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Geotechnical Investigation
Proposed Residential Subdivision
1055 Klondike Road
Ottawa, Ontario

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Geotechnical Investigation
Proposed Residential Subdivision
1055 Klondike Road
Ottawa, Ontario

April 4, 2018
Project: 64153.85

experience • knowledge • integrity

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April 4, 2018 File: 64153.85 – R0

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Attention: Mark Bissett, P.Eng., Senior Project Manager

Re: Geotechnical Investigation
Proposed Residential Subdivision
1055 Klondike Road
Ottawa, Ontario

Enclosed is our geotechnical investigation report for the above noted project, in accordance with our proposal. This report was prepared by Gregory Davidson, B.Eng., E.I.T. and Brent Wiebe, P.Eng.

Greg Davidson, B.Eng., E.I.T.

Brent Wiebe P.Eng.
Senior Geotechnical Engineer

Enclosure:
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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for a proposed residential subdivision at 1055 Klondike Road in Ottawa, Ontario (refer to Key Plan, Figure 1). The purpose of the investigation was to identify the subsurface conditions at the site by means of a limited number of boreholes and, based on the results of the factual information obtained, to provide engineering guidelines and recommendations on the geotechnical design aspects of this project, along with construction considerations that could influence design decisions.

This investigation was carried out in general accordance with our proposal dated February 15, 2018.

2.0 PROJECT DESCRIPTION AND BACKGROUND INFORMATION

Plans are being prepared for the development of 1055 Klondike Road into a residential subdivision. It is understood that the proposed development is still in preliminary design stages, but will consist of a residential subdivision with an internal roadway and municipal water and sewer services. It is understood that the residences will be a mixture of single family homes, duplexes, townhomes and low rise buildings. The site currently consists of grass fields with a gravel laneway through the centre to an abandoned farm house. The site is bounded by a meandering creek to the north and west, grass fields and a residential house to the east and Klondike Road to the south. There is a residential property located at the southwest corner of the site.

2.1 Previous Investigation by GEMTEC formerly Houle Chevrier Engineering Ltd.

GEMTEC Consulting Engineers and Scientists Limited (GEMTEC), formerly Houle Chevrier Engineering Ltd. previously conducted a preliminary geotechnical investigation and slope stability assessment at this site. The results of our previous investigations are summarized in our report titled: “Preliminary Geotechnical Investigation, Proposed Residential Subdivision, 1055 Klondike Road, Ottawa, Ontario”, dated April 13, 2017. The results of that investigation should be read in conjunction with this report.

In preparation for submission for Draft Plan approval, additional boreholes are required to conform with City of Ottawa guidelines as well as a slope stability assessment of the site and a Phase One ESA. The results of the slope stability assessment and Phase One ESA will be provided separately and should be read in conjunction with this report.

The relevant borehole information from the 2017 investigation at this site is provided on the Record of Borehole sheets in Appendix A.

The approximate locations of the boreholes advanced as part of previous investigation are provided on the Borehole Location Plan, Figure 2.

2.2 Site Geology

Surficial geology and drift thickness maps of the Ottawa area indicate that the site is underlain by alluvial sediments of fluvial terraces, sand, silt and clay. Bedrock geology maps indicate that the site is underlain by interbedded sandstone and dolostone bedrock of the March formation at depths of about 5 to 10 metres.

3.0 METHODOLOGY

3.1 Geotechnical Investigation

The field work for this investigation was carried out on March 8 and 9, 2018. At that time, five (5) boreholes, numbered 18-1 to 18-5 inclusive, were advanced at the site by George Downing Estate Drilling Ltd. to depths ranging from about 5.9 to 10.2 metres below surface grade (elevations 68.2 to 71.8 metres, geodetic datum).

Standard penetration tests (SPT) were carried out in the boreholes and samples of the soils encountered were recovered using a 50 millimetre diameter split barrel sampler.

The field work was observed throughout by a member of our engineering staff who directed the drilling operations and logged the samples and boreholes.

Three (3) standpipe piezometeres were installed at borehole locations 18-1, 18-3 and 18-5 to facilitate groundwater level measurements and sampling, if required.

Following completion of the drilling, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples were submitted for moisture content, grain size distribution testing and Atterberg limits. One (1) sample of the soil recovered from borehole 18-2 was sent to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel.

The results of the boreholes are provided on the Record of Borehole sheets in Appendix A. The approximate locations of the boreholes are shown on the Borehole Location Plan, Figure 2. The results of the laboratory classification tests on the soil samples are provided on Figures B1 to B3, inclusive, in Appendix B. The results of the chemical analysis of a sample of soil relating to corrosion of buried concrete and steel are provided in Appendix C.

The borehole locations were selected by GEMTEC and positioned on site relative to existing features. The ground surface elevations at the location of the boreholes were determined using a Trimble R10 global positioning system. The elevations are referenced to geodetic datum and are considered to be accurate within the tolerance of the instrument.

4.0 SUBSURFACE AND FOUNDATION CONDITIONS

4.1 General

The subsurface conditions described below indicate the conditions at the specific test locations only. Boundaries between zones are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the frequency and recovery of samples, the method of sampling and the uniformity of the subsurface conditions. Subsurface conditions at other than the test locations may vary from the conditions encountered in boreholes.

The soil descriptions in this letter are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgment and GEMTEC does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The results of the boreholes are provided on the Record of Boreholes sheet in Appendix A. The following presents an overview of the subsurface conditions.

4.2 Topsoil

A layer of topsoil was encountered from surface grade at borehole locations 18-1, 18-2 and 18-4. The topsoil is composed of dark brown to brown silty sand/ sandy silt with organic material. The thickness of the topsoil layer ranges from 180 to 310 millimetres.

Moisture content testing carried out on a sample of the topsoil material indicates a moisture content of about 37 percent.

4.3 Granular Driveway Material

A layer of grey crushed sand and gravel was encountered from surface grade at borehole 18-3 which was advanced through the existing driveway at this site. The granular material has a thickness of about 150 millimetres.

4.4 Fill Material

Fill material was encountered below the granular driveway material at borehole 18-3 and from surface grade at borehole 18-5. The fill material can be described as dark brown to brown silty sand with organic material and grey brown silty clay with pockets of dark brown organic material. The fill material extends to depths of about 0.9 and 3.3 metres below surface grade (elevations 74.6 and 77.9 metres, geodetic datum) at the location of boreholes 18-3 and 18-5, respectively.

4.5 Silt and Sand

Deposits of silty sand were encountered below the topsoil and fill material at all borehole locations. The silty sand can be described as grey brown to brown silty sand with trace roots at borehole 18-3 and with trace wood at borehole location 18-5.

The thickness of the sand ranges from 0.8 to 2.0 metres and extends to depths of about 1.0 to 4.7 metres below surface grade (elevations 73.1 to 77.3 metres, geodetic datum).

Standard penetration tests carried out in the silty sand deposit gave N values ranging between 3 and 8 blows per 0.3 metres of penetration, which reflects a very loose to loose relative density.

The results of grain size distribution testing on a sample of the silt and sand is provided on Figure B1 in Appendix B and summarized in Table 4.1.

Table 4.1 – Summary of Grain Size Distribution Testing (Silt and Sand)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt & Clay (%)
18-1	2	0.8 – 1.4	0	51	49

Moisture content testing carried out on samples of the silt and sand indicate moisture contents ranging from about 16 to 24 percent.

A native deposit of brown silty sand with trace wood was encountered in borehole 18-5 at a depth of about 3.3 metres below ground surface. The silty sand has a thickness of about 1.5 metres and extends to a depth of about 4.7 metres below surface grade (elevation 73.1 metres, geodetic datum).

4.6 Sand

Native deposits of sand were encountered below the silty sand at borehole locations 18-2 to 18-4 inclusive, at depths ranging from about 0.9 to 1.1 metres below surface grade. The sand can be described as brown, fine to medium grained with trace to some silt and was layered with grey brown silty sand at borehole 18-2.

The thickness of the sand ranges from 0.6 to 1.1 metres and extends to depths of about 2.1 to 3.1 metres below surface grade (elevations 75.5 to 76.3 metres, geodetic datum).

Standard penetration tests carried out in the sand deposit gave N values ranging between 5 and 10 blows per 0.3 metres of penetration, which reflects a loose to compact relative density.

Moisture content testing carried out on samples of the native sand indicate moisture contents ranging from about 10 to 19 percent.

4.7 Weathered Crust (Silt and Clay)

Native deposits of weathered, grey brown silt and clay with trace amounts of sand (weathered crust) were encountered underlying the sand and silty sand at all borehole locations. The weathered crust was encountered at depths ranging from about 2.1 to 4.7 metres below surface grade (elevations 73.1 to 76.3 metres, geodetic datum). Where fully penetrated, the weathered crust has a thickness ranging from about 3.0 to 4.6 metres and extends to depths ranging from about 6.1 to 7.7 metres below surface grade (elevations 70.1 to 72.3 metres, geodetic datum).

Borehole 18-1 was terminated within the weathered crust at a depth of about 5.9 metres below ground surface (elevation 71.8 metres, geodetic datum).

Standard penetration tests carried out in the weathered crust gave N values ranging between 2 and 12 blows per 0.3 metres of penetration. Based on our experience with native clays in the Ottawa region, N values of 2 or greater reflect a stiff to very stiff consistency.

In situ vane shear strength testing carried out in the silt and clay indicates undrained shear strength values of greater than 100 kilopascals which indicates a very stiff consistency.

The results of grain size distribution testing on samples of the silty clay weathered crust is provided on Figure B2 in Appendix B and summarized in Table 4.2.

Table 4.2 – Summary of Grain Size Distribution Testing (Weathered Crust)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
18-3	5	3.1 – 3.7	0	3	55	43
18-4	4	2.3 – 2.9	0	3	41	56

The results of Atterberg limit testing carried out on samples of the silty clay weathered crust are provided on Figure B3 in Appendix B and summarized in Table 4.3.

Table 4.3 – Summary of Atterberg Limit Testing (Weathered Curst)

Borehole	Sample Number	Sample Depth (metres)	Water Content (%)	LL (%)	PL (%)	PI (%)
18-3	5	3.1 – 3.7	43.3	54.6	22.8	31.9
18-4	4	2.3 – 2.9	45.7	48.9	21.5	27.5

The Atterberg limit testing indicates that the material has low to high plasticity.

Moisture content testing carried out on samples of the weathered crust indicate moisture contents ranging from about 43 to 55 percent.

4.8 Silty Clay

The grey brown silty clay weathered crust transitions to grey silty clay at depths of about 6.1 to 7.6 metres below surface grade in boreholes 18-2 to 18-4, inclusive. Where fully penetrated the grey silty clay has a thickness ranging from about 2.3 to 3.3 metres.

Borehole 18-3 was terminated within the grey silty clay at a depth of about 8.2 metres below surface grade (elevation 70.6 metres, geodetic datum).

In situ vane strength tests carried in the grey silty clay gave undrained shear strength values ranging from about 57 to 65 kilopascals, which reflects a stiff consistency. The remoulded vane shear test values generally ranged from 10 to 15, indicating that the sensitivity of the silty clay is medium to sensitive.

The results of grain size distribution testing on a sample of the grey silty clay is provided on Figure B2 in Appendix B and summarized in Table 4.4.

Table 4.4 – Summary of Grain Size Distribution Testing (Silty Clay)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
18-2	8	6.1 – 6.7	0	1	49	50

The results of Atterberg limit testing carried out on a sample of the silty clay weathered curst are provided on Figure B3 in Appendix B and summarized in Table 4.5.

Table 4.5 – Summary of Atterberg Limit Testing (Silty Clay)

Borehole	Sample Number	Sample Depth (metres)	Water Content (%)	LL (%)	PL (%)	PI (%)
18-2	8	6.1 – 6.7	54.9	43.8	21.6	22.2

The Atterberg limit testing indicates that the material has low plasticity.

Moisture content testing carried out on samples of the silty clay indicate moisture contents ranging from about 52 to 55 percent.

It should be noted that the moisture content of the silty clay is above the liquid limit value at the tested depth (approximate elevation of 72.6 metres, geodetic datum).

4.9 Glacial Till

Glacial till was encountered below the native silty clay at borehole locations 18-2 and 18-4 at depths of about 9.1 and 8.4 metres below surface grade, respectively. Glacial till is a heterogeneous mixture of all grain sizes, which at this site, can be described as grey sand and silt with some clay, gravel and cobbles.

Boreholes 18-2 and 18-4 were terminated within the glacial till due to sampler and auger refusal at depths of about 10.2 and 8.6 metres below surface grade (elevations 68.2 and 69.1 metres, geodetic datum), respectively. It is possible that the refusal depths represent the surface of bedrock, however, it should be noted that practical refusal can occur on cobbles/boulders within a glacial till deposits, or on bedrock.

Standard penetration tests carried out in the glacial till gave N values ranging between 15 to 27 blows per 0.3 metres of penetration, and 50 blows per 0.1 metres of penetration, which reflects a compact to very dense relative density.

Moisture content testing carried out on samples of the glacial till indicates moisture contents ranging from about 9 to 11 percent.

4.10 Auger Refusal

Borehole 18-5 was terminated at a depth of about 7.7 metres below surface grade (elevation 70.1 metres, geodetic datum) due to auger refusal on possible bedrock.

4.11 Groundwater

No groundwater seepage was observed in the open boreholes prior to backfilling. However, samples recovered from boreholes 18-2 and 18-4, became saturated at depths of about 2.3 metres below surface (elevations 76.0 and 75.3 metres geodetic), respectively.

Standpipe piezometers were installed in boreholes 18-1, 18-3 and 18-5 to measure stabilized groundwater conditions. The groundwater levels were observed at depths ranging from about 2.0 to 6.3 metres below surface grade (elevations 72.3 to 75.7 metres, geodetic datum) on March 15, 2018.

It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring or following periods of precipitation.

4.12 Soil Chemistry Relating to Corrosion

The results of chemical testing of a sample of soil from test pit 18-2 sample 5 are provided in Appendix D and summarized below:

- pH 7.34
- Sulphate Content 64 micrograms per gram
- Chloride Content 54 micrograms per gram
- Resistivity 39.1 Ohm metre
- Conductivity 256 microsiemens per centimetre

5.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the boreholes advanced as part of this investigation and the project requirements. It is stressed that the information in the following sections is provided for the guidance of the designers and is intended for this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities of this site or adjacent properties, and/or resulting from the introduction onto the site from materials from offsite sources are outside the terms of reference for this report and have not been investigated or addressed.

5.2 Grade Raise Restrictions

Much of the site is underlain by a deposit of silty clay, which has a somewhat limited capacity to support loads imposed by grade raise fill material and, to a lesser extent, the foundations of the residential dwellings.

The placement of fill material must therefore be carefully controlled so that the stress imposed by the fill material does not result in excessive consolidation of the grey silty clay deposit. The settlement response of the silty clay deposit to the increase in stress caused by fill material and groundwater lowering is influenced by variables such as the existing effective overburden pressure, the past pre-consolidation pressure for the silty clay, the compressibility characteristics of the silty clay, and the presence or absence of drainage paths, etc. It is well established that the settlement response of silty clay deposits can be significant when the stress increase is at or near the difference between the preconsolidation pressure (P_c) and the existing overburden stress (σ_{vo}').

The site grade raise restriction is also dependent on the footing depth. We have assumed that the footing depths for the residential buildings are 1.5 metres below existing grade for assessment purposes.

The proposed grading plan for the subdivision was not available at the time of submission of this report. In general, it is considered that the native deposits at this site can likely accommodate a grade raise of 2 metres above original ground surface, which should be sufficient for the majority of the site. However, it is noted that a significant elevation change occurs over the central portion of this property. In particular, the existing grade across the proposed cul-de-sac at the end of the internal roadway changes by over 5 metres from the southwest to northeast. Based on the results of the borehole investigations to date it is considered possible that this grade raise could be accommodated without the need for light weight fill but this will likely need to be confirmed through on site monitoring following the fill placement.

It is recommended that the proposed grading plan for the subdivision be reviewed by GEMTEC prior to detailed design. Following this review, we will provide recommendations on fill placement in conjunction with settlement monitoring plates in advance of construction (6 months to 1 year, or more if possible) for areas where significant grade raise is required. Surcharge loading may also be suggested depending on the proposed grade raise.

Based on the results of the settlement monitoring, guidelines on the use of light weight fill (if required) could be provided for the final design of the development.

5.3 Proposed Residential Buildings

5.3.1 Excavation

The excavations for the foundations will be through topsoil, sand, and weathered silty clay crust. The sides of the excavations should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the shallow native overburden deposits can be classified as Type 3 and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical extending upwards from the base of the excavation.

Excavation of the native silty clay soils above or below the groundwater should not present any significant excavation constraints. In contrast, excavation in the native silty sand or sand below the groundwater level could present constraints. Groundwater inflow from silty sand and sand deposits could cause sloughing of the sides of the excavation and disturbance to the soils at the bottom of the excavation, flatter side slopes and or drainage measures may be required if excavation is required below the groundwater level in these deposits.

All foundation excavations should be undertaken with an excavator equipped with a smooth ditching bucket to minimize disturbance of the subgrade soils which are sensitive to disturbance from construction activities. Based on our previous experiences at sites underlain by weathered silty clay crust, it is possible that the upper 0.3 to 0.5 metres of the weathered silty clay may be affected by past frost action and may unavoidably "peel" during excavation. If this occurs, an allowance should be made to remove and replace any disturbed silty clay with compacted granular material within the building areas.

5.3.2 Groundwater Pumping and Management

Possible groundwater inflow from the overburden deposits into the excavations could be controlled by pumping from filtered sumps within the excavations. It is not expected that short term pumping during excavation will have any significant affect on nearby structures and services.

Suitable detention and filtration will be required before discharging water. The contractor should be required to submit an excavation and groundwater management plan for review.

5.3.3 Subgrade Preparation and Placement of Engineered Fill

Any existing topsoil, organic material, fill, and/or disturbed soil should be removed from below the proposed structures.

Imported granular material (engineered fill) should be used to raise the grade in areas where the proposed founding level is above the level of the native soil, or where subexcavation of material is required below proposed founding level. The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular

B Type II and should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. To allow spread of load beneath the footings, the engineered fill should extend horizontally at least 0.3 metres beyond the footings and then down and out from the edges of the footings at 1 horizontal to 1 vertical, or flatter. The excavations should be sized to accommodate this fill placement.

5.3.4 Spread Footing Design

In general, the native sand and silty clay and or engineered fill above the native soils is considered suitable to support residential structures founded on conventional spread footing foundations. For this project, we have assumed that the footings will not extend to a depth beyond the weathered crust. The topsoil fill materials and the deposit of brown silty sand with trace wood (encountered in borehole 18-5) are not considered suitable for the support of the proposed structures or concrete floor slabs and should be removed from the proposed building areas.

Provided that the grade raise restrictions above are followed, footings for residential buildings could be sized using preliminary bearing pressures provided in Table 5.1.

Table 5.1 – Allowable Bearing Pressures for Foundations

Subgrade Material	Allowable Bearing Pressure for Foundations
Native Sand	90
Weathered Silty Clay	100
Engineered fill material, over undisturbed native deposits	150

The foundation bearing value should be determined on a lot by lot basis by geotechnical personnel at the time of construction.

Some of the native soils at this site are sensitive to construction operations, from ponded water and frost action. The construction operations should therefore be carried out in a manner that minimizes disturbance of the subgrade surfaces.

The post construction total and differential settlement of footings should be less than 25 and 15 millimetres, respectively, provided that all loose or disturbed soil is removed from the bearing surfaces and provided that any engineered fill material is compacted to the required density.

5.3.5 Frost Protection of Foundations

All exterior footings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated exterior footings adjacent to surfaces which are cleaned of snow cover during the winter months should be provided with a minimum of 1.8 metres of earth cover. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. Further details regarding the insulation of foundations could be provided at the detailed design stage, if necessary.

5.3.6 Foundation Wall Backfill and Drainage

In accordance with the Ontario Building Code, the following alternatives could be considered for drainage of the foundation walls:

- Damp proof the exterior of the foundation walls and backfill the walls with free draining, non-frost susceptible sand or sand and gravel such as that meeting OPSS requirements for Granular B Type I or II. OR

- Damp proof the exterior of the foundation walls and install an approved proprietary drainage system on the exterior of the foundation walls and backfill the walls with native material or imported soil. It is pointed out that the moisture content of the native material may be above the optimum moisture content for compaction. As such, in areas where hard surfacing will abut the buildings, it is suggested that imported sand or sand and gravel be used for foundation backfill material to reduce the potential for post construction settlement of the backfill and damage to the hard surfacing.

The backfill should be compacted in maximum 300 millimetres thick lifts to at least 95 percent of the standard Proctor dry density value using suitable vibratory compaction equipment.

A perforated plastic foundation drain with a surround of clear crushed stone should be installed on the exterior of the foundation walls. A nonwoven geotextile should be placed between the top of the clear stone and any sandy foundation wall backfill material to avoid loss of sand backfill into the voids in the clear stone (and possible post construction settlement of the ground around the houses). The top of the drain should be located below the bottom of the floor slab. The drain should outlet to a sump from which the water is pumped or should drain by gravity to a storm sewer or other suitable outlet.

5.3.7 Garage Foundation and Pier Backfill

The backfill against isolated (unheated) walls or piers should consist of free draining, non-frost susceptible material, such as sand/sand and gravel meeting OPSS Granular B Type I or II requirements. The backfill should be compacted in maximum 300 millimetres thick lifts to at least 95 percent of the standard Proctor dry density value using suitable vibratory compaction equipment.

Other measures to prevent frost jacking of these foundation elements could be provided, if required.

5.3.8 Concrete Slab Support

To provide predictable settlement performance of the undeground slab, all topsoil, fill material, disturbed soil, and other deleterious materials should be removed from the slab area.

The base for the floor slab should consist of 19 millimetre clear crushed stone. Allowance should be made for between 150 and 200 millimetres of base material. Nominal compaction of the clear stone is recommended to consolidate the material into place.

If clear crushed stone is used below the floor slab, underfloor drains are not considered essential provided that drains are installed to link any hydraulically isolated areas in the basement. The drains should outlet by gravity to a sump from which the water is pumped or drained by gravity to a storm sewer or other suitable outlet.

The ACI 302.1R-04 “Guide for Concrete Floor and Slab Construction” should be referenced for design purposes.

A polyethylene vapour retarder is recommended below the floor slabs.

5.3.9 Foundation Wall Support

In accordance with the City of Ottawa document titled: “Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines” and discussed in Section 6.0 of this report and in the separate letter to be provided, reinforcement of foundations walls for all structures will be required if deciduous tree planting is to be carried out in proximity to the foundations.

Foundation walls at this site will require at least nominally, two upper and two lower 15M bars for reinforcement.

5.3.10 Seismic Site Classification

According to Table 4.1.8.4.A of the Ontario Building Code, 2012, Site Class D should be used for the seismic design of the structures bearing on native soils or on engineered fill material over native soils.

In our opinion the soils at this site are not considered to be liquefiable or collapsible under seismic loads.

5.4 Site Services

5.4.1 Excavation

Based on the available subsurface information, the excavations for the services within the site will be carried out through topsoil, fill material, silty clay, clayey silt and possibly glacial till.

The sides of the excavations within overburden soils should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act.

According to the Act, the soils at this site can be classified as Type 3 soils. Therefore, for design purposes, allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation slopes within the native soils at this site. As an alternative to sloping the excavations, all services installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose.

The groundwater inflow should be controlled throughout the excavation and pipe laying operations by pumping from sumps within the excavation. Notwithstanding, some disturbance and loosening of the subgrade materials could occur, and allowance should be made for subexcavation and additional pipe bedding (sub-bedding) material, as discussed later in this report.

5.4.2 Groundwater Pumping

Possible groundwater inflow from the overburden deposits into the excavations could be controlled by pumping from filtered sumps within the excavations. It is not expected that short term pumping during excavation will have any significant affect on nearby structures and services.

The groundwater handling should be carried out in accordance with provincial and local regulations. To reduce the groundwater pumping requirements, we suggest that the excavation be planned for the dry period of the year (i.e. June to September).

Suitable detention and filtration will be required before discharging water. The contractor should be required to submit an excavation and groundwater management plan for review.

Depending on the depth of proposed services and groundwater level at the time of construction, an Environmental Activity and Sector Registry (EASR) in accordance with Environmental Protection Act Part II or a Category 3 Permit to Take Water may be required. Further details could be provided as the design progresses.

5.4.3 Pipe Bedding and Cover

The bedding for the sanitary sewers, storm sewers and watermains should be in accordance with OPSD 802.010/802.013 and 802.031/802.033 for flexible and rigid pipes, respectively. The pipe bedding should consist of at least 150 millimetres of well graded crushed stone meeting OPSS requirements for Granular A. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A and Granular B Type II material. Since the source of recycled material cannot be determined, it is suggested that any granular materials used in the service trenches be composed of virgin (i.e., not recycled) material only.

Allowance should be made for subexcavation of any existing fill, organic deposits, or disturbed material encountered at subgrade level.

Allowance should be made to place a subbedding layer composed of 150 to 300 millimetres of OPSS Granular B Type II in areas where wet clayey silt is encountered at the pipe subgrade level to reduce the potential for disturbance.

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

The use of clear crushed stone should not be permitted for the installation of site services, since it could exacerbate groundwater lowering of the overburden materials due to “French Drain” effects.

The subbedding, bedding and cover materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.4.4 Trench Backfill

The general backfilling procedures should be carried out in a manner that is compatible with the future use of the area above the service trenches.

In areas where the service trench will be located below or in close proximity to existing or future roadway areas, acceptable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetration in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent section of roadway. Where native backfill is used, it should match the native materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I. The depth of frost penetration in areas that are kept clear of snow and where trench backfill consists of broadly graded shattered rock fill or earth fill is expected to be about 1.8 metres. It is our experience, however, that the frost penetration can be as much as 2.4 metres when the trench backfill consists solely of relatively open graded rock fill. Where cover requirements are not practicable, the pipes could be protected from frost using a combination of earth cover and insulation. Further details regarding insulation could be provided, if required.

It is anticipated that most of the inorganic overburden materials encountered during the subsurface investigation will be acceptable for reuse as trench backfill. Topsoil or other organic material should be wasted from the trench. As indicated above, the moisture content of the grey silty clay is above the liquid limit. As such, this material may become disturbed during excavation and may not be suitable for use as trench backfill. An assessment of the adequacy of this material to be reviewed as trench backfill could be made by a geotechnical personnel at the time of construction. If blast rock is used as backfill within the service trench, it should be mostly 300 millimetres, or smaller, in size and should be well graded. To prevent ingress of fine material into voids in the blast rock, the upper surface of the blast rock should be covered with a thin layer of well graded crushed stone (e.g. OPSS Granular B Type II).

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, curbs, driveways, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. Rock fill should be placed in maximum 500 millimetre thick lifts and compacted with a large drum roller, the haulage and spreading equipment, or a combination of both. The specified density for compaction of the backfill materials may be reduced where the trench backfill is not located below or in close proximity to existing or future areas of hard surfacing and/or structures.

The weathered crust and silty clay from the excavations may have moisture contents above optimum for compaction. Furthermore, most of the overburden deposits at this site are sensitive to changes in moisture content. Unless these materials are allowed to dry, the specified densities will not likely be possible to achieve and, as a consequence, some settlement of these backfill materials could occur. Consideration could be implementing one or a combination of the following measures to reduce post construction settlement above the trenches, depending on the weather conditions encountered during the construction:

- Allow the overburden materials to dry prior to compaction.
- Reuse any wet materials in the lower part of the trenches and make provision to defer final paving of any roadways for 6 months, or longer, to allow some the trench backfill settlement to occur and thereby improve the final roadway appearance.
- Reuse any wet materials outside hard surfaced areas and where post construction settlement is less of a concern (such as landscaped areas).

The soils that exist at this site are highly frost susceptible and are prone to significant ice lensing. In order to carry out the work during freezing temperatures and maintain adequate performance of the trench backfill as a roadway subgrade, the service trenches should be opened for as short a time as practicable and the excavations should be carried out only in lengths which allow all of the construction operations, including backfilling, to be fully completed in one working day. The sides of the trenches should not be allowed to freeze. In addition, the backfill should be excavated, stored and replaced without being disturbed by frost or contaminated by snow or ice.

5.4.5 Seepage Barriers

The granular bedding in the service trench could act as a “French Drain”, which could promote groundwater lowering. As such, we suggest that seepage barriers be installed along the service trenches at strategic locations at a horizontal spacing of about 100 metres and where the property meets Klondike Road. The seepage barriers should begin at subgrade level and extend vertically through the granular pipe bedding and granular surround to within the native backfill materials, and horizontally across the full width of the service trench excavation. The seepage barriers could consist of 1.5 metre wide dykes of compacted silty clay. The silty clay should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. The locations of the seepage barriers could be provided as the design progresses.

5.5 Roadway

5.5.1 Subgrade Preparation

In preparation for roadway construction at this site, all surficial topsoil, fill material and any soft, wet or deleterious materials should be removed from the proposed roadway areas.

Prior to placing granular material for the internal roads, the exposed subgrade should be inspected and approved by geotechnical personnel. Any soft areas should be subexcavated and replaced with suitable (dry) earth borrow or well shattered and graded rock fill material that is frost compatible with the materials exposed on the sides of the area of subexcavation.

Similarly, should it be necessary to raise the roadway grades at this site, material which meets OPSS specifications for Select Subgrade Material, earth borrow or well shattered and graded rock fill material may be used.

The select subgrade material or earth borrow should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment. Rock fill should also be placed in thin lifts and suitably compacted either with a large drum roller, the haulage and spreading equipment, or a combination of both.

Truck traffic should be avoided on the native soil subgrade or the trench backfill within the roadways especially under wet conditions.

5.5.2 Pavement Structure

For the access roadways and parking lots, the following minimum pavement structure should be used:

- 100 millimetres of hot mix asphaltic concrete (40 millimetres of Superpave 12.5 (Traffic Level B) over 60 millimetres of Superpave 19.0 (Traffic Level B)), over
- 150 millimetres of OPSS Granular A base over
- 450 millimetres of OPSS Granular B, Type II subbase

It is noted that the above pavement structure meets City of Ottawa Standard Drawing No. R-27 (Rural Local Roadway Cross Section Over Earth) requirements.

The above pavement structure assumes that any trench backfill is adequately compacted, and that the access roadway and parking lot subgrade surfaces are prepared as described in this report. If the subgrade surfaces become disturbed or wetted due to construction operations or precipitation, the granular subbase thickness given above may not be adequate and it may be necessary to increase the thickness of the subbase and/or to incorporate a woven geotextile separator between the subgrade surfaces and the granular subbase material. The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction.

If the granular pavement materials are to be used by construction traffic, it may be necessary to increase the thickness of the granular subbase layer, install a woven geotextile separator between the roadway subgrade surface and the granular subbase material, or a combination of both, to prevent pumping and disturbance to the subbase material. The contractor should be made responsible for their construction access.

5.5.3 Asphalt Cement Type

Performance grade PG 58-34 asphalt cement should be specified for Superpave asphaltic concrete mixes.

5.5.4 Pavement Transitions

As part of the roadway reconstruction, the new pavement will abut the existing pavement at Klondike Road. The following is suggested to improve the performance of the joint between the new and the existing pavements:

- Neatly saw cut the existing asphaltic concrete;
- Remove the asphaltic concrete and slope the bottom of the excavation within the existing granular base and subbase at 1 horizontal to 1 vertical, or flatter, to avoid undermining the existing asphaltic concrete.
- To avoid cracking of the asphaltic concrete due to an abrupt change in the thickness of the roadway granular materials where new pavement areas join with the existing pavements, the granular depths should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the existing pavement structure.
- Remove (mill off) 40 to 50 millimetres of the existing asphaltic concrete to a distance of 300 millimetres at the joint and tack coat the asphaltic concrete at the joint in accordance with the requirements in OPSS 310.

5.5.5 Pavement Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. The subgrade surfaces should be crowned and shaped to drain to the ditches and/or catch basins to promote drainage of the pavement granular materials.

Catch basins should be equipped with minimum 3 metre long stub drains extending in two directions at the subgrade level.

5.5.6 Granular Material Compaction

The granular base and subbase materials should be compacted in maximum 300 millimetre thick lifts to at least 98 percent of the standard Proctor maximum dry density value.

6.0 SENSITIVE MARINE CLAY SOILS – EFFECTS OF TREES

Much of the site is underlain by silty clay, a material which is known to be susceptible to shrinkage with a change/reduction in moisture content. Research by the Institute for Research in Construction (formerly the Division of Building Research) of the National Research Council of Canada has shown that trees can cause a reduction of moisture content in the silty clays in the Ottawa area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations, or hard surfaced areas. Therefore, deciduous tree planting should be carried in accordance with the guidelines identified in the City of Ottawa document titled: "Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines".

As specified in the guideline, a separate letter will be provided regarding tree planting in silty clay soils, specific to this site. The letter will include the results of the specified laboratory testing and a corresponding map with locations of areas of low/ medium/ highly sensitive clay soils.

The effects of trees (both existing and proposed) on the dwellings should be considered in the landscape plan for this development.

7.0 ADDITIONAL CONSIDERATIONS

7.1 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the sample of soil recovered from borehole 18-2 sample 5 was 64 micrograms per gram. According to Canadian Standards Association (CSA) "Concrete Materials and Methods of Concrete Construction", the concentration of sulphate can be classified as low. Therefore any concrete in contact with the native soil could be batched with General Use (GU) cement. The effects of freeze thaw in the presence of de-icing chemical (sodium chloride) use on the roadway should be considered in selecting the air entrainment and the concrete mix proportions for any concrete.

Based on the resistivity and pH of the sample, the soil in this area can be classified as non-aggressive towards unprotected steel. It should be noted that the corrosivity of the soil/groundwater could vary throughout the year due to the application sodium chloride for de-icing.

7.2 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, etc.) will cause ground vibration on and off of the site. The vibrations will attenuate with distance from the source, but may be felt at nearby structures. However, the magnitude of the vibrations is expected to be much less than what is required to cause damage to the nearby structures or services that are in good condition.

7.3 Winter Construction

The soils that exist at this site are highly frost susceptible and are prone to significant ice lensing. In the event that construction is required during freezing temperatures, the soil below the footings and floor slab should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

Any service trenches should be opened for as short a time as practicable and the excavations should be carried out only in lengths which allow all of the construction operations, including backfilling, to be fully completed in one working day. The materials on the sides of the trenches should not be allowed to freeze. In addition, the backfill should be excavated, stored and replaced without being disturbed by frost or contaminated by snow or ice.

7.4 Excess Soil Management Plan

This report does not constitute an excess soil management plan. The disposal requirements for excess soil from the site have been assessed and are provided in the separate Phase One ESA report.

7.5 Design Review and Construction Observation

The design details of the proposed residential development were not available to us at the time of preparation of this letter. It is recommended that the design drawings be reviewed by the geotechnical engineer as the design progresses to ensure that the guidelines provided in this report have been interpreted as intended.

Due to the variable nature of the deposits on this site and the spacing between the borehole locations, the allowable bearing pressures and grade raises should be considered preliminary. The final allowable bearing pressure and grade raise restriction should be determined on a lot by lot basis.

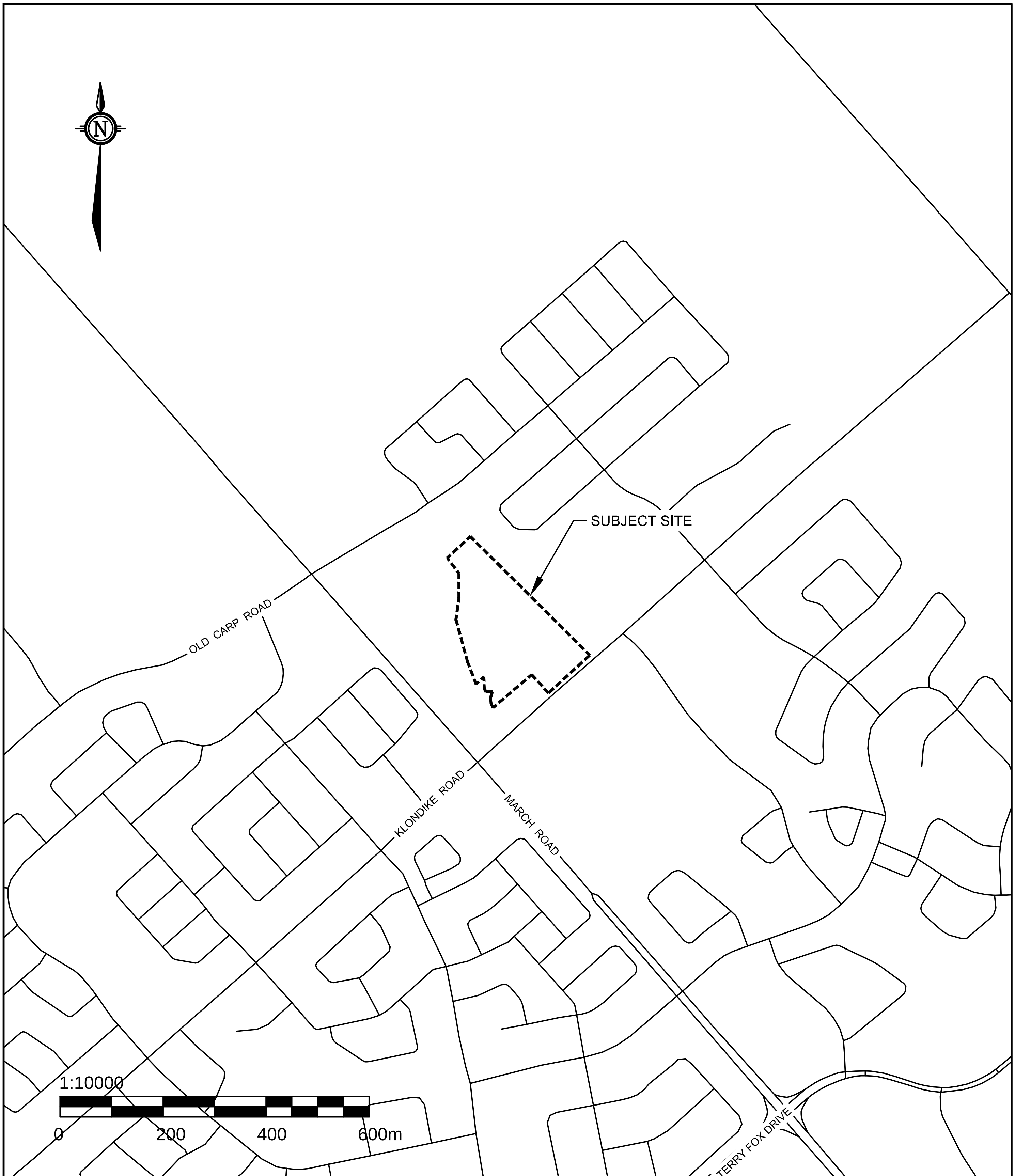
All footing surfaces and any engineered fill areas for the houses should be inspected by GEMTEC to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of granular materials beneath the foundations should be inspected to ensure that the materials used conform to the grading and compaction specifications. The subgrade surfaces for the site services and roadways should be inspected by geotechnical personnel. In-situ density testing should be carried out on the service pipe bedding and backfill and the roadway granular materials.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

Greg Davidson, B.Eng., E.I.T.

LICENSED PROFESSIONAL ENGINEER
B.D. WIEBE
100060438
26 April 2018
PROVINCE OF ONTARIO

Brent Wiebe, P.Eng.
Senior Geotechnical Engineer
P:\0. Files\64100\64153.85\Report\64153.85_RPT01_V01_2018-04-04.docx



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Project			Drawing		
GEOTECHNICAL INVESTIGATION 1055 KLONDIKE ROAD OTTAWA, ONTARIO			KEY PLAN		
Drwn By	Chkd By	Date	Project No.	Revision No.	
S.L.	G.D.	APRIL 2018	64153.85	0	**FIGURE 1**

LEGEND

- BH 18-1 77.61 — BOREHOLE (current investigation by GEMTEC)
  - GROUND SURFACE ELEVATION IN METRES GEODETIC DATUM
- BH 17-1 78.30 — BOREHOLE (previous investigation by GEMTEC, formerly Houle Chevrier Engineering Ltd.(2017))
- 72.52 — ELEVATION (current investigation by GEMTEC)
- A A — SLOPE CROSS SECTION (current investigation by GEMTEC)

Scale 1:1000

0 20 40 60m

GEMTEC
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Client		Project
NOVATECH		64153.85

Location	
1055 KLONDIKE ROAD OTTAWA, ONTARIO	

Drwn by	Chkd by	
S.L.	G.D.	BOREHOLE LOCATION PLAN

Date	Rev.	
APRIL 2018	0	**FIGURE 2**

APPENDIX A

Record of Borehole Sheets
List of Abbreviations and Terminology

Report to: Novatech
Project: 64153.85 (April 4, 2018)

RECORD OF BOREHOLE 18-1

PROJECT: 64153.85
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: March 9, 2018

SHEET 1 OF 1
DATUM:
SPT HAMMER: 63.5 kg; drop 0.76 metres

BORING METHOD: 150 mm Diameter Power Auger

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
0	Ground Surface	77.69			
	Dark brown silty sand, some organic material (TOPSOIL)	77.38 0.31	1	50 D.O.	4
	Brown SILT and SAND				
1			2	50 D.O.	4
2		75.40 2.29	3	50 D.O.	6
	Very stiff to stiff, grey brown SILT and CLAY (WEATHERED CRUST)		4	50 D.O.	5
3			5	50 D.O.	5
4			6	50 D.O.	3
5			7	50 D.O.	4
		71.75 5.94	8	50 D.O.	3
6	End of borehole				
7					
8					
9					
10					
11					
12					

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^{-5} 10^{-4} 10^{-3} 10^{-2}

WATER CONTENT, PERCENT: Wp — W — Wl : 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

- Above ground protector
- Bentonite
- Filter sand
- 50 mm diameter, 3m length slotted PVC screen
- Groundwater level observed at about 2.0 metres below surface grade (elevation 75.7 metres, geodetic datum) on March 15, 2018.

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/03/15	2.03 ∇	75.66

DEPTH SCALE
1 to 60

GEMTEC

LOGGED: AN
CHECKED:

RECORD OF BOREHOLE 18-2

PROJECT: 64153.85
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: March 8, 2018

SHEET 1 OF 1
DATUM:
SPT HAMMER: 63.5 kg; drop 0.76 metres

BORING METHOD: 150 mm Diameter Power Auger

SOIL PROFILE

DESCRIPTION	ELEV. (m)	DEPTH (m)
Ground Surface	78.38	
Brown sandy silt with organic material (TOPSOIL)	78.13	0.25
Grey brown SILT and SAND	77.34	1.04
Brown, fine to medium grained SAND, trace to some silt, layered with grey brown SILTY SAND	76.27	2.11
Very stiff to stiff, grey brown SILT and CLAY (WEATHERED CRUST)	72.28	6.10
Stiff, grey SILTY CLAY	69.24	9.14
Compact, grey sand and silt, trace to some clay, some gravel and cobbles (GLACIAL TILL)	68.17	10.21
Sampler refusal End of borehole		

SAMPLES

NUMBER	TYPE	BLOWS/0.3m
1	50 D.O.	4
2	50 D.O.	7
3	50 D.O.	5
4	50 D.O.	4
5	50 D.O.	3
6	50 D.O.	3
7	50 D.O.	7
8	50 D.O.	2
9	50 D.O.	2
10	50 D.O.	15
11	50 D.O.	27

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^{-5} 10^{-4} 10^{-3} 10^{-2}
WATER CONTENT, PERCENT: Wp ⊢ ○ W ⊣ Wl: 20 40 60 80
ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Borehole backfilled with auger cuttings

Soil becomes saturated at 2.29 metres below ground surface.

DEPTH SCALE 1 to 60

GEMTEC

LOGGED: AN
CHECKED:

RECORD OF BOREHOLE 18-3

PROJECT: 64153.85
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: March 9, 2018

SHEET 1 OF 1
DATUM:
SPT HAMMER: 63.5 kg; drop 0.76 metres

DEPTH SCALE METRES | BORING METHOD: 150 mm Diameter Power Auger

SOIL PROFILE

DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	78.79
Grey, crushed sand and gravel, trace silt (DRIVEWAY MATERIAL)	78.64 / 0.15
Dark brown and brown silty sand, some gravel, and organic material (FILL MATERIAL)	77.88 / 0.91
Brown SILT and SAND	76.30 / 2.49
Brown, fine to medium grained SAND, trace to some silt	75.74 / 3.05
Very stiff to stiff, grey brown SILT and CLAY (WEATHERED CRUST)	71.16 / 7.63
Firm to stiff, grey Silty Clay	70.56 / 8.23
End of borehole	

SAMPLES

NUMBER	TYPE	BLOWS/0.3m
1	50 D.O.	46
2	50 D.O.	7
3	50 D.O.	7
4	50 D.O.	5
5	50 D.O.	4
6	50 D.O.	3
7	50 D.O.	4
8	50 D.O.	3
9	50 D.O.	2
10	50 D.O.	
11	50 D.O.	1

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^{-5} 10^{-4} 10^{-3} 10^{-2}
WATER CONTENT, PERCENT: Wp — W — Wl : 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Above ground protector
Bentonite
Auger cuttings
Bentonite
Filter sand
50 mm diameter, 3m length slotted PVC screen

Groundwater level observed at about 6.3 metres below surface grade (elevation 72.5 metres, geodetic datum) on March 15, 2018.

GROUNDWATER OBSERVATIONS		
DATE	DEPTH (m)	ELEV. (m)
18/03/15	6.33 ∇	72.46

DEPTH SCALE
1 to 60

GEMTEC

LOGGED: AN
CHECKED:

BOREHOLE LOG 6415385_BOREHOLE LOGS_GNT_V01_2018-03-14.GPJ 2015.GDT 27/3/18

RECORD OF BOREHOLE 18-4

PROJECT: 64153.85
SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2
DATUM:

BORING DATE: March 8, 2018
SPT HAMMER: 63.5 kg; drop 0.76 metres

BORING METHOD: 150 mm Diameter Power Auger

SOIL PROFILE

DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	77.61
Dark brown silty sand / sandy silt, some organic material (TOPSOIL)	77.43 / 0.18
Brown SILT and SAND, trace roots	76.49 / 1.12
Brown, fine to medium grained SAND, trace to some silt	75.48 / 2.13
Very stiff to stiff, grey brown SILT and CLAY (WEATHERED CRUST)	71.51 / 6.10
Stiff, grey SILTY CLAY	69.23 / 8.38
Grey sand and silt, some gravel, possible cobbles (GLACIAL TILL)	69.05 / 8.56
Auger refusal on inferred bedrock End of borehole	

SAMPLES

NUMBER	TYPE	BLOWS/0.3m
1	50 D.O.	3
2	50 D.O.	7
3	50 D.O.	10
4	50 D.O.	4
5	50 D.O.	4
6	50 D.O.	2
7	50 D.O.	2
8	50 D.O.	W.H.
9	50 D.O.	1
10	50 D.O.	50 for 0.1m

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^{-5} 10^{-4} 10^{-3} 10^{-2}

WATER CONTENT, PERCENT: Wp —○W— Wl : 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Borehole backfilled with auger cuttings

Soil becomes saturated at 2.29 metres below ground surface.

DEPTH SCALE 1 to 60

GEMTEC

LOGGED: AN

CHECKED:

RECORD OF BOREHOLE 18-5

PROJECT: 64153.85

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: March 8, 2018

SHEET 1 OF 1

DATUM:

SPT HAMMER: 63.5 kg; drop 0.76 metres

BORING METHOD: 150 mm Diameter Power Auger

DESCRIPTION	ELEV. (m) / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
Ground Surface	77.80			
Grey brown silty clay, with dark brown pockets, some organic material (FILL MATERIAL)		1	50 D.O.	8
		2	50 D.O.	5
		3	50 D.O.	3
		4	50 D.O.	4
Brown silty sand, trace wood	74.55 / 3.25	5	50 D.O.	5
		6	50 D.O.	8
Very stiff to stiff, grey brown SILT and CLAY (WEATHERED CRUST)	73.08 / 4.72	7	50 D.O.	12
		8	50 D.O.	5
		9	50 D.O.	3
		10	50 D.O.	2
		11	50 D.O.	50 for 0.13m
Auger refusal on inferred bedrock End of borehole	70.06 / 7.74			

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80)

SHEAR STRENGTH Cu, kPa: nat. V - + ; rem. V - ⊕ ; Q - ● ; U - ○ (20, 40, 60, 80)

HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-5}, 10^{-4}, 10^{-3}, 10^{-2})

WATER CONTENT, PERCENT: Wp — W — Wl (20, 40, 60, 80)

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION:
- Above ground protector
- Bentonite
- Filter sand
- 50 mm diameter, 3m length slotted PVC screen
- Groundwater level observed at about 5.5 metres below surface grade (elevation 72.3 metres, geodetic datum) on March 15, 2018.

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/03/15	5.47 ∇	72.33

DEPTH SCALE METRES: 0 – 12

DEPTH SCALE 1 to 60

GEMTEC

LOGGED: AN

CHECKED:

RECORD OF BOREHOLE 17-1

PROJECT: 60616.46
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: March 27, 2017

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 metres

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DEPTH (m)	DESCRIPTION	ELEV. (m)
0.00	Ground Surface	78.30
0.15	Dark brown silty sand (TOPSOIL)	78.15
0.15 – 2.29	Brown fine to coarse grained SAND, some silt	76.01
2.29 – 3.81	Very stiff, grey brown SILTY CLAY (Weathered crust)	74.49
3.81 – 9.60	Very stiff to firm, grey SILTY CLAY	68.70
9.60	End of Borehole	

SAMPLES

NUMBER	TYPE	BLOWS/0.3m
1	50 D.O.	8
2	50 D.O.	9
3	50 D.O.	10
4	50 D.O.	4
5	50 D.O.	4
6	50 D.O.	4
7	50 D.O.	4
8	50 D.O.	3
9	50 D.O.	2
10	50 D.O.	1
11	50 D.O.	1

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^{-5}, 10^{-4}, 10^{-3}, 10^{-2}
WATER CONTENT, PERCENT: Wp — W — Wl ; 20, 40, 60, 80
ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: Backfilled with soil cuttings

DEPTH SCALE
1 to 50

Houle Chevrier Engineering

LOGGED: M.L.
CHECKED:

RECORD OF BOREHOLE 17-2

PROJECT: 60616.46
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: March 27, 2017

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 metres

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
Ground Surface	78.43			
Dark brown silty sand (TOPSOIL)	78.28 / 0.15			
Brown fine to coarse grained SAND		1	50 D.O.	4
		2	50 D.O.	6
		3	50 D.O.	6
Very stiff, grey brown SILTY CLAY (Weathered crust)	75.84 / 2.59	4	50 D.O.	5
		5	50 D.O.	4
Very stiff to firm, grey SILTY CLAY	74.62 / 3.81	6	50 D.O.	4
		7	50 D.O.	3
		8	50 D.O.	3
		9	50 D.O.	2
		10	50 D.O.	1
End of Borehole	68.83 / 9.60			

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^{-5} 10^{-4} 10^{-3} 10^{-2}
WATER CONTENT, PERCENT: Wp W Wl; 20 40 60 80
ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION:
Bentonite seal
Filter Sand
25 mm Diameter, 0.6 metres long well screen
Soil cuttings

DEPTH SCALE 1 to 50

Houle Chevrier Engineering

LOGGED: M.L.
CHECKED:

BOREHOLE LOG GINT LOGS MARCH 28 2017.GPJ HOULE CHEVRIER 2015.GDT 30/3/17

RECORD OF BOREHOLE 17-3

PROJECT: 60616.46

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

DATUM: Geodetic

BORING DATE: March 27, 2017

SPT HAMMER: 63.5 kg; drop 0.76 metres

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (m)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface	72.74				
		Dark brown silty sand (TOPSOIL)	72.59 0.15				
		Very stiff, grey brown SILTY CLAY (Weathered crust)		1	50 D.O.	7	
1				2	50 D.O.	11	
2				3	50 D.O.	7	
				4	50 D.O.	6	
3				5	50 D.O.	4	Bentonite seal; Filter Sand; 25 mm Diameter, 0.6 metres long well screen
4			68.74 4.00	6	50 D.O.	>50 for 75 mm	
		Brown silty sand, some clay with small gravel (Glacial Till)	4.07				
		End of Borehole Practical Auger Refusal					

Columns also present (no data recorded): STRATA PLOT; 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^{-5}, 10^{-4}, 10^{-3}, 10^{-2}); WATER CONTENT, PERCENT Wp W Wl (20, 40, 60, 80); ADDITIONAL LAB. TESTING.

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

DEPTH SCALE

1 to 50

Houle Chevrier Engineering

LOGGED: M.L.

CHECKED:

RECORD OF BOREHOLE 17-4

PROJECT: 60616.46
SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2
DATUM: Geodetic

BORING DATE: March 27, 2017
SPT HAMMER: 63.5 kg; drop 0.76 metres

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DEPTH (m)	SOIL PROFILE DESCRIPTION	ELEV. (m) / DEPTH (m)
0	Ground Surface	78.14
0.00–0.10	Dark brown silty sand (TOPSOIL)	78.04 / 0.10
0.10–2.44	Brown fine to coarse grained SAND	75.70 / 2.44
2.44–3.81	Very stiff, grey brown SILTY CLAY (Weathered crust)	74.33 / 3.81
3.81–8.46	Very stiff to firm, grey SILTY CLAY	69.68 / 8.46
8.46	End of Borehole Practical Auger Refusal	

SAMPLE NUMBER	TYPE	BLOWS/0.3m
1	50 D.O.	4
2	50 D.O.	8
3	50 D.O.	12
4	50 D.O.	7
5	50 D.O.	5
6	50 D.O.	6
7	50 D.O.	3
8	50 D.O.	3
9	50 D.O.	3
10	50 D.O.	1

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^{-5} 10^{-4} 10^{-3} 10^{-2}

WATER CONTENT, PERCENT: Wp — W — Wl ; 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: Backfilled with soil cuttings

DEPTH SCALE 1 to 50

Houle Chevrier Engineering

LOGGED: M.L.

CHECKED:

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

AS	auger sample
CA	casing sample
CS	chunk sample
BS	Borros piston sample
GS	grab sample
DO	drive open
MS	manual sample
RC	rock core
ST	slotted tube
TO	thin-walled open Shelby tube
TP	thin-walled piston Shelby tube
WS	wash sample

PENETRATION RESISTANCE

Standard Penetration Resistance, N

The number of blows by a 63.5 kg hammer dropped 760 millimetre required to drive a 50 mm drive open sampler for a distance of 300 mm. For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

Dynamic Penetration Resistance

The number of blows by a 63.5 kg hammer dropped 760 mm to drive a 50 mm diameter, 60^{o} cone attached to 'A' size drill rods for a distance of 300 mm.

WH

Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drill rig.

PM

Sampler advanced by manual pressure.

SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	sieve and hydrometer analysis
U	unconfined compression test
Q	undrained triaxial test
V	field vane, undisturbed and remoulded shear strength

SOIL DESCRIPTIONS

Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	over 100

LIST OF COMMON SYMBOLS

c_u	undrained shear strength
e	void ratio
C_c	compression index
c_v	coefficient of consolidation
k	coefficient of permeability
I_p	plasticity index
n	porosity
u	pore pressure
w	moisture content
w_L	liquid limit
w_P	plastic limit
ϕ^1	effective angle of friction
γ	unit weight of soil
γ^1	unit weight of submerged soil
σ	normal stress

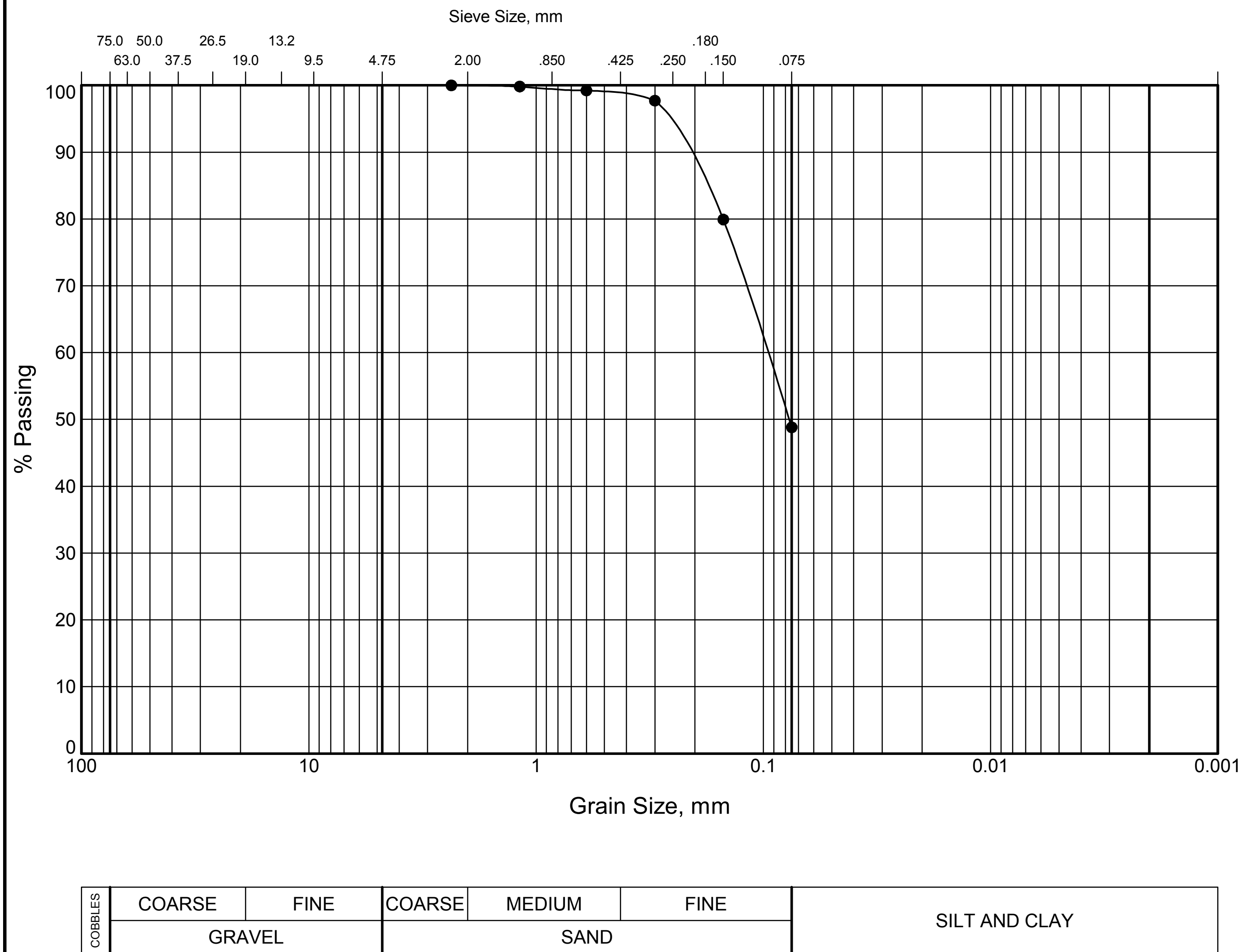
APPENDIX B

Laboratory Test Results
Figures B1 to B3

Report to: Novatech
Project: 64153.85 (April 4, 2018)

GRAIN SIZE DISTRIBUTION

FIGURE B1



Legend	Borehole	Sample	Depth (m)	% Gravel	% Sand	% Silt & Clay
●	18-1	2	0.8 - 1.4	0	51	49

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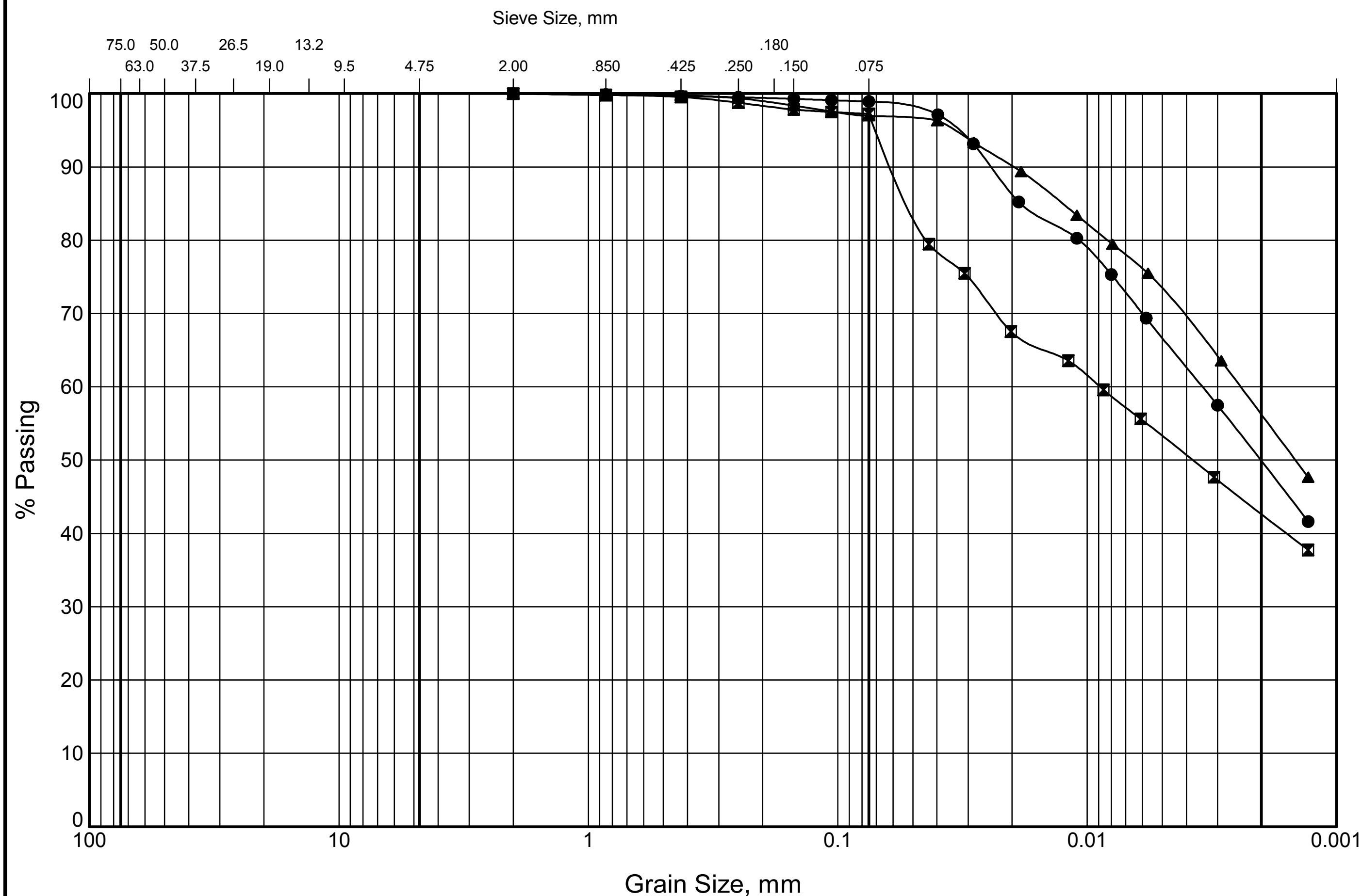
Date: March 2018

Project: 64153.85

GRAIN SIZE DISTRIBUTION

FIGURE B2



COBBLES	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
	GRAVEL		SAND				

Legend	Borehole	Sample	Depth (m)	% Gravel	% Sand	% Silt	% Clay
●	18-2	8	6.1 - 6.7	0	1	49	50
⧗	18-3	5	3.1 - 3.7	0	3	55	43
▲	18-4	4	2.3 - 2.9	0	3	41	56

SOILS GRAIN SIZE GRAPH UNIFIED % (HYDRO) 6415385_BOREHOLE LOGS_GNT_V01_2018-03-14.GPJ 9 2011.GDT 27/3/18

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Date: March 2018

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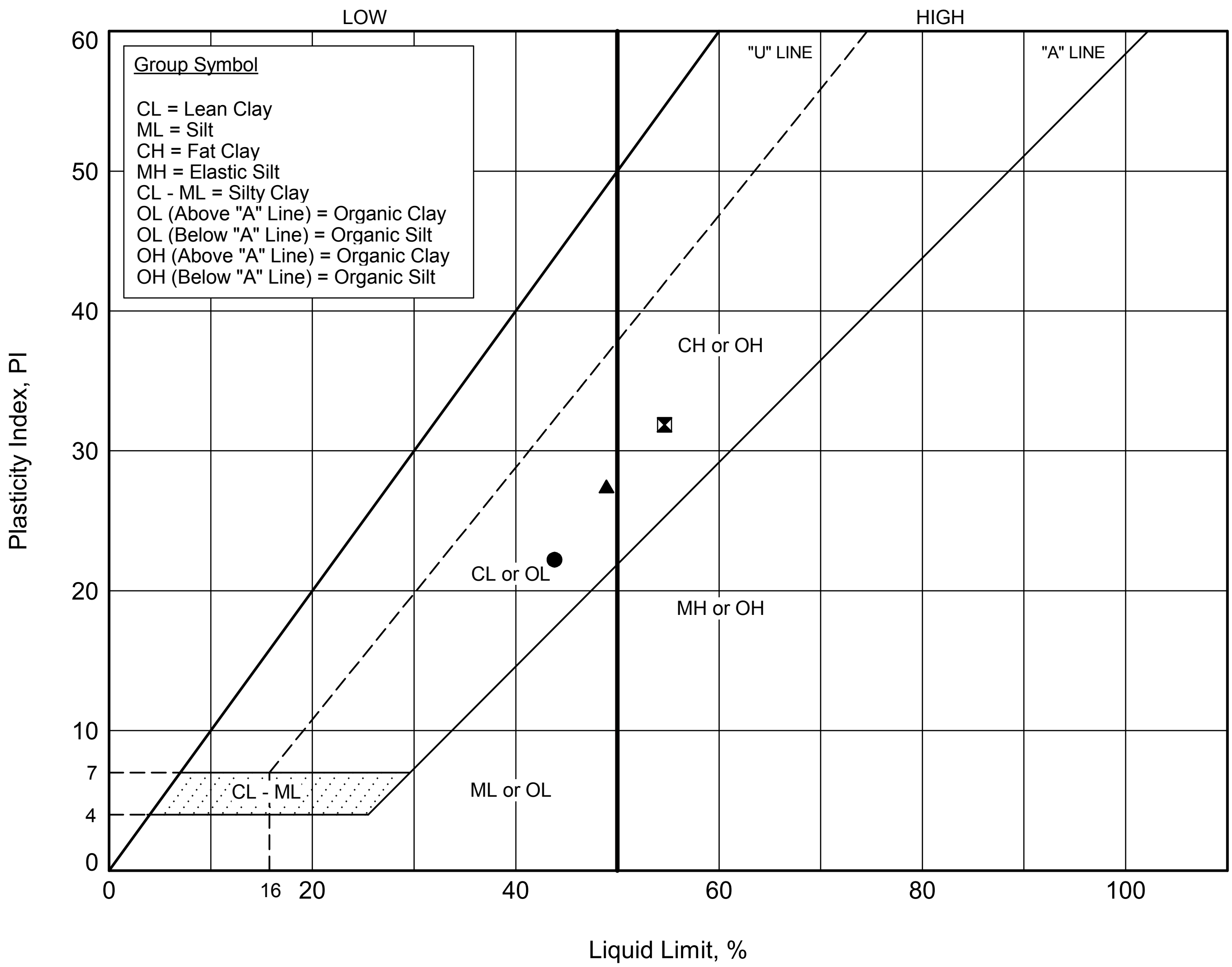
PLASTICITY CHART

FIGURE B3



Legend	Borehole	Sample	Depth (m)	Water Content %	LL %	PL %	PI %
●	18-2	8	6.1 - 6.7	54.9	43.8	21.6	22.2
⊠	18-3	5	3.1 - 3.7	43.3	54.6	22.8	31.9
▲	18-4	4	2.3 - 2.9	45.7	48.9	21.5	27.5

HCE ATTERBERG LIMITS 6415385_BOREHOLE LOGS_GNT_V01_2018-03-14.GPJ 9 2011.GDT 27/3/18

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Date: March 2018

Project: 64153.85

APPENDIX C

Chemical Analysis of Soil Samples
Samples Relating to Corrosion
(Paracel Laboratories Ltd. Order No. 1811506)

Report to: Novatech
Project: 64153.85 (April 4, 2018)

PARACEL LABORATORIES LTD.

Order #: 1811506

Certificate of Analysis

Client: GEMTEC Consulting Engineers and Scientists Limited

Client PO:

Report Date: 21-Mar-2018

Order Date: 16-Mar-2018

Project Description: 64153.85

	Client ID:	18-2 SA5	-	-	-
	Sample Date:	14-Mar-18	-	-	-
	Sample ID:	1811506-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	65.7	-	-	-
General Inorganics					
Conductivity	5 uS/cm	256	-	-	-
pH	0.05 pH Units	7.34	-	-	-
Resistivity	0.10 Ohm.m	39.1	-	-	-
Anions					
Chloride	5 ug/g dry	54	-	-	-
Sulphate	5 ug/g dry	64	-	-	-

experience • knowledge • integrity

civil	civil
geotechnical	géotechnique
environmental	environnementale
field services	surveillance de chantier
materials testing	service de laboratoire des matériaux

expérience • connaissance • intégrité