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REPORT ON

Supplemental Geotechnical Investigation Royal Ridge Subdivision Trim Road and Old Montreal Road Ottawa, Ontario

Submitted to:

Claridge Homes
210 Gladstone Avenue
Ottawa, Ontario
K2P 0Y6

REPORT

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1.0 INTRODUCTION

This report presents the results of a supplemental geotechnical investigation carried out for the proposed Royal Ridge Subdivision located south of Old Montreal Road, east of Trim Road, and north of Watters Road in Ottawa, Ontario.

The purpose of this investigation was to supplement the existing subsurface information by means of four boreholes. Based on an interpretation of the factual information obtained, along with the existing subsurface information available for this site, engineering guidelines are provided on the geotechnical design aspects of the project, including construction considerations which could affect design decisions.

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 to develop the Royal Ridge Subdivision which is located on lands south of Old Montreal Road, east of Trim Road and north of Watters Road in Ottawa, Ontario (see Key Plan in Figure 1).

The site measures approximately 500 by 150 metres in plan dimension and is proposed for development with a conventional suburban residential subdivision (i.e., single family homes and townhouse blocks). The ground surface across the site slopes from south to north between about elevations 84 and 74 metres. Natural slopes are also present along the northern boundary of the site, adjacent to Old Montreal Road, and the southeast corner of the site near Watters Road (along a tributary to Cardinal Creek).

An initial geotechnical investigation was carried out by Golder Associates for this site in 2004. The results of that investigation were provided in a report to the Regional Group titled "Geotechnical Investigation, Proposed Residential Development, Russell Finlay Lands, Trim Road, Queen Street, Watters Road, Ottawa, Ontario" dated September 2004 (report number 04-1120-146). A previous investigation for the proposed reconstruction of Regional Road 57 was also carried out across this site in 1988. The results of that investigation were provided in a report to Kostuch Engineering Limited titled "Preliminary Geotechnical Investigation, Proposed Reconstruction of Regional Road 57, Cumberland, Ontario" dated July 1988 (report number 881-2108).

Based on the results of the above report, the subsurface conditions are expected to consist of an extensive deposit of sensitive silty clay. Bedrock in the vicinity of the site is indicated to consist of shale of the Billings Formation.

It is understood that the layout for the development has been updated, since the previous report was prepared, and therefore this supplemental investigation was undertaken to provide revised geotechnical recommendations specific to the new subdivision layout.

3.0 PROCEDURE

The field work for this supplemental investigation was carried out between November 26 and 30, 2009. At that time, four boreholes (numbered 09-1 to 09-4, inclusive) were put down at the approximate locations shown on the Site Plan, Figure 2. One additional borehole, numbered 09-1A, was advanced adjacent to borehole 09-1 to obtain additional geotechnical information at depth. The borings were advanced using a track mounted hollow stem auger drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. The boreholes were advanced to depths ranging from about 7.9 to 10.7 metres below the existing ground surface.

Standard penetration tests were carried out in the boreholes at regular intervals of depths and samples of the soils encountered were recovered using drive open sampling equipment. In situ vane testing was carried out where possible in the silty clay to determine the undrained shear strength of the deposit.

The field work was supervised by an experienced technician from our staff who located the boreholes, directed the drilling operations, logged the boreholes and samples, directed the in situ testing, and took custody of the soil samples retrieved.

On completion of the drilling operations, samples of the soils encountered in the boreholes were transported to our laboratory for examination by the project engineer and for laboratory testing. The laboratory testing included natural water content and Atterberg limit tests.

One soil sample from borehole 09-1 was submitted to EXOVA Accutest Laboratories Ltd. for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

The borehole locations were selected, picketed and surveyed in the field by Golder Associates prior to the commencement of the field work using a Trimble R8 Global Position System (GPS) survey unit. The borehole locations and elevations are referenced to Geodetic datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the boreholes put down for the present investigation are shown on the Record of Borehole Sheets in Appendix A. The subsurface conditions encountered in test pits and boreholes put down for previous investigations by Golder Associates are shown on the Record of Boreholes and Test Pits in Appendix B. The results of the laboratory water content and Atterberg limit testing on the selected soil samples are given on the Record of Borehole Sheets. The results of the basic chemical analysis on soil samples are provided in Appendix C.

Note: Boreholes from previous investigations in the area by Golder Associates Ltd. and others are also shown on the Site Plan, for information purposes only, and are not discussed further in this report.

In general, the subsurface conditions on this site consist of an extensive deposit of sensitive silty clay.

The following sections present a summarized overview of the subsurface conditions encountered in the boreholes and test pits.

4.2 Topsoil

A layer of topsoil was encountered at ground surface at all of the borehole and test pit locations, with the exception of previous borehole 3. The thickness of the topsoil ranges from 50 to 330 millimetres.

4.3 Sensitive Silty Clay

The topsoil is underlain by a deposit of sensitive silty clay. The upper portion of the silty clay has been weathered to a stiff grey brown crust. The weathered zone generally extends to 8 to 9 metres depth. Standard penetration test N values between about 1 and 13 blows per 0.3 metres of penetration were measured in the weathered silty clay. In situ vane testing carried out in the lower portions of the weathered crust gave undrained shear strengths ranging from 52 to in excess of 130, kilopascals indicating a stiff to very stiff consistency for the weathered crust.

The results of Atterberg limit testing carried out on two samples of the weathered silty clay gave plasticity index values of 35 and 40 percent and liquid limit values of 60 and 70 percent, indicating a highly plastic soil. Water contents ranging from 30 to 48 percent were measured in the weathered silty clay.

At the boreholes 09-1A and 3 and test pit 04-6, grey silty clay is present below the depth of weathering (7.9, 5.2 and 3.7 metres, respectively). The grey silty clay at these locations was proven to depths of 10.7, 14.9 and 3.9 metres, respectively. The results of in-situ vane testing in this material typically gave undrained shear strengths ranging from about 56 to greater than 96 kilopascals, indicating a stiff to very stiff consistency.

The results of Atterberg limit testing carried out on one sample of the unweathered silty clay gave a plasticity index value of 18 percent and a liquid limit value of 43 percent, indicating a soil of intermediate plasticity. The measured natural water contents of two samples of the grey silty clay were 50 and 64 percent.

4.4 Groundwater

The groundwater levels in the standpipes at boreholes 09-1 and 09-4 were measured on January 15, 2010. At that time the groundwater level were 0.3 and 2.6 metes below the existing ground surface (i.e., elevations 74.1 and 81.1 metres).

It should be noted that 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 this project based on our interpretation of the borehole and test pit information as well as the project requirements, and is subject to the limitations in the "Important Information and Limitations of This Report" attachment which follows the text of this report.

5.2 Site Grading

In general, the subsurface conditions consist of topsoil overlying an extensive deposit of stiff to very stiff sensitive silty clay.

The unweathered silty clay at depth has potentially limited capacity to accept additional load from the weight of grade raise fill placed across the site and from the foundations of houses without undergoing significant consolidation settlement. To leave sufficient remaining capacity for the silty clay to support house foundations with reasonable footing sizes, the thickness of grade raise fill on this site will need to be limited.

In making the site grading assessment, certain assumptions have been made regarding the footing depths, width, and loads, as discussed subsequently in Section 5.3 of this report.

Based on the above, it is considered that the grade across this site should be raised no higher than 2.5 metres above existing ground surface level. It is understood that the proposed grades for this site are within the limits of this restriction.

As a general guideline regarding the site grading, the preparation for filling of the site should include stripping the existing topsoil for predictable performance of structures and services. The topsoil is not suitable as general fill and should be stockpiled separately for re-use in landscaping applications only. In areas with no proposed structures, services, or roadways, the topsoil may be left in place provided some settlement of the ground surface following filling can be tolerated.

5.3 Foundations

It is considered that the proposed residences may be supported on spread footings on or within the silty clay.

As discussed in the preceding section, the silty clay deposit has limited capacity to accept the combined load from site grading fill and foundation loads. The allowable bearing pressures for spread footing foundations at this site are therefore based on limiting the stress increases on the compressible, grey silty clay at depth to an acceptable level so that foundation settlements do not become excessive. Four important parameters in calculating the stress increase on the grey silty clay are:

- The thickness of soil below the underside of the footings and above the unweathered grey silty clay;
- The size (dimensions) of the footings;
- The amount of surcharge in the vicinity of the foundations due to landscape fill, underslab fill, floor loads, etc., as described in Section 5.2; and,
- The effects of groundwater lowering caused by this or other construction.

Provided that the amount of fill material placed on this site is restricted to the permissible maximum grade raise given in Section 5.2, spread footing foundations can be designed using a maximum allowable bearing pressure of 75 kilopascals, for up to 1.0 metre wide footings.

The post construction total and differential settlements of footings sized using the above maximum allowable bearing pressure 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, the provided allowable bearing pressure corresponds 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 allowable bearing pressure given above should be the full dead load plus sustained live load.

Note: If the design of any house foundations to Part 4 (rather than Part 9) of the Ontario Building Code is required, then the following parameters may be used (for footings up to 1.0 metre wide).

- Serviceability Limit States bearing resistance = 75 kilopascals
- Ultimate Limit States factored bearing resistance = 150 kilopascals

5.4 Seismic Design

The seismic design provisions of the 2006 Ontario Building Code depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or rock below founding level. Based on the 2006 Ontario Building Code methodology, the soil profile for this site would be a Site Class D (for any structures requiring Part 4 design).

5.5 Basement and Garage Floor Slabs

In preparation for the construction of the basement floor slabs, all loose, wet, and disturbed material should be removed from beneath the floor slab. Provision should be made for at least 200 millimetres of 19 millimetre crushed clear stone to form the base of the basement floor slabs.

To prevent hydrostatic pressure build up beneath the basement floor slabs, it is suggested that the granular base for the floor slabs be positively drained. This could be achieved by providing a hydraulic link between the under-floor fill material and the exterior drainage system.

The backfill material inside the garage should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment. The granular base for the garage floor slab should consist of at least 150 millimetres of Granular A compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

5.6 Frost Protection

All exterior perimeter foundation elements or 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 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.

Insulation of the bearing surface with high density insulation could be considered as an alternative to earth cover for frost protection. The details for footing insulation could be provided if and when required.

5.7 Basement Walls and Foundation Wall Backfill

The soils at this site are highly frost susceptible and should not be used as backfill directly against exterior, unheated or well insulated foundation elements. To avoid problems with frost adhesion and heaving, these foundation elements should either be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular B Type I or, alternatively, a bond break such as the Platon system sheeting could be placed against the foundation walls.

Drainage of the wall backfill should be provided by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, wrapped in geotextile, which leads by gravity drainage to an adjacent storm sewer or sump pit. Conventional damp proofing of the basement walls is appropriate with the above design approach.

Where design of basement walls in accordance with Part 4 of the 2006 Ontario Building Code is required, 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 base magnitude of $K_o\gamma H$, where:

K_o = The lateral earth pressure coefficient in the 'at rest' state, use 0.5;

γ = The unit weight of the granular backfill, use 22 kilonewtons per cubic metre; and,

H = The height of the basement wall in metres.

If Platon System sheeting or similar water barrier product is used against the foundation walls, then hydrostatic groundwater pressures should also be considered in the calculation of the lateral earth pressures.

5.8 Basement Excavations

Excavation for the basement constructions will be through weathered silty clay.

The weathered silty clay would generally be classified as a Type 3 soil in accordance with the Occupational Health and Safety Act of Ontario (OHSA). Accordingly side slopes in this material should be cut back at 1 horizontal to 1 vertical.

No unusual problems are anticipated in excavating in the overburden using conventional hydraulic excavating equipment.

Some groundwater inflow into the excavations could potentially be expected. However, for the planned excavation depths, it should be possible to handle the groundwater inflow by pumping from well filtered sumps in the excavations.

5.9 Site Servicing

Excavation for the installation of site services will be within the silty clay.

As described above, the stiff to very stiff silty clay would generally be classified as a Type 3 soil in accordance with the OHSA of Ontario. Accordingly excavations in the silty clay can be made with side slopes at 1 horizontal to 1 vertical. Alternatively, excavations within the overburden could also be carried out within a fully braced steel trench box, which would minimize the width of the excavation. The use of a trench box will not however eliminate the potential for disturbance outside the trench box limits. Good construction practices using trench boxes can limit the potential zone of disturbance to within about 0.5 metres of the outside of the trench box walls.

Some groundwater inflow into the trenches should be expected. However, it should be possible to handle the groundwater inflow by pumping from well filtered sumps established in the floor of the excavations, provided suitably sized pumps are used.

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 does occur, it may be necessary to place a sub-bedding layer consisting of compacted OPSS Granular B Type II beneath the Granular A or to thicken the Granular A bedding. 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 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 could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from 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 standard Proctor maximum dry density.

It should generally be possible to re-use the weathered silty clay as trench backfill. Where the trench will be covered with hard surfaced areas, the type of native 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 standard Proctor maximum dry density using suitable compaction equipment.

The high water content of the unweathered grey silty clay makes this soil difficult to handle and compact. If grey silty clay is excavated during installation of the site services, this material should be wasted or should only be used as backfill in the lower portion of the trenches to limit the amount of long term settlement of the pavement surface.

Impervious dykes or cut-offs should be constructed in the service trenches at about 150 metre intervals to reduce groundwater lowering at the site due to the "french drain" effect of the granular bedding and surround for the service pipes. It is important that these barriers extend from trench wall to trench wall and that they fully penetrate the granular materials to the trench bottom. The dykes should be at least 1.5 metres wide and could be constructed using relatively dry (i.e., compactable) grey brown silty clay from the weathered zone.

5.10 Pavement Design

In preparation for pavement construction, all topsoil, disturbed, or otherwise deleterious materials should be removed from the roadway areas.

Pavement areas requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material. These materials should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

The surface of the pavement subgrade should be crowned to promote drainage of the roadway granular structure. Perforated pipe sub-drains should be provided at subgrade level extending from the catch basins for a distance of at least 3 metres longitudinally, parallel to the curb in two directions.

The pavement structure for local roads without bus or truck traffic should consist of:

Pavement Component	Thickness (millimetres)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	400

The pavement structure for collector roadways which will carry bus and truck traffic should consist of:

Pavement Component	Thickness (millimetres)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	500

The granular base and subbase materials should be uniformly compacted as per OPSS 310, Method A. The asphaltic concrete should be compacted in accordance with the procedures outlined in OPSS 310.

The composition of the asphaltic concrete pavement in collector roadways should be as follows:

Superpave 12.5 mm Surface Course - 40 millimetres

Superpave 19 mm Base Course - 50 millimetres

The asphalt cement should consist of PG 58-34 and the design of the mixes should be based on a Traffic Category B for local roads and Category C for collector roads.

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 density 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.

In addition, it is understood that cuts up to about 3.7 metres are proposed for the roadways at the north end of the site (i.e., portions of Antigonish Avenue and Challenge Crescent) which would expose the frost susceptible native silty clay subgrade, below the existing frost depth, to frost penetration for the first time. If not protected against frost penetration, the roadways in this area could heave severely in the winter and subsequently settle in the spring. The freezing and thawing cycles will ultimately result in shrinkage of the clay. The resulting pavement distortions could be severe and highly non-uniform. Therefore, the pavement subgrade in this area should be insulated with high density insulation (Dow High Load 60).

The insulation should extend laterally beneath all hard surfacing (i.e., beneath the pavement, curbs and sidewalks) in areas where the cut for the roadway is more than 0.3 metres in depth. At the longitudinal limit of the insulation as well as at the lateral extent of the insulation adjacent to driveways, a special frost taper detail will be required to avoid severe differential frost heaving of the pavement surface above the insulation. A schematic of that detail is provided on Figure 3.

5.11 Vortechs Structures

It is understood that two vortechs structures are proposed to be installed at this site, the first of which is to be located just north of lot 1. The second is to be located within the easement area adjacent to Lot 31.

Excavation for the installation of vortechs structures will be within the silty clay. As mentioned previously, the stiff to very stiff silty clay would generally be classified as a Type 3 soil in accordance with the OHSA of Ontario. Accordingly excavations in the silty clay can be made with side slopes at 1 horizontal to 1 vertical. It would be preferable for the excavation to be made and the structures backfilled prior to the foundations of the adjacent houses being constructed. If that sequence is not feasible, the further assessment of the excavation side slope stability will be required.

Some groundwater inflow into the excavations should be expected. However, it should be possible to handle the groundwater inflow by pumping from well filtered sumps established in the floor of the excavations, provided suitably sized pumps are used.

Since the structures are essentially buried concrete tanks, it is considered that foundations can be designed as raft foundations supported on or within the silty clay.

The raft foundations for the vortech structures founded within the silty clay may be design based on a gross bearing resistance at Serviceability Limit States (SLS) of 150 kilopascals and an Ultimate Limit States (ULS) factored gross bearing resistance of 200 kilopascals. The design value for SLS is based on 25 millimetres of total settlement and 15 millimetres of differential settlement.

The permissible SLS contact pressure and corresponding settlement estimates are dependant upon the soil at or below founding level not being disturbed during construction. The silty clay subgrade may be wet and sensitive to disturbance. Consideration should be given to covering the subgrade with a mud slab of lean concrete immediately following inspection and approval.

The SLS bearing resistance corresponds to a settlement resulting from consolidation of the silty clay. Consolidation of 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 permissible SLS contact pressure given above should be the full dead load plus sustained live load. The factored dead load plus full factored live load should be used in conjunction with the ULS factored bearing resistance.

The SLS bearing resistance value and settlement estimates are based on the structure not resulting in a long-term lowering of the groundwater level. These structures should therefore be designed to be water-tight and the backfill should not be provided with a drainage system.

The deflections and the resulting forces and bending moments in the vortechs raft slab foundations to be used in its structural design could be determined by structural analysis using a modulus of subgrade reaction, ks, 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 walls) and 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.

The modulus of subgrade reaction may therefore be assumed to be in the range of 3 to 30 megapascals per metre. The structural design of the slab at any location should be determined based on whichever value causes the larger effect, since the maximum and minimum values may govern for different locations and load effects.

Once the foundation geometry and the actual distribution of load is known, then the resulting settlement can be calculated and the modulus value can be updated/refined.

To avoid ground settlements around the foundations, which could affect site grading and drainage, all of the backfill materials should consist of sand or sand and gravel conforming to the requirements for OPSS Granular B, Type I, placed in maximum 0.3 metre thick lifts, and compacted to at least 95 percent of the material's standard Proctor maximum dry density.

Furthermore, the soils at this site are frost susceptible and should not be used as backfill within the depth of potential frost penetration (1.5 metres) to avoid problems with frost adhesion and heaving.

The vortechs structure walls should be designed to resist lateral earth pressures calculated using a triangular distribution of the stress, which may be determined as follows:

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

Where: $\sigma_h(z)$ = The lateral earth pressure at depth 'z' (kPa);

z = The depth below ground surface (m);

K_o = The at rest pressure coefficient, use 0.5;

γ = The unit weight of the backfill soil (kN/m^3)
use 22 kilonewtons per cubic metre; and,

q = The surcharge due to live loads on the ground surface above the structure (kPa).

The value of the surcharge due to live loading (q) should consider the potential construction loads from equipment or materials. A value of no less than 15 kilopascals could be reasonable.

Hydrostatic water pressures should be considered for the portion of the foundation walls below the measured groundwater level (i.e., elevation 74.1 metres at Vortechs 1 and elevation 81.1 metres at Vortechs 2).

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:

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

Where: K_{AE} = The seismic earth pressure coefficient, use 0.8; and,

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

According to the National Building Code of Canada the site-specific zonal acceleration ratio for Ottawa is 0.42. Since the structure wall would be essentially un-yielding, the horizontal seismic coefficient, kh, used in the calculation of the seismic pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., kh = 0.63). The corresponding value of the seismic earth pressure coefficient (KAE) would therefore be 0.8.

In addition, the potential hydrodynamic pressures from the groundwater should be considered under seismic loading conditions for that portion of the foundation wall below the groundwater level. The additional hydrodynamic pressure may be calculated using the following expression:

$$p(z) = 1.5\, k_h\, \gamma_w\, (hz)^{1/2}$$

Where: $p(z)$ = The additional hydrodynamic water pressure, at depth z;

k_h = The design horizontal ground acceleration, use 0.42,

γ_w = The unit weight of water, use 9.81 kM/m^3; and,

h = The total depth of water.

All of the above lateral earth pressure equations and parameters are given in an unfactored format.

5.12 Slope Stability Considerations

As mentioned previously, natural slopes exist at the north boundary of the site along Old Montreal Road as well as at the southeast corner of the site adjacent to Watters Road (i.e., along the Cardinal Creek tributary). In addition a cut slope section is also proposed along the western boundary of the site for the future realignment of Trim Road. A slope stability assessment for these slopes has also been carried out by Golder Associates, the results of which are presented in a separate report.

5.13 Above Ground and In Ground Pools

No special geotechnical considerations are necessary for the installation of in-ground pools, provided that the pool (including piping) does not extend deeper than the house footing level.

Due to the additional loads that would be imposed by the construction of *above-ground pools*, these should be located no closer than 2 metres from the outside wall of the house. In addition, the installation of an above-ground pool should not be permitted to alter the existing grades within 2 metres of the house. Provided these restrictions are adhered to, no further geotechnical assessment should be required for above-ground pools.

5.14 Cardinal Creek Outlet

A storm sewer outlet is to be provided to Cardinal Creek along the north side of the embankment that carries Old Montreal Road over the creek valley. It is understood that a series of drop structure manholes will be used to convey the water to an outlet structure which outlets just downstream of the culvert that carries Cardinal Creek through the embankment.

The proposed arrangement is acceptable from a geotechnical point of view, subject to the following guidance:

- Cardinal Creek is known to be a dynamic water course with a history of active erosion and slope instability. Erosion protection measures around the outlet structure, possibly including the opposite bank as well, should be provided, as required.
- The embankment should be reinstated to match its existing geometry (which is presumably stable). The backfill should be placed in maximum 300 mm thick lifts which are compacted to at least 95 % of the material's standard Proctor maximum dry density. The surface should be protected against erosion, such as with an erosion control blanket.
- The bearing surface for the outlet control structure should be inspected by qualified geotechnical personnel, to confirm its suitability to support the structure.

5.15 Corrosion and Cement Type

Soil samples from borehole 09-1 and previous borehole 3 were submitted to EXOVA Accutest Laboratories Ltd. for basic chemical analysis related to potential corrosion of buried steel elements and potential sulphate attack on buried concrete elements. The results of the testing are provided in Appendix C. 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.

5.16 Trees

The silty clay on this site is highly sensitive to water depletion by trees of high water demand during periods of dry weather. When trees draw water from the silty clay, the silty clay undergoes shrinkage which can result in settlement of adjacent structures. The zone of influence of a tree is considered to be approximately equal to the height of the tree. Therefore trees which have a high water demand should not be planted closer to structures than the ultimate height of the trees. Table 1 provides a list of the common trees in decreasing order of water demand and, accordingly, decreasing risk of potential effects on structures.

5.17 Test Pits

Where the test pits for the previous investigation have been excavated within the zone of influence of the proposed building footprints or adjacent to the building footprints, the disturbed backfill soils in the test pits are unsuitable for vertical or lateral support of foundations or floor slabs. Foundations or floor slabs supported on the backfill soils could experience unacceptable settlements. The backfill materials should therefore be removed and replaced with compacted engineered fill. The engineered fill material used within the test pits should consist of OPSS Granular B Type II or Granular A. These materials should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

Based on the current development layout and the recorded test pit locations, this issue is a particular concern for Lots 14 and 15, adjacent to test pit 04-3. However careful inspection of the subgrades of all the lots in the areas of the previous test pits will be necessary.

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 as well as sewer bedding and backfill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction point of view.

The groundwater level monitoring devices (i.e., standpipe piezometers or wells) installed at the site will require decommissioning at the time of construction in accordance with Ontario Regulation 128/03. However, it is expected that most of the wells will either be destroyed during construction or can be more economically abandoned as part of the construction contract. If that is not the case or is not considered feasible, abandonment of the monitoring wells can be carried out separately.

Yours truly,

GOLDER ASSOCIATES LTD.

S Trickey

Susan Trickey, EIT

Mike Cunningham, P.Eng.
Associate

M. I. CUNNINGHAM

SAT/MIC/cg

n:\active\2009\1121 - geotechnical\09-1121-0163 claridge russell findlay lands ottawa\09-1121-0163-3000 rpt-001.docx

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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. 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 can not 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 can not 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

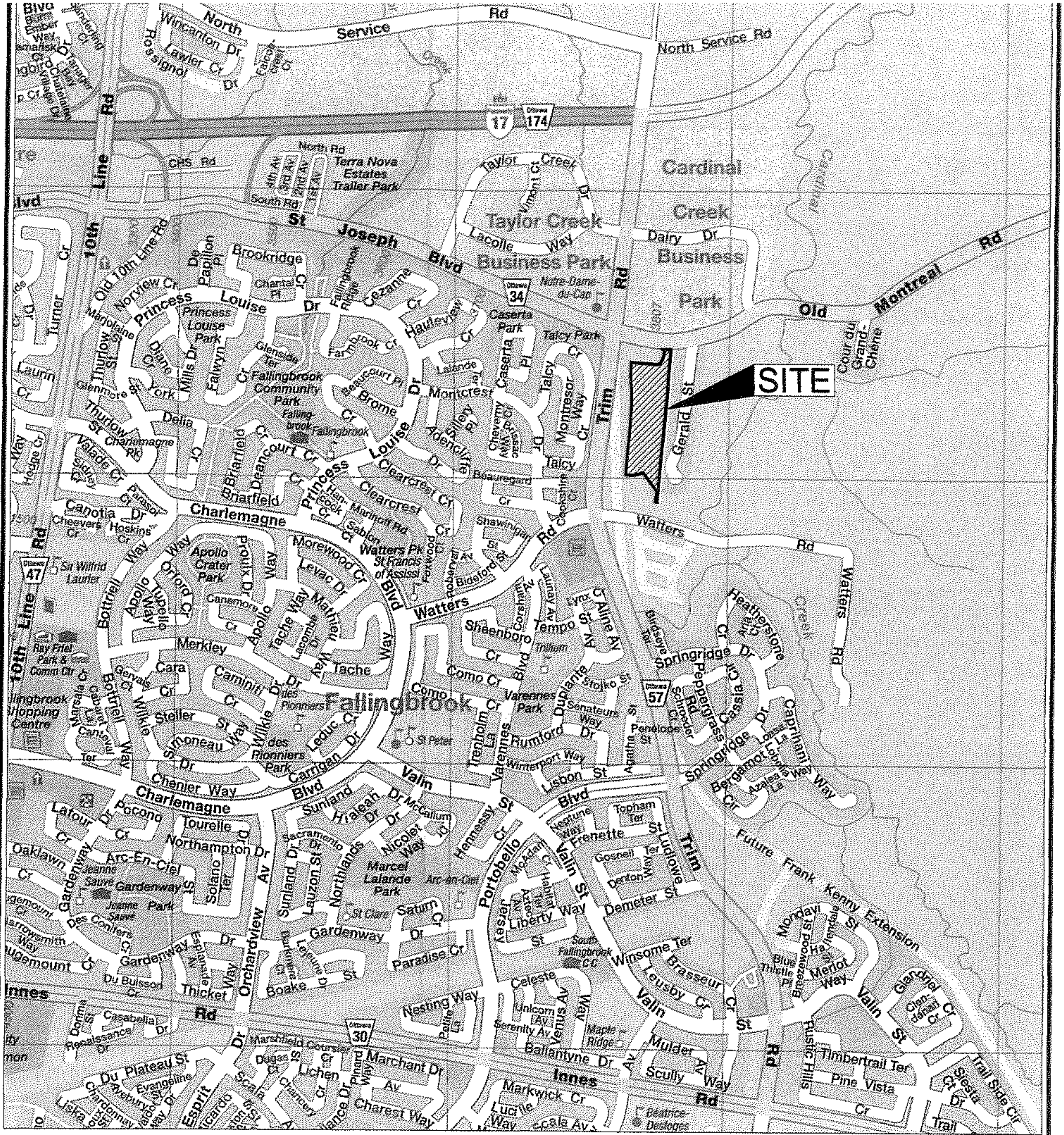
Larch

Evergreen Conifers

Spruce

Fir

Pine



NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 09-1121-0163-3000

Golder Associates
Ottawa, Ontario, Canada

SCALE	1:20,000	
DATE	29 JAN. 10	
DESIGN		
CAD	J.M.	
CHECK	SAT	
REVIEW	MIL	

TITLE

KEY PLAN

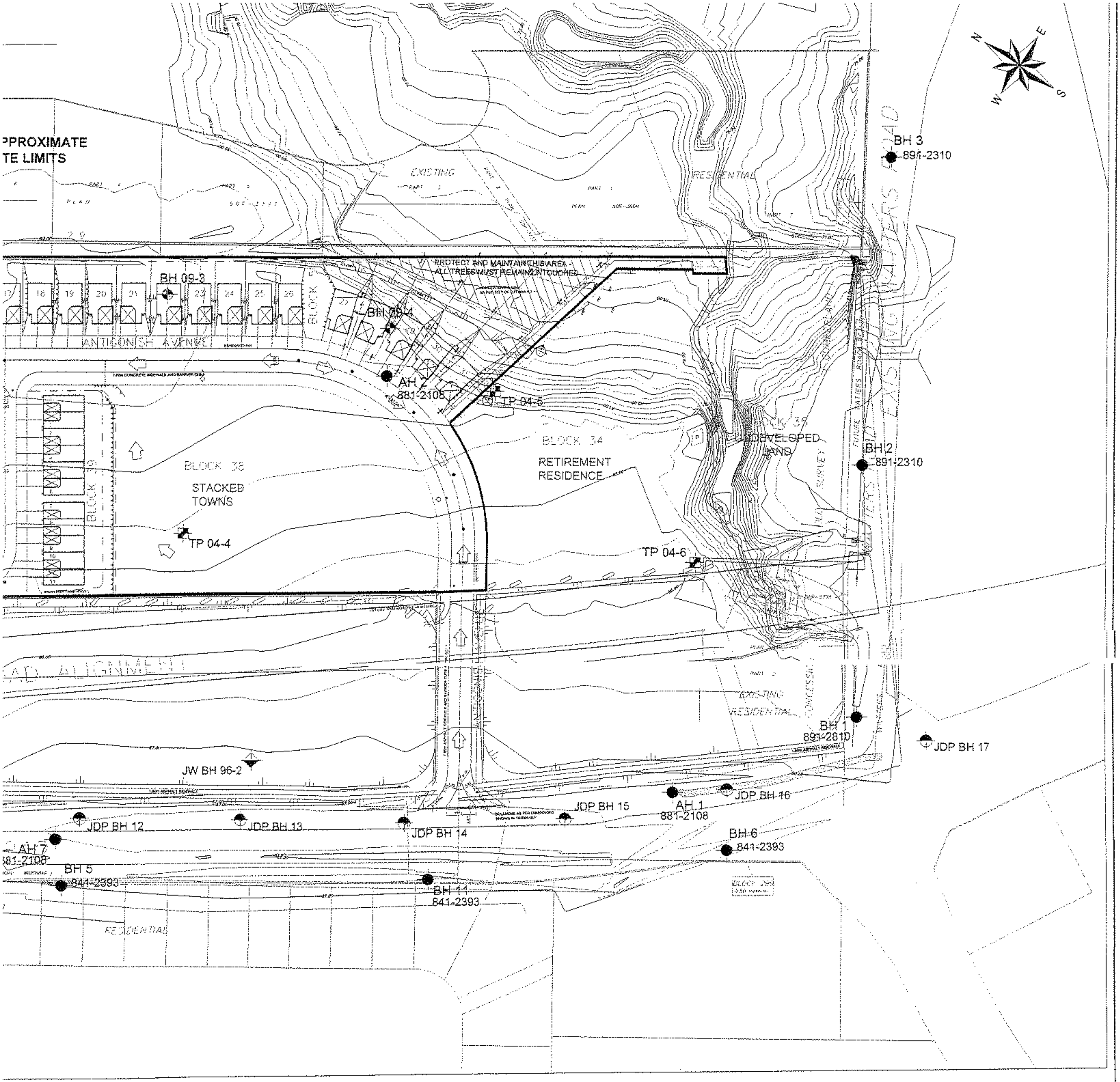
ROYAL RIDGE
OTTAWA, ONTARIO

FIGURE 1

FILE No. 0911210163-1000-01.dwg

PROJECT No. 09-1121-0163 REV.

PLOT DATE: January 29, 2010
FILENAME: N:\Active\2009\1121 – Geotechnical\09-1121-0163 Claridge Russell Findlay lands Ottawa\ACAD\Phase 1000\0911210163-1000-01.dwg



ERENCE:

PLAN PROVIDED IN ELECTRONIC FORMAT BY CLARIDGE HOMES

E:

DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING
ER ASSOCIATES LTD. REPORT No. 09-1121-0163-3000

REV	DATE	DES	REVISION DESCRIPTION	CAD	CHK	RVW
△						

PROJECT

GEOTECHNICAL INVESTIGATION
ROYAL RIDGE
OTTAWA, ONTARIO

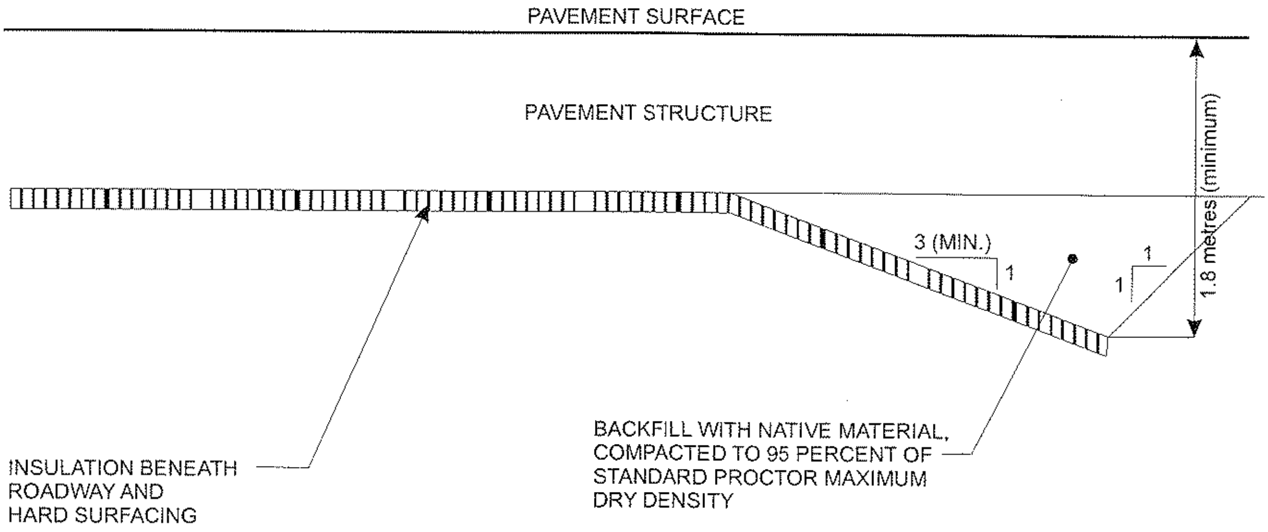
TITLE

SITE PLAN

PROJECT No.	09-1121-0163	FILE No. 0911210163-02A.dwg	
DESIGN		SCALE 1:1,000	REV.

FROST TAPER DETAIL FOR EDGE OF SUBGRADE INSULATION

FIGURE 3



NOTE:

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 09-1121-0163-3000

Date: JAN. 2010

Project: 09-1121-0163-3000

Golder Associates

Drawn: J.M.

Chkd: SAT

APPENDIX A

List of Abbreviations and Symbols
Record of Borehole Sheets

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual Tube sample

II. 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.) drive open Sampler for a distance of 300 mm (12 in.)

DD- Diamond Drilling

Dynamic Penetration Resistance; 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, 60^0 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

Peizo-Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60^0 conical tip and a projected 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 (PWP) and friction along a sleeve are recorded Electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	**N** Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	**C_u or S_u**	
	Kpa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limited
w_l	liquid limit
C	consolidaiton (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, G_s)
DS	direct shear test
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	field vane test (LV-laboratory vane test)
γ	unit weight

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

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
F	factor of safety
V	volume
W	weight

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
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma''$-u)
σ'_{vo}	initial effective overburden stress
$\sigma_1\sigma_2\sigma_3$	principal stresses (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 ($D_R= p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation

*	Density symbol is p. Unit weight symbol is γ where γ=pg(i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	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 (overconsolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio=σ'_p/σ'_{vo}

(d) Shear Strength

$\tau_p\tau_r$	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction=tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength (ϕ=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

Notes: 1. $\tau=c'\sigma'$ tan |'
2. Shear strength=(Compressive strength)/2

PROJECT: 09-1121-0163

RECORD OF BOREHOLE: BH 09-1

SHEET 1 OF 1

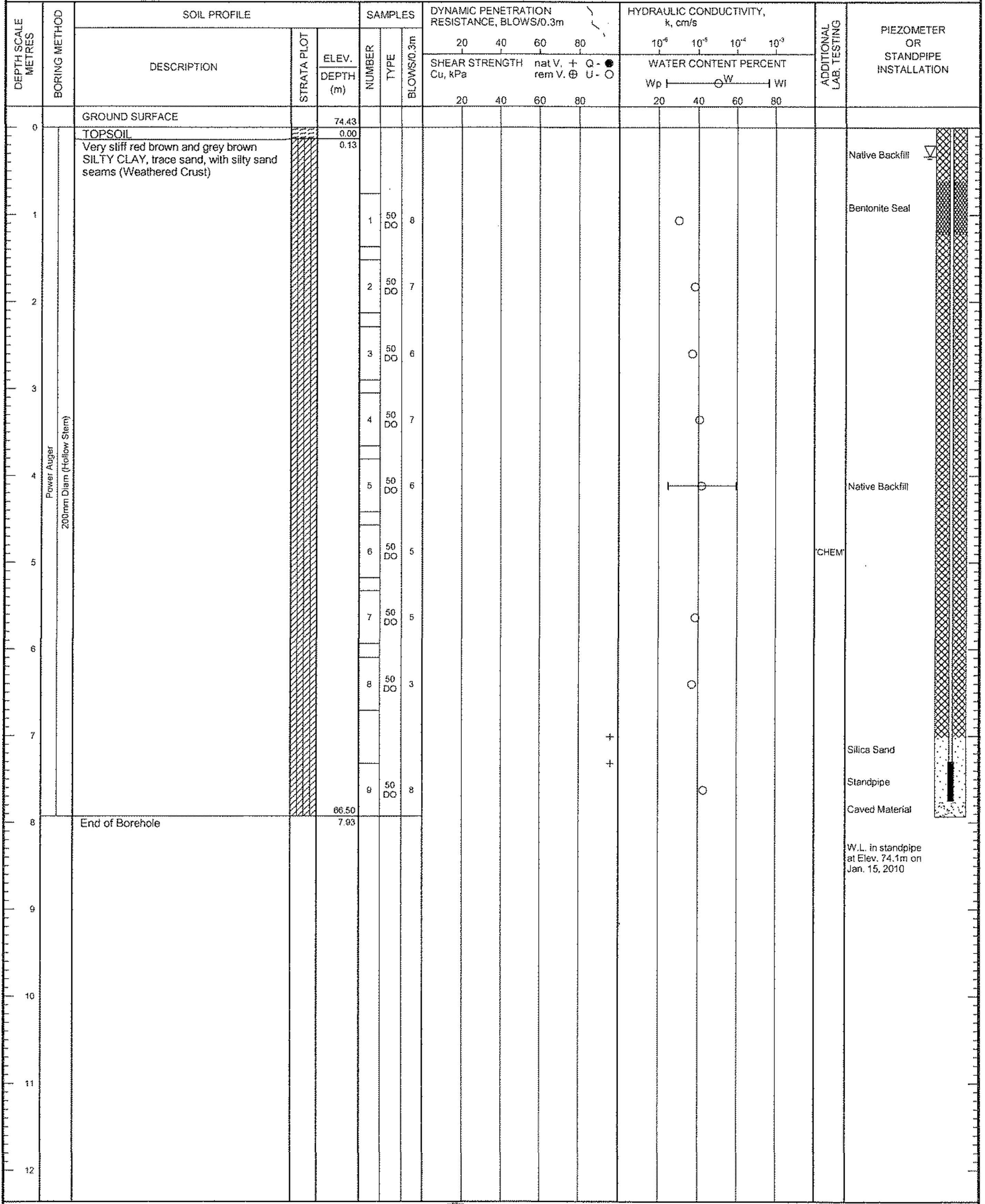
LOCATION: See Site Plan

BORING DATE: Nov. 27, 2009

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm



DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	74.43
TOPSOIL	0.00
Very stiff red brown and grey brown SILTY CLAY, trace sand, with silty sand seams (Weathered Crust)	0.13
	66.50
End of Borehole	7.93

NUMBER	TYPE	BLOWS/0.3m
1	50 DO	8
2	50 DO	7
3	50 DO	6
4	50 DO	7
5	50 DO	6
6	50 DO	5
7	50 DO	5
8	50 DO	3
9	50 DO	8

DEPTH SCALE
1 : 60

Golder Associates

LOGGED: R.I.
CHECKED: SAT

MIS-BHS 001 0911210163.GPJ GAL-MIS.GDT 1/29/10

PROJECT: 09-1121-0163

RECORD OF BOREHOLE: BH 09-1A

SHEET 1 OF 1

LOCATION: See Site Plan

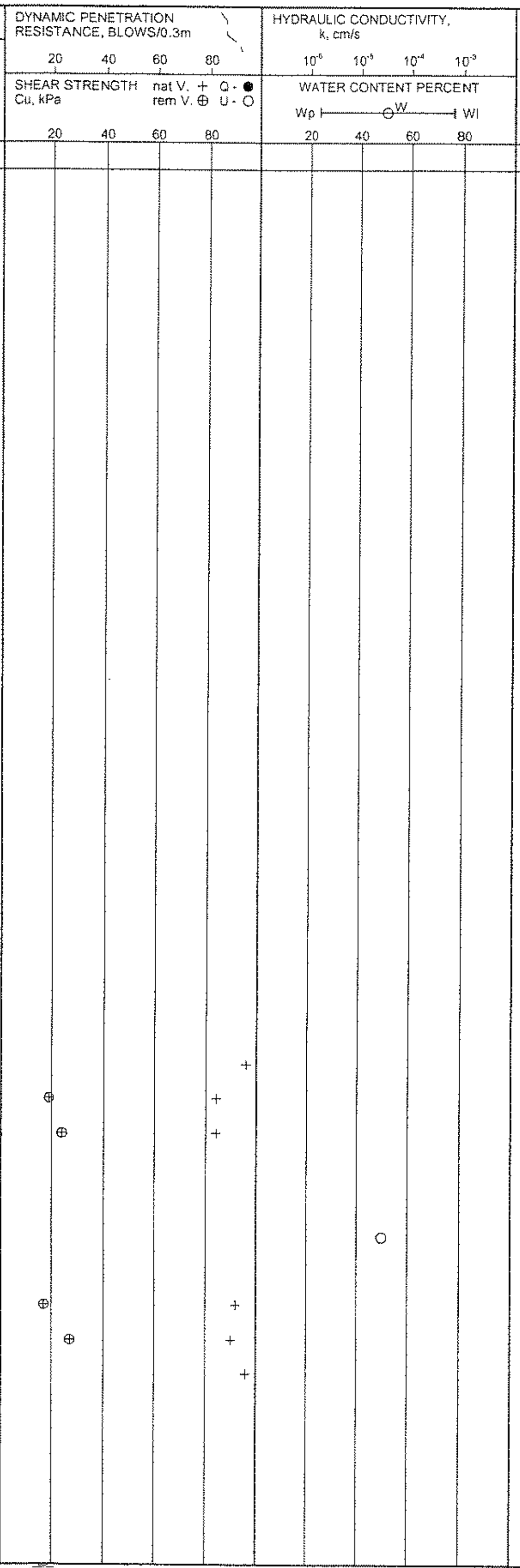
BORING DATE: Nov. 30, 2009

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/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		74.43					
0 1 2 3 4 5 6 7	Power Auger 200mm Diam (Hollow Stem)	For Stratigraphy See Record of Borehole BH 09-1		0.00					
8		Very stiff grey brown and red brown SILTY CLAY, trace sand, with silty sand seams (Weathered Crust)	▨	66.81 7.62					
9 10		Stiff to very stiff grey SILTY CLAY	▨	66.50 7.93	1	50 DO	2		
11 12		End of Borehole		63.76 10.67					



DEPTH SCALE

1 : 60

Golder Associates

LOGGED: R.I.

CHECKED: SIA7

PROJECT: 09-1121-0163

RECORD OF BOREHOLE: BH 09-2

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Nov. 26, 2009

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/0.3m	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
		GROUND SURFACE		80.92							
0	Power Auger 200mm Diam (Hollow Stem)	TOPSOIL		0.00							
		Very stiff to stiff grey brown and red brown SILTY CLAY, trace sand, with silty sand seams and rootlets (Weathered Crust)		0.13							
1					1	50 DO	9				
					2	50 DO	9				
2											
					3	50 DO	11				
3											
					4	50 DO	6				
4					5	50 DO	5				
					6	50 DO	5				
5											
					7	50 DO	3				
6											
					8	50 DO	5				
7					9	50 DO	4				
					10	50 DO	5				
8				72.69							
		End of Borehole		8.23							
9											
10											
11											
12											

MIS-BHS 001 0911210163.GPJ GAL-MIS.GDT 1/29/10

DEPTH SCALE

1 : 60

Golder Associates

LOGGED: R.I.

CHECKED: SAT

PROJECT: 09-1121-0163

RECORD OF BOREHOLE: BH 09-3

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Nov. 26, 2009

DATUM: Geodetic

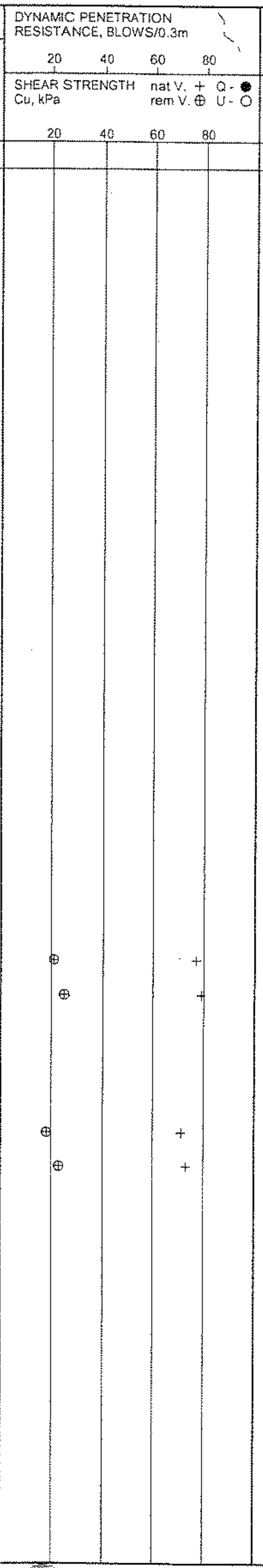
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger 200mm Diam (Hollow Stem)

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	83.24
TOPSOIL	0.00
Very stiff to stiff red brown and grey brown SILTY CLAY, trace sand, with silty sand seams and rootlets (Weathered Crust)	82.91 0.33
End of Borehole	74.40 8.84

NUMBER	TYPE	BLOWS/0.3m
1	50 DO	8
2	50 DO	8
3	50 DO	8
4	50 DO	5
5	50 DO	5
6	50 DO	5
7	50 DO	4
8	50 DO	3
9	50 DO	1



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

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10 11 12

DEPTH SCALE
1 : 60

Golder Associates

LOGGED: R.I.

CHECKED:

PROJECT: 09-1121-0163

RECORD OF BOREHOLE: BH 09-4

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Nov. 26, 2009

DATUM: Geodetic

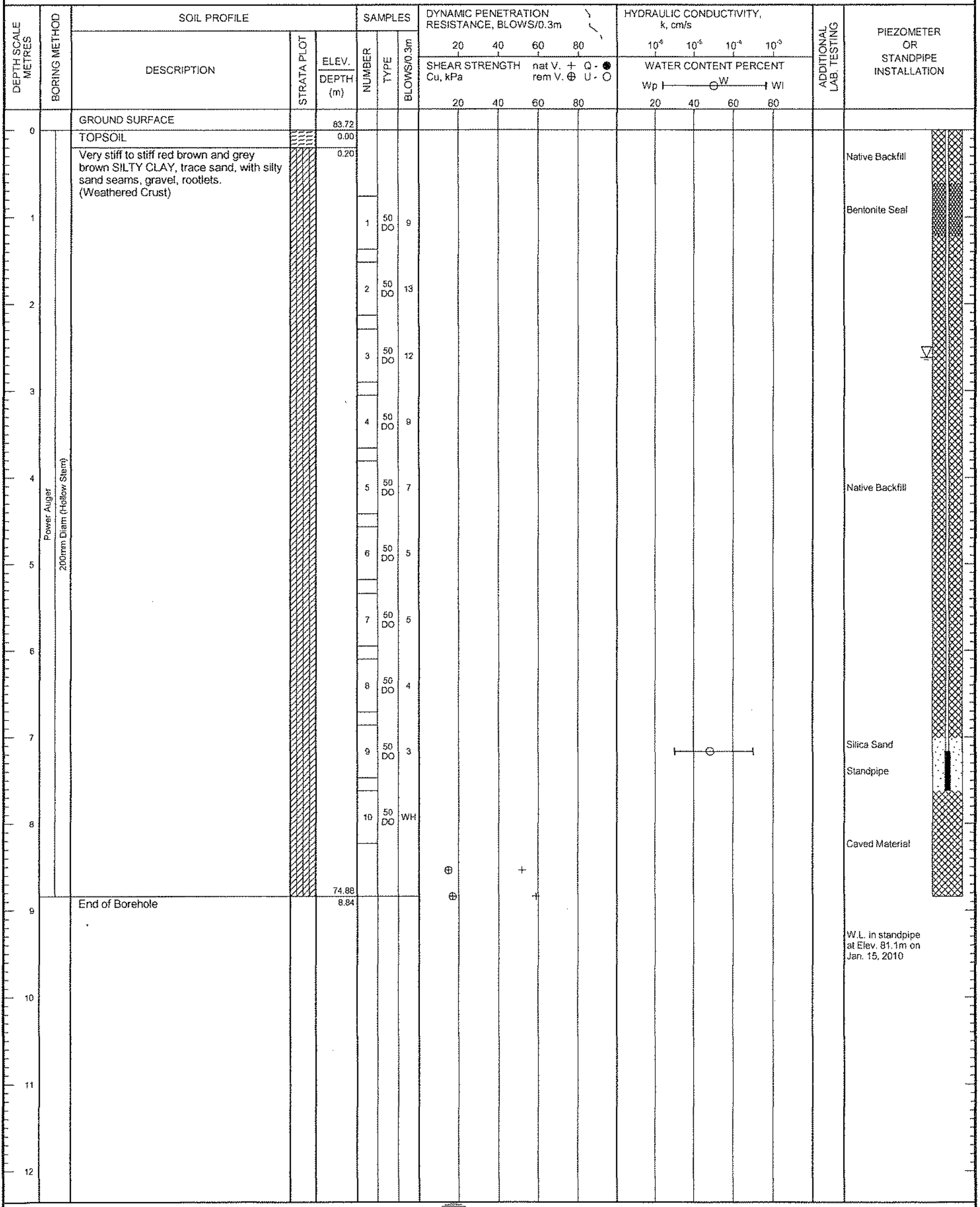
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200mm Diam (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	83.72
TOPSOIL	0.00
Very stiff to stiff red brown and grey brown SILTY CLAY, trace sand, with silty sand seams, gravel, rootlets. (Weathered Crust)	0.20
	74.88
End of Borehole	8.84

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 DO	9
2	50 DO	13
3	50 DO	12
4	50 DO	9
5	50 DO	7
6	50 DO	5
7	50 DO	5
8	50 DO	4
9	50 DO	3
10	50 DO	WH



W.L. in standpipe at Elev. 81.1m on Jan. 15, 2010

DEPTH SCALE
1 : 60

Golder Associates

LOGGED: R.I.
CHECKED: SAT

APPENDIX B

Record of Boreholes and Test Pits Previous Investigations by Golder Associates Ltd.

RECORD OF BOREHOLE 3

SHEET 1 of 2

LOCATION See Figure 2

BORING DATE Apr.22&25,1988

DATUM GEODETIC

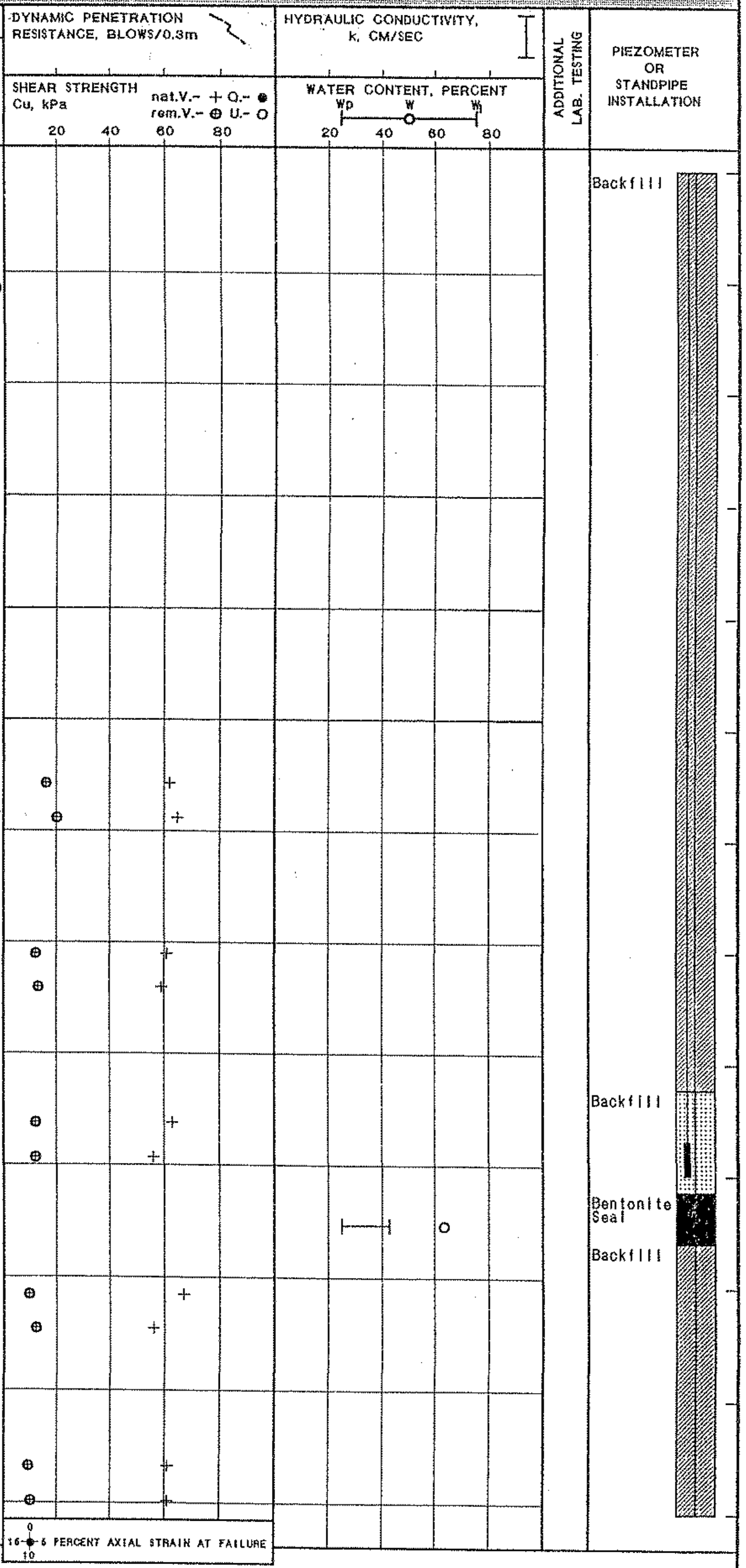
SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

BORING METHOD: Power Auger 200mm Diam (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (M)
Ground Surface	77.91 0.00
Very stiff red brown and grey brown SILTY CLAY, some silty fine sand seams (Weathered Crust)	72.73 5.18
Stiff grey SILTY CLAY, some silty fine sand layers	70.14 7.77
Stiff red brown SILTY CLAY	69.38 8.53
Stiff grey and red brown, trace black mottling SILTY CLAY, occasional silt band	65.91 12.00
Hole Continued	

SAMPLES NUMBER	TYPE	BLOWS/0.3M
1	50 DO	10
2	50 DO	7
3	50 DO	9
4	50 DO	6
5	50 DO	4
6	50 DO	4
7	50 DO	3
8	50 DO	1
9	50 DO	1
10	50 DO	1



DEPTH SCALE 1 : 60

Golder Associates

LOGGED S.L

CHECKED CW

RECORD OF BOREHOLE 3

SHEET 2 of 2

LOCATION See Figure 2
BORING DATE Apr.22&25,1988
DATUM GEODETIC
SAMPLER HAMMER, 63.5kg, DROP, 760mm
PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 881-2108

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3M	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○	HYDRAULIC CONDUCTIVITY, k, CM/SEC; WATER CONTENT, PERCENT Wp W Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
12		Hole Continued		65.91							
	Power Auger 200mm Diam (Hollow Stem)	Stiff grey and red brown SILTY CLAY, some black organic mottling		12.00	11	50 DO	2				Backfill
13											Bentonite Seal Backfill
14					12	50 DO	1				
				62.97							
15		End of Hole		14.94							
16											Standpipes Destroyed
17											
18											
19											
20											
21											
22											
23											
24											

0
15 -⊕- 5 PERCENT AXIAL STRAIN AT FAILURE
10

DEPTH SCALE
1 : 60

Golder Associates

LOGGED S.L
CHECKED

RECORD OF BOREHOLE AH-2

SHEET 1 of 1

LOCATION See Figure 2

BORING DATE April 22,1988

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	TYPE	BLOWS/0.3M	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○	HYDRAULIC CONDUCTIVITY, k, CM/SEC; WATER CONTENT, PERCENT Wp W Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200mm Diam (Hollow Stem)	Ground Surface		83.70							
		TOPSOIL		[illegible] 0.09							
1		Very stiff red brown SILTY CLAY, grey silty fine sand (Weathered Crust)			1	AS	-				
		End of Hole		82.18 1.52							
2											
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											

0
15 -●- 5 PERCENT AXIAL STRAIN AT FAILURE
10

DEPTH SCALE
1 : 60

Golder Associates

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CT 8 3

RECORD OF TEST PITS

Test Pit Number	Elevation	Description	
TP 04-1	0.00 – 0.05	TOPSOIL	
(Elev. 73.98)	0.05 – 3.50	Very stiff red brown SILTY CLAY (weathered crust)	
	3.50	End of test pit	
		No free water	
		Cu > 130 kPa throughout	
TP 04-2	0.00 – 0.05	TOPSOIL	
(Elev. 75.34)	0.05 – 3.80	Very stiff red brown SILTY CLAY (weathered crust), trace gravel to 3.0 metres	
		End of test pit	
	3.80	No free water	
		Cu > 130 kPa throughout	
TP 04-3	0.00 – 0.08	TOPSOIL	
(Elev. 82.67)	0.08 – 2.40	Very stiff red brown SILTY CLAY, trace small cobbles, occasional grey silty sand pockets (weathered crust)	
	2.40 – 3.45	Very stiff red brown silty clay (weathered crust)	
	3.45	End of test pit	
		No free water	
		Cu > 130 kPa throughout	
TP 04-4	0.00 – 0.05	TOPSOIL	
(Elev. 84.58)	0.05 – 3.50	Very stiff red brown SILTY CLAY, occasional grey silty sand pockets, trace gravel (weathered crust)	Cu > 130 kPa throughout
	3.50 – 3.90	Stiff red brown SILTY CLAY, occasional grey silty sand seams (weathered crust)	Cu = 90 kPa @ 3.7 m Cu = 90 kPa @ 3.9 m
	3.90	End of test pit	
		No free water	

RECORD OF TEST PITS (continued)

Test Pit Number	Elevation	Description	
TP. 04-5	0.00 – 0.30	TOPSOIL	
(Elev. 82.91)	0.30 – 3.50	Very stiff red brown SILTY CLAY, occasional grey silty sand pockets below 3.3 metres (weathered crust)	
	3.50	End of test pit	
		No free water	
		Cu = 120 kPa throughout	
TP 04-6	0.00 – 0.16	TOPSOIL	
(Elev. 85.80)	0.16 – 3.70	Very stiff to stiff red brown SILTY CLAY, occasional grey silty sand pockets, (weathered crust)	Cu > 130 kPa @ 2.1 m Cu = 110 kPA @ 2.4 m Cu = 100 kPa @ 2.8 m Cu = 86 kPa @ 3.1 m Cu = 110 kPa @3.4 m
	3.70 – 3.90	Stiff grey SILTY CLAY	Cu = 56 kPA @ 3.90 m
	3.90	End of test pit	
		No free water	

APPENDIX C

Results of Chemical Analysis

REPORT OF ANALYSIS

Client: **Golder Associates Ltd. (Ottawa)**
32 Steacie Drive
Kanata, ON
K2K 2A9
Attention: **Ms. Susan Trickey**

Report Number: 2929584
Date: 2009-12-09
Date Submitted: 2009-12-02

Project: 09-1121-0163

P.O. Number:
Matrix: Soil

Chain of Custody Number: **106477**

		LAB ID:	764552					**GUIDELINE**		
		Sample Date:	2009-11-27							
		Sample ID:	BH09-1 SA# 6							
PARAMETER	**UNITS**	**MRL**						**TYPE**	**LIMIT**	**UNITS**
Chloride	%	0.002	<0.002							
Electrical Conductivity	mS/cm	0.05	0.20							
pH			8.2							
Resistivity	ohm-cm	1	5000							
Sulphate	%	0.01	0.02							

MRL = Method Reporting Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration

Comment:

APPROVAL:
Lorna Wilson
Agriculture Lab Supervisor

Client: **Golder Associates Ltd.**
1796 Courtwood Cr.
Ottawa, ON
K2C 2B5
Attention: **Mr. Gerry Webb**

Report Number: 2416542
Date: 2004-09-07
Date Submitted: 2004-08-31

Project: 041120146

P.O. Number:
Matrix: Soil

		LAB ID:	339357					**GUIDELINE**		
		Sample Date:	2004-08-24							
		Sample ID:	04-2 Sal							
PARAMETER	**UNITS**	**MDL**						**TYPE**	**LIMIT**	**UNITS**
Chloride	%	0.001	0.003							
Electrical Conductivity	mS/cm	0.01	0.36							
pH			8.0							
Sulphate	%	0.01	0.02							

MDL = Method Detection Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration

Comment:

APPROVAL:
Lorna Wilson
Agriculture Lab Supervisor