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Geotechnical Investigation
Proposed Warehouse Building
9460 Mitch Owens Road
5606, 5630, 5592 Boundary Road

Ottawa, Ontario

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Submitted to:

Touchstone Contracting and Engineering Ltd.
P.O Box 124
Greely, Ontario
K4P 1N4

Geotechnical Investigation
Proposed Warehouse Building
9460 Mitch Owens Road
5606, 5630, 5592 Boundary Road

Ottawa, Ontario

December 18, 2018
Project: 60369.15

experience • knowledge • integrity

GEMTEC Consulting Engineers and Scientists Limited
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December 18, 2018 File: 60369.15

Touchstone Contracting and Engineering Ltd.
P.O Box 124
Greely, Ontario
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Attention: Mr. David Kurosky

Re: Geotechnical Investigation
Proposed Warehouse Building
9460 Mitch Owens Road
5606, 5630, 5592 Boundary Road
Ottawa, Ontario

Please find enclosed the supplemental geotechnical investigation report for the above noted project. This report was prepared by Mr. Johnathan A. Cholewa, Ph.D., P.Eng., and reviewed by Mr. Craig Houle, M.Eng., P.Eng..

Do not hesitate to contact the undersigned if you have any questions or require additional information

Johnathan A. Cholewa, Ph.D., P.Eng.

Craig Houle, M.Eng., P.Eng.
Senior Geotechnical Engineer

Enclosures
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1.0 INTRODUCTION

This report presents the results of a subsurface investigation carried out for the proposed warehouse building to be constructed at 9460 Mitch Owens Road, and 5606, 5630, and 5592 Boundary Road 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 boreholes 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.

2.0 BACKGROUND

2.1 Site Description

The proposed development site is located at 9460 Mitch Owens Road, and 5606, 5630, and 5592 Boundary Road in Ottawa, Ontario (see Key Plan, Figure 1). The site is currently vacant; however, a north-south aligned gravel surfaced access roadway currently extends through the site. Imported fill material has been placed over portions of the site.

2.2 Project Description

It is understood that consideration is being given to constructing a warehouse building at 9460 Mitch Owens Road, and 5606, 5630, and 5592 Boundary Road in Ottawa, Ontario. Based on the preliminary information provided to us, the proposed building will occupy a footprint of about 4,992 square metres and will have a finished floor level at elevation 78.9 metres. The exterior finished grade around the perimeter will be at about elevation 77.7 metres, or about 1.2 metres below the planned finished floor level. The proposed underside of footing elevation is at 76.9 metres. The site is to be serviced by a drilled water well and septic disposal system.

The proposed development also includes: gravel and asphalt surfaced access roadways and parking areas, exterior concrete slabs on grade, stormwater ponding areas, and an underground water storage tank for fire fighting.

A copy of the most current site development plan is provided in the Appendix B.

2.3 Previous Investigation

A previous geotechnical investigation was carried out at the site by St. Lawrence Testing and Inspection Co. Ltd. (SLTI) on April 7, 2010. The findings of the SLTI ground investigation are described in their report titled, “Property at County Roads 8 & 41, Edwards, ON, Geotechnical Subsurface Investigation, Report No. 10C75”, dated April 27, 2010.

As part of the previous subsurface investigation carried out by SLTI, a total of four (4) boreholes were advanced at the site. At the time of preparation of this report, the exact locations of these previous boreholes were not available. The boreholes were advanced to depths ranging from about 3.7 to 22.6 metres below ground surface. Standard penetration tests were carried out in the upper 2.2 to 3.7 metres of the boreholes and samples of the soils encountered were recovered using a 50 millimetre diameter drive open sampler. Dynamic cone penetration testing was carried out in two (2) of the boreholes from about 2.2 metres to about 21.0 and 22.6 metres below ground surface. Vane shear testing was not carried out by SLTI within the silty clay layer. In addition, laboratory oedometer consolidation testing of the silty clay layer was not carried out.

The subsurface conditions encountered in the SLTI boreholes consisted of a surficial topsoil layer generally underlain by silty sand followed by sensitive silty clay deposits. At between about 18.3 and 18.9 metres below ground surface, the number of blows required to advance the cone during the dynamic cone penetration tests increased from 3 to between 20 and 113 blows per 0.3 metres of penetration. These results reflect a material having a dense to very dense relative density and/or the presence of cobbles/boulder obstructions.

2.4 Canadian Geotechnical Research Site No. 1

In March 1955, the National Research Council of Canada began measuring settlement of the new Accommodation Building constructed at HMCS Gloucester. The site, known as Canadian Geotechnical Research Site No. 1, is located just north of Mitch Owens Road approximately 7.4 kilometres west of the proposed commercial development. One of the objectives of this research was to obtain a long term performance record of a light structure constructed on a very compressible, near normally consolidated clay soil. The Accommodation Building was built in August 1954. The structure consisted of slab on grade construction with footings on the original ground. The finished floor elevation was about 1.4 metres higher than the original ground surface and the floor slab was supported on imported sand and gravel (Crawford and Bozozuk, 1990). The soil conditions at Geotechnical Research Site No. 1 are similar to those at this site.

Survey points were established around the exterior of the building and in a grid on the floor slab. The survey points were referenced to a benchmark set on bedrock. The initial survey was carried out on March 3, 1955. After 1 year, the slab had settled about 70 millimetres. Twenty years later the slab had a settled 300 millimetres, increasing to 330 millimetres after 33 years (Crawford and Bozozuk, 1990).

2.5 Review of Geology Maps

Geology maps from the urban geology database of Canada's National Capital Region (Geological Survey of Canada, Open File 2878, 1994) indicate that the subsurface conditions are expected to consist of overburden deposits of sand underlain by marine deposits of silty clay. The bedrock is mapped as shale of the Carlsbad formation at depths of between 15 and 25 metres.

3.0 SUBSURFACE INVESTIGATION

The field work for this investigation was carried out on June 9 and 19, 2010. During that time, five (5) boreholes, numbered 10-1 to 10-5, inclusive, were advanced using a hollow stem auger drill rig supplied and operated by George Downing Estate Drilling Ltd. of Grenville-sur-la-Rouge, Quebec. Details of the boreholes are provided below:

- Two (2) boreholes, numbered 10-1 and 10-2, were advanced to about 6.7 and 15.9 metres below ground surface, respectively.
- Three (3) boreholes, numbered 10-3 to 10-5, were advanced to between 2.1 and 2.9 metres depth.

Standard penetration tests were carried out in the boreholes and samples of the soils encountered were recovered using a 50 millimetre diameter split barrel sampler. The standard penetration tests were carried out in general accordance with ASTM D1586. In situ vane shear strength testing was carried out where possible in the clayey deposits to measure the undrained shear strength. The vane shear testing was carried out in general accordance with ASTM D2573. Well screens were sealed in boreholes 10-1 and 10-2 to measure the groundwater levels.

The field work was supervised throughout by a member of our engineering staff, who located the boreholes, logged the samples and observed the in-situ testing. Following the field work, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of the soil were tested for water content, grain size distribution, and Atterberg limits. The grain size distribution testing was carried out in accordance with ASTM D6913. Moisture content testing was carried out in accordance with LS-701 and ASTM D2216. The Atterberg limit testing was carried out in accordance with LS-703/704.

Five (5) relatively undisturbed Shelby tube samples of the softer portions of the clayey deposits were obtained from boreholes 10-1 and 10-2. The samples were recovered using a piston sampler supplied by George Downing Estate Drilling Ltd. Laboratory vane shear strength testing was carried out in selected Shelby tube samples. A Shelby tube sample recovered from borehole 10-1 at a depth of about 4.9 metres below ground surface was selected for oedometer consolidation testing, laboratory classification and laboratory vane shear strength testing. The consolidation testing was carried out in general accordance with ASTM D2435.

The approximate locations of the boreholes are shown on the Borehole Location Plan, Figure 2. Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix A. The soil descriptions are based, in part, on the visual examination and manual test procedures described in ASTM D2488. The results of the soil classification testing from samples recovered from the boreholes are provided on the Record of Borehole sheets and on Figures A1 to A2 in Appendix A.

The borehole locations were determined relative to existing site features by GEMTEC Consulting Engineers and Scientists Limited (GEMTEC), formerly Houle Chevrier Engineering Ltd. The borehole elevations were measured relative to the top of the culvert located on the subject property south of Mitch Owens Road. The elevation of this point was arbitrarily taken as 100.0 metres, local datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

The soil and groundwater conditions logged in the boreholes are given on the Record of Borehole sheets in Appendix A. The borehole 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. Subsurface conditions at other than the borehole locations may vary from the conditions encountered in the boreholes. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site.

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 boreholes advanced during this investigation.

4.2 Fill Material

In borehole 10-5, fill material composed of dark brown silty sand with gravel and clay was encountered from ground surface. The fill material has a thickness of approximately 1.4 metres. A standard penetration test carried out in the fill material gave an N value of 4 blows per 0.3 metres of penetration, which reflects a very loose to loose relative density.

During a site reconnaissance on November 14, 2012, fill material was visually observed over the western portion of the site north of boreholes 10-1 and 10-3, and south of boreholes 10-2 and 10-4. Based on our visual assessment, the fill material has a thickness of about 0.5 to 1.0 metres. The physical and chemical composition of the fill material observed on November 14, 2012 is unknown.

4.3 Silty Sand

A deposit of brown silty sand, ranging in thickness from 0.4 to 1.1 metres, was encountered from ground surface in boreholes 10-1 to 10-4, inclusive, and below the fill material in borehole 10-5. Based on one standard penetration test conducted in borehole 10-5 the silty sand deposit has a loose relative density.

4.4 Silty Clay

Deposits of silty clay were encountered in all of the boreholes beneath the silty sand. The silty clay was not fully penetrated as part of this investigation; however, based on the results of the previous investigation by SLTI, the silty clay deposit is about 18 metres thick.

The upper part of the silty clay is weathered reddish grey brown. Standard penetration tests carried out in the weathered, silty clay gave N values ranging from 5 to 6 blows per 0.3 metres of penetration, which reflect a stiff to very stiff consistency. The weathered silty clay has a thickness of between about 0.6 to 1.4 metres and extends to depths of about 1.8 to 2.4 metres below ground surface (elevation 98.1 to 98.4 metres, local datum).

Below the weathered zone, the silty clay is grey in colour. The grey silty clay contains occasional silty sand and silt seams. Standard penetration tests carried out in the grey silty clay gave N values of “static weight of hammer (WH)” to 2 blows per 0.3 metres of penetration. In situ vane shear strength tests carried out in the grey silty clay in boreholes 10-1 and 10-2 gave undrained shear strengths of 17 to 46 kilopascals, which indicate a soft to firm consistency. Between 2.6 and 11.6 metres below ground surface, the undrained shear strengths ranged between 17 and 29 kilopascals. Between 11.6 to 15.9 metres below ground surface, the undrained shear strengths steadily increased from 31 to 46 kilopascals. Laboratory vane shear strength tests carried out in the Shelby tube samples of the softest portions of the grey silty clay recovered from boreholes 10-1 and 10-2 gave undrained shear strength values of 17 to 33 kilopascals, which indicate a soft to firm consistency.

The results of one (1) laboratory oedometer consolidation test carried out on the relatively undisturbed thin walled Shelby tube piston sample of the grey silty clay recovered from borehole 10-1 are provided in Table 4.1.

Table 4.1 – Consolidation Test Results

Borehole	Sample Depth (metres)	Estimated Apparent Past Preconsolidation Pressure (kilopascals)	Calculated Existing Vertical Effective Stress (kilopascals)	Initial Void Ratio	Recompression Index	Compression Index
10-1	4.6 - 5.2	70	41	0.99	0.04	0.28

A graph of the variation in void ratio with applied stress from the consolidation test carried out by GEMTEC is provided on Figure A3 in Appendix A. It is noted that the softer portions of the silty clay deposits, based on the in-situ vane shear strength tests, were targeted for consolidation testing.

The results of an Atterberg limit test carried out on a sample of the grey silty clay from borehole 10-1 gave a liquid limit of 61 percent, a plastic limit of 25 percent and a corresponding plasticity index of 36 (see Figure A2). This testing indicates that the silty clay has high plasticity. The water content of the sample tested was 92 percent, which is much greater than the measured liquid limit value.

A grain size distribution curve for a sample of the silty clay recovered from borehole 10-1 is provided on Figure A1.

4.5 Groundwater Levels

The groundwater level in the well screens installed in boreholes 10-1 and 10-2 were at 0.6 and 1.6 metres below ground surface (elevation 99.3 and 98.3 metres, local datum) on June 17, 2010, respectively. The groundwater conditions at the other borehole locations were not observed.

The groundwater levels are expected to vary seasonally and may be higher during wet periods of the year such as the early spring or following periods of precipitation, particularly within the upper silty sand deposits.

5.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

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 Grade Raise Restrictions

The site is underlain by deposits of sensitive silty clay, which has a limited capacity to support loads imposed by grade raise fill material, pavement structures and foundations for the buildings. The placement of fill material on this site must therefore be carefully planned and controlled so that the stress imposed by the fill material does not result in excessive consolidation of the silty clay deposit. Concrete slabs, granular base materials, overall grade raise and pavement structures are considered grade raise filling. The settlement response of the silty clay deposits due to the increase in stress caused by fill material is influenced by variables such as the existing effective overburden pressure, the past preconsolidation 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 near or above the preconsolidation pressure.

Based on the results of the subsurface investigation in conjunction with the oedometer consolidation test results, the maximum thickness of any grade raise filling is provided in Table 5.1. The grade raise restrictions provided in Table 5.1 have been calculated in order to limit the total settlement of the ground to about 25 millimetres in the long term.

Table 5.1 – Site Grade Raise Restrictions

Material Type	Maximum Unit Weight (kN/m^3)	Maximum Thickness of Grade Raise Fill Material above Native Soil (metres)
OPSS Granular A or Granular B Type II	22.0	0.40
General Fill (e.g., silty sand)	18.5	0.50
19 millimetre Clear Crushed Stone	16.0	0.60

At least 2.4 metres beyond the building footprint, and where settlements in excess of 25 millimetres can be tolerated (e.g., within access roadways, parking areas, landscaped areas), a grade raise of up to 1 metre above native soil would result in settlements that could potentially be accommodated by on-going site maintenance (e.g., padding, crack sealing, etc.).

Additional comments on site grading are provided below:

- The maximum permissible grade raise within the building footprint depends, in part, on the thickness of the concrete floor slab. For a 250 millimetre thick floor slab, the maximum combined thickness of concrete and granular base material is 450 millimetres, assuming that the granular base material is composed of 19 millimetre clear stone.

- Where the grade raise restriction cannot be achieved, consideration could be given to the use of expanded polystyrene (EPS) blocks, which are specifically manufactured for this purpose, to make up the additional depth of grade raise.

- In areas where the thickness of the existing fill material on the property exceeds the site grade raise restriction, and where ground settlements in excess of 25 millimetres cannot be tolerated, it will be necessary to remove the fill material such that the thickness of the remaining fill material is at, or below, the site grade raise restriction. In the vicinity of the building, the fill removal described above will have to include an area extending 2.4 metres beyond the perimeter of the foundation.

- For design purposes, we have assumed that the groundwater lowering due to the development at this site will be at most 0.5 metres (causing an increase in effective stress of 5 kilopascals greater than that which presently exists).

5.3 Warehouse Building

5.3.1 Excavation

The excavations for the building foundations will be carried out through topsoil, fill material, silty sand, and reddish brown silty clay. Based on the results of the investigation, and assuming relatively shallow excavations are required (i.e., not exceeding about 1 metre depth), it is not expected that the excavations will extend into the underlying grey silty clay. For excavation exceeding 1.2 metres in depth, the sides of 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 fill material and native silty sand 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.

Based on the measured groundwater levels, and assuming that a perched groundwater table will likely be present within the silty sand above the silty clay deposits, excavation below the groundwater level may be required, which could present some constraints. There is potential for disturbance to the silty sand on the sides and bottom of the excavation and relatively flat side slopes may be required to prevent sloughing of material into the excavation. The groundwater inflow should be controlled throughout the excavation by pumping from sumps within the excavation. Notwithstanding, some disturbance and loosening of the subgrade materials could occur where excavation below the groundwater level is required, and allowance should be made for subexcavation of any disturbed subgrade soil.

5.3.2 Foundation Design

In general, the native silty sand and reddish brown silty clay are considered suitable to support lightly loaded spread footing foundations. All organic material, topsoil, and loose or water softened soils should be removed from within the proposed footing areas. The existing fill material should also be removed from within the footprint of the proposed building.

In areas where proposed founding level is above the level of the native soil, or where subexcavation of disturbed material is required below proposed founding level, the following comments are provided:

- Imported granular material meeting OPSS requirements for Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type I, Granular B Type II, or Granular A could be used below the footings. The use of 19 millimetre clear stone could also be considered. Any clear stone should be tightly wrapped in a non-woven geotextile where it is in direct contact with sandy soils.
- The imported granular material should be placed in accordance with the site grade raise restrictions.
- OPSS Granular B Type I, Granular B Type II, and Granular A should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. The clear stone should be nominally compacted in maximum 300 millimetre thick lifts with at least 2 passes of a diesel plate compactor.
- Where subexcavation of greater than 0.5 metres of native soil is required, and where the native soil is replaced with heavier OPSS Granular B Type I, Granular B Type II, and Granular A, the net stress increase on the underlying silty clay should be reviewed by geotechnical personnel to confirm that the recommendations provided in this report are still valid.
- To allow spread of load beneath the footings, the imported granular material should extend horizontally at least 0.3 metres beyond the footings and then down and out from this point at 1 horizontal to 1 vertical, or flatter. The excavations should be sized to accommodate this fill placement.

The bearing pressures for strip or pad footing foundations at this site are based on the necessity to limit the stress increase on the softer grey silty clay layer to an acceptable level so that foundation settlements will not be excessive. Four important parameters in calculating the stress increase on the grey silty clay beneath the weathered crust are:

- The thickness of the soil beneath the base of the foundation and the surface of the grey silty clay;
- The size, type (i.e. pad or strip) and loading of the foundation;
- The amount of surcharge (fill, etc.) in the vicinity of the foundation; and
- The amount of post-development groundwater lowering at the site.

From a spread footing design perspective, it is preferable to maximize the vertical separation between the underside of the footings and the surface of the softer, grey silty clay to distribute the foundation loads onto the softer, grey silty clay at depth. This can be achieved by founding the structures as high as practical within the soil profile and minimizing the amount of fill (surcharge) on the site.

For preliminary planning and design purposes, the following could be considered for the design of the residential building foundations:

Table 5.2 – Foundation Bearing Capacity

Type of Footing	Maximum Underside of Footing Elevation (metres)	Maximum Size of Footing (metres)	Gross Geotechnical Reaction at Serviceability Limit State[1,2,3] (kilopascals)	Factored Geotechnical Resistance at Ultimate Limit State (kilopascals)
Exterior Strip	76.9 or above	0.9	45	150
Exterior Pad	76.9 or above	2.4 square	50	175
Interior Pad	76.9 or above	2.4 square	60	175

Notes:
1. Provided that any loose or disturbed soil is removed from the bearing surfaces, the total and differential settlement of the foundation at SLS should be less than 20 and 25 millimetres, respectively. These settlements, and the bearing capacities provided in Table 5.2, assume that the fill materials are placed in accordance with the site grade raise restrictions.
2. The gross geotechnical reaction at SLS does not include the weight of the footing and excavated soil, but assumes that the footings are backfilled in accordance with the site grade raise restrictions (refer to Table 5.2).
3. From a geotechnical perspective, it is acceptable to size the footings using 100 percent of the dead and snow loads, and 50 percent of the live load.

For the purpose of this assessment we have considered a long term groundwater lowering at the site equal to 0.5 metres below the measured groundwater level. The bearing pressures provided above do not consider a sustained floor load. If the sustained floor loading is to be significant (i.e., greater than 4.8 kilopascals), it may be necessary to reduce the bearing pressures provided above and/or reduce the permissible grading limits below the slab area.

There are many other possible combinations of founding depths, footing sizes, and thickness of grade raise fills which might be suitable for this site. All other alternatives must be checked by the geotechnical engineer to ensure that overstressing of the softer silty clay soil does not occur, as this could result in excessive settlement and cracking/distress of the structure.

In accordance with City of Ottawa standards, the allowable bearing pressure and grade raise restriction should be verified at the time of construction by advancing a series of auger probe holes from founding elevation.

5.3.3 Frost Protection of Foundations

As previously indicated, it is preferable to found the structure as high as practical within the soil profile. For this case, the required frost protection could be provided by means a combination of earth cover and extruded polystyrene insulation. An insulation detail is provided on Figure 3.

Extruded polystyrene insulation is not required for exterior footings provided with at least 1.5 metres of earth cover or isolated (unheated) piers that are provided with at least 1.8 metres of earth cover. In all areas which are to be kept clear of snow, at least 1.8 metres of earth cover or the equivalent in polystyrene insulation, is recommended.

5.3.4 Seismic Design

5.3.4.1 Potential for Liquefaction

The subject site is underlain predominately by deposits of silty clay, followed granular deposits, possibly glacial till, at about 18 to 19 metres below ground surface. According to the criteria discussed in the Commentary on CSA S6-14 Canadian Highway Bridge Design Code, soils with a plasticity index of greater than 12 percent can be considered non-liquefiable for design purposes. Furthermore, according to the Bray et al. (2004) criteria for the liquefaction assessment of fine grained soils, the sample of silty clay tested from borehole 10-1 is “not susceptible” to liquefaction (i.e., the plasticity index is greater than 20 percent). As such, based on the results of the investigation, in our opinion, the overburden deposits at this site are not susceptible to liquefaction.

5.3.4.2 Seismic Site Class

In accordance with the 2012 Ontario Building Code, seismic Site Class E should be used for design purposes.

5.3.5 Foundation Wall Backfill

5.3.5.1 General

The native soil deposits at this site are highly frost susceptible and should not be used as backfill against foundations, piers, etc. To avoid frost adhesion and possible heaving, the foundations should be backfilled with imported, free-draining, non-frost susceptible granular material meeting OPSS Granular B Type I or II requirements. The use of 19 millimetre clear stone could also be considered. Any clear stone should be tightly wrapped in a non-woven geotextile where it is in direct contact with sandy soils. The imported granular material should be placed in accordance with the site grade raise restrictions.

Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment. Where future landscaped areas will exist next to the proposed structure and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value. The clear stone should be nominally compacted in maximum 300 millimetre thick lifts with at least 2 passes of a diesel plate compactor

Where areas of hard surfacing (concrete, sidewalk, pavement, etc.) abut the proposed building, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible native materials to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from the bottom of the excavation or 1.5 metres below finished grade, whichever is less, to the underside of the granular base/subbase material for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.

Perimeter foundation drainage is not considered necessary for slab on grade structures at this site provided that the floor slab level is above the finished exterior ground surface level at the building.

5.3.5.2 EPS Foundation Backfill

As previously indicated, where the grade raise restriction cannot be achieved, consideration could be given to the use of expanded polystyrene (EPS) blocks to make up the additional depth of grade raise. The following general comments are provided regarding the use of EPS blocks as foundation backfill:

- All EPS material should meet the designation in ASTM D6817, Standard Specification for Rigid Cellular Polystyrene Geofoam.
- It is recommended that EPS should be placed above the groundwater level and 1:100 year storm elevation. Alternatively, where this requirement cannot be met, the upper surface of the light weight fill material should be weighted down to reduce the potential for buoyant uplift from occurring during a storm event.
- Where required, the EPS should extend at least 2.4 metres from the edge of the exterior footings. EPS will also likely be required for backfill below the slab on grade where the grade raise restrictions cannot be achieved.

It should be noted that, depending on the proposed grades, the use of EPS may be required in other areas of the site where the grading limits are exceeded and where ground settlements in excess of 25 millimetres cannot be tolerated.

The types of EPS material for the proposed development, including general specifications and installation guidelines, could be provided upon request as the design progresses.

5.3.6 Slab on Grade Support

To prevent excessive settlement of the floor slab, all topsoil, fill material, and other deleterious materials should be removed from below the proposed slab. The subgrade surface should then be proof rolled with a static drum roller under dry conditions. Any soft areas that are evident from the proof rolling should be subexcavated and replaced in accordance with our recommendations provided in the second paragraph of Section 5.3.2.

If necessary, the grade within the proposed building could be raised, where necessary, with granular material meeting OPSS requirements for Granular A, Granular B Type II, or 19 millimetre clear stone. The granular base for the proposed slab on grade should consist of at least 150 millimetres of OPSS Granular A or 19 millimetre clear crushed stone. The placement of the engineered fill below the slab should comply with the requirements given in Table 5.1. It is noted that the concrete for the slab on grade is considered grade raise fill.

A grade raise of more than the allowable limits specified in Table 5.1. will require the use of EPS blocks to make up the difference where the depth of the grade raise fill exceeds these limits. The slab on grade and granular base layer could then be constructed on the surface of the EPS blocks. The types of EPS material for use below the slab on grade could be provided as required. A modulus of subgrade reaction for the EPS material, which can be used by the structural engineer for design of the slab, could be provided by the product manufacturer.

The granular base materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density value. The clear stone should be nominally compacted in maximum 300 millimetre thick lifts with at least 2 passes of a diesel plate compactor

Underfloor drainage is not considered necessary provided that the floor slab level is above the finished exterior ground surface level.

Where interior areas of the building will be unheated, thermal protection for the subgrade will be required below the floor slab. It should be noted that EPS provides adequate frost protection of the underlying subgrade soils if it is used below the proposed building. Further details on the insulation requirements could be provided, if necessary.

Control joints (either saw cut or hand formed) could be installed in the concrete slab to reduce the potential for random cracking of the slab due to shrinkage. The control joints should be at least one quarter of the depth of the slab and must be made before uncontrolled shrinkage of the slab occurs. For saw cut joints, cutting should be carried out as soon as the concrete surface has hardened sufficiently to resist raveling (usually within 6 to 18 hours of placing the concrete). The spacing of the control joints should be provided by the structural engineer.

The surface of the concrete should be protected from early drying and extreme temperatures during the initial set and curing using appropriate wet curing methods.

5.3.7 Effects of Trees on the Foundations

The site is underlain by silty clay, a material which is known to be susceptible to shrinkage with a 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 deciduous trees can cause a reduction of moisture content in the silty clays found in the area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations and hard surfaced areas.

To eliminate the potential of settlement related damage, we suggest that, for buildings founded above 3.5 metres depth, no deciduous trees should be permitted closer to the building (or any ground supported structures which may be affected by settlement) than the ultimate height of the trees. For groups of trees or trees in rows, the separation distance should be increased to 1.5 times the ultimate height of the trees. The effects of trees (both existing and proposed) on the buildings should be considered in the landscape plan for this development.

If reduced tree setbacks are desired, the City of Ottawa guidelines for tree planting in sensitive marine clay soils (2017) could be considered to reduce the risk of possible damage to foundations. Mitigation measures described in the City of Ottawa document include:

- Plant small to medium sized trees (i.e., trees with a mature height of less than 14 metres);
- Nominally reinforce the foundation walls to provide ductility (minimum two (2) upper and two (2) lower 15M bars in the foundation wall);
- Provide grading around the tree that promotes drainage to the root zone; and
- Provide the tree with the minimum volume of uncompacted soil determined by a landscape architect.

It is noted that these mitigation measures minimize, not eliminate, the potential for ground settlement beneath foundations due to soil shrinkage.

5.3.8 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.

5.4 Septic Bed

It will be necessary to import granular material for the septic leaching bed, which will present some constraints for a geotechnical point of view. The following general comments are provided regarding the construction of leaching beds in these areas:

- Where required, the grade raise fill material below the leaching beds (i.e., below the septic sand) should consist of material which meets OPSS specifications for Select Subgrade Material. Guidelines for the placement of fill material below the leaching beds are provided below:
    - Prior to placing the grade raise fill material, all surficial topsoil and any soft, wet or deleterious materials should be removed from the subgrade surface.
    - The fill material should extend at least 0.3 metres beyond the footprint of the bed and then down and out from this point at 1 horizontal to 1 vertical, or flatter;
    - The grade raise fill material should be nominally compacted (i.e., with the haulage and spreading equipment);
    - The type of fill material (i.e., grain size distribution) should be consistent across the footprint of the leaching bed.

- In order to limit the total settlement of the ground to about 25 millimetres in the long term, the septic sand, and any grade raise fill material below the septic sand, should be placed in accordance with the grade raise restrictions provided in Table 5.1. Based on our experience, header and distribution piping within a conventional leaching bed can tolerate some uniform settlement; the piping is less tolerable to differential settlement. If a settlement in excess of 25 millimetres can be tolerated, a grade raise of up to 1 metre above native soil could potentially be accommodated provided that the fill material is of uniform thickness below, and within 2.4 metres, of the header and distribution piping (in order to minimize differential settlement).

- Some re-leveling of pipes may be required over time due to the settlement of the leaching bed.

We recommend that the final septic and grading plans for the site be reviewed by a geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended.

5.5 Stormwater Ponding Areas

Based on the preliminary information provided to us, stormwater ponding areas will be constructed at various locations across the site. For the shallow ponding areas, the final side slopes should be sloped at 3 horizontal to 1 vertical, or flatter. Suitable erosion control measures will depend on the anticipated flow velocities, but could be achieved using topsoil and seed (or sod). For higher velocities, consideration could be given to suitably sized rip rap, turf reinforcement mats, vegetation in conjunction with a photodegradable erosion control blanket, or a combination thereof.

5.6 Underground Water Storage Tank

5.6.1 Excavation

It is understood that the development plans include the construction of an underground storage tank. Based on the preliminary information provided to us, the water tank will have an inside depth of 1.9 metres and be constructed partially below ground surface. As such, the excavation for the tank will be carried out through topsoil, fill material, silty sand, and silty clay. Based on the results of the investigation, it is possible that the excavation will extend into the underlying grey silty clay.

The following comments are provided on the excavation of the water tank:

- The sides of 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 overburden 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.
- Based on the measured groundwater levels, and assuming that a perched groundwater table will likely be present within the silty sand above the silty clay deposits, excavation below the groundwater level will likely be required, which could present some constraints. There is potential for disturbance to the silty sand on the sides and bottom of the excavation and relatively flat side slopes may be required (e.g., 3 horizontal to 1 vertical) to prevent sloughing of material into the excavation.
- It is anticipated that groundwater inflow from the overburden deposits into the excavations could be handled from within the excavations. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and utilities.
- Depending on the depth of the excavation, saturated silty sand or grey silty clay may be encountered at subgrade level. These deposits are susceptible to weakening under vibration and/or repeated loading. We recommend that a contingency allowance be made for a 300 millimetre thick subbedding layer of OPSS Granular B Type II granular material and a woven geotextile separator meeting OPSS 1860 Class I requirements in the event that the subgrade soils are disturbed during construction. Where saturated silty sand or grey silty clay are encountered, a 75 to 100 millimetre (minimum) thick layer of low strength concrete, placed over the approved subgrade surface immediately after it is exposed, could be considered to reduce the potential for subgrade disturbance.

5.6.2 Foundation Design

In areas where subexcavation of disturbed material is required below proposed founding level, the following comments are provided:

- Imported granular material meeting OPSS requirements for Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type I, Granular B Type II, or Granular A could be used below the tank. The use of 19 millimetre clear stone could also be considered. Any clear stone should be tightly wrapped in a non-woven geotextile where it is in direct contact with sandy soils.
- The imported granular material should be placed in accordance with the site grade raise restrictions.
- OPSS Granular B Type I, Granular B Type II, and Granular A should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. The clear stone should be nominally compacted in maximum 300 millimetre thick lifts with at least 2 passes of a diesel plate compactor.
- Where subexcavation of greater than 0.5 metres of native soil is required, and where the native soil is replaced with heavier OPSS Granular B Type I, Granular B Type II, and Granular A, the net stress increase on the underlying silty clay should be reviewed by geotechnical personnel to confirm that the recommendations provided in this report are still valid.
- To allow spread of load beneath the footings, the imported granular material should extend horizontally at least 0.3 metres beyond the footings and then down and out from this point at 1 horizontal to 1 vertical, or flatter. The excavations should be sized to accommodate this fill placement.

The tank will be underlain by deposits of silty clay which have a limited capacity to support additional loads. The bearing pressures for the tank foundations will depend on the depth of the excavation and the amount of surcharge (fill) in the vicinity of tank. For preliminary planning and design purposes, the (gross) bearing resistance at SLS may be taken as 40 kilopascals. The factored bearing resistance at ULS may be taken as 100 kilopascals.

5.6.3 Backfill

Our comments on backfill provided in Section 5.3.5.1 apply equally to the backfill for the water tank. To avoid unbalanced loading, backfilling should be carried out concurrently on all sides of the structure.

The proposed structure should be designed to resist the following earth and groundwater pressures, assuming that groundwater level is at ground surface:

$$P_o = K_o (\gamma_s - \gamma_w) (H) + \gamma_w (H)$$

Where,

P_o	= At rest earth pressure at the bottom of the shoring (kilopascals)
K_o	= At rest earth pressure coefficient (0.50)
γ_s	= Unit weight of backfill material (22 kilonewtons per cubic metre)
γ_w	= Unit weight of water (10 kilonewtons per cubic metre)
H	= Height of wall (metres)

The increase in earth pressure acting on the walls during seismic shaking, could be provided, if required, as the design progresses.

5.6.4 Buoyant Uplift of the Proposed Structures

The proposed structure should be designed to resist hydrostatic uplift due to hydrostatic pressures below the base of the structure. The groundwater level should be assumed at ground surface during spring thaw conditions to assess buoyant conditions. To resist hydrostatic uplift, the dead weight of the structure may be sufficient. However, if this not the case, one option that could be considered to reduce the potential for uplift of the structure due to buoyant effects would be to increase the dead weight of the structure. This could be achieved by thickening/oversizing the base of the slab or walls to add additional mass.

Alternatively, a second option to consider would be to extend the base of the structure beyond the walls, so that the uplift loads are resisted by the weight of the backfill material. Table 5.3 provides buoyant unit weights for imported granular materials.

Table 5.3 – Unit Weight for Buoyant Uplift

Material Type	Approximate Compacted Unit Weight (kN/m^3)	Approximate Submerged Unit Weight (kN/m^3)
OPSS Granular A or Granular B Type II	22	12
19 millimetre Clear Crushed Stone	16	6

We suggest that a factor of safety of at least 1.2 be used to resist uplift due to hydrostatic pressure below the base.

5.6.5 Frost Protection Requirements

The structures should be founded at least 1.8 metres below finished grade for frost protection purposes. If the tank will be founded within 1.8 metres of finished grade, the subgrade surface should be protected from frost using extruded polystyrene insulation. Further, if the water tank is left empty, even temporarily, during the winter period, it may be necessary to protect the subgrade surface from frost using a layer of extruded polystyrene insulation, even if the tank is founded below 1.8 metres depth. An insulation detail could be provided if required.

5.7 Flexible Pavement Design

5.7.1 Post-Construction Settlement

As previously indicated, consideration could be given to raising the grade within the gravel and asphalt surfaced areas by up to 1 metre above native soil. For this case, the long-term settlement of the ground surface will be in excess of 25 millimetres, but the settlements could potentially be accommodated by on-going site maintenance (e.g., padding, crack sealing, etc.). Grading within 2.4 metres of building foundations, concrete slabs, or other areas where settlements are required to be less than 25 millimetres, should be carried out in accordance with the grade raise restrictions provided in Table 5.1.

5.7.2 Subgrade Preparation

In preparation for the construction of roadways and parking areas at this site, all surficial topsoil, and any loose/soft, wet, organic or deleterious materials should be removed from the proposed subgrade surface. This need not include removal of the existing fill material.

Prior to placing granular fill the exposed subgrade should be proof rolled with a large (minimum 10 tonne) vibratory steel drum roller under dry conditions and inspected and approved by geotechnical personnel. Any soft areas that are evident from the proof rolling should be sub-excavated and replaced with suitable earth borrow that is frost compatible with the excavated material. If wet subgrade conditions exist, the proof rolling should be omitted as this would likely result in disturbance to the subgrade.

The grade below the new roadway could be raised using a material which meets OPSS specifications for Granular B Type I or II, Select Subgrade Material (SSM), or suitable earth borrow. Abrupt changes in the frost susceptibility of the grade raise fill material should be avoided. If unavoidable, frost tapers may be required to prevent localized differential frost heaving of the pavement structure. The grade raise fill 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. It is noted that most SSM and earth borrow materials are sensitive to changes in moisture content, precipitation and frost heaving. As such, unless the earth material placement is planned during the dry period of the year (June to September), precipitation and freezing conditions may restrict or delay adequate compaction of these materials.

5.7.3 Design Sections

5.7.3.1 Asphaltic Concrete Surfacing

It is suggested that the following minimum pavement structure be used for light duty parking areas:

- 50 millimetres of Superpave 12.5 Traffic Level B incorporating PG 58-34 asphalt cement,
- 150 millimetres of OPSS Granular A, over
- 300 millimetres of OPSS Granular B Type II (100 millimetre minus crushed stone), over
- Geotextile separation (OPSS 1860 Class II Woven Geotextile)

For any areas which will be used by heavy trucks or fire trucks, the following pavement structure is suggested:

- 90 millimetres of Superpave 12.5 Traffic Level D incorporating PG 64-34 asphalt cement, placed in two (2) 45-millimetre-thick layers, over
- 150 millimetres of OPSS Granular A, over
- 450 millimetres of OPSS Granular B Type II, over
- Geotextile separation (OPSS 1860 Class II Woven Geotextile)

Our recommendation for Traffic Level D incorporating PG 64-34 asphalt cement for the heavy duty pavement areas could be reviewed as the design progresses. Depending on the traffic volumes, it may be possible to use Traffic Level B incorporating PG 58-34 asphalt cement in the heavy duty pavement areas.

5.7.3.2 Gravel Surfacing

For gravel surfaced areas which will be used by heavy trucks or fire trucks, the following pavement structure is suggested:

- 150 millimetres of OPSS Granular M or OPSS Granular A, over
- 450 millimetres of OPSS Granular B Type II, over
- Geotextile separation (OPSS 1860 Class II Woven Geotextile)

From a geotechnical perspective, OPSS Granular M base material is preferred. Depending on the traffic volumes, it may be possible to reduce the subbase thickness provided above.

5.7.4 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.

5.7.5 Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. Perimeter ditching around the paved areas is suggested and the granular base and subbase materials for the paved areas should extend horizontally to the ditch areas.

5.7.6 Effects of Soil Disturbance and Construction Traffic

The above pavement structure assumes that the roadway subgrade surface is prepared as described in this report. If the roadway subgrade surface becomes disturbed or wetted due to construction operations or precipitation, the Granular B Type II 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 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 B Type II to prevent pumping and disturbance to the subbase material. The contractor should be made responsible for their construction access.

5.7.7 Pavement Transitions

Where the new pavement will abut 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.

5.8 Rigid Pavement Design

It is understood that a rigid (i.e., concrete) pavement solution is being considered around the perimeter of the proposed building. Based on our experience, rigid pavements are generally less tolerable to settlement (relative to asphaltic concrete and gravel surfacing). As such, we recommend that the grading below, and within a zone extending at least 2.4 metres beyond the edge of the rigid pavement, be carried out in accordance with the grade raise restrictions provided in Table 5.1. Where the grade raise restriction cannot be achieved, consideration could be given to the use of EPS blocks to make up the additional depth of grade raise. The types of EPS material for the use below the rigid pavement, including general specifications and installation guidelines, could be provided upon request as the design progresses.

The following comments are provided for the design and construction of the rigid pavement:

- In preparation for the construction of rigid pavement, the subgrade surface should be prepared as described in Section 5.7.2.

- We recommend that the Portland Cement Concrete (PCC) have a thickness of at least 200 millimetres. CSA Exposure Class C-2 with a 28-day compressive strength of 32 MPa is also recommended.
- The PCC should be placed on at least 150 millimetres of OPSS Granular A. The granular base materials should be compacted in maximum 300 millimetre thick lifts to at least 98 percent of the standard Proctor maximum dry density value.
- Installation of the PCC pavement should follow OPSD 560.010 and 552.010. The requirement for dowelling could be assessed as the design progresses.
- Thermal protection of the rigid pavement is recommended and could be achieved using sheets of extruded polystyrene insulation. The type of insulation for use below the slab on grade could be provided as required. It should be noted that EPS provides adequate frost protection of the underlying subgrade soils if it is used below the rigid pavement.
- It is noted that EPS is potentially soluble in hydrocarbons. Where EPS is used below the rigid pavement, it is general practice to cover the outside surface of the EPS with polyethylene sheeting to protect the EPS from accidental release and infiltration of fuel.

6.0 ADDITIONAL CONSIDERATIONS

6.1 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 that required to cause damage to the nearby structures or services.

6.2 Monitoring Well Abandonment

All monitoring wells installed as part of this investigation should be decommissioned by a licensed well technician. The well abandonment could be carried out in advance of or during construction.

6.3 Disposal of Excess Soil

It is noted that 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, including naturally occurring sources of contamination, are outside the terms of reference for this report.

6.4 Design Review and Construction Observations

It is recommended that the final 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.

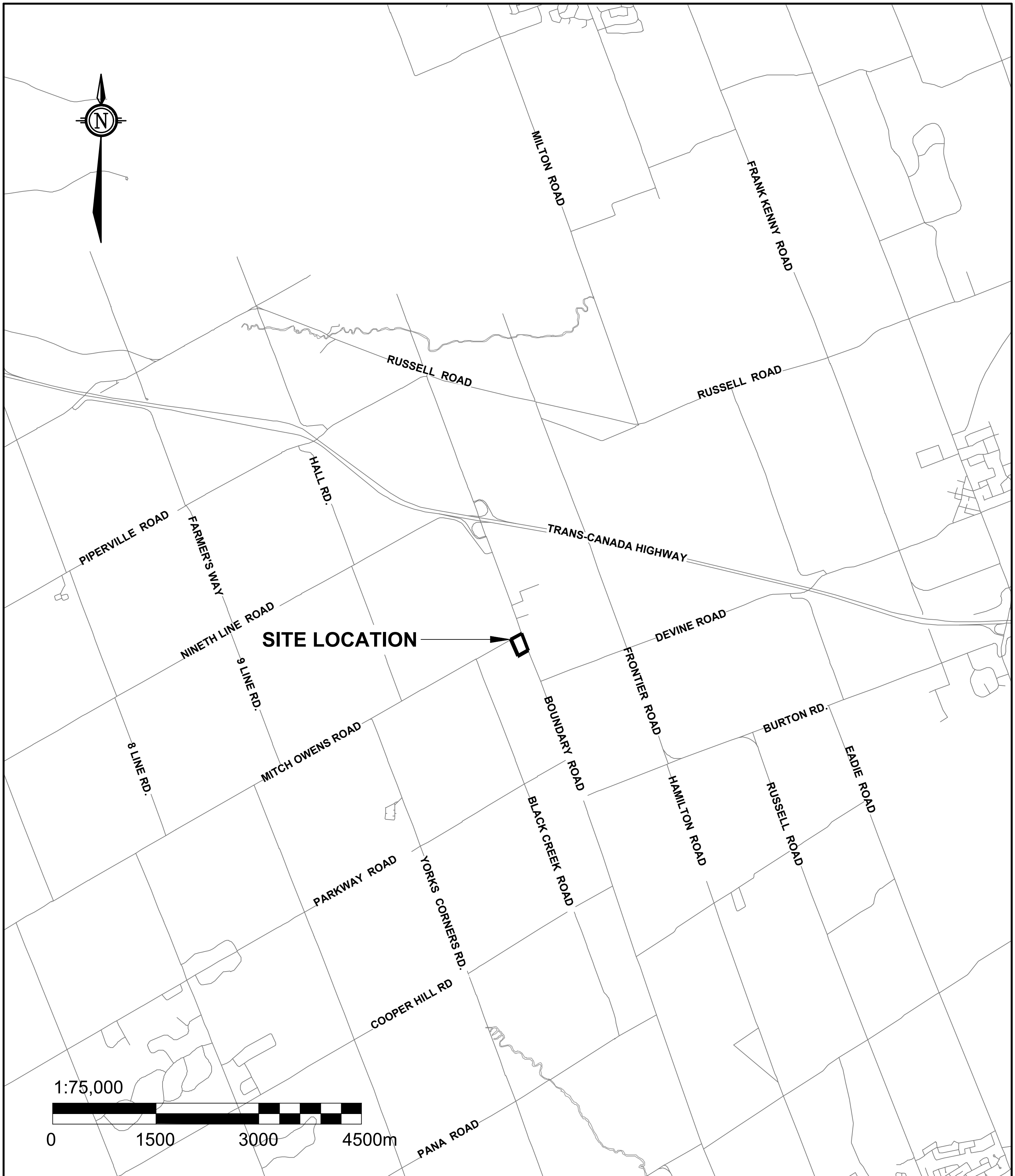
The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent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

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.

LICENSED PROFESSIONAL ENGINEER
J. A. CHOLEWA
100119055
18 Dec 2018
PROVINCE OF ONTARIO

Johnathan A. Cholewa, Ph.D., P.Eng.

Craig Houle, M.Eng., P.Eng.,
Principal



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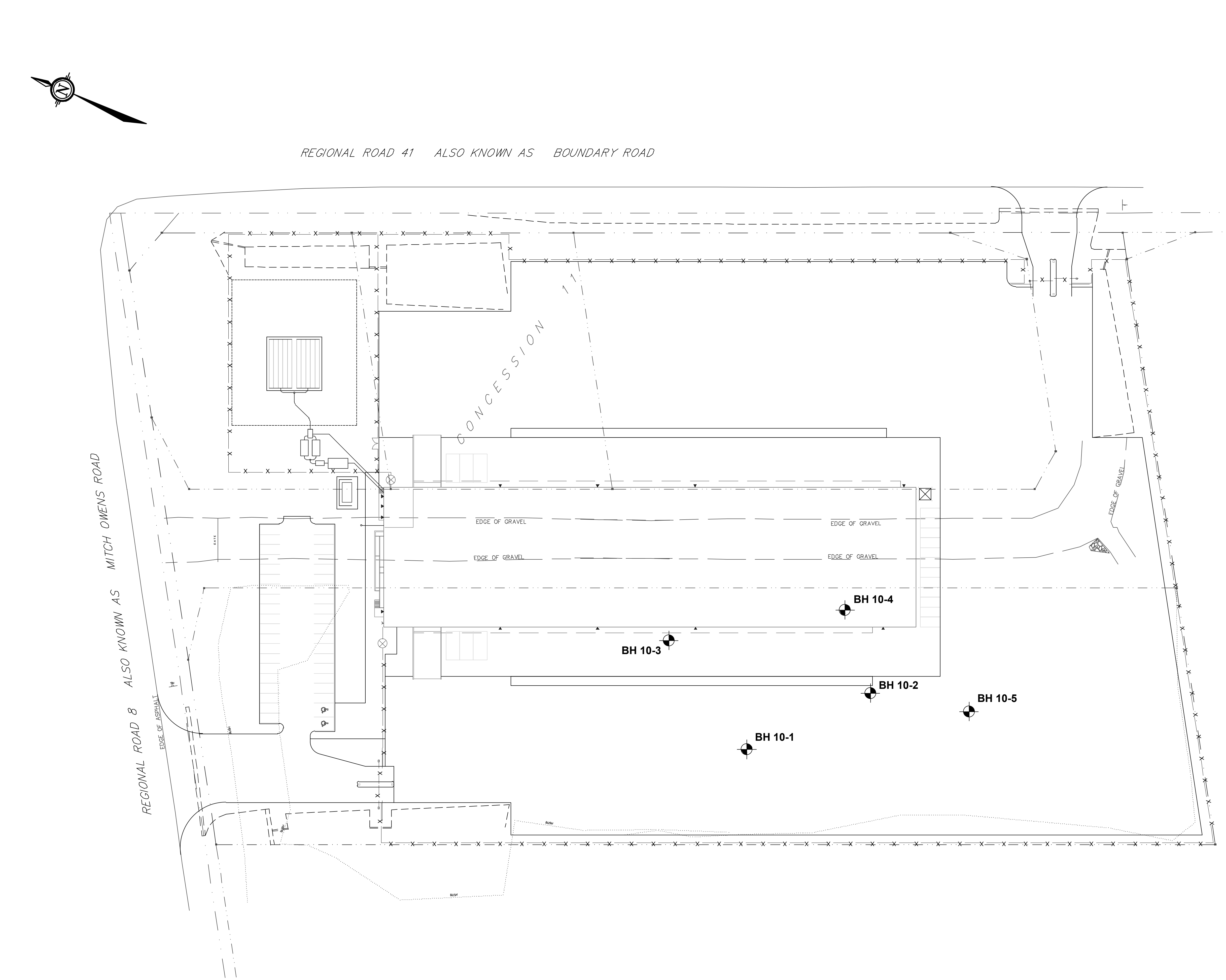
Project				Drawing	
GEOTECHNICAL INVESTIGATION PROPOSED WAREHOUSE BUILDING 9460 MITCH OWENS RD., 5606, 5630, 5592 BOUNDARY RD. OTTAWA ONTARIO				KEY PLAN	
Drwn By	Chkd By	Date	Project No.	Revision No.	
P.C.	J.C.	DECEMBER 2018	60369.15	0	**FIGURE 1**

P:\0. Files\60300\60369.15\Drafting\Drawings\60369.15_FG1_V01_2018-12-13.dwg, FIGURE 1, 18/12/13 2:13:46 PM



LEGEND

BH 10-1 **BOREHOLE LOCATION IN PLAN**
(current investigation by Houle Chevrier Engineering Ltd., 2010)

Scale 1:1000

0 20 40 60m

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Client: TOUCHSTONE CONTRACTING AND ENGINEERING LTD.	Project: 60369.15
Location: 9460 MITCH OWENS RD, 5606, 5630, 5592 BOUNDARY RD. OTTAWA, ON	
Drwn by: P.C. / Chkd by: J.C.	BOREHOLE LOCATION PLAN
Date: DECEMBER 2018 / Rev.: 0	**FIGURE 2**

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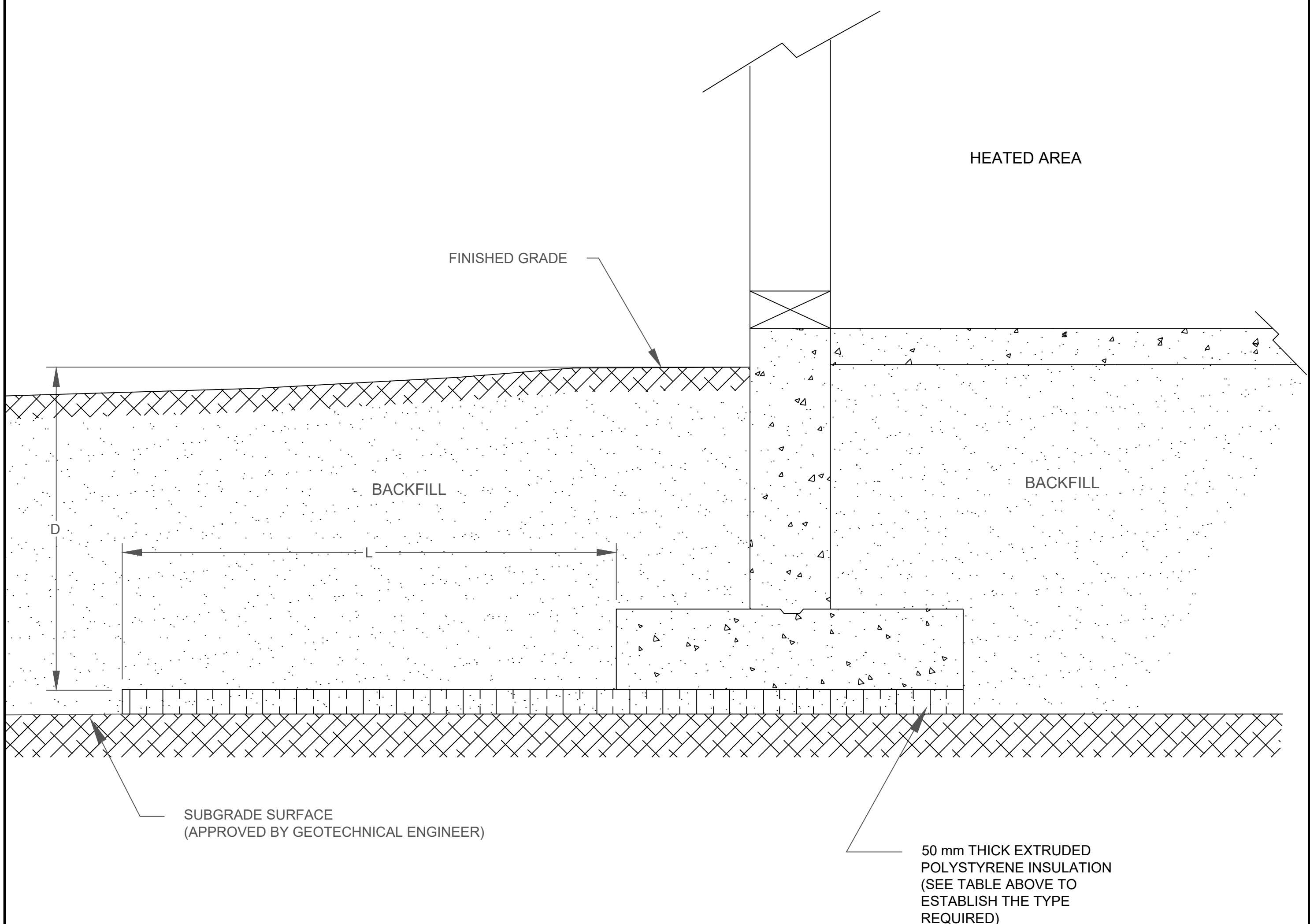
LEGEND

D — DEPTH OF EARTH COVER ABOVE BOTTOM OF FOOTING, 0.3 m (min.)

L — REQUIRED PROJECTION OF INSULATION IN AREAS WITH LESS THAN 1.5m OF EARTH COVER.

APPLIED BEARING PRESSURE	TYPE OF INSULATION
<70 kPa	DOW SM
<100 kPa	DOW HI 40
<150 kPa	DOW HI 60



NOT TO SCALE

NOTES

1) INSULATION JOINTS TO BE TIGHTLY BUTT JOINED OR SHIP LAPPED.

2) FOR ADEQUATE FROST PROTECTION, D + L $\geq$ 1.5 metres.

3) THE SURFACE BENEATH THE INSULATION SHOULD BE FLAT TO ENSURE THAT SPLITTING OR BREAKAGE OF THE SHEETS DOES NOT OCCUR.

4) ALL WORK SHOULD BE INSPECTED BY THE ENGINEER.

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Project: GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE BUILDING
9460 MITCH OWENS RD., 5606, 5630, 5592 BOUNDARY RD.
OTTAWA ONTARIO

Drawing: FOOTING INSULATION DETAIL

Drwn By	Chkd By	Date	Project No.	Revision No.	
P.C.	J.C.	DECEMBER 2018	60369.15	0	**FIGURE 3**

APPENDIX A

Lists of Descriptive Terms
Record of Borehole Sheets
Figures A1 to A3

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

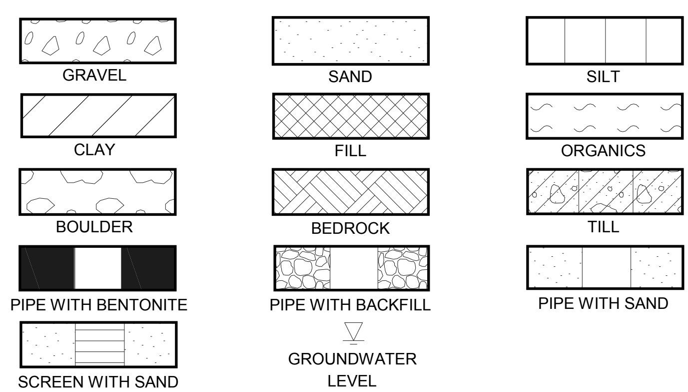
PENETRATION RESISTANCE

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.

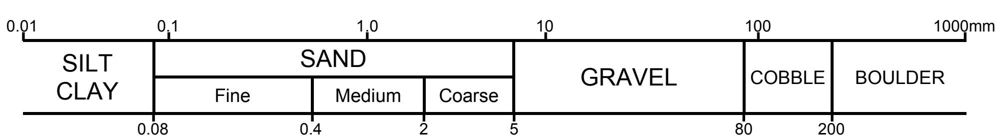
Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).

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

PROJECT: 10-203

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: June 9, 2010

RECORD OF BOREHOLE 10-1

SHEET 1 OF 1

DATUM: Local

SPT HAMMER: 63.5 kg; drop 0.76 m

DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE: DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m) | SAMPLES: NUMBER, TYPE, BLOWS/0.3m | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat. V - +, Q - ●, rem. V - ⊕, U - ○) (20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-7}, 10^{-6}, 10^{-5}, 10^{-4}); WATER CONTENT, PERCENT (Wp, W, Wl) (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: Power Auger, 200 mm Diameter Hollow Stem

Description	Elev. / Depth (m)
Ground Surface	99.88
Brown SILTY SAND	
Stiff, reddish brown SILTY CLAY	99.43 / 0.45
Soft, grey SILTY CLAY - occasional grey silty sand seams throughout	98.05 / 1.83
End of borehole	93.17 / 6.71

Number	Type	Blows/0.3m	Additional Lab. Testing
1	50 D.O.	6	
2	50 D.O.	2	
3	T.P.	PH	C, MH
4	T.P.	PH	

Piezometer or standpipe installation:

Bentonite Seal

2" Diameter slotted well screen,

Groundwater level is 0.62 metres below orginal ground surface. Sampled on June 17, 2010.

DEPTH SCALE 1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED: [signature]

RECORD OF BOREHOLE 10-2

PROJECT: 10-203

LOCATION: See Borehole Location Plan, Figure 2

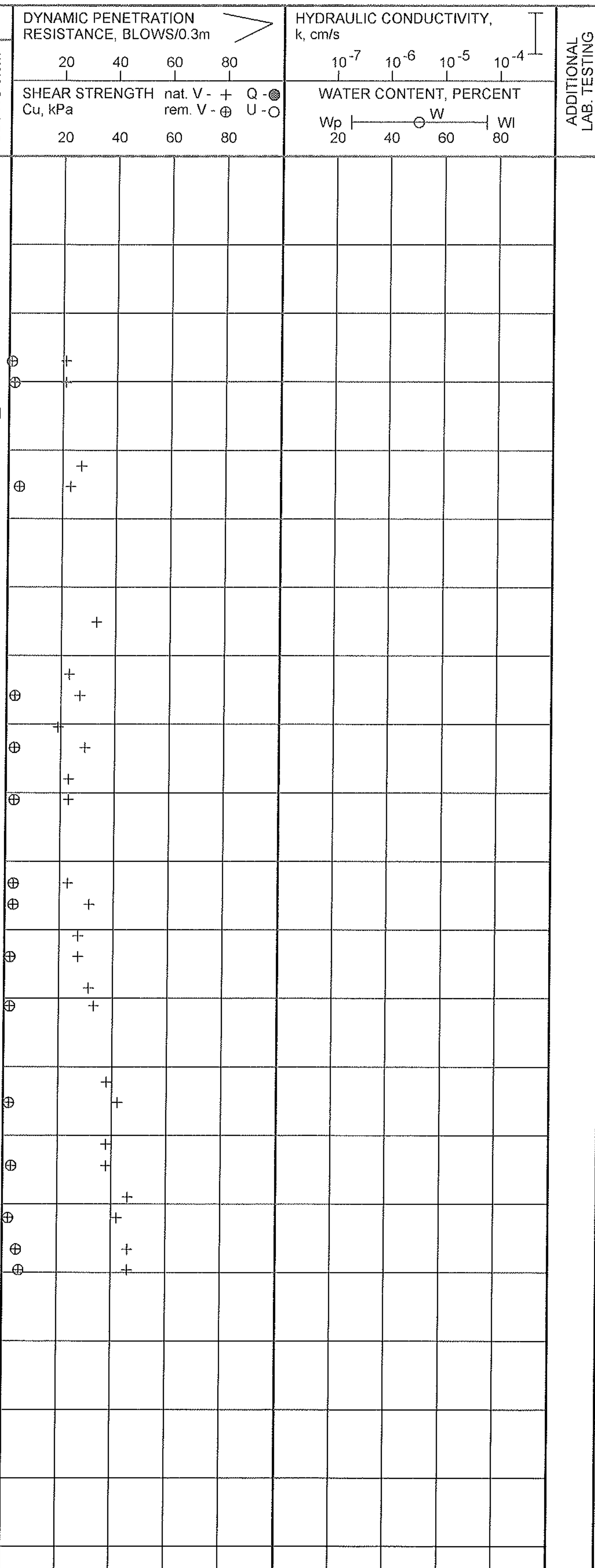
BORING DATE: June 9, 2010

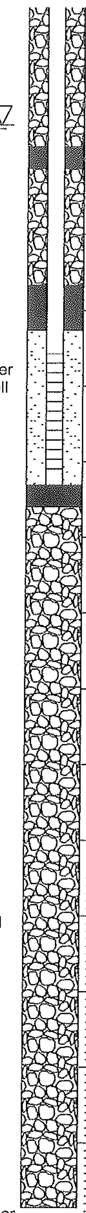
SHEET 1 OF 1

DATUM: Local

SPT HAMMER: 63.5 kg; drop 0.76 m

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	99.89				
		Brown SILTY SAND	99.48 / 0.41				
1		Stiff, reddish brown SILTY CLAY		1	50 D.O.	5	
2		Soft to firm, Grey SILTY CLAY	98.06 / 1.83	2	50 D.O.	2	
3		- occasional silty sand seams throughout		3	T.P.	PH	
4							Bentonite seal
5				4	50 D.O.	1	2" Diameter slotted well screen
6				5	50 D.O.	1	
				6	T.P.	PH	
9				7	T.P.	PH	
12		Firm, dark grey SILTY CLAY	87.85 / 12.04	8	50 D.O.	WH	Native Soil
16		End of borehole	84.04 / 15.85				Groundwater level is 1.55 metres below orginal ground surface. Sampled on June 17, 2010.





DEPTH SCALE

1 to 100

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED: JC

RECORD OF BOREHOLE 10-3

PROJECT: 10-203
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: June 10, 2010

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

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-7} 10^{-6} 10^{-5} 10^{-4}) / WATER CONTENT, PERCENT Wp — W — Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger, 200 mm Diameter Hollow Stem	Ground Surface		100.16							
		Brown SILTY SAND									
1		Stiff, reddish brown SILTY CLAY		99.09 1.07	1	50 D.O.	9				
2		Soft to firm, grey SILTY CLAY, trace silt seams		98.27 1.89	2	50 D.O.	3				
					3	50 D.O.	WH				
3		End of borehole		97.26 2.90							Groundwater conditions not observed
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE
1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.
CHECKED: JE

RECORD OF BOREHOLE 10-4

PROJECT: 10-203
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: June 10, 2010

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

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-7} 10^{-6} 10^{-5} 10^{-4}); WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface		100.04							
		Brown SILTY SAND									
1		Stiff, reddish brown SILTY CLAY, trace silt seams		98.97 1.07	1	50 D.O.	6				
		Soft to firm, Grey SILTY CLAY, trace silt seams		98.36 1.68	2	50 D.O.	2				
2		End of borehole		97.91 2.13							Groundwater conditions not observed
3											
4											
5											
6											
7											
8											
9											
10											

BOREHOLE_RECORD GINT LOGS 10-203.GPJ HCE DATA TEMPLATE.GDT 1/25/11

DEPTH SCALE
1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.
CHECKED: hc

RECORD OF BOREHOLE 10-5

PROJECT: 10-203

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: June 10, 2010

SHEET 1 OF 1

DATUM: Local

SPT HAMMER: 63.5 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-7} 10^{-6} 10^{-5} 10^{-4}); WATER CONTENT, PERCENT Wp —○W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger, 200 mm Diameter Hollow Stem	Ground Surface		100.87							
		Loose, dark brown silty sand some gravel, trace clay [Fill Material]									
1					1	50 D.O.	4				
		Loose, brown SILTY SAND		99.45 / 1.42							
2		Stiff, reddish brown SILTY CLAY		99.04 / 1.83	2	50 D.O.	6				
		Soft to firm, grey SILTY CLAY, trace sandy silt seams		98.43 / 2.44	3	50 D.O.	1				
3		End of borehole		97.97 / 2.90							Groundwater conditions not observed
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

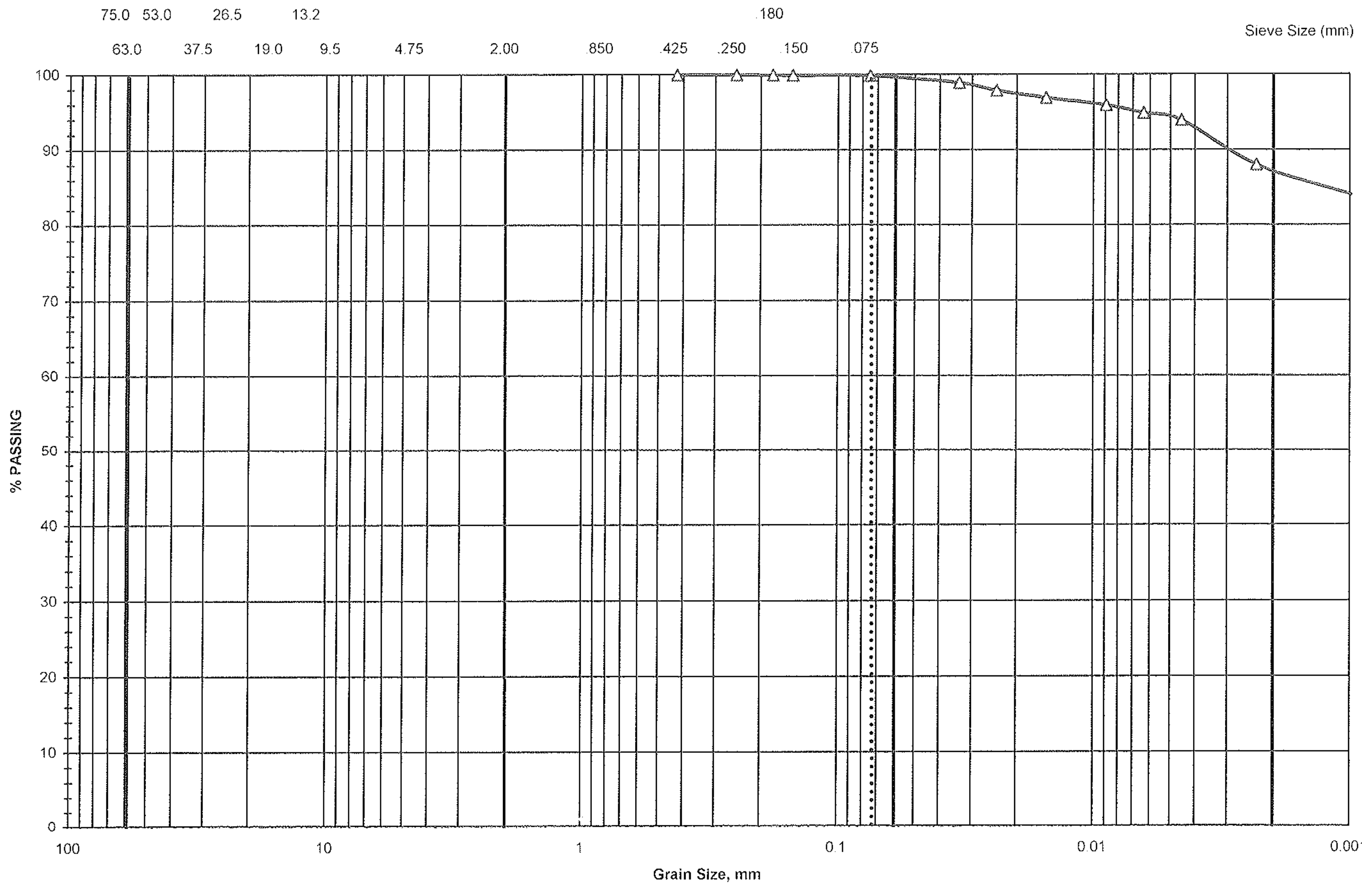
1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED: JC

GRAIN SIZE ANALYSIS	FIGURE A1



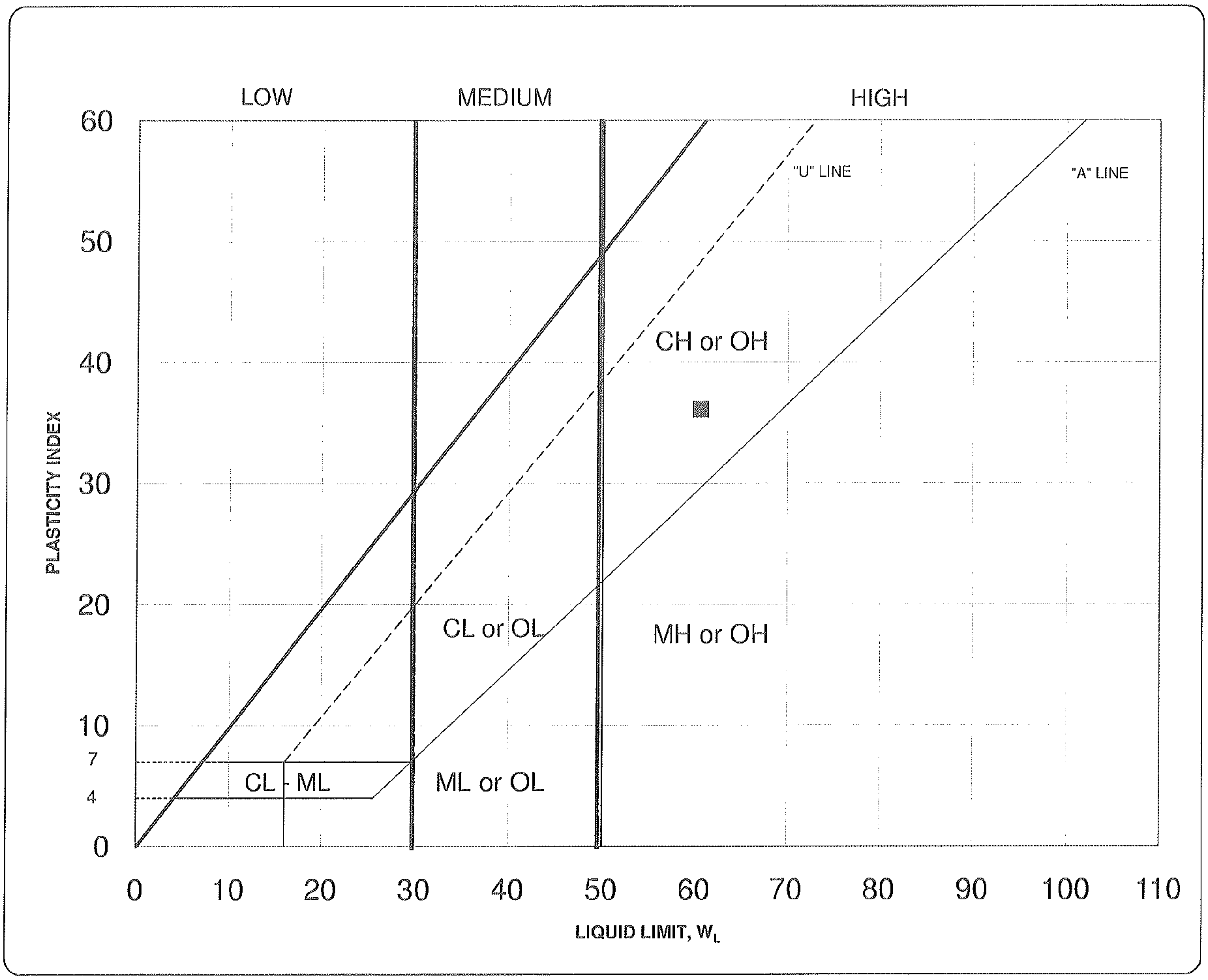
	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	CLAY
	GRAVEL			SAND			SILT			
Modified M.I.T. Classification										

Bore Hole	Sample	Depth (m)	Legend
10-1	2	4.57 - 5.18	△

Houle Chevrier Engineering

Date: February 2011
Project : 10-203

ATTERBERG LIMITS	FIGURE A2

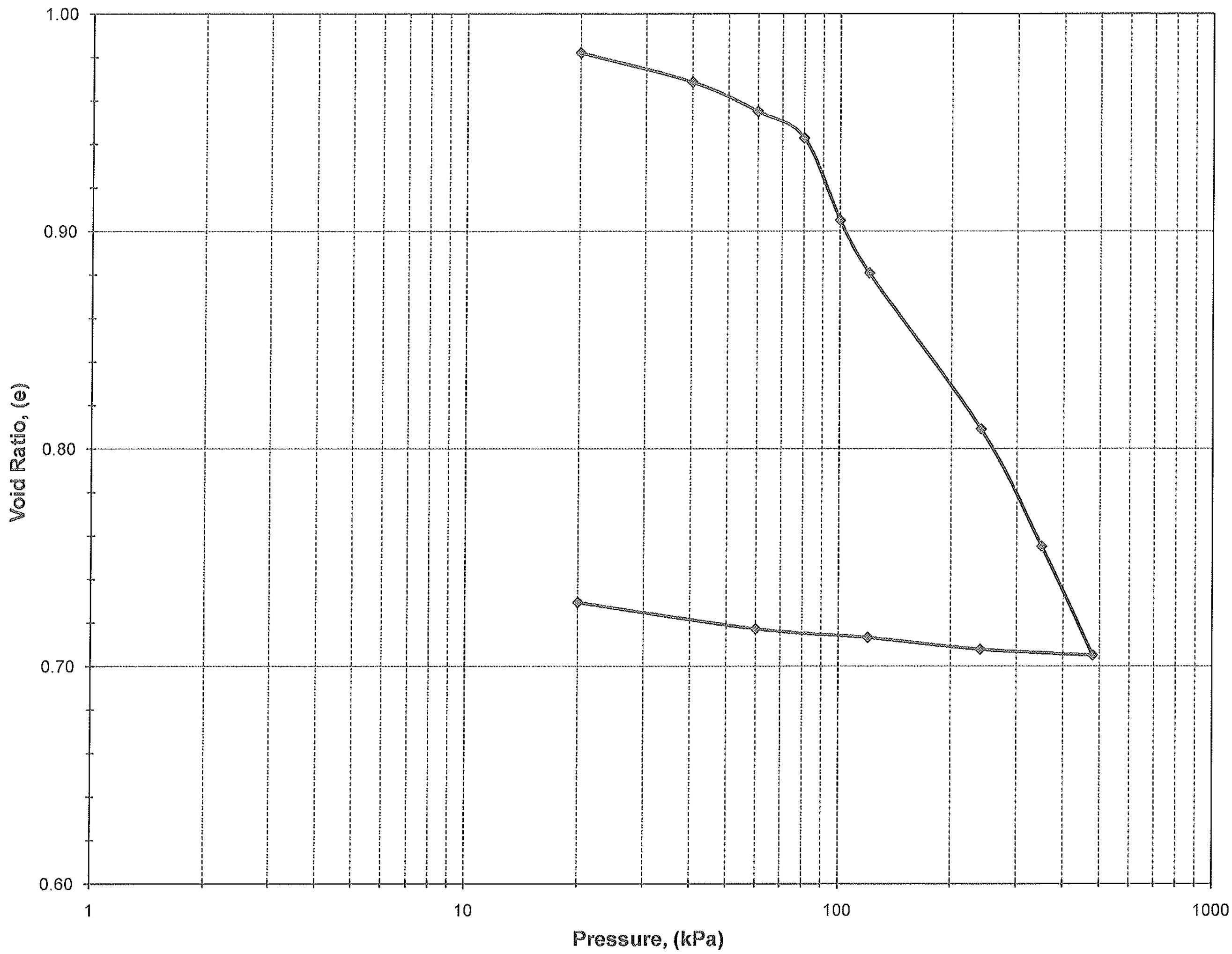


Bore Hole	Sample	Depth (m)	Legend	Moisture Content, %
10-1	3	4.57 - 5.18	■	92.4

Houle Chevrier Engineering

Date: February 2011
Project: 10-203

CONSOLIDATION TEST RESULTS | FIGURE A3



Borehole	Sample	Depth (m)
BH 10-1	3	4.57 - 5.18

Determined Properties:

W	92
W_L	61
W_p	25

Test Results:

C_r	0.04	
C_c	0.28	
σ'_p	70	kPa

Houle Chevrier Engineering

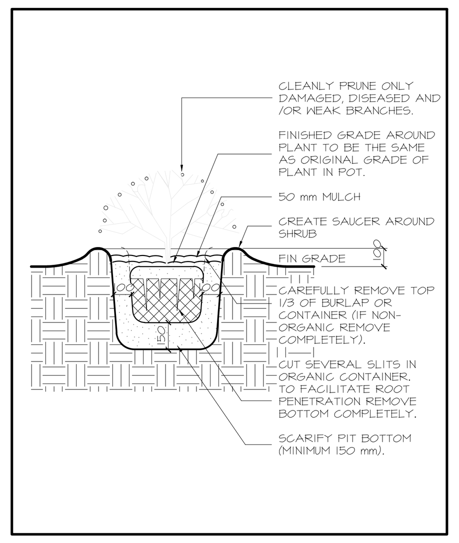
Date: February 2011
Project: 10-203

APPENDIX B

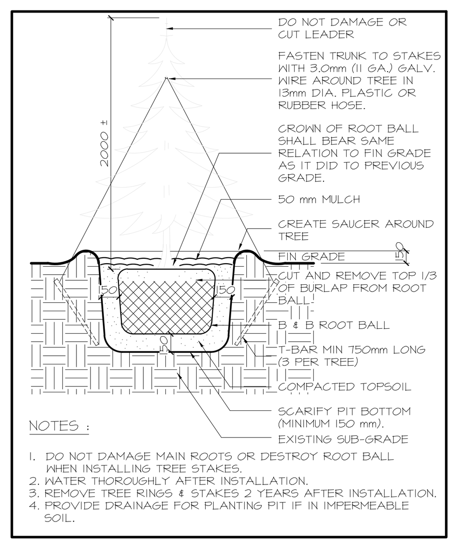
Current Site Plan

KEY	NAME	SIZE HT/SPR	COND	QUAN
AS	ACER SACCHARUM SUGAR MAPLE	75 mm cal	W/B	
JS	JUNIPERUS SABINA SAVIN JUNIPER	500 mm spread	POT	
GT	GLEDITSIA TRIACANTHOS SHADEMASTER HONEYLOCUST	70 mm cal	W/B	
PP	PICEA PUNGENS GREEN COLORADO SPRUCE	2000 mm high	W/B	
PS	PINUS SYLVESTRIS SCOTS PINE	2000 mm high	W/B	
PST	PINUS STROBUS WHITE PINE	2000 mm high	W/B	
SR	SYRINGA RETICULATA JAPANESE TREE LILAC	50 mm cal	W/B	

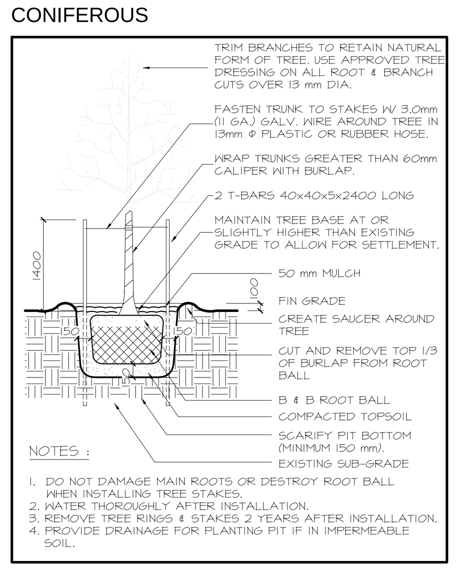
PLANTING LIST



SHRUB

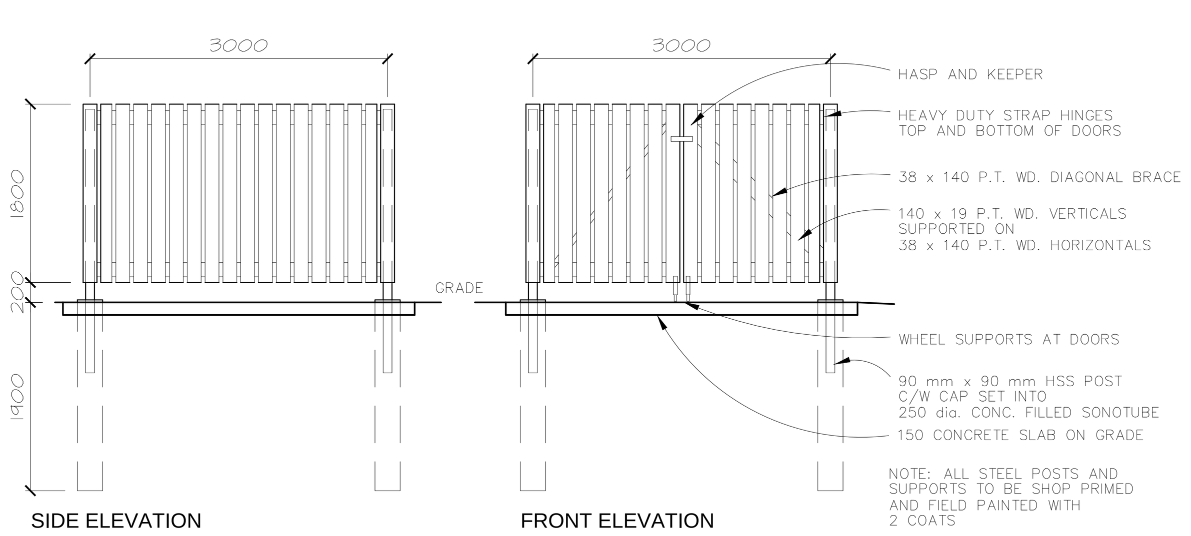


CONIFEROUS

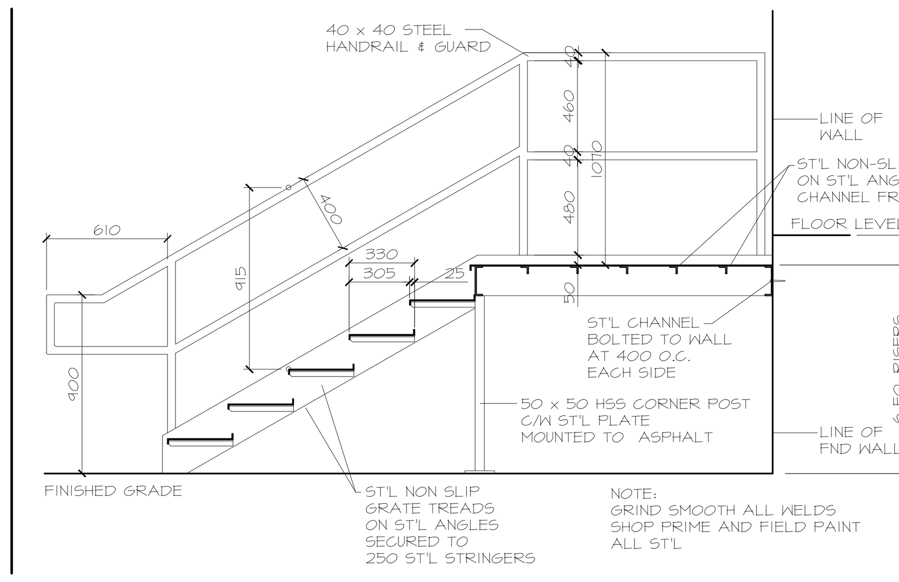


DECIDUOUS

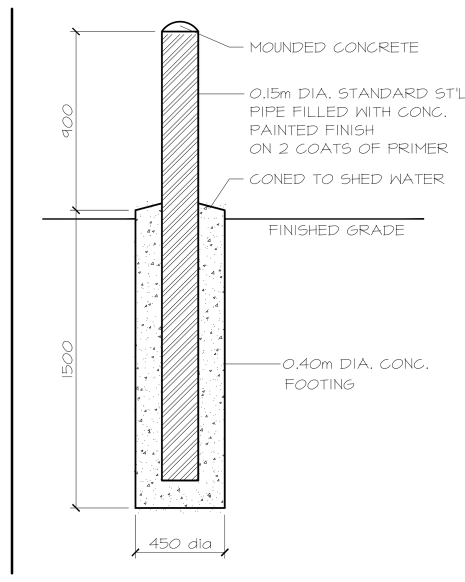
A | PLANTING DETAILS
A1.1 | 1 : 50



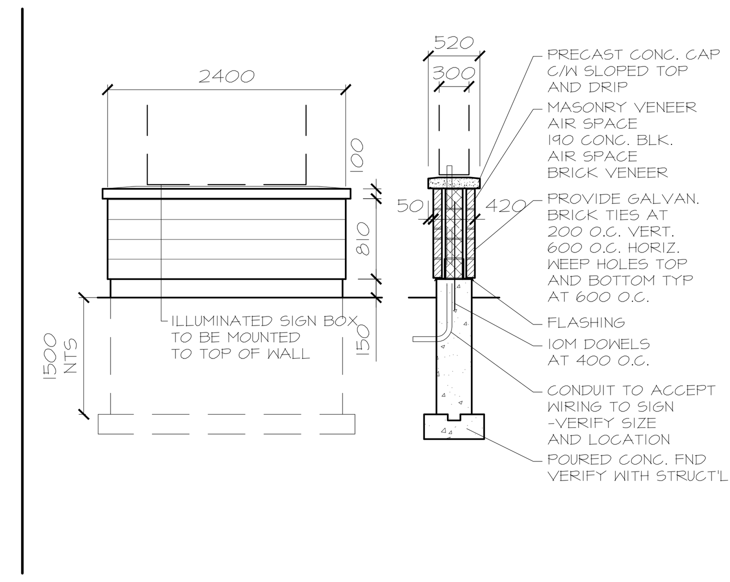
B | GARBAGE ENCLOSURE
A1.1 | 1 : 50



C | EXTERIOR STEEL STAIR
A1.1 | 1 : 25

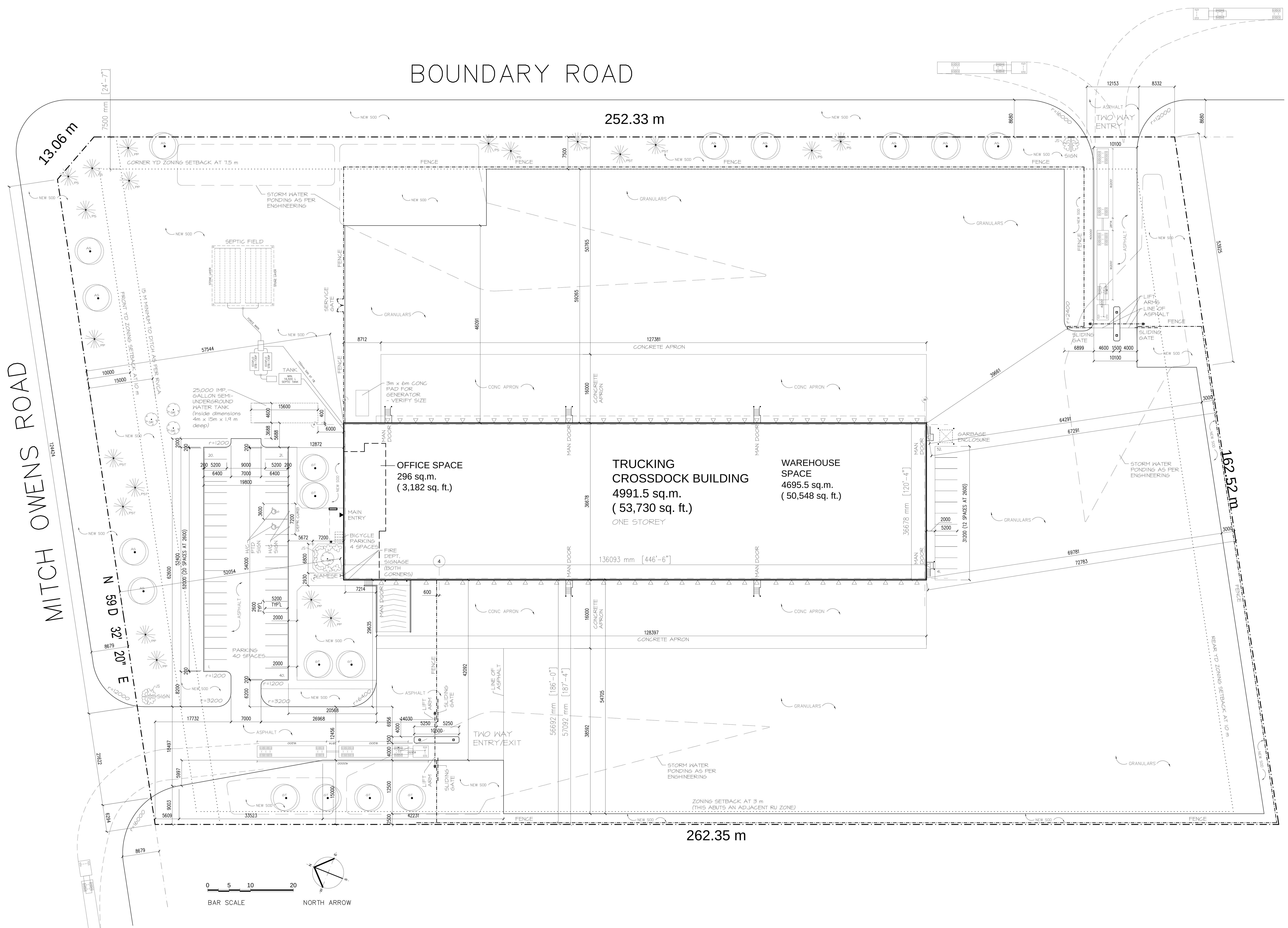


D | EXTERIOR STEEL BOLLARD
A1.1 | 1 : 25



E | SITE ENTRANCE SIGNAGE (x2)
A1.1 | 1 : 50

j. guy monette
architecture and urban design
308a fairmont ave. ottawa, on, K1Y 1Y8
613.859.6483 guymonette@rogers.com



1 | SITE PLAN
A1.1 | 1 : 400 metric

BUILDING STATISTICS:

BLDG FOOTPRINT = 4991.5 SQ. M. 11.8 % COVERAGE

BLDG AREA =	OFFICE AREA	GROUND FLR	296 SQ. M.
	WAREHOUSE AREA	GROUND FLR	4695.5 SQ. M.
	TOTAL GROSS FLOOR AREA		4991.5 SQ. M.
BLDG HEIGHT =	+/- 9.83 M		

SITE STATISTICS:

CURRENT ZONING - RG [784r]

PROPOSED USE: TRUCK TRANSPORT TERMINAL WAREHOUSE

LOT AREA = 42,066 SQ. M. (10.39 acres)

SETBACKS
- front yd 15m
- corner side yd 12m
- interior yd 8m
- (this zone abuts an RU zone)
- rear yd 15m
- max bldg hteight 15m
- max lot coverage 50%

AREA OF:

ASPHALT PAVEMENT	3,181	SQ.M.
CONCRETE APRONS, PADS	4,116	SQ.M.
GRANULARS	18,392	SQ.M.
SIDEWALKS	306	SQ.M.
SOD, PLANTERS	11,079	SQ.M.

PARKING (AREA 'D' ON SCHEDULE 1A)

TYPICAL PARKING STALL TO BE 2.6 m WIDE x 5.2 m IN LENGTH

TYP. HANDICAPPED PARKING STALL TO BE 3.6 m WIDE x 5.2 m IN LENGTH

OFFICE SPACE	2.4 PER 100 SQ. M. G.F.A. 7 SPACES REQUIRED
WAREHOUSE SPACE	.8 PER 100 SQ. M. G.F.A. 38 SPACES REQUIRED

TOTAL 45 SPACES REQUIRED AND 52 SPACES ARE PROVIDED INCLUDING 2 H/C SPACES

BICYCLE PARKING 4 SPACES PROVIDED

TYPICAL BICYCLE PARKING STALL TO BE 0.6 m WIDE x 1.8 m IN LENGTH

LOADING SPACE (AREA 'D' ON SCHEDULE 1A)

UNDER 500 SQ. M, 1 SPACE REQUIRED AND 0 OVERSIZED SPACES ARE REQUIRED
LOADING SPACE TO BE 3.5 x 7 m

REVISION	No	DATE	PAR
Issued for Site Plan Control	0	17Dec2018	bjs

SEAL

PROJET

NEW WAREHOUSE TRUCKING FACILITY

9460 Mitch Owens;
5606,5630,5592 Boundary Road
Ottawa, ON

TITRE DU DESSIN

SITE PLAN

D07-14-18-0006

DESIGNED BY	CHECKED BY
Brian Sindall	GUY MONETTE
DRAWN BY	SCALE
STAFF	As Noted
PROJECT No.	DRAWING No.
2018-00	A1.1

experience • knowledge • integrity

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

expérience • connaissance • intégrité