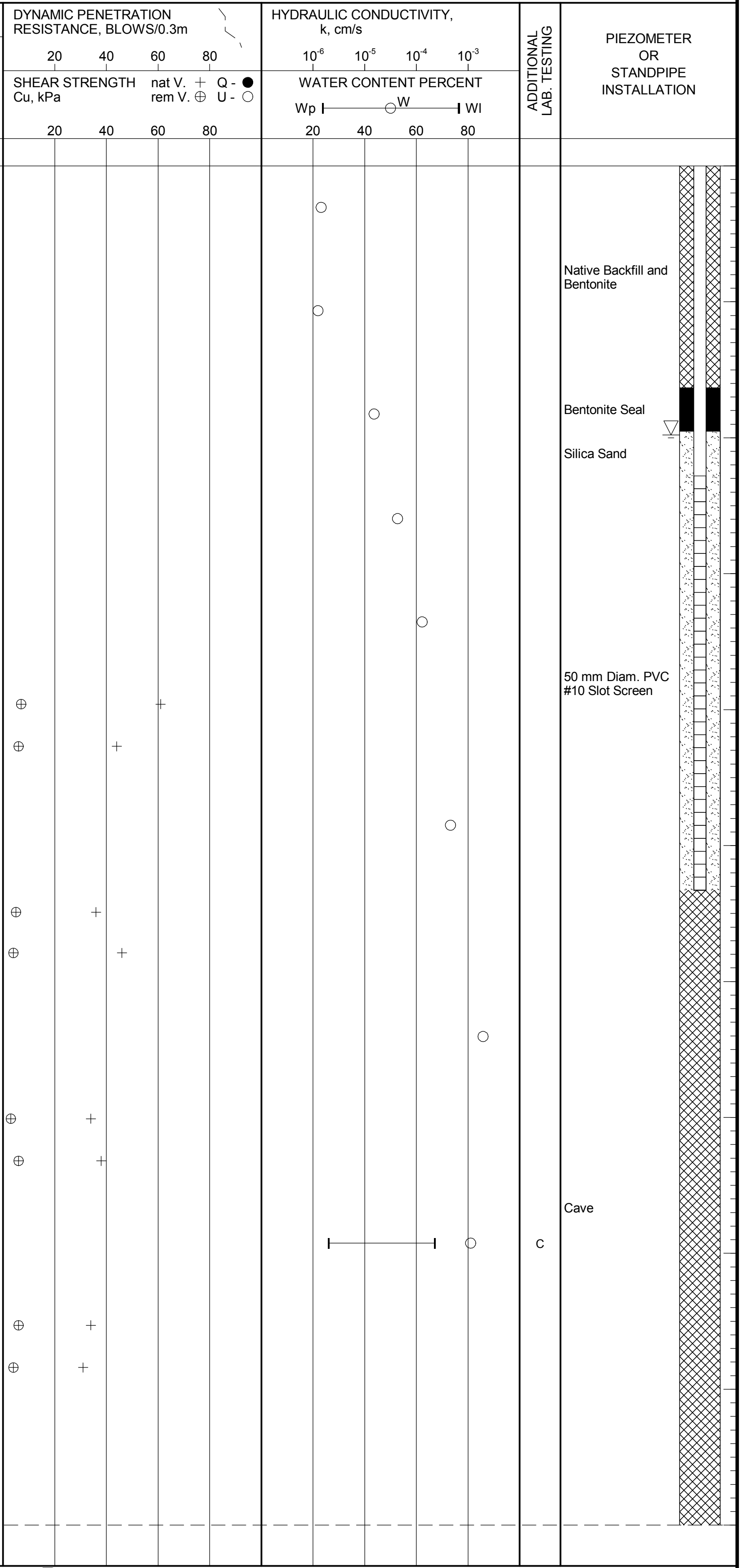
BORING DATE: October 26-27, 2016

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
		GROUND SURFACE		88.60				
0	Power Auger 200 mm Diam. (Hollow Stem)	FILL - (CL) SILTY CLAY, trace gravel; grey brown, contains roots; cohesive, w>PL	▨	0.00	1	SS	10	
1		(CL/CI) SILTY CLAY; grey brown (WEATHERED CRUST); cohesive, w~PL, very stiff to stiff	▨	87.84 0.76	2	SS	13	
2					3	SS	8	
3					4	SS	4	
					5	SS	1	
4		(CI/CH) SILTY CLAY; grey with black mottling; cohesive, w>PL, firm to stiff	▨	84.94 3.66				
5					6	TP	PH	
6					7	SS	WH	
8					8	TP	PH	C
9					9	SS	WH	
10		*CONTINUED NEXT PAGE*						



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: DG

CHECKED: KPH

PROJECT: 1664873

RECORD OF BOREHOLE: 16-2

SHEET 2 OF 2

LOCATION: See Site Plan

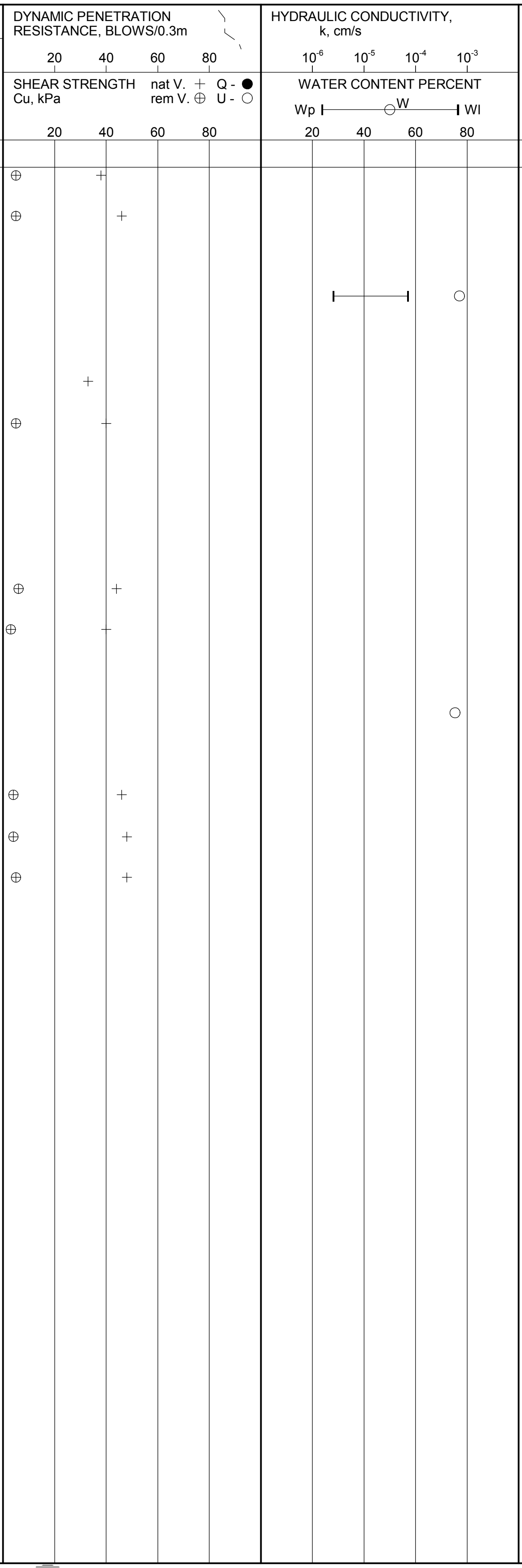
BORING DATE: October 26-27, 2016

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---							
10	Power Auger 200 mm Diam. (Hollow Stem)	(CI/CH) SILTY CLAY; grey with black mottling; cohesive, w>PL, firm to stiff	▨						
11					10	TP	PH	C	
12					11	SS	WH		Cave
13									
14					12	SS	WH		
15		End of Borehole		73.36 15.24					WL in Screen at Elev. 86.62 m on November 10, 2016
16									
17									
18									
19									
20									



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: DG

CHECKED: KPH

PROJECT: 1664873

RECORD OF BOREHOLE: 16-3

SHEET 1 OF 4

LOCATION: See Site Plan

BORING DATE: October 27-28, 2016

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
		GROUND SURFACE	88.73				
0	Power Auger 200 mm Diam. (Hollow Stem)	FILL - (CL) SILTY CLAY, trace gravel and sand; grey brown, contains roots; cohesive, w~PL	0.00	1	SS	6	
1				2	SS	7	
		(CL/CI) SILTY CLAY, trace sand; grey brown (WEATHERED CRUST); cohesive, w~PL, very stiff	87.21 1.52	3	SS	6	
2				4	SS	11	
3				5	SS	4	
		(CI/CH) SILTY CLAY to CLAY; grey, with black mottling; cohesive, w>PL, firm to stiff	85.07 3.66	6	SS	1	
5				7	SS	WH	
6				8	TP	PH	C
8				9	SS	WH	
9				10	TP	PH	
10							

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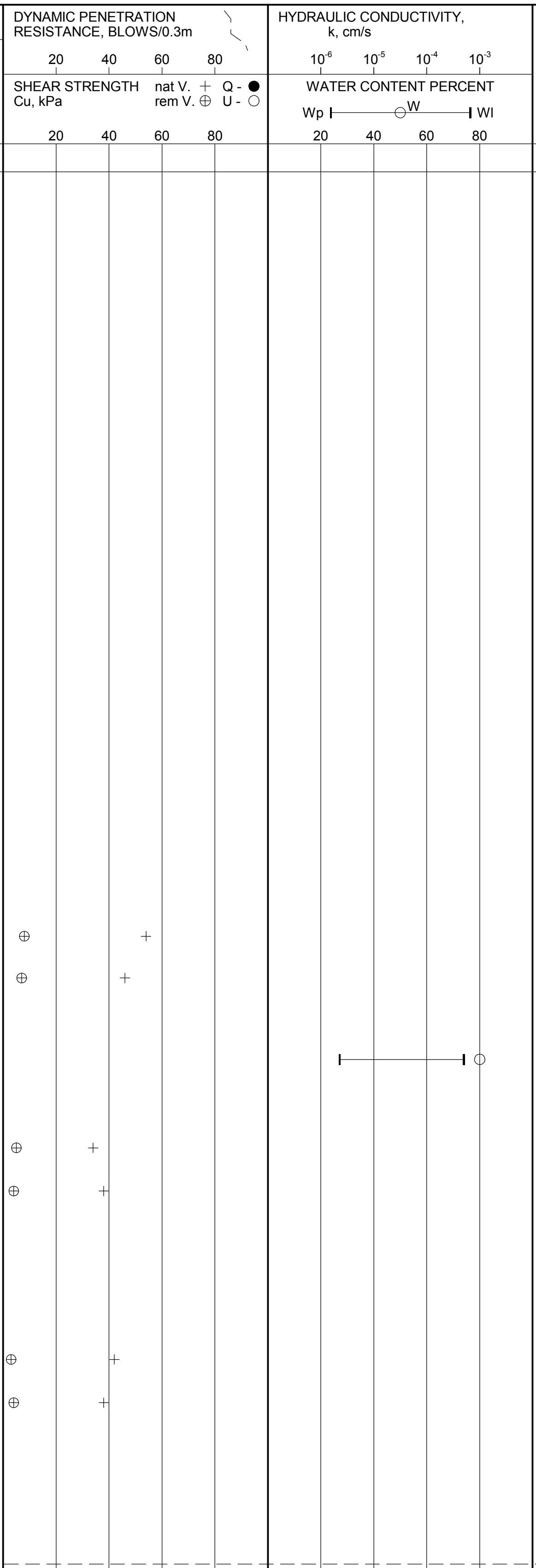
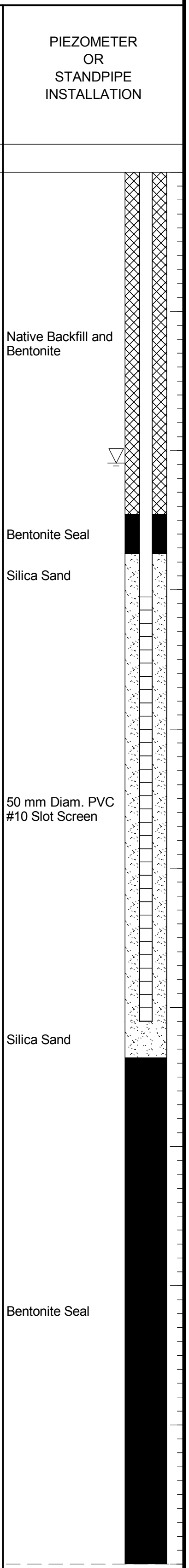




DEPTH SCALE

1 : 50

Golder Associates

LOGGED: DG

CHECKED: KPH

PROJECT: 1664873

RECORD OF BOREHOLE: 16-3

SHEET 2 OF 4

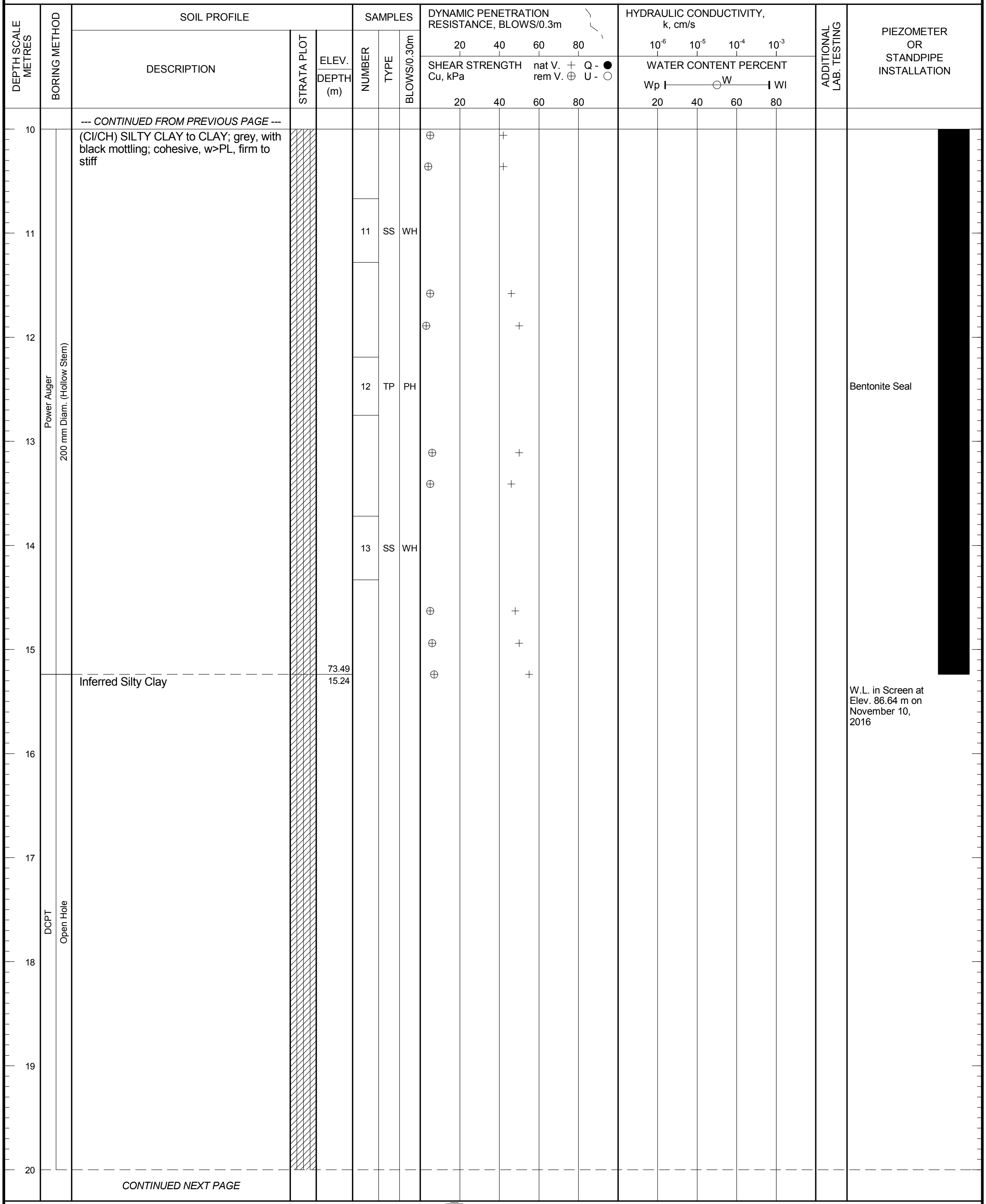
LOCATION: See Site Plan

BORING DATE: October 27-28, 2016

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm



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(CI/CH) SILTY CLAY to CLAY; grey, with black mottling; cohesive, w>PL, firm to stiff

73.49
15.24

Inferred Silty Clay

NUMBER	TYPE	BLOWS/0.30m
11	SS	WH
12	TP	PH
13	SS	WH

Bentonite Seal

W.L. in Screen at Elev. 86.64 m on November 10, 2016

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

LOGGED: DG
CHECKED: KPH

PROJECT: 1664873

RECORD OF BOREHOLE: 16-3

SHEET 3 OF 4

LOCATION: See Site Plan

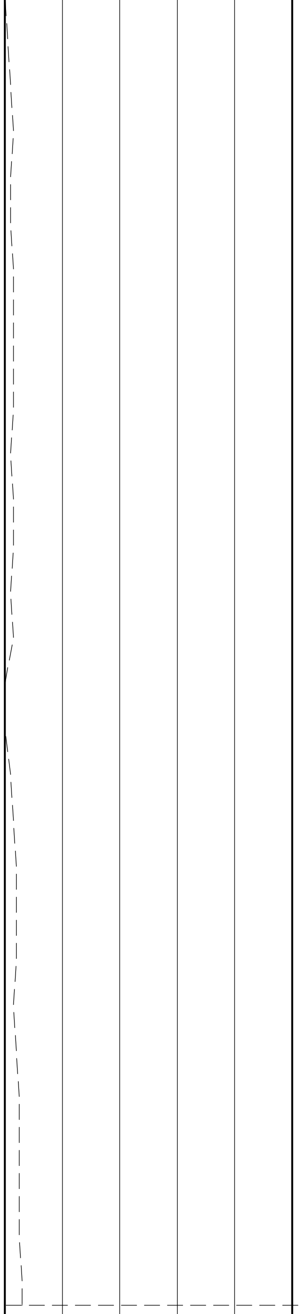
BORING DATE: October 27-28, 2016

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
20	DCPT Open Hole	Inferred Silty Clay	▨								
21											
22											
23											
24											
25											
26											
27											
28											
29											
30											
		CONTINUED NEXT PAGE									



DEPTH SCALE

1 : 50

LOGGED: DG

CHECKED: KPH

PROJECT: 1664873

RECORD OF BOREHOLE: 16-3

SHEET 4 OF 4

LOCATION: See Site Plan

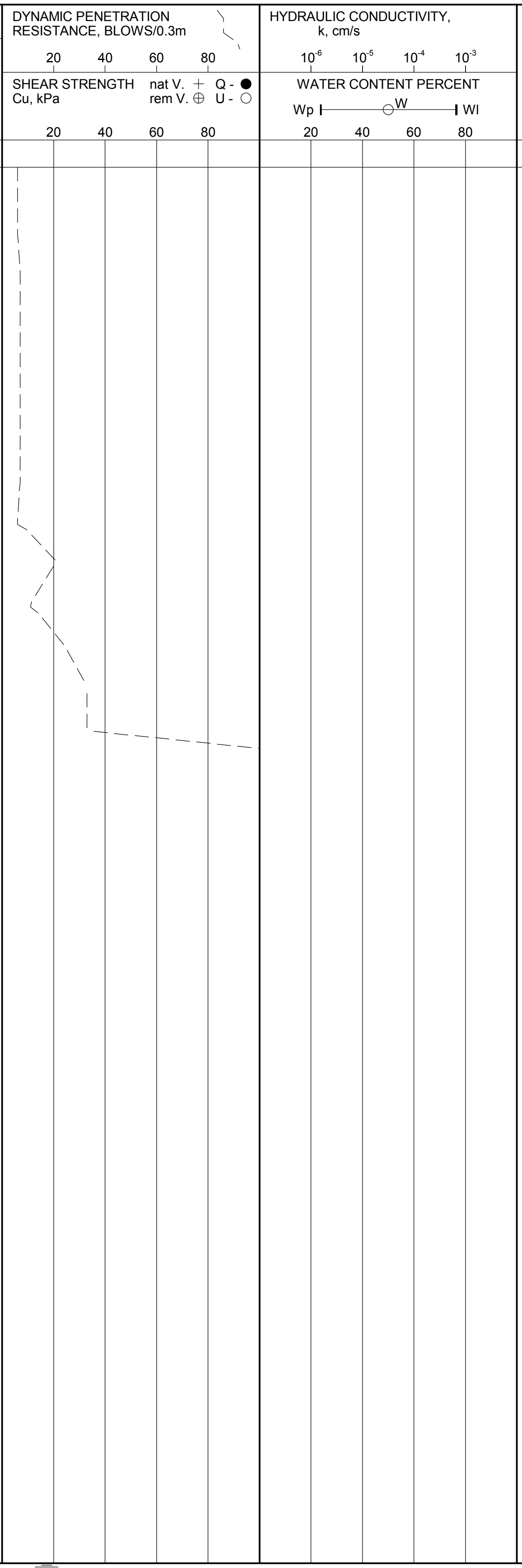
BORING DATE: October 27-28, 2016

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---							
30	DCPT Open Hole	Inferred Silty Clay	▨	54.44 34.29					
		End of Borehole DCPT Refusal							
31									
32									
33									
34									
35									
36									
37									
38									
39									
40									



DEPTH SCALE
1 : 50

LOGGED: DG
CHECKED: KPH

APPENDIX B

Results of Basic Chemical Analysis
Paracel Laboratories Report 1650222

	Client ID:	16-02	-	-	-
	Sample Date:	05-Nov-16	-	-	-
	Sample ID:	1650222-01	-	-	-
	MDL/Units	Water	-	-	-
General Inorganics					
Conductivity	5 uS/cm	1430 [1]	-	-	-
pH	0.1 pH Units	7.5 [1]	-	-	-
Resistivity	0.01 Ohm.m	7.00 [1]	-	-	-
Anions					
Chloride	1 mg/L	69 [1]	-	-	-
Sulphate	1 mg/L	145 [1]	-	-	-

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