REPORT

Geotechnical Investigation Residential Development Spring Valley Trails - Zen Blocks

Ottawa, Ontario

Submitted to:

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the proposed 'Zen' residential blocks within the Spring Valley Trails residential development in Ottawa, Ontario.

The purpose of this subsurface investigation was to determine the general soil and groundwater conditions across the site by means of a limited number of boreholes. Based on an interpretation of the factual information obtained, 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 'Zen' residential blocks within the Spring Valley Trails residential development in Ottawa, Ontario (see Key Plan inset, on Figure 1).

The overall development site is located on lands south of Navan Road and Renaud Road. The property is bounded to the south by a former CN Rail line and the Mer Bleue Conservation area, to the east by the WSI landfill, to the west by Phases 1 and 2 of this development, and to the north by Navan Road and Renaud Road.

The proposed 'Zen' blocks will be located along Rolling Meadow Crescent within Block 165 of the development. The site measured approximately 130 metres by 60 metres in plan dimension, although somewhat irregular in shape. The site slopes down from the north to the south with an approximate 2 metre grade difference (i.e., from about elevation 78 to 76 metres), and from the east to the west with a grade difference of about 3 meters (i.e., from about elevation 79 to 76 metres).

It is understood that this 'Zen' blocks will be 3-storeys in height with one basement level and no garages.

Golder has carried out several previous subsurface investigations on the overall development site. The results of one previous investigation, which has some boreholes that were advanced within close proximity to the 'Zen' study area, are presented in the following report:

- Report to Claridge Homes by Golder Associates Ltd. titled "*Geotechnical Investigation, Proposed Residential Development, Lands South of Navan Road and Fourth Line Road, Ottawa, Ontario*" dated October 2006 (Report No. 05-1120-041).

The results of the previous investigation indicate that the subsurface conditions within the study area generally consist of a thick deposit of sensitive silty clay. A surficial, discontinuous cap of sandy soil is also indicated to overly the clay.

3.0 PROCEDURE

The fieldwork for this investigation was carried out between October 8 and 9, 2008. During that time, three boreholes (numbered 08-204 to 08-206, inclusive) were put down at the approximate locations shown on the Site Plan, Figure 1. The boreholes were advanced using a track-mounted auger drill rig supplied and operated by Marathon Underground Constructors Corporation of Greely, Ontario. These boreholes were advanced to a depth of about 7.6 metres below the existing ground surface.

An additional investigation was carried out on February 20, 2019 for additional sampling for the tree planting restrictions. During that time, three test pits (numbered 19-01, 19-02, and 19-03) were advanced at the approximate locations shown on the Site Plan, Figure 1. The test pits were advanced using a rubber tire back hoe supplied and operated by Glenn Wright Excavating of Ottawa, Ontario. The test pits were advanced to depths ranging from about 4.3 to 4.5 metres below the existing ground surface.

Standard penetration tests were carried out in the boreholes at regular intervals of depth and samples of the soils encountered 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.

Standpipe piezometers were sealed into boreholes 08-205 and 08-206 to allow subsequent measurement of the groundwater levels across the site. The groundwater level in the standpipe piezometer installed at borehole 08-205 was measured on October 17, 2008.

The fieldwork was supervised by experienced personnel from our staff who located the boreholes and test pits, directed the drilling and excavating 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 determinations, Atterberg limit testing, and grain size distribution testing.

The borehole and test pit locations were marked in the field, and subsequently surveyed by Golder personnel. The position and ground surface elevation at each borehole location were determined using a Trimble R8 GPS survey unit. The elevations are referenced to Geodetic datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

The Record of Borehole sheets for the investigation are provided in Appendix A.

The subsurface conditions beneath the study area generally consist of topsoil and/or sand overlying firm to very stiff grey brown silty clay crust underlain by generally soft to firm grey silty clay.

The following sections present an overview of the subsurface conditions encountered in the boreholes.

4.2 Topsoil and Fill

Topsoil exists at the ground surface at boreholes 08-205 and 08-206 and test pits 19-01 19-02, and 19-03 and below the fill in borehole 08-204. The topsoil thickness ranges from about 100 to 500 millimetres.

An approximately 0.2 metre thick layer of silty clay fill exists extending from the ground surface at borehole 08-204.

4.3 Silty Sand and Sand

Deposits of silty sand and sand exist below the topsoil and fill, where encountered, in borehole 08-206 and test pit 19-01 and 19-03. The sandy deposits ranges from about 1.1 to 2.4 metres in thickness, and extend down to depths ranging from about 1.1 to 2.9 metres below the existing ground surface.

Standard penetration test carried out within the sandy deposit gave a SPT ‘N’ value of 12 blows per 0.3 metres of penetration, indicating a compact state of packing.

4.4 Silty Clay

Silty clay underlies the topsoil, fill, and sand in all of the boreholes.

The upper portion of the silty clay has been weathered to a grey brown crust at all locations, except at test pit 19-01. The weathered crust extends to depths ranging from 1.7 to 2.4 metres below the existing ground surface.

Standard penetration tests carried out within the weathered silty clay gave SPT ‘N’ values of ‘weight of hammer’ to 6 blows per 0.3 metres of penetration. In situ vane testing carried out in the lower portion of the weathered crust gave an undrained shear strength value of 88 kilopascals. The results of this in situ testing indicate that the weathered crust has a very stiff to firm consistency.

The silty clay below the depth of weathering is grey in colour. The unweathered grey silty clay was not fully penetrated but was proven to extend to a depth of about 7.6 metres below the ground surface.

The results of in situ vane testing in the grey silty clay gave undrained shear strength values ranging from about 19 to 65 kilopascals. The results of this in situ testing indicate the unweathered portion of the silty clay deposit has a soft to stiff consistency, but more typically, soft to firm consistency.

The results of Atterberg limit testing carried out on three samples of the grey silty clay gave plasticity index values ranging from about 7 to 45 percent and liquid limit values ranging from about 25 to 71 percent, indicating a soil of low to high plasticity. The results of the Atterberg Limit tests are provided on Figure 2.

Water contents ranging from about 30 to 81 percent were measured in the grey silty clay.

The results of grain size distribution testing on one sample of the silty clay are provided on Figure 3.

4.5 Groundwater

Standpipe piezometers were installed in boreholes 08-205 and 08-206 to allow for subsequent measurement of the groundwater levels at the site. However, only the groundwater level in the piezometer in borehole 08-205 was measured on October 17, 2008 as indicated in the table below.

Borehole Number	Ground Surface Elevation	Water Level Depth (m)	Water Level Elevation (m)	Date of Measurement
08-205	77.58	1.88	75.70	October 17, 2008

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 information as well as the project requirements, and is subject to the limitations in the "Important Information and Limitations of This Report" which follows the text of this report.

5.2 Site Grading

The subsurface conditions beneath the site generally consist of topsoil and/or sand overlying firm to very stiff grey brown silty clay crust underlain by generally soft to firm grey silty clay. It is understood that because of the grade change across the site that the proposed residences will be within both cut (up to about 0.4 metres in depth) and fill sections.

The unweathered compressible grey silty clay deposit, which is located beneath the site, has limited capacity to accept additional load from the weight of grade raise fill and from the foundations of houses without undergoing consolidation settlements. Therefore, to leave sufficient remaining capacity for the silty clay to support house foundations, with reasonable footing sizes, the thicknesses of grade raise fill 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, a maximum allowable grade raise of 0.5 metres is permitted on this site. This maximum allowable grade raise was calculated assuming that any fill required for site grading (above original grade) would have a unit weight of no more than 19 kilonewtons per cubic metre and that the porches would be backfilled with expanded polystyrene (EPS). Well graded sand, sand and gravel, glacial till, and crushed stone typically have a higher unit weight and, if these materials are to be used, the permissible grade raises would be reduced and would need to be re-evaluated. The maximum allowable grade raise was also based on the preliminary underside of footing elevations provided by IBI Group and will need to be reviewed/confirmed once final underside of footing elevations have been established.

If the grading restrictions given above cannot be accommodated, the following three options could be considered:

1) The additional required grade raising above the limit given above could be accomplished using EPS light weight fill.

2) The area could be pre-loaded and allowed to settle in advance of house construction. The subgrade settlements would need to be monitored to establish when sufficient settlements had occurred such that house construction could proceed. To reduce the time required for the pre-loading, it is likely that a temporary surcharge above the existing grade would need to be considered, however in either case the pre-load time could be months or years.

3) The residences could be supported on driven steel piles, which derive their support from more competent bearing soils (i.e., glacial till) at depth below the "softer" layers of silty clay. However, the depth to more competent strata is not known and the potential settlements of the services, relative to the 'Zen' blocks, could be problematic.

4) A test fill area could be established to monitor the settlements of the site and confirm the compressibility of the silty clay deposit. This option is only considered appropriate where the proposed grading will only marginally exceed the permitted levels.

Additional geotechnical guidelines would need to be provided if any of the above options are selected. Additional investigation could also be required (in particular for Options 2, 3 and 4) before those guidelines could be finalized.

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

5.3 Foundations

As discussed in the previous section, the unweathered silty clay deposit beneath this site has limited capacity to accept the combined load from site grading fill and foundation loads. The allowable bearing pressures for spread footing foundations are therefore based on limiting the stress increases on the unweathered compressible 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 “softer” unweathered 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.

Preliminary underside of footing elevations were provided by IBI Group as follows:

Building	Underside of Footing Elevation (m)
A	76.35
B	75.54
C	75.30
D	75.05

Based on the above and the existing subsurface information across the site, the spread footings for the residences will be founded within either the lower portion of the weathered silty clay or directly on the soft to firm compressible silty clay. It is considered that the proposed residences can potentially be supported on strip footings on or within the native silty clay designed using the maximum allowable bearing pressures summarized in the table below for the various building blocks.

Building	Footing Width (metres)	Maximum Allowable Bearing Pressure (kilopascals)
A	0.6	75
B	Up to 0.8	31
	0.9 to 1.6	28
C	Up to 1.6	26
D	Up to 0.6	33
	0.7 to 1.0	24
	1.1 to 1.6	19

The maximum allowable bearing pressures provided above were calculated based on the preliminary underside of footing elevations provided by IBI Group as well as assuming the grade raise on site is limited to 0.5 metres as discussed in Section 5.2, and that any fill required for site grading (above original grade) around the foundations has a unit weight of no more than 19 kilonewtons per cubic metre (i.e., native silty clay or clear crushed stone) and that the porches would be backfilled with EPS. The above maximum allowable bearing pressures will need to be reviewed/confirmed once the final underside of footing levels for the residences have been established.

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 subgrade at or below founding level is not disturbed during construction.

The tolerance of the house foundations to accept those settlements could be increased by providing nominal amounts of reinforcing steel in the top and bottom of the foundation walls.

The provided maximum allowable bearing pressures for footings founded within the silty clay corresponds to settlement resulting from consolidation of these deposits. 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 for houses founded in the silty clay should be the full dead load plus sustained live load.

If the maximum allowable bearing pressures provided above are not considered feasible the following options could be considered:

1) The silty clay adjacent to the exterior foundation walls could be subexcavated and replaced with EPS to 'unload' the underlying silty clay in the zone of influence of the foundations and provide additional capacity for the compressible silty clay to accept foundation loads. However the feasibility of this option and the details of the EPS fill placement will need to be confirmed on a case-by-case basis and can only be established once the grade level and footing levels have been confirmed.

2) The residences could be supported on raft foundations, which spread the foundation loads over the entire structure footprint.

3) The residences could be supported on driven steel piles, as described in Section 5.2 above.

Additional geotechnical guidelines would need to be provided if any of the above options are selected. Additional investigation may also be required before those guidelines could be finalized.

5.4 Seismic Considerations

5.4.1 Seismic Site Class

The subsurface conditions beneath the site generally consist of topsoil and/or silty sand/sand overlying firm to very stiff grey brown silty clay crust underlain by generally soft to firm grey silty clay.

The seismic design provisions of the 2012 Ontario Building Code depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or bedrock below founding level. Based on the 2012 Ontario Building Code methodology, the soil stratigraphy on the site could be assigned a Site Class of E (for any structures requiring design under Part 4 of the Ontario Building Code).

5.4.2 Liquefaction Assessment

It is considered that the fine grained silty clay at this site is not potentially liquefiable during an earthquake.

5.5 Basement Excavations

Excavation for basements will be through the surficial layers of topsoil, fill, silty sand/sand and into soft to firm silty clay.

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

The silty sand/sand, weathered silty clay and firm to very stiff silty clay (in the upper portion of the deposit) 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 these materials should be cut back at a minimum of 1 horizontal to 1 vertical. If the water table is encountered within the sandy deposits, side slopes of 3 horizontal to 1 vertical will be required (i.e., Type 4 soil). Should the excavations reach the soft silty clay, side slopes as flat as 3 horizontal to 1 vertical would be required (Type 4 soil).

Some groundwater inflow into the basement excavations should be expected. For the expected excavation depths, it should be possible to handle the groundwater inflow by pumping from well filtered sumps in the excavations.

Where the subgrade is found to be wet and sensitive to disturbance, consideration should be given to placing a mud slab of lean concrete over the subgrade (following inspection and approval by geotechnical personnel).

Based on the drawings provided, the nearest structures or services would be more than about 5 metres from the edge of the excavations and the excavations will therefore not have any potential impacts on adjacent properties and infrastructure, since those properties and infrastructure will be outside the excavations zones of influence.

5.6 Basement 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 slabs. 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 material be positively drained. This could be achieved by providing a hydraulic link between the underslab fill material and the exterior drainage system.

The backfill within the porches should consist of expanded polystyrene (EPS) sheets or blocks with a unit weight of no more than 1.0 kilonewtons per cubic metre placed on a thin levelling pad of sand. EPS meeting the requirements for EPS 19 as defined by ASTM D6817/D6817M-17 should be suitable for this purpose. The surface of the EPS backfill should be covered with a Class II non-woven geotextile conforming to Ontario Provincial Standard Specification (OPSS) 1860 prior to placement of the underslab granular backfill. The underslab granular backfill should consist of a maximum thickness of 300 millimetres of clear crushed stone. The EPS backfill should extend from footing level up to the underside of the porch slab base material.

The general groundwater level at this site is within about 1.9 metres of the existing ground surface. The sandy soils at this site are somewhat permeable and therefore ideally, from a constructability perspective, excavations below the groundwater level in this soil should be avoided. However, if/where the groundwater level is encountered above subgrade level, a geotextile would be required between the clear stone underslab fill and the sandy subgrade soils, to avoid loss of fine soil particles from the subgrade soil into the voids in the clear stone and ultimately into the drainage system. In the extreme case, loss of fines into the clear stone could cause ground loss beneath the slab and plugging of the drainage system. Where a geotextile is required, it should consist of a Class II non-woven geotextile with a Filtration Opening Size (FOS) not exceeding 100 microns, in accordance with OPSS 1860.

5.7 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 and/or 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.

5.8 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, a bond break such as the Platon system sheeting should 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 2012 Ontario Building Code is required, walls backfilled with native 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 backfill, use 19 kilonewtons per cubic metre; and,

H = The height of the basement wall in metres.

Provided the foundation walls are drained, as described above, hydrostatic groundwater pressures need not be considered in the calculation of the lateral earth pressures.

5.9 Site Servicing

Excavation for the installation of site services will be through fill, topsoil, silty sand/sand, firm to very stiff silty clay and will likely extend into the soft silty clay.

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

As described previously, the silts sand/sand and firm to very stiff silty clay would generally be classified as a Type 3 soil in accordance with OHSA. If the water table is encountered within the sandy deposits, side slopes of 3 horizontal to 1 vertical will be required (i.e., Type 4 soil per OHSA). The underlying soft silty clay would also be classified as a Type 4 soil. Accordingly, excavations which do not penetrate into the soft silty clay or below the water table in the sand deposits may be made with side slopes at 1 horizontal to 1 vertical. Should the excavations reach the soft silty clay, side slopes of 3 horizontal to 1 vertical would be required. Alternatively, the excavations could be carried out using steeper side slopes with all manual labour carried out within a fully braced, steel trench box for worker safety. The stability of braced excavations which extend into the soft grey silty clay should be assessed individually based on the length, width, and depth of the trench box. For example, the factor of safety against basal instability of a braced excavation 3 metres wide by 10 metres long to 4 metres depth would be about 2, which is acceptable. Shorter, narrower or shallower trenches will therefore also be stable. Further guidance on trenches that exceed the above limits can be provided. In these areas, excavated materials should not be stock piled beside the excavations, even temporarily.

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 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 or sandy soils on the trench walls 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 material's standard Proctor maximum dry density.

It should generally be possible to re-use sandy soils and silty clay from the weathered zone 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 material's standard Proctor maximum dry density using suitable compaction equipment.

The high moisture 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 roadway surface. If the grey silty clay is used in trenches under roadways, some long term settlement of the pavement surface should be expected.

Impervious dykes or cut-offs should be constructed at 100 metre intervals in the service trenches 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.

Excavations deeper than about 1.9 metres will likely extend below the groundwater level, depending on the time of construction. Groundwater inflow into the excavated trenches should feasibly be handled by pumping from sumps within the excavations. The rate of groundwater inflow from the silty clay is expected to be low, with moderate inflows, occurring from the sandy fill above the silty clay. The actual rate of groundwater inflow to the trench 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 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 excavations. 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.

5.10 Pavement Design

In preparation for pavement construction, all topsoil, disturbed, or otherwise deleterious materials (i.e., fill materials containing organic material) 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 material's 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 the interior 'local' roadways which will not experience 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	375

If any of the roadways would be categorized as 'collector' roadways and/or will experience bus or truck traffic (other than school buses, garbage trucks, moving trucks, etc.), then additional pavement design recommendations will need to be provided.

The granular base and subbase materials should be uniformly compacted as per OPSS 501, Method A. The asphaltic concrete should be compacted in accordance with the procedures outlined in OPSS 310.

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

- Superpave 12.5 mm Surface Course - 40 millimetres
- Superpave 19 mm Base Course - 50 millimetres

The asphaltic cement should consist of PG 58-34 and the design of the mixes should be based on a Traffic Category B.

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.

5.11 Corrosion and Cement Type

No soil samples were submitted for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements as part of this investigation. However, the results of this testing within adjacent phases of the development typically indicate that concrete made with Type GU Portland cement should be acceptable for substructures as well as a low to moderate potential for corrosion of exposed ferrous metal.

5.12 Trees

The results of grain size distribution testing on one sample of the silty clay are provided on Figure 3. The grain size distribution test results indicate that the percentage of the soil particles finer than 0.475 millimetres in diameter is 100 percent. The results of Atterberg limit testing carried out on three samples of the silty clay gave plasticity index values ranging from about 7 to 45 percent and liquid limit values ranging from about 25 to 71 percent. The results of the grain size distribution and Atterberg limit testing indicate that the soil is a silty clay of low to high plasticity.

The results of the shrinkage test are provided in Table 2 and indicate that the silty clay at this site has a shrinkage limit of about 19 and a shrinkage ratio of about 1.8.

The Atterberg limit testing on the three samples of the silty clay from the current investigation are provided in the table below:

Test Pit / Sample Number	Ground Surface Elevation (m)	Sample Depth / Elevation (m)	Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Modified Plasticity Index
19-01 / 6	79.0	4.1 / 74.9	81	71	26	45
19-02 / 3	78.0	3.1 / 74.9	30	25	18	7
19-03 / 1	78.1	2.6 / 75.5	69	20	20	41
					Average	**31**

Based on the results of the laboratory testing, the soil on the east side of the site, as shown on Figure 1, have plasticity indices greater than about 40 percent and only small trees of low water demand can be planted and the setback distance must be at least 7.5 metres.

On the west side of the site, it should be acceptable to reduce the setback distances for small size (mature tree height up to 7.5 metres) and medium size (mature tree height 7.5 to 14 metres) trees to 4.5 meters from the foundations within the residential development. However, in accordance with current City guidelines, the following conditions must also be met:

- The underside of footing elevation must be 2.1 metres or greater below the lowest finished grade;

- Available soil volume must be provided for small and medium trees as per the guidelines;
- Tree species must be very low to moderate Potential Subsistence Risk;
- The foundation walls should be reinforced at least nominally, to provide ductility; and,
- The grading must promote drainage towards the tree root zone.

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.

Golder Associates should be retained to review the final grading plan and specifications for this project prior to construction to ensure that the guidelines in this report have been adequately interpreted.

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 903. 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.

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SAT/WC/mvrd
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IMPORTANT INFORMATION AND LIMITATIONS
OF THIS REPORT

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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.

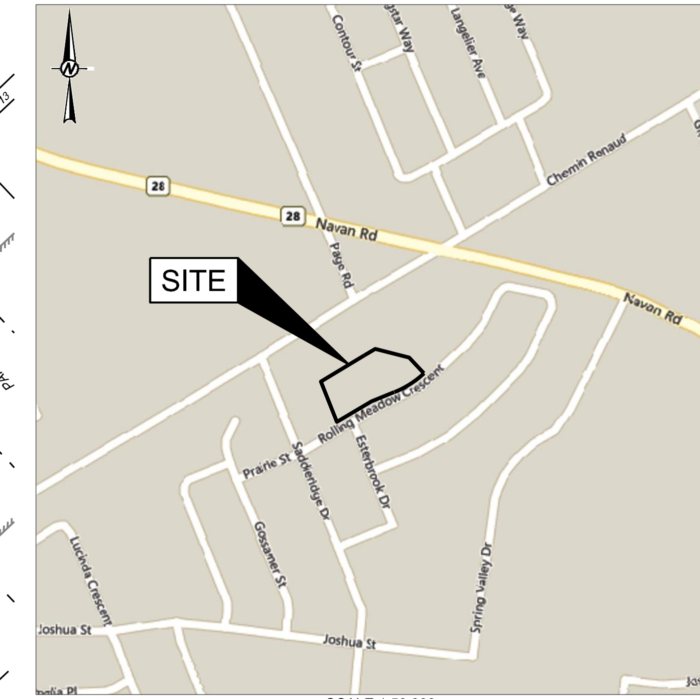
Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

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.

KEY MAP



SCALE 1:50,000

LEGEND

APPROXIMATE PROPOSED TEST PIT LOCATION

APPROXIMATE BOREHOLE LOCATION

AREA WITH TREE PLANTING RESTRICTIONS

REFERENCE(S)

1. BASE PLAN SUPPLIED BY RODERICK LAHEY ARCHITECT INC. ON APRIL 19, 2018, FILE NO 1721 Zen Site Plans 2018.dwg
2. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: UTM ZONE 18, VERTICAL DATUM: CGVD28

0 10 20

1:500 METRES

CLIENT
CLARIDGE HOMES

PROJECT
GEOTECHNICAL INVESTIGATION
SPRING VALLEY TRAILS - ZENS
OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT

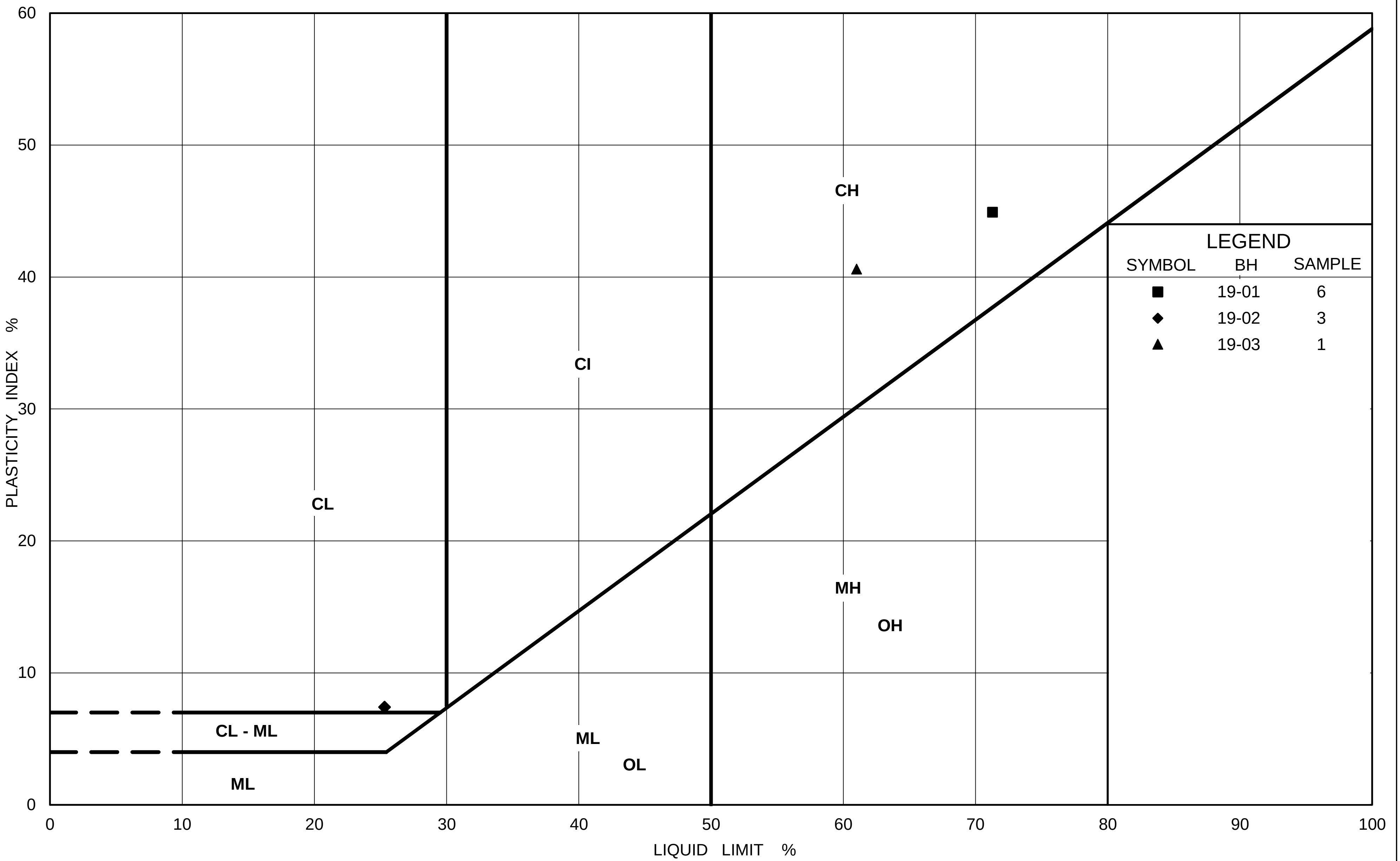
GOLDER

YYYY-MM-DD	2019-03-15
DESIGNED	----
PREPARED	JM/ZS
REVIEWED	WAM
APPROVED	WC

PROJECT NO.	CONTROL	REV.	FIGURE
07-1121-0232	0002	A	1

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

Path: \\golder.gds\gal\ottawa\Active\Spatial_IM\ClaridgeHomes\SpringValleyTrails\CAD\07112110232_Claridge_SpringValleyTrails\40_PROD\0002_GeotechInvestigation\ | File Name: 07112110232-0002-BG-0001.dwg | Last Edited By: zsauve Date: 2019-03-19 Time:3:37:54 PM | Printed By: ZSauve Date: 2019-03-19 Time:3:38:25 PM



GOLDER

PLASTICITY CHART
SILTY CLAY TO CLAY

Figure : 2

Project No.: 07-1121-0232/7000

Compiled By : CW Checked By : MI

GRAIN SIZE DISTRIBUTION	FIGURE 3

Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
19-03	1	2.60-2.90

Project: 07-1121-0232/7000 **Golder Associates**

Created by: CW
Checked by: MI

TABLE 1

RECORD OF TEST PITS

Test Pit Number Elevation (Metres)	Depth (metres)	Description
19-01	0.0 – 0.1	TOPSOIL – (SM) SILTY SAND; brown; non-cohesive, moist
(79.02 metres)	0.1 – 0.5	FILL – (SM) gravelly SILTY SAND, dark brown; non-cohesive, moist
	0.5 – 2.9	(SP) SAND, grey-brown; non-cohesive, moist
	2.9 – 4.5	(CI/CH) SILTY CLAY to CLAY; grey with red-brown mottling; cohesive, w>PL
	4.5	END OF TEST PIT
		Note: water seepage at 2.9 m depth upon completion.

Sample	Depth (m)	Lab Testing
1	2.6 – 2.8	
2	2.9 – 3.1	
3	3.1 – 3.5	
4	3.5 – 3.7	
5	3.7 – 4.0	
6	4.1 – 4.5	w= 81% LL= 71%, PI= 43%

Test Pit Number Elevation (Metres)	Depth (metres)	Description
19-02	0.0 – 0.2	TOPSOIL – (SM) SILTY SAND; dark brown; non-cohesive, moist
(77.67 metres)	0.2 – 2.1	(CI/CH) SILTY CLAY to CLAY, trace sand; grey brown with red-brown mottling; highly fissured (WEATHERED CRUST); non-cohesive, w>PL
	2.1 – 4.3	(CL) SILTY CLAY; grey with red-brown mottling; cohesive, w>PL
	4.3	END OF TEST PIT
		Note: water seepage at 1.5 m depth upon completion.

Sample	Depth (m)	Lab Testing
1	2.4 – 2.7	
2	2.7 – 3.1	
3	3.1 – 3.4	w = 30% LL = 25%, PI = 7%
4	3.4 – 3.7	w = 34%
5	3.7 – 3.9	
6	3.9 – 4.3	

TABLE 1

RECORD OF TEST PITS

Test Pit Number Elevation (Metres)	**Depth (metres)**	**Description**
19-03 (78.15 metres)	0.0 – 0.5	TOPSOIL - (SM) SILTY SAND; dark brown; non-cohesive, moist
	0.5 – 1.1	(SP) SAND; brown; non-cohesive, moist
	1.1 – 2.3	(CI/CH) SILTY CLAY to CLAY, trace sand; grey brown with red-brown mottling; (WEATHERED CRUST); non-cohesive, w>PL
	2.3 – 4.3	(CI/CH) SILTY CLAY to CLAY; grey with red-brown mottling; silty sand layers; cohesive, w>PL
	4.3	END OF TEST PIT

Note: water seepage at 1.5 m depth upon completion.

Sample	Depth (m)	Lab Testing
1	2.6– 2.9	w= 69% LL= 61%, PI= 41%
2	2.9 – 3.3	
3	3.3 – 3.6	
4	3.6 – 4.0	
5	4.0 – 4.3	

GOLDER

SHRINKAGE LIMIT DETERMINATIONS
ASTM D4943

Borehole Number	TP19-02	
Sample Number	4	
Depth, m	3.4-3.7	
Shrinkage Dish Number	1	2
Mass of the dry soil pat, g	20.22	20.04
Mass of dry soil pat + shrinkage dish, g	43.39	42.30
Mass of shrinkage dish, g	23.17	22.26
Volume of shrinkage dish, cm^3	13.40	13.33
Mass of wet soil + shrinkage dish, g	49.60	48.40
Moisture content of the soil	30.71	30.44
Mass of dry soil pat before waxing, g	20.22	20.04
Volume of dry soil pat + wax, cm^3	14.90	16.11
Mass of dry soil pat + wax in air, g	23.79	24.73
Mass of dry soil pat + wax in water, g	8.89	8.62
Mass of wax, g	3.57	4.69
Volume of wax, cm^3	3.86	5.07
Specific gravity of wax	0.925	0.925
Volume of dry soil pat, cm^3	11.04	11.04
SHRINKAGE LIMIT, SL	**19.04**	**19.01**
SHRINKAGE RATIO, R	**1.83**	**1.82**

		Date Tested:	3/08/19
Project Number:	07-1121-0232	Tested By:	Frank M
Project Name CLARIDGE(CARSON)/SPRING VALLEY/OTT			

Notes:

Test carried out using wax method (Microsere Wax 5214)

Checked By: LM

APPENDIX A

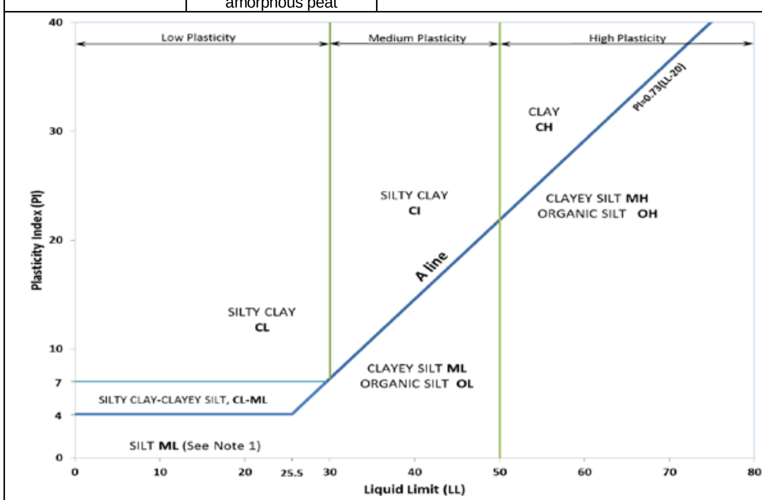
List of Abbreviations and Symbols Record of Borehole Sheets

METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil		Laboratory Tests	Field Indicators: Dilatancy	Field Indicators: Dry Strength	Field Indicators: Shine Test	Field Indicators: Thread Diameter	Field Indicators: Toughness (of 3 mm thread)	Organic Content	USCS Group Symbol	Primary Name
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS	(Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
					Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
					Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
				Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
					None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS	(PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% **(see Note 2)**	CL	SILTY CLAY
				Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
				Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures								30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat								75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.
Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.
For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.
For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.
A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

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)
> 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.). Values reported are as recorded in the field and are uncorrected.

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
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
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 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 to 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 the effects of overburden pressure.
2. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

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'[1,2] (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.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

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
$\sigma_1, \sigma_2, \sigma_3$	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

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

(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 = $(w_l - w_p)$
NP	non-plastic
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 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. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

PROJECT: 07-1121-0232

RECORD OF BOREHOLE: 08-204

SHEET 1 OF 1

LOCATION: N 5032526.8 ;E 381695.0

BORING DATE: October 8 2008

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	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		76.39							
0	Power Auger 200 mm Diam. (Hollow Stem)	Grey brown SILTY CLAY (FILL)		0.00							
		TOPSOIL		76.15 0.24							
		Very stiff to firm, grey brown to red brown SILTY CLAY (WEATHERED CRUST)		75.93 0.46							
1					1	SS	6				
					2	SS	WH				
2		Soft to firm, grey SILTY CLAY		74.50 1.89							
3											
4											
5											
6					3	SS	WH				
7											
		End of Borehole		68.77 7.62							
8											
9											
10											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: D.J.S.

CHECKED: S.A.T.

RECORD OF BOREHOLE: 08-205

PROJECT: 07-1121-0232

SHEET 1 OF 1

LOCATION: N 5032570.9 ;E 381718.9

BORING DATE: October 8 2008

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	77.58				
0	TOPSOIL	0.00				
	Very stiff to firm, grey brown SILTY CLAY (WEATHERED CRUST)	77.28 / 0.30				Bentonite Seal
1			1	SS	6	
	Soft to firm, grey SILTY CLAY	75.87 / 1.71	2	SS	WH	
3						Native Backfill
5						Benonite Seal
						Silica Sand
6						Standpipe
			3	SS	WH	Benonite Seal
7						Cave
	End of Borehole	69.96 / 7.62				
8						WL in Screen at Elev. 75.70 m on Oct. 17, 2008

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

DEPTH SCALE 1 : 50

GOLDER

LOGGED: D.J.S.

CHECKED: S.A.T.

PROJECT: 07-1121-0232

RECORD OF BOREHOLE: 08-206

SHEET 1 OF 1

LOCATION: N 5032572.3 ;E 381786.6

BORING DATE: October 9, 2008

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	78.91				
TOPSOIL	0.00				Bentonite Seal
Yellow brown SILTY SAND	78.61 / 0.30				
Loose, brown SAND, trace silt	78.36 / 0.55	1	SS	12	
Compact, brown SILTY SAND	77.81 / 1.10				
Very stiff to stiff, grey brown SILTY CLAY (WEATHERED CRUST)	77.54 / 1.37	2	SS	5	
Stiff to firm, grey SILTY CLAY, with occasional sand seams	76.47 / 2.44				Native Backfill
Firm, grey SILTY CLAY	75.56 / 3.35	3	SS	WH	Benonite Seal; Silica Sand; Standpipe; Cave
End of Borehole	71.29 / 7.62				

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

DEPTH SCALE 1 : 50

GOLDER

LOGGED: D.J.S.

CHECKED: S.A.T.

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