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Geotechnical Investigation Fernbank Crossing Residential Subdivision
Phase 5
Ottawa, Ontario

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Submitted to:

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Geotechnical Investigation Fernbank Crossing Residential Subdivision Phase 5

Ottawa, Ontario

March 7, 2019
Project: 64153.97-R01

experience • knowledge • integrity

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March 7, 2019

File: 64153.97-R01

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240 Michael Cowpland Drive
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Attention: Mr. Mark Bissett, P.Eng.

Re: Geotechnical Investigation
Fernbank Crossing Residential Subdivision
Phase 5
Ottawa, Ontario

Please find enclosed the revised report summarizing the findings of our geotechnical investigation for the proposed Fernbank Crossing Phase 5 Residential Subdivision located in Ottawa (Stittsville), Ontario.

Kelsey Holkestad, B.Eng., E.I.T.

Brent Wiebe, P.Eng.
VP Operations - Ontario

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation conducted by GEMTEC Consulting Engineers and Scientists Limited (GEMTEC) for Phase 5 of the Fernbank Crossing residential subdivision located in Ottawa, Ontario. The purpose of the investigation was to identify the general subsurface conditions at the site by means of a limited number of test pits and, based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations that could influence design decisions.

This investigation was carried out in accordance with our proposal dated July 5, 2018.

This report should be read in conjunction with our geotechnical reports titled, “Geotechnical Investigation, Fernbank Crossing Residential Subdivision, Phase 3 and 4, Ottawa, Ontario”, dated May, 2015, and “Geotechnical Investigation, Fernbank Crossing Residential Subdivision, Phase 4, Ottawa, Ontario”, dated March 13, 2017.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Project Description

Plans are being prepared to construct Phase 5 of the Fernbank Crossing Residential Subdivision. Phase 5 of the subdivision includes areas of land bordered by residential lots on Shinny Avenue to the north, Tim Sheehan Place to the east, Fernbank Road to the south and future development to the west. The site location is provided on the Key Plan, Figure 1.

It is understood that the proposed development will consist of residential units with an internal roadway system. It is assumed that the proposed residences will be townhouses of conventional wood frame construction with full depth concrete basements. Water, sanitary and storm services will be part of the proposed development.

The existing site consists of a generally flat vegetated field containing grass and shrubs, including trees along the west perimeter. The east edge of the site slopes downwards towards an area outside of the proposed Phase 5 development, which has been stripped of topsoil and is currently undergoing development.

2.2 Previous Experience at the Subject Site

GEMTEC, formerly Houle Chevrier Engineering Ltd., carried out previous geotechnical investigations in 2012, and throughout 2014 to 2018 for Phases 1 to 4 of the Fernbank Crossing development, including the Abbott Street Extension. The information obtained from those investigations was used during the development of recommendations provided in this report.

2.3 Review of Geology Maps

Geology maps of the Ottawa area indicate that the subsurface conditions within the site are characterized primarily by glacial till having a thickness of about 3 to 10 metres over interbedded limestone and dolostone of the Gull River Formation. Fill material, associated with the existing development, is also expected to be encountered at the subject site.

The maps are consistent with the findings of our previous geotechnical investigations in this area.

3.0 SUBSURFACE INVESTIGATION

The field work for this investigation was carried out on August 14, 2018. At that time, four (4) test pits, numbered 18-1 to 18-4, inclusive, were advanced at the site to refusal (depths ranging from about 2.5 to 3.9 metres below surface grade) using a 470 Hitachi shovel supplied and operated by Thomas Cavanagh Construction Limited of Ashton, Ontario.

The subsurface and groundwater conditions encountered in the test pits were identified by visual and tactile observation. The test pits were loosely backfilled with the excavated materials and tamped with the bucket of the excavator. As such, the test pits represent areas of soil disturbance.

The field work was supervised throughout by members of our engineering staff, who located the test pits and logged the subsurface conditions. Following the field work, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Select samples of the soil were tested for water content and grain size testing. One (1) soil sample recovered from test pit 18-3 was sent for basic chemical testing relating to corrosion of buried concrete and steel.

The approximate locations of the test pits are shown on the Test Pit Location Plan, Figure 2. Descriptions of the subsurface conditions logged in the test pits are provided on the Record of Test Pit sheets in Appendix A. The results of the laboratory classification testing are provided on the Record of Test Pit sheets and in Appendix B. The results of the chemical analysis relating to corrosion of buried steel and concrete on the soil sample collected are provided in Appendix C.

The test pit locations were selected and determined relative to existing site features by GEMTEC personnel. Elevations were measured using our Trimble R10 GPS equipment. The coordinates of the test pits are referenced to NAD83 (CSRS) Epoch 2010, vertical network CGVD2013 and are considered to be accurate within the tolerance of the instrument.

4.0 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, the soil and groundwater conditions identified in the test pits are given on the Record of Test Pit sheets (Appendix A). The logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of exploration, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at other than the test pit locations may vary from the conditions encountered in the test pits. 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 groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities in the area.

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

The following presents an overview of the subsurface conditions encountered in the test pits advanced as part of this investigation.

4.2 Topsoil

A surficial layer of topsoil was encountered from ground surface in test pits 18-1 and 18-3. The topsoil consists of dark brown silty sand and has a thickness of about 80 millimetres at the test pit locaitons.

4.3 Silty Sand

Native deposits of brown to grey brown silty sand were encountered beneath the topsoil in test pits 18-1 and 18-3, and at ground surface in test pit 18-2. The thickness of the silty sand ranges from 0.7 to 1.4 metres. Organic material was present in the sandy silt in boreholes 18-1 and 18-3 to depths ranging between 0.5 to 0.7 metres below ground surface (elevations 104.9 to 105.1 metres, geodetic).

Two (2) grain size distribution tests were undertaken on selected samples of sandy silt/silty sand from test pits 18-1 and 18-2. The results are provided in Appendix B and are summarized in Table 4.1.

Table 4.1 – Summary of Grain Size Distribution Test (Silty Sand)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
18-1	4	0.7 – 0.8	13	42	34	11
18-2	2	0.3 – 0.6	23	41	25	11

The moisture content of samples of the silty sand range from 7 to 13 percent.

4.4 Glacial Till

Native deposits of glacial till were encountered below the silty sand/ sandy silt layer in test pits 18-1 to 18-3 at depths ranging from 0.8 to 1.4 metres below ground surface, and at surface in test pit 18-4 (elevation 103.3 to 105.3 metres, geodetic datum). The glacial till has a thickness ranges from 2.0 to 3.1 metres. The glacial till can be generally described as grey brown silty sand, some gravel, trace to some clay, with cobbles and boulders.

One (1) grain size distribution test was undertaken on a selected sample of glacial till from test pit 18-3. The results are provided in Appendix B and are summarized in Table 4.2.

Table 4.2 – Summary of Grain Size Distribution Test (Glacial Till)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
18-3	4	0.8 – 1.0	8	47	33	12

The moisture content of a sample of the glacial till is 10 percent.

4.5 Inferred Bedrock

Practical refusal occurred on inferred bedrock in all of the test pits at depths ranging from 2.9 to 3.9 metres below ground surface (elevations 100.8 to 102.7 metres, geodetic). It should be noted that practical auger refusal could occur as a result of cobbles or boulders in the glacial till deposit and may not necessarily be representative of the upper surface of the bedrock.

4.6 Groundwater Levels

Groundwater seepage was not encountered in any of the test pits during the field investigation on August 14, 2018.

It should be noted that groundwater seepage within test pit excavations do not represent stabilized groundwater conditions. Groundwater levels may also be higher during wet periods of the year, such as the early spring or fall or following periods of heavy precipitation.

4.7 Soil Chemistry Relating to Corrosion

The results of chemical testing on a soil sample recovered from test pit 18-3 are provided in Appendix C and are summarized in Table 4.3.

Table 4.3 – Summary of Corrosion Testing

Parameter	Test Pit 18-3
Chloride Content (µg/g)	5
Resistivity (Ohm.m)	94.4
pH	7.84
Sulphate Content (µg/g)	<5

5.0 GEOTECHNICAL DESIGN GUIDELINES

5.1 General

The information in the following sections is provided for the guidance of the design engineers and is intended for the design of 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 off site sources are outside the terms of reference for this report.

5.2 Site Grade Raise Restrictions

The subsurface conditions at this site consist of silty sand/sandy silt underlain by glacial till and bedrock. Based on this information, there are no grade raise restrictions for Phase 5 of this subdivision, from a geotechnical perspective.

5.3 Proposed Houses

5.3.1 Overburden Excavation

Overburden excavation will be carried out through topsoil, silty sand/sandy silt, and glacial till. 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 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 soils above the groundwater should not present any excavation constraints. In contrast, excavation in the native silt and sandy silt below the groundwater level could present constraints. Groundwater inflow from silt, sandy silt, and silty 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 silt and sandy silt deposits.

5.3.2 Bedrock Excavation

If required, excavation of the upper portions of the bedrock can likely be carried out using large excavation equipment in conjunction with line drilling and pneumatic hoe ramming equipment. Any excavation within bedrock could be excavated near vertical. To protect workers, any loose rock should be removed from the sides of the excavation. For the bedrock at this site, it is suggested that allowance be made for line drilling 50 millimetre diameter holes on 300 millimetre centres to assist with bedrock excavation. Nevertheless, some overbreak or underbreak of the bedrock should be expected.

The vibration effects of hoe ramming are usually minor and localized. Monitoring of the hoe ramming should be carried out, at least initially, to measure the vibrations to ensure that they are below acceptable threshold values.

5.3.3 Groundwater Pumping

Based on our observations on site, groundwater inflow from the overburden deposits into the excavations should be controlled by pumping from filtered sumps within the excavations. It is not expected that short term pumping during excavation will have any significant effect on nearby structures and services.

Although, we do not expect significant groundwater inflow into excavations for residential units, a Permit to Take Water (PTTW) may be required for pumping from within the excavations for services, depending on the invert depths. The owner currently possesses a Category 3 PTTW (Permit No. 5313-8WYKWW) for this development.

5.3.4 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.

The test pits represent areas of disturbed soil. Any test pits which are within the building footprints should be subexcavated and backfilled with engineered fill material as described above. The sides of the subexcavated test pits should be sloped at 1 horizontal to 1 vertical, or flatter.

5.3.5 Spread Footing Design

In our opinion the foundations at this site should be designed in accordance with Part 9 of the Ontario Building Code.

The proposed houses could be founded on spread footings bearing on or within the native soils/bedrock or on engineered fill above the native soils/bedrock. The topsoil and any fill materials are not considered suitable for the support of the proposed houses or concrete floor slabs and should be removed from the proposed building areas.

Based on the results of the investigation, the allowable bearing pressures listen in Table 5.1 may be used to size the spread footing foundations:

Table 5.1 – Allowable Bearing Pressures for Foundations

Subgrade Material	Allowable Bearing Pressure for Foundations
Silty sand/sandy silt	100
Glacial till	150
Engineered fill material, over undisturbed native deposits	150
Bedrock	250

The bearing values provided above are based on a maximum width of 1 metre for strip footings and a maximum width of 2 metres for pad footings.

It is pointed out that the deposits of silty sand/sandy silt near or below the groundwater level may become disturbed following excavation. If disturbance to these deposits occurs, one solution would be to wait several days to allow the water pressures to dissipate. The groundwater level could also be lowered in advance of excavation by pumping from sump pits, possibly combined with ditching around the perimeter of the excavations.

The post construction total and differential settlement of footings on the overburden deposits or on engineered fill material above the overburden deposits 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 placed and compacted in accordance with recommendations in this report.

The post construction settlement of footings founded on bedrock should be negligible, provided that all loose soil is removed from the bearing surfaces prior to placing concrete.

There may be areas on this site where the subgrade material at founding level transitions from overburden to bedrock. To reduce the potential for cracking of basement foundation walls above abrupt transitions from overburden to bedrock, it is suggested that the foundation walls be suitably reinforced for a distance of at least 3 metres from the transition.

5.3.6 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.

5.3.7 Basement Foundation Wall Backfill and Drainage

In accordance with the Ontario Building Code 2017, the following alternatives could be considered for drainage of the basement 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 material on the exterior of the foundation walls and backfill the walls with native material or imported soil.

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.

5.3.8 Garage Foundation and Pier Backfill

To avoid adfreeze and possible jacking (heaving) of the foundation walls due to adfreeze between the unheated garage foundation walls and the wall backfill, the interior and exterior of the garage foundation walls should be backfilled with free draining, non-frost susceptible sand or sand and gravel such as that meeting OPSS requirements for Granular B Type I or II. The backfill within the garage 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.

Alternatively, the interior of the garages could be filled with 19 millimetre clear crushed stone. In areas where the subgrade consists of silty sand/sandy silt a suitable nonwoven geotextile should be placed over the subgrade prior to the placement of clear stone to prevent ingress of fines into voids in the clear stone and possible settlement/cracking of the slab. The clear stone should be nominally compacted (2 to 3 passes with vibratory compaction equipment) in maximum 500 millimetre thick lifts.

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.

5.3.9 Basement Concrete Slab Support

To provide predictable settlement performance of the basement slab, all topsoil, 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. The clear stone should be nominally compacted (2 to 3 passes with vibratory compaction equipment).

In areas where the subgrade consists of silty sand/sandy silt, a suitable nonwoven geotextile should be placed over the subgrade prior to the placement of clear stone to prevent ingress of fines into voids in the clear stone and possible settlement/cracking of the slab.

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 sewer.

Basement floor slabs should be constructed in accordance with guidelines provided in ACI 302.1R-04 “Guide for Concrete Floor and Slab Construction”.

A polyethylene vapour barrier should be installed below the basement floor slabs.

5.3.10 Seismic Site Classification

Based on the results of our investigation and the 2012 Ontario Building Code, Site Class C should be used for the seismic design of the structures at this site.

In our opinion, there is no potential for liquefaction of the overburden deposits at this site.

5.3.11 Tree Planting

Based on the subsurface conditions at the test pit locations, there is no requirement to adhere to the City of Ottawa Guideline for Tree Planting in Sensitive Marine Clay Soils. The soils encountered are not susceptible to shrinkage due to moisture reduction.

5.4 Site Services

5.4.1 Overburden Excavation

Based on the available subsurface information, the excavations for the services within the site will be carried out through topsoil, silty sand/sandy silt, glacial till, and possibly bedrock.

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, most of 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 results of this investigation indicate that glacial till exists at relatively shallow depths (ground surface to 1.4 metres below ground surface). We do not anticipate any significant constraints with excavation of the glacial till above or below the groundwater level. However, if encountered, excavation below the groundwater level within silty sand/sandy silt could result in some disturbance to the soils at the bottom of the excavation and relatively flat side slopes may be required to prevent sloughing of material into the excavation unless the groundwater level is lowered in advance of excavation. It is our experience that excavation for site service installations to shallow depth within these deposits can usually be carried out within a braced steel trench box specifically designed for this purpose, in combination, where necessary, with steel plates advanced along the sides of the trench box to below the level of excavation. In this case, 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 Bedrock Excavation

In bedrock, the excavation for flexible service pipes should be in accordance with OPSD 802.013 for bedrock. The excavation for rigid service pipes should be in accordance with OPSD 802.033 for bedrock.

Guidelines for bedrock removal are provided in Section 5.3.2 of this report.

5.4.3 Groundwater Pumping

Groundwater inflow from the overburden deposits should be controlled by pumping from within the excavations.

Groundwater inflow from the bedrock into the excavations for the site services should be expected and should be handled by pumping from within the excavations. Significant groundwater pumping should be anticipated in areas where fractured bedrock is encountered. As indicated above, it is understood that the owner has a Category 3 PTTW for this site.

The groundwater should be detained and filtered before it is released into any ditches or creeks.

5.4.4 Pipe Bedding

The bedding for the sanitary sewers, storm sewers and watermains should be in accordance with Ontario Provincial Standard Drawing (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 Ontario Provincial Standard Specification (OPSS) 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 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 silt, sandy silt, or silty sand 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 on this project, 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.

5.4.5 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. If on-site 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.

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 silty sand/sandy silt from the lower part of 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).

5.4.6 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. 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 weathered silty clay. The weathered 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.

5.4.7 Servicing Blocks

The following information was provided to us for the Service Block between Lot 126 and Block 128.

- The 200 millimetre diameter sanitary sewer has an invert elevation of about 101.90 metres;
- The 900 millimetre diameter storm sewer has an invert elevation of about 102.32 metres;
- The 200 millimetre diameter drainage lead has an invert elevation of about 102.95 metres.

Since the underside of footing elevations for Lot 126 and the dwelling on the edge of Block 128 are 103.48 and 103.52 metres, respectively, the excavations for these services, should repairs be required after the houses are constructed would extend a maximum of about 1.7 metres below underside of footing level. The horizontal distance between the edges of the house foundations and the easement is at least 1.2 metres. Based on the width of the easement and the above parameters, excavation within the zone of influence of the foundations will not be required.

Based on the soil conditions encountered in this area, the soils can be classified as Type 3 according the Occupation Health and Safety Act. As such, 1 horizontal to 1 vertical side slopes are required for excavations in these soils. However, based on our experience and typical construction practices, steel trench boxes would be used for any repairs/service to these pipes. Therefore, we do not have any concerns related to space constraints when repairing or servicing these pipes.

5.5 Internal Roadway

5.5.1 Subgrade Preparation

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

Prior to placing granular material for the roadway, the exposed subgrade should be heavily proof rolled with a large (10 tonne) vibratory steel drum roller under dry conditions and inspected and approved by geotechnical personnel. Any soft areas evident from the proof rolling 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 or Earth Borrow material 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 be placed in maximum 500 millimetre thick lifts and suitably compacted either with a large drum roller, the haulage and spreading equipment, or a combination of both.

5.5.2 Pavement Structure

For the internal roadway within this phase of the residential development, 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
- 375 millimetres of OPSS Granular B, Type II subbase

5.5.3 Effects of Soil Disturbance

The above pavement structures assume that any trench backfill is adequately compacted and that the roadway subgrade surface is prepared as described in this report. If the roadway subgrade surface is disturbed or wetted due to construction operations or precipitation, the granular thickness given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or to incorporate a woven geotextile separator between the roadway subgrade surface and the granular subbase material. The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction. In our experience, a geotextile will likely be required in most cases where the subgrade consists of overburden, if the roadway construction is planned during the wet period of the year (such as the spring or fall).

Similarly, if the granular pavement materials are to be used by construction traffic, it may be necessary to increase the thickness of the Granular B Type II, 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.

5.5.4 Granular Material Compaction

The pavement granular materials should be compacted in maximum 300 millimetre thick lifts to at least 98 percent of standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.5.5 Asphaltic Concrete Types

The asphaltic concrete should consist of 40 millimetres of Superpave 12.5 over one 60 millimetre lift of Superpave 19.0. Performance grade PG 58-34 asphaltic cement should be specified.

5.5.6 Transition Treatments and Frost Tapers

Where the new pavement structure will abut the existing pavement, the depths of the granular materials should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement.

Granular frost tapers should be installed in accordance with OPSD 205.030 in areas where there is an abrupt transition from bedrock to overburden.

5.5.7 Pavement Drainage

The subgrade surface should be shaped and crowned to promote drainage of the roadway granular materials.

Adequate drainage if the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. As such it is recommended that catch basins be provided with perforated stub drains extending 3 metres out from the catch basins in two directions parallel to the roadway. These drains should be installed at the bottom of the subbase layer.

6.0 ADDITIONAL CONSIDERATIONS

6.1 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the soil sample recovered from test pit 18-3 is less than 5 micrograms per gram. According to Canadian Standards Association (CSA) “Concrete Materials and Methods of Concrete Construction”, the concentration of sulphate in the soil can be classified as low. For low exposure conditions, any concrete that will be in contact with the native soil or groundwater should be batched with General Use cement. The design of any concrete should take into consideration freeze thaw effects and the presence of chlorides.

Based on the conductivity and pH of the soil sample recovered from test pit 18-3, the soil can be classified as non-aggressive towards unprotected steel. The manufacturer of any buried steel elements that will be in contact with the soil and groundwater should be consulted to determine the durability of the product used. It is noted that the corrosivity of the soil and groundwater could vary throughout the year due to the application sodium chloride for de-icing.

6.2 Swimming Pools

As per Section 5.2, based on our investigation, there are no grade raise restrictions for Phase 5 of this subdivision, from a geotechnical perspective. Subsequently, we do not have any concerns, from a geotechnical perspective, for the installation of in-ground or above-ground pools on lots within Phase 5 of the subdivision.

6.3 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, bedrock removal, 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. We recommend that preconstruction surveys be carried out on any adjacent structures to mitigate potential claims.

6.4 Winter Construction

In the event that construction is required during freezing temperatures, the soil below the proposed houses should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

Any open excavations should be opened for as short a time as practicable. The materials on the sides of the excavation 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.

Provision must be made to prevent freezing of any soil below the level of any existing structures or services. Freezing of the soil could result in heaving related damage to structures or services.

6.5 Excess Soil Management Plan

This report does not constitute an excess soil management plan. The disposal requirements for excess soil from the site have not been assessed.

6.6 Design Review and Construction Observation

The details for the proposed construction were not available to us at the time of preparation of this report. 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.

The engagement of the services of GEMTEC during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design.

The subgrade surfaces for the proposed houses, utilities and roadways should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

In accordance with City of Ottawa requirements, all foundation subgrades and footings should be inspected and approved by geotechnical personnel. The placement and compaction of engineered fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

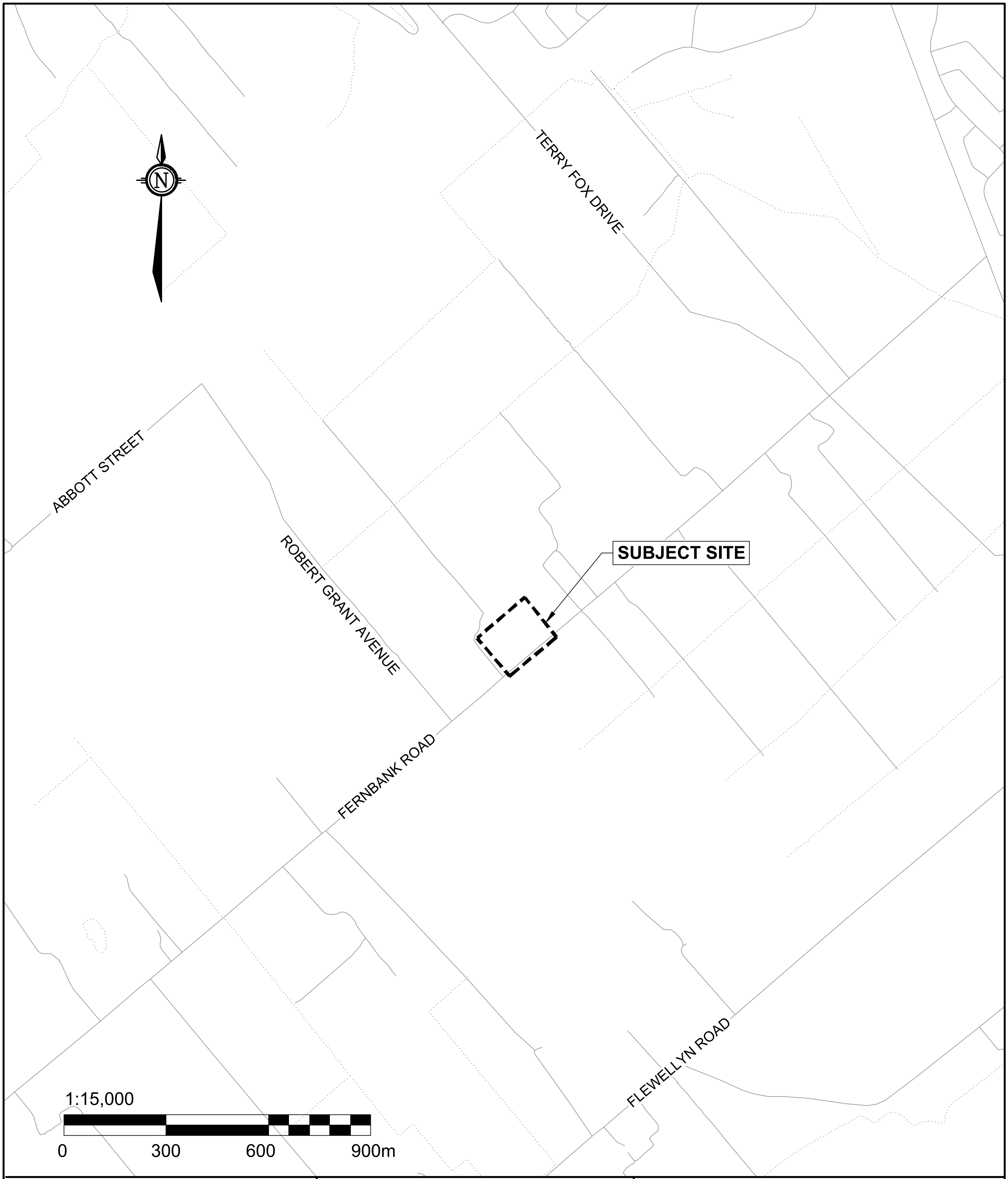
7.0 CLOSURE

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.

Kelsey Holkestad, B.Eng., EIT

LICENSED PROFESSIONAL ENGINEER
B.D. WIEBE
100060438
07 Mar 2019
PROVINCE OF ONTARIO

Brent Wiebe, P.Eng.
VP Operations - Ontario



GEMTEC
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Project
GEOTECHNICAL INVESTIGATON
FERNBANK CROSSING
RESIDENTIAL SUBDIVISION - PHASE 5
OTTAWA, ONTARIO

Drawing
KEY PLAN

Drwn By	Chkd By	Date	Project No.	Revision No.	
P.C.	K.H.	MARCH 2019	64153.97	0	**FIGURE 1**

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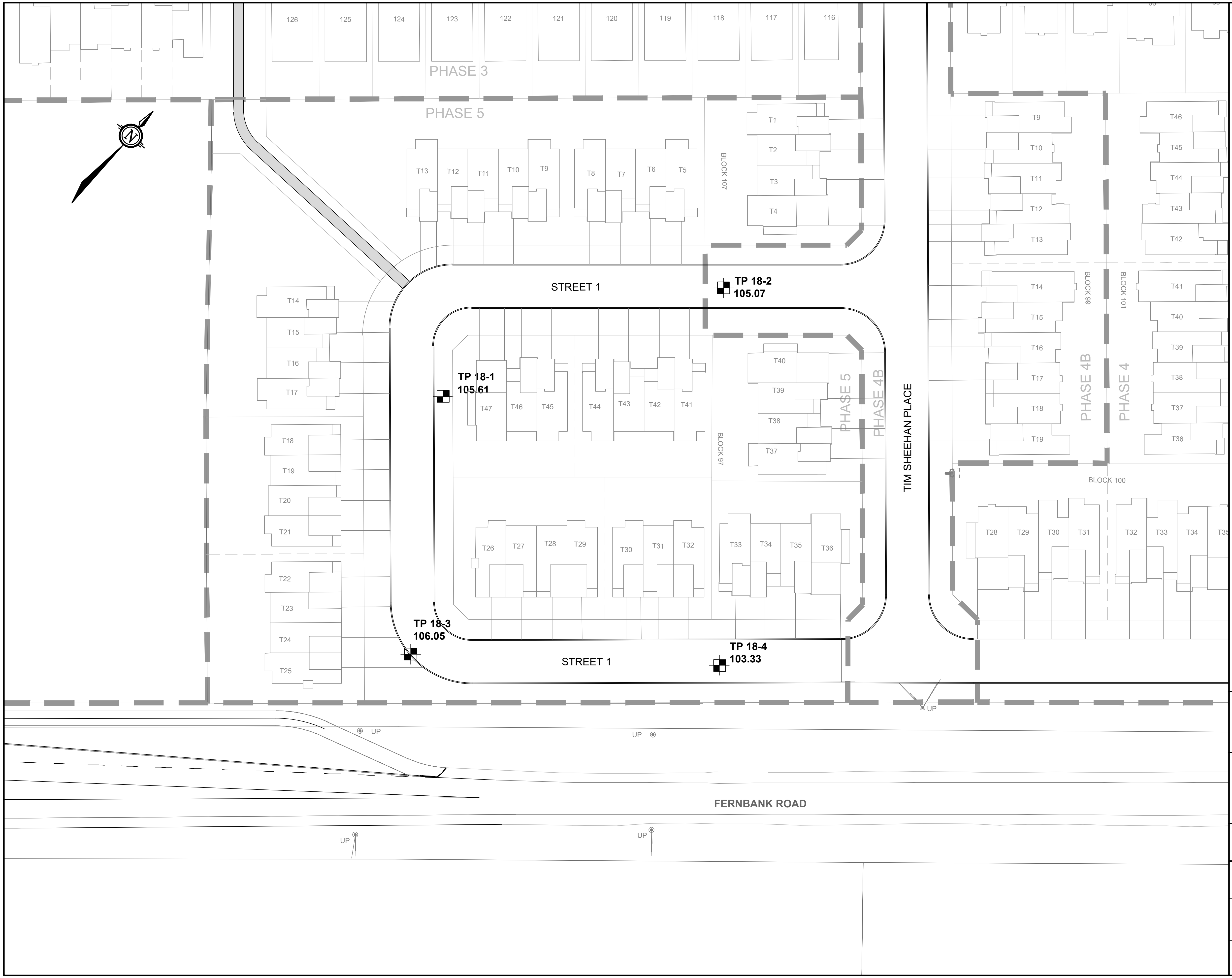


LEGEND

▚ TEST PIT LOCATION IN PLAN
(current investigation by GEMTEC)

TP 18-1 ← TEST PIT ID
105.61 ← GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM

Scale 1:750
0 15 30 45m

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

32 Steacie Drive
Ottawa, ON K2K 2A9
Tel: (613) 836-1422
www.gemtec.ca
ottawa@gemtec.ca

Client NOVATECH	Project 64153.97
Location FERNBANK CROSSING - PHASE 5 OTTAWA, ON	
Drwn by P.C.	Chkd by K.H.
TEST PIT LOCATION PLAN	
Date MARCH 2019	Rev. 0
FIGURE 2	

P:\0. Files\64100\64153.97\Drafting\Drawings\64153.97_FG2_V01_2018-08-17.dwg, FIGURE 2, 18/11/30 2:50:56 PM

APPENDIX A

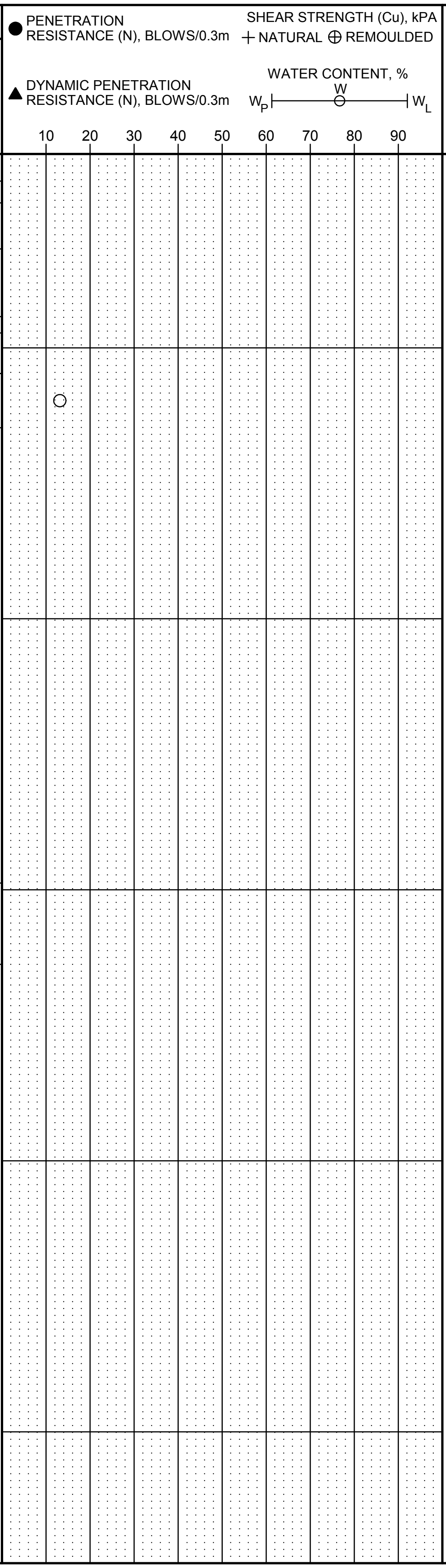
Record of Test Pit Sheets
List of Abbreviations and Terminology

RECORD OF BOREHOLE 18-1

CLIENT: Novatech
PROJECT: Fernbank Crossing - Phase 5
JOB#: 64153.97
LOCATION: See Test Pit Location Plan, Figure 2

SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Aug 14 2018

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Excavator 470 Hitachi	Ground Surface	105.62						
		Dark brown sandy silt, with organic material (TOPSOIL)	105.54 0.08	1	GS				Backfilled with soil cuttings
		Brown SILTY SAND, trace to some clay and gravel, trace organic material	104.71 0.91	2	GS				
				3	GS				
				4	GS			MH	
1 2		Grey brown silty sand, some gravel, trace to some clay, with cobbles and boulders (GLACIAL TILL)	102.72 2.90	5	GS				Groundwater was not observed in open test pit
3 4 5		End of test pit Excavator refusal on inferred bedrock							



GEO - BOREHOLE LOG 64153.97_GNT_V01_2018-08-15.GPJ GEMTEC 2018.GDT 30/8/18

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LOGGED: K.H.
CHECKED: B.W.

RECORD OF BOREHOLE 18-2

CLIENT: Novatech
PROJECT: Fernbank Crossing - Phase 5
JOB#: 64153.97
LOCATION: See Test Pit Location Plan, Figure 2

SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Aug 14 2018

● PENETRATION RESISTANCE (N), BLOWS/0.3m
▲ DYNAMIC PENETRATION RESISTANCE (N), BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA + NATURAL ⊕ REMOULDED
WATER CONTENT, % W_P ⊢ W ⊖ ⊣ W_L
10 20 30 40 50 60 70 80 90

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface	105.07						
	Excavator 470 Hitachi	Brown to grey brown SILTY SAND, some clay, gravel and cobbles		1	GS				
				2	GS			MH	
1									
		Grey brown silty sand, some gravel, trace to some clay, with cobbles and boulders (GLACIAL TILL)	103.70 1.37						
2									Backfilled with soil cuttings
3									
				3	GS				
		End of test pit Excavator refusal on inferred bedrock	101.41 3.66						Groundwater was not observed in open test pit
4									
5									

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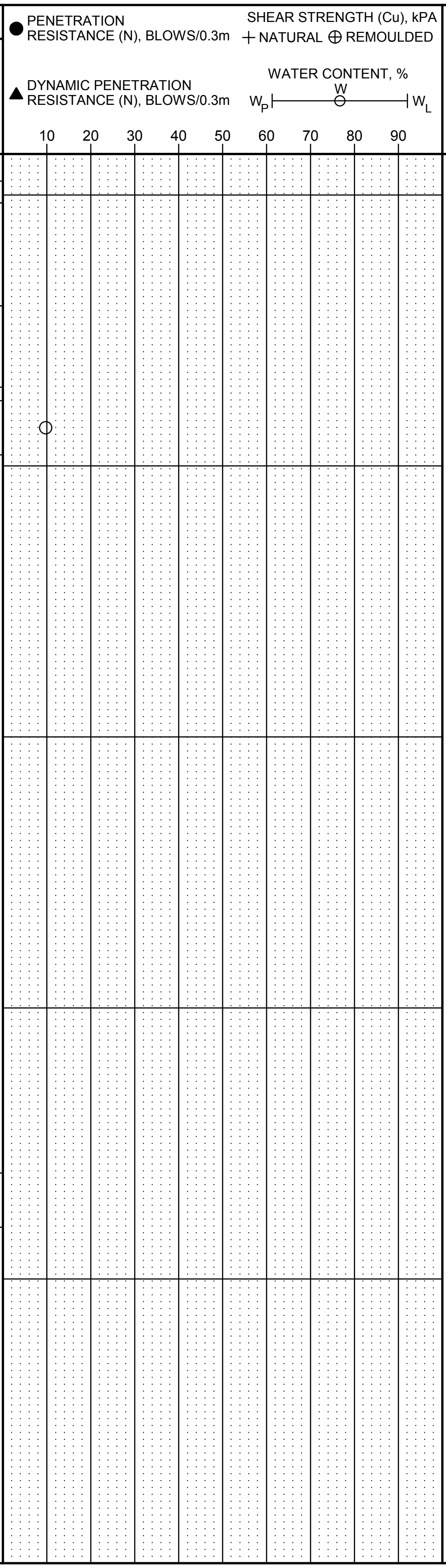
CHECKED: B.W.

RECORD OF BOREHOLE 18-3

CLIENT: Novatech
PROJECT: Fernbank Crossing - Phase 5
JOB#: 64153.97
LOCATION: See Test Pit Location Plan, Figure 2

SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Aug 14 2018

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Excavator 470 Hitachi	Ground Surface	106.05						
		Dark brown sandy silt, with organic material (TOPSOIL)	105.97 0.08	1	GS				Backfilled with soil cuttings
		Dark brown SILTY SAND, trace clay and gravel, with roots		2	GS				
			105.29 0.76	3	GS				
1		Grey brown silty sand, some gravel, trace to some clay, with cobbles and boulders (GLACIAL TILL)		4	GS			MH	
2									
3			102.19 3.86	5	GS				Groundwater was not observed in open test pit
4		End of test pit Excavator refusal on inferred bedrock							
5									



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LOGGED: K.H.
CHECKED: B.W.

RECORD OF BOREHOLE 18-4

CLIENT: Novatech
PROJECT: Fernbank Crossing - Phase 5
JOB#: 64153.97
LOCATION: See Test Pit Location Plan, Figure 2

SHEET: 1 OF 1
DATUM: CGVD2013
BORING DATE: Aug 14 2018

● PENETRATION RESISTANCE (N), BLOWS/0.3m
▲ DYNAMIC PENETRATION RESISTANCE (N), BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA + NATURAL ⊕ REMOULDED
WATER CONTENT, % W_P ⊢—○—⊣ W_L (W)
10 20 30 40 50 60 70 80 90

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Excavator 470 Hitachi	Ground Surface		103.33						
		Grey brown silty sand, some gravel, trace to some clay (GLACIAL TILL)			1	GS				
1		Grey brown silty sand, some gravel, trace to some clay, with cobbles and boulders (GLACIAL TILL)		102.57 0.76						Backfilled with soil cuttings
2										
		End of test pit Excavator refusal on inferred bedrock		100.79 2.54						Groundwater was not observed in open test pit
3										
4										
5										

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LOGGED: K.H.
CHECKED: B.W.

ABBREVIATIONS AND TERMINOLOGY USED ON RECORDS OF BOREHOLES AND TEST PITS

SAMPLE TYPES	
AS	Auger sample
CA	Casing sample
CS	Chunk sample
BS	Borros piston sample
GS	Grab sample
MS	Manual sample
RC	Rock core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled open shelby tube
TP	Thin-walled piston shelby tube
WS	Wash sample

SOIL TESTS	
w	Water content
PL, w_p	Plastic limit
LL, w_L	Liquid limit
C	Consolidation (oedometer) test
D_R	Relative density
DS	Direct shear test
G_S	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
UC	Unconfined compression test
γ	Unit weight

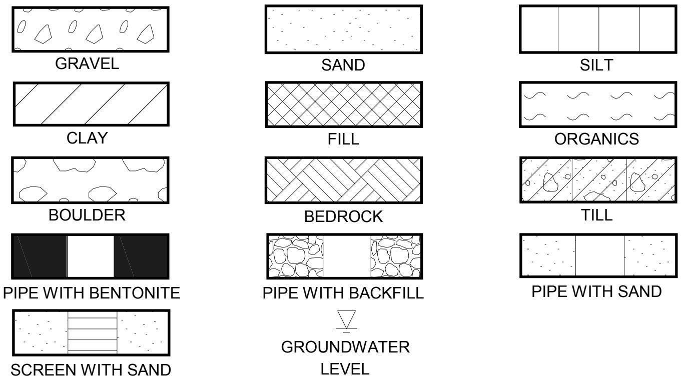
PENETRATION RESISTANCE

Standard Penetration Resistance, N
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 millimetres (30 in.) required to drive a 50 mm split spoon sampler for a distance of 300 mm (12 in.). 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.

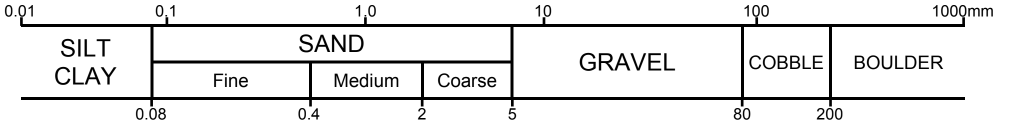
Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).

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

COHESIONLESS SOIL Compactness		COHESIVE SOIL Consistency	
SPT N-Values	Description	Cu, kPa	Description
0-4	Very Loose	0-12	Very Soft
4-10	Loose	12-25	Soft
10-30	Compact	25-50	Firm
30-50	Dense	50-100	Stiff
>50	Very Dense	100-200	Very Stiff
		>200	Hard



GRAIN SIZE



DESCRIPTIVE TERMINOLOGY

(Based on the CANFEM 4th Edition)

0	10	20	35
TRACE	SOME	ADJECTIVE	noun > 35% and main fraction
trace clay, etc	some gravel, etc.	silty, etc.	sand and gravel, etc.

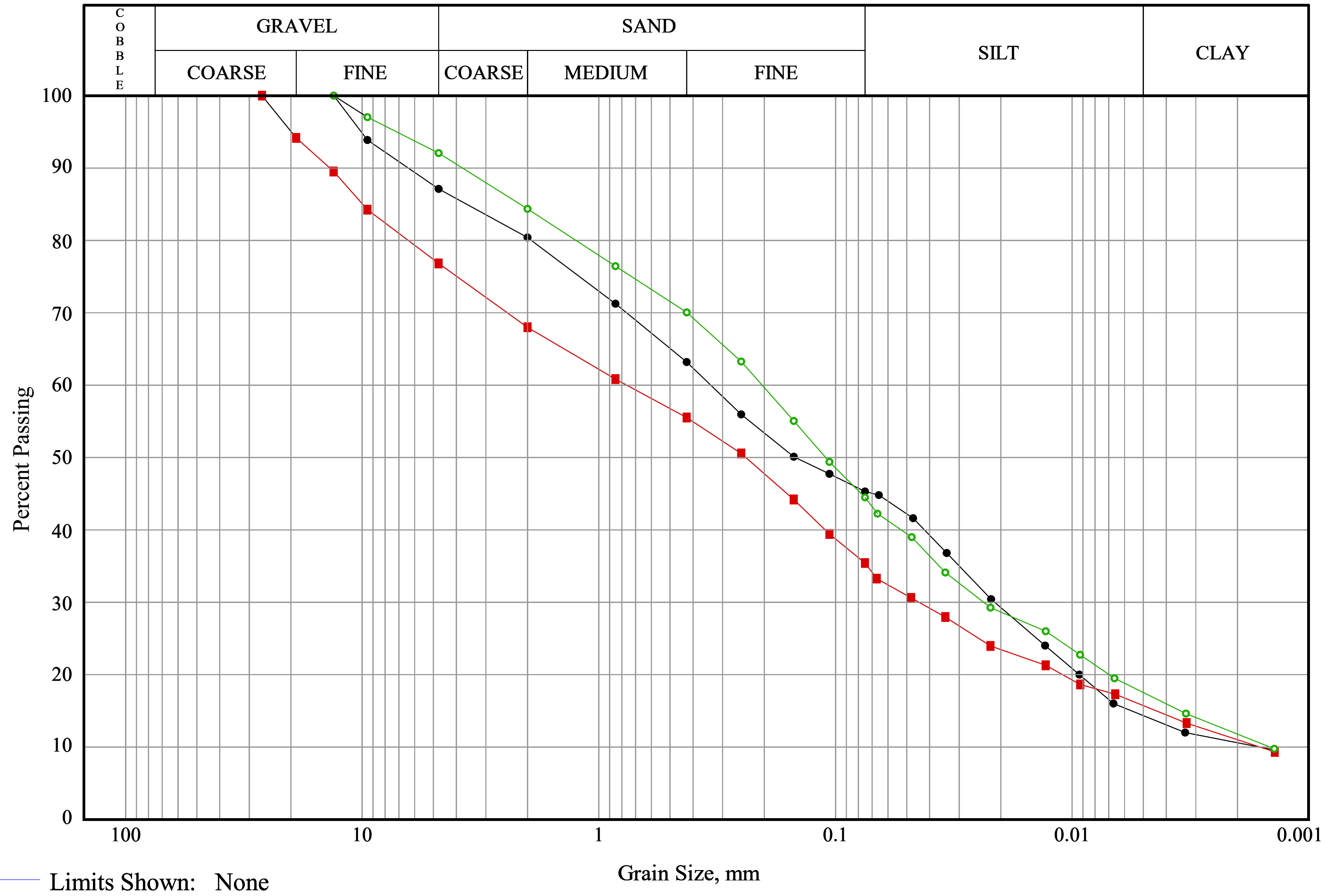
GEMTEC

descriptive terms.pub
Modified May 2018

APPENDIX B

Results of Laboratory Index Testing

GEMTEC Consulting Engineers and Scientists	Client: Novatech Project: Fernbank Crossing Subdivision Project #: 6415397	Soils Grading Chart



Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
●		18-1	4	0.7-0.8	12.9	41.8	31.0	14.3
■		18-2	2	0.3-0.6	23.2	41.4	19.7	15.7
○		18-3	4	0.8-2.1	7.9	47.6	26.9	17.5

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75μm
●	Silty sand , some gravel, some clay		0.00	0.01	0.02	0.15	0.34	3.61	31.0
■	Gravelly sand , some silt, some clay		0.00	0.00	0.04	0.24	0.76	9.96	19.7
○	Silty sand , some clay , trace gravel		0.00	0.00	0.02	0.11	0.20	2.15	26.9

APPENDIX C

Chemical Analyses of Soil Samples Relating to Corrosion Order No. 1833414

Certificate of Analysis
Client: GEMTEC Consulting Engineers and Scientists Limited
Client PO: 64153.97

Report Date: 20-Aug-2018
Order Date: 15-Aug-2018
Project Description: 64153.97

	Client ID:	TP 18-3 SA-4	-	-	-
	Sample Date:	08/14/2018 09:00	-	-	-
	Sample ID:	1833414-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	90.3	-	-	-
General Inorganics					
pH	0.05 pH Units	7.84	-	-	-
Resistivity	0.10 Ohm.m	94.4	-	-	-
Anions					
Chloride	5 ug/g dry	5	-	-	-
Sulphate	5 ug/g dry	<5	-	-	-

experience • knowledge • integrity

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

expérience • connaissance • intégrité