Geotechnical Investigation

Pearl Investments Ltd.

Project Name:
Proposed Residential Development
32 Chesterfield Avenue,
London, Ontario

Project Number:
LON-21013388-A0

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1. Introduction and Background

1.1 Introduction

EXP Services Inc. (EXP) was retained by **Pearl Investments Ltd.** to carry out a geotechnical investigation and slope stability assessment and prepare a geotechnical report relating to the proposed development at 32 Chesterfield Avenue, in London, Ontario, hereinafter referred to as the 'Site'. It is understood that the existing lot is to be subdivided into seven (7) single-family residences with associated paved access and parking and services is proposed at the Site.

The Site encroaches on regulated Lands of Upper Thames River Conservation Authority (UTRCA) and will require approval from the conservation authority.

Based on an interpretation of the factual test hole data and a review of soil and groundwater information from test holes advanced at the site, EXP has provided geotechnical engineering guidelines to support the proposed Site development. A slope assessment for the northern sides of the Site were also conducted to determine the erosion hazard limits (development setback).

1.2 Terms of Reference

The geotechnical investigation was generally completed in accordance with the scope of work outlined through EXP's proposal P20-128 dated April 28, 2021. Authorization to proceed with this investigation was received from Subramanian Suppiah of **Pearl Investments Ltd.** through email communications dated August 5, 2021.

The purpose of the investigation was to examine the subsoil and groundwater conditions at the site by advancing a series of boreholes at the locations chosen by EXP and illustrated on the attached Borehole Location Plan (**Drawing 1**).

Based on an interpretation of the factual borehole data, and a review of soil and groundwater information from test holes advanced at the site, EXP Services Inc. has provided engineering guidelines for the geotechnical design and construction of the proposed development. More specifically, this report provides comments on site preparation, excavations, dewatering, foundations, slab-on-grade, basement construction, bedding and backfill, earthquake design considerations, pavement recommendations, curbs and sidewalks, and erosion hazard limits.

This report is provided on the basis of the terms of reference presented above, and on the assumption that the design will be in accordance with applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning geotechnical aspects of the codes and standards, this office should be contacted to review the design.

The information in this report in no way reflects on the environmental aspects of the soil. Should specific information in this regard be needed, additional testing may be required.

Reference is made to **Appendix F** of this report, which contains further information necessary for the proper interpretation and use of this report.

2. Methodology

2.1 Field Work

The drilling program was carried out on September 2nd, 2021. In general, the geotechnical investigation consisted of the advancement of three (3) boreholes at the locations denoted on **Drawing 1** as BH1 to BH3, inclusive.

Prior to the drilling, buried service clearances were obtained for the test hole locations by EXP.

The boreholes were completed by a specialist drilling subcontractor under the full-time supervision of EXP geotechnical staff. The boreholes were advanced utilizing a track-mounted drill rig equipped with continuous flight solid and hollow stem augers, soil sampling and soil testing equipment. In each borehole, disturbed soil samples were recovered at depth intervals of 0.75 m to 1.5 m using conventional split spoon sampling equipment and Standard Penetration Test (SPT) methods or auger samples.

During the drilling, the stratigraphy in the boreholes was examined and logged in the field by EXP geotechnical personnel.

Short-term groundwater levels within the open boreholes were observed. These observations pertaining to groundwater conditions at the test hole locations are recorded in the borehole logs found in **Appendix A**. Following the drilling, the boreholes were backfilled with the excavated materials and bentonite, to satisfy the requirements of O.Reg. 903.

Representative samples of the various soil strata encountered at the test locations were taken to our laboratory in London for further examination by a Geotechnical Engineer and laboratory classification testing. Laboratory testing for this investigation comprised routine moisture content determinations, with results presented on the borehole logs found in **Appendix A**.

Samples remaining after the classification testing will be stored for a period of three months following the issuance of report. After this time, they will be discarded unless prior arrangements have been made for longer storage.

Borehole locations were determined in the field based on the development plan provided by Philip Agar Architect Inc. The borehole elevations were surveyed to the finished floor of the existing house (elevation 247.853 m) by EXP personnel.

In addition to the drilling, Site reconnaissance was completed on August 13, 2021 to review the slopes located on the north side of the Site. Select slope profiles identified on Site were reviewed using the 'Slope Stability Rating Chart' (created by MNR), which summarizes the site observations and empirically scores various elements of the slope profile which contribute to slope stability, to provide an assessment of the potential for slope instabilities at the Site. Two rating charts were completed at three locations on the Site. Cross Section A-A scored 31 while Cross Section B-B' scored 35, indicating slight potential for instability. The rating chart for the cross section examined is provided in **Appendix D** for review and consideration.

2.2 Review of Topographic Data

Ontario Digital Terrain (DTM) lidar-derived topographic data (2017) was utilized by EXP to create cross sections of the slope at select locations. A survey was carried out at the Site by EXP personnel in May 2024 to delineate the top and toe of the existing slope. The Slope cross sections and topographic mapping used was reviewed and used in the slope assessment to determine the stable slope setback and development setback from the top of the existing slope. Using engineering judgement and technical experience, various cross sections (which are considered to be representative of typical site conditions) have been reviewed.

Examination of factors of safety using Morgenstern Price methods were carried out and analysed by computer methods utilizing the Slope/W computer program. Topographic information used for the slope sections is taken from the Ontario Digital Terrain (DTM) lidar-derived data (2017). Soil strength parameters used in the analyses were based on our observations and experience with similar soil and groundwater conditions, and are consistent with typical values in literature sources.

3. Site and Subsurface Conditions

3.1 Site Description

The subject area comprises approximately 0.6 hectares on the east side of Chesterfield Avenue in London Ontario. The Site is currently occupied by an abandoned residence and pool and is generally vegetated with urban lawn and frequent mature trees. There is a slope located along the northwest site boundary that traverses across the northern portion of the property. The Site is bounded by Veronica Avenue to the south, residential development to the east, woodlot and grasslands to the north and Chesterfield Avenue to the southwest. The Thames River is located approximately 80 north of the Site. The Site is generally flat with a minor gradient down, from south to north, approaching the slope. The Site at the base of the slope in generally level.

The slope is approximately 8 to 9 m in height and is generally vegetated by mature trees and shrubbery. An open drain is located at the base of the slope on the west side of the Site that runs north to the Thames River. The slope and table lands on the east side of the Site has been graded to create an access ramp down to the base of the slope. Select photos of the Site are attached in **Appendix B**.

The following sections provide a summary of the soil and groundwater conditions.

3.2 Soil Stratigraphy

The detailed stratigraphy encountered in each test hole is shown on the borehole logs found in **Appendix A** and summarized in the following paragraphs. It must be noted that the boundaries of the soil indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for geotechnical design and should not be interpreted as exact planes of geological change.

3.2.1 Topsoil

All boreholes were surfaced with a layer of topsoil. The topsoil thickness typically was 200 mm.

It should be noted that topsoil quantities should not be established from the information provided at the test hole locations only. If required, a more detailed analysis (involving additional shallow test pits) is recommended to accurately quantify the amount of topsoil to be removed for construction purposes.

3.2.2 Fill

Fill was encountered beneath the topsoil in each borehole. The fill varied in composition from silt to sand and extended to depths ranging from 1.4 to 1.5 m below ground surface (bgs). The fill was generally brown to black in colour and contained trace organics. The fill was typically in a very loose to compact state with SPT N values ranging from 3 to 13 blows per 300 mm split spoon sampler penetration. The fill was in a damp to moist state based on tactile observations and in-situ moisture contents of 7 to 12 percent.

3.2.3 Till

The predominant soil encountered within the boreholes was till. The till varied in composition from clayey silt till to silt till. Clayey silt till was encountered in BH1 underlying silt. The clayey silt till was generally brown, becoming grey near 3.5 m bgs. The clayey silt till contained trace sand and was in a stiff to very stiff state with SPT N values of 12 to 16. The clayey silt till was generally in a moist state with in-situ moisture contents ranging from 11 to 19 percent.

Silt till was encountered in boreholes BH2 and BH3 underlying the fill materials. The silt till was generally brown becoming grey with depth and contained some sand and trace to some gravel. The silt till was generally compact with SPT N values of 13 to 26. Laboratory testing of the silt till yielded *in situ* moisture contents of 11 to 14 percent, indicative of damp to moist conditions. Borehole BH3 was terminated in the silt till at a dept of 3.5 m bgs.

3.2.4 Sand/Silt

Sand/silt strata were encountered in BH1 and BH2. In BH1 silt was encountered beneath the fill and extending to a depth of approximately 2.9 m bgs. The silt was brown, compact (based n SPT N values of 11 to 22) and damp (in-situ moisture contents of 11 to 22 percent).

The sand/silt layer in BH2 was encountered beneath the silt till and extended to termination depth of 6.6 m bgs. The sand was brown, contained trace silt and was fine to medium grained. The sand was in a compact state with SPT N value of 26. The sand had in-situ moisture content of 7 percent, indication moist conditions. The silt layer was encountered underlying the sand in BH2 and was brown in colour, contained some sand and was dilatant. The silt was in a very dense state with SPT N value of 53. The silt was wet with in-situ moisture contents of 19 percent.

3.2.5 Sand and Gravel

A layer of sand and gravel was encountered in BH1 beneath the till at a depth of 9.3 m bgs. The sand and gravel was grey, in a compact relative density with SPT N value of 11 and described as wet based on tactile observations and in-situ moisture contents of 11 percent. Borehole BH1 was terminated in the sand and gravel at 9.6 m bgs.

3.3 Groundwater Conditions

Details of the groundwater conditions observed within the test holes are provided on the attached borehole logs. Upon completion of drilling, the open boreholes were examined for the presence of groundwater and groundwater seepage. Groundwater was measured near 8.2 m bgs in borehole BH1 upon completion of drilling. All other boreholes were dry upon completion of drilling.

It is noted that insufficient time was available for the measurement of the depth to the stabilized groundwater table prior to backfilling the test hole. It is also noted that the depth to the groundwater table may vary in response to climatic or seasonal conditions, and, as such, may differ at the time of construction, with higher levels in wet seasons. Capillary rise effects should also be anticipated in fine-grained soil deposits.

4. Geotechnical Discussion and Recommendations

It is understood that the existing lot is to be subdivided into seven (7) single-family residences with associated paved access and parking and services is proposed at the Site.

The property is located within Upper Thames River Conservation Authority (UTRCA) regulated lands and will require approval from the conservation authority.

The following sections of this report provide geotechnical comments and recommendations regarding site preparation, excavations and dewatering, foundations, slab-on-grade and basement design, bedding and backfill, earthquake design considerations, pavement design and curbs and sidewalks.

4.1 Site Preparation

Prior to placement foundations and/or engineered fill, all surficial topsoil, vegetation and/or otherwise deleterious materials should be stripped. Thicker areas of topsoil may be anticipated in areas with trees and/or heavy vegetative cover. It is anticipated that the surficial topsoil may be stockpiled on site for possible reuse as landscaping fill.

Following the removal of the topsoil and unsuitable materials described above and prior to fill placement, the exposed subgrade should be inspected by a Geotechnical Engineer. Any loose or soft zones noted in the inspection should be over-excavated and replaced with approved fill.

It is recommended that construction traffic be minimized on the finished subgrade, and that the subgrade be sloped to promote surface drainage and runoff.

In the building areas where the grade will be raised, the fill material should comprise imported granular or approved onsite (excavated) material. The fill material should be inspected and approved by a Geotechnical Engineer and should be placed in maximum 300 mm (12 inch) thick lifts and uniformly compacted to 100 percent Standard Proctor Maximum Dry Density (SPMDD) within 3 percent of optimum moisture content. The geometric requirements for engineered fill are provided on **Drawing 4**.

The natural and inorganic fill materials on site would be suitable for reuse as engineered fill. The material should be examined and approved by a Geotechnical Engineer prior to reuse.

In areas along the proposed roadways, fill material used to raise grades may comprise of onsite excavated soils, or imported granular fill approved by an engineer. The fill should be placed in maximum 300 mm (12 inch) thick loose lifts and uniformly compacted to 95/98 percent SPMDD within 3 percent of optimum moisture content to provide adequate stability for the new pavements.

In situ compaction testing should be carried out during the fill placement to ensure that the specified compaction is being achieved.

If imported fill material is utilized at the site, verification of the suitability of the fill may be required from an environmental standpoint. Conventional geotechnical testing will not determine the suitability of the material in this regard. Analytical testing and environmental site assessment may be required at the source. This will best be assessed prior to the selection of the material source. A quality assurance program should be implemented to ensure that the fill material will comply with the current Ministry of Environment, Conservation and Parks (MECP) standards for placement and transportation. The disposal of excavated materials must also conform to the MECP Guidelines and requirements. EXP can be of assistance if an assessment of the materials is required.

4.2 Excavation and Groundwater Control

4.2.1 Excess Soil Management

It should be noted that Ontario Regulation 406/19 made under the Environmental Protection Act (November 28, 2019) was implemented on January 1, 2021. The new regulation dictates the testing protocol that will be required for the management and disposal of Excess Soils. As set forth in the Regulation, specific analytical testing protocols will need to be implemented and followed based on the volume of soil to be managed. The testing protocols are specific as to whether the soils are stockpiled or in situ. In either scenario, the testing protocols are far more onerous than have been historically carried out as part of standard industry practices. These decisions should be factored in and accounted for prior to the initiation of the project-defined scope of work. EXP would be pleased to assist with the implementation of a soil management and testing program that would satisfy the requirements of Ontario Regulation 406/19.

The following is the regulated sampling and testing regiment.

Table 1 – Recommended Ex-Situ (e.g., Stockpiles)

Soil Volume	Sampling Frequency
<130 m^3	Minimum of 3
>130 - 220 m^3	4
>220 - 5000 m^3	5-32*
>5000 m^3	N = 32 + (Volume – 5000) / 300

*refer to stockpile sampling frequency in O.Reg. 153/04 for specifics. Essentially, one sample for every 150 m^3 after 800 m^3

Table 2 – Recommended *In-Situ*

Soil Volume	Sampling Frequency
<600 m^3	Minimum of 3
>600 m3 - 10,000 m^3	1 sample per every additional 200 m^3
>10,000 m3 – 40,000 m^3	1 sample per every additional 450 m^3
>40,000 m^3	1 sample per every additional 2000 m^3

Soil Analytical Testing Requirements:

- Samples to be tested for a minimum of Petroleum Hydrocarbons (PHCs) – Fractions F1-F4, Benzene, Toluene, Ethylbenzene & Xylenes (BTEX), Metals & Hydrides, including Electrical Conductivity (EC) and Sodium Absorption Ration (SAR), only if from an area where de-icing has historically occurred.
- Any potential Contaminant of Concern identified in past uses report (comes into effect January 1, 2022)
- Leachate analysis (not required for volumes under 350 m^3: between 350 m^3 and 600 m^3 (minimum of 3); greater than 600 m3 (10 % of samples). Note, leachate not required unless address and Area of Potential Environmental Concern (APEC), as identified in the past uses report (January 1, 2022).

4.2.2 Excavations

All work associated with design and construction relative to excavations must be carried out in accordance with Part III of Ontario Regulation 213/91 under the Occupational Health and Safety Act. Based on the results of the geotechnical investigation and in accordance with Section 226 of Ontario Regulation 213/91, the very loose to compact fill and firm to stiff clayey silt till soils are classified as <u>Type 4</u> soils. The compact silt till, sand silt, sand and gravel, stiff to very stiff clayey silt till are classified as Type 3 soils. The very dense silt soils are classified as <u>Type 2</u> soils.

Where excavations extend into or through <u>Type 4</u> soil, excavation side slopes must be cut back at an inclination of about 3H:1V from the base of the excavation or flatter. Where excavations extend into or through <u>Type 3</u> soil, excavation side slopes must be cut back at an inclination of about 1H:1V or flatter from the base of the excavation. Where excavations extend into or through <u>Type 2</u> soil may be cut vertical in the bottom 1.2 m (4 ft.) and cut back at an inclination of 1 horizontal to 1 vertical above that level. Should groundwater egress loosen the side slopes, slopes of 3H:1V or flatter will be required. Geotechnical inspection at the time of excavation can confirm the soil type present.

Although not encountered, it should be noted that the presence of cobbles and boulders in natural glacial deposits may influence the progress of excavation and construction.

4.2.3 Excavation Support

The recommendations for side slopes given in the above section would apply to most of the conventional excavations expected for the proposed development. However, in areas adjacent to buried services that are located above the base of the excavations, side slopes may require support to prevent possible disturbance or distress to these structures. This concept also applies to connections to existing services. In granular soils above the groundwater and in cohesive natural soils, bracing will not normally be required if the structures are behind a 45-degree line drawn up from the toe of the excavation. In wet sandy or silty soils, the setback should be about 3H to 1V if bracing is to be avoided.

For support of excavations such as for any deep manholes, shoring such as sheeting or soldier piles and lagging can be considered. The design and use of the support system should conform to the requirements set out in the most recent version of the Occupational Health and Safety Act for Construction Projects and approved by the Ministry of Labour. Excavations should conform to the guidelines set out in the proceeding section and the Safety Act.

The shoring should also be designed in accordance with the guidelines set out in the Canadian Foundation Engineering Manual, 4th Edition. Soil-related parameters considered appropriate for a soldier pile and lagging system are shown below.

Where applicable, the lateral earth pressure acting on the excavation shoring walls may be calculated from the following equation:

$$P = K (\gamma h+q)$$

where, P = lateral earth pressure in kPa acting at depth h;

γ = natural unit weight, a value of 20.5 kN/m3 may be assumed;

h = depth of point of interest in m;

q = equivalent value of any surcharge on the ground surface in kPa.

The earth pressure coefficient (K) may be taken as 0.25 where small movements are acceptable and adjacent footing or movement sensitive services are not above a line extending at 45 degrees from the bottom edge of the excavation; 0.35 where utilities, roads, sidewalks must be protected from significant movement; and 0.45 where adjacent building footings or movement sensitive services (gas and water mains) are above a line of 60 degrees from the horizontal extending from the bottom edge of the excavation.

For long term design, a K at rest (K_o) of a minimum of 0.5 should be considered.

The above expression assumes that no hydrostatic pressure will be applied against the shoring system. It should be recognized that the final shoring design will be prepared by the shoring contractor. It is not possible to comment further on specific design details until this design is completed.

If the shoring is exposed to freezing temperatures, appropriate insulation may be provided to prevent outward movement.

The performance of the shoring must be checked through monitoring for lateral movement of the walls of the excavation to ensure that the shoring movements remain within design limits. The most effective method for monitoring the shoring movements can best be devised by this office when the shoring plans become available. The shoring designer should however assess the specific site requirements and submit the shoring plans to the engineer for review and comment.

4.2.4 Construction Dewatering

As stated in Section 3.3, groundwater was observed in Borehole BH1 at a depth of 8.2 m bgs upon completion of drilling. All other boreholes were dry upon completion of drilling.

Significant groundwater infiltration is not anticipated within excavations to conventional depths. Any minor groundwater infiltration can likely be accommodated using conventional sump pumping techniques; however, due to sandy soil presence, it is recommended that the sump pits be lined with a suitable geotextile filter fabric and the pump inlet be set in a clear stone, which should fill the sump pit completely. The use of an unfiltered system will result in migration of sandy soil particles that will loosen the soil deposits. If groundwater infiltration persists, more extensive dewatering measures may be required. EXP would be pleased to provide further information in this regard, upon request.

The collected water should be discharged a sufficient distance away from the excavated area to prevent the discharge water from returning to the excavation. Sediment control measures should be provided at the discharge point of the dewatering system. Caution should also be taken to avoid any adverse impacts to the environment.

Although not anticipated for service excavations to conventional depths, it is important to mention that for any projects requiring positive groundwater control with a removal rate of 50,000 liters to less than 400,000 liters per day, an Environmental Activity and Sector Registry (EASR) will be required. Permit to Take Water (PTTW) applications are required for removal rates more than 400,000 L per day and will need to be approved by the MECP per Sections 34 and 98 of the Ontario Water Resources Act R.S.O. 1990 and the Water Taking and Transfer Regulation O. Reg. 387/04. It is noted that a standard geotechnical investigation will not determine all the groundwater parameters which may be required to support the application.

4.3 Building Foundations

4.3.1 Conventional Strip and Spread Footings

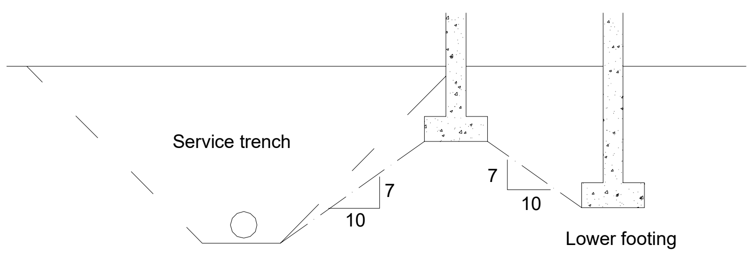
It is understood that the existing lot is to be subdivided into seven (7) single-family. The proposed residences can be supported on conventional spread and strip footings founded below the topsoil and unsuitable soils on the natural competent subgrade soils, or engineered fill. The following section provides options for conventional foundations.

The following allowable bearing pressures (net stress increase) can be used on the natural, undisturbed compact silt and silt till at depths below 1.5 m bgs:

Bearing Resistance at Serviceability Limit States (SLS)	145 kPa (3,000 psf)
Factored Bearing Resistance at Ultimate Limit States (ULS)	215 kPa (4,500 psf)

4.3.2 Foundations - General

Footings at different elevations should be located such that the higher footings are set below a line drawn up at 10 horizontal to 7 vertical from the near edge of the lower footing. This concept should also be applied to service excavation, etc. to ensure that undermining is not a problem.



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

Provided that the footing bases are not disturbed due to construction activity, precipitation, freezing and thawing action, etc., and the aforementioned bearing pressures are not exceeded, the total and differential settlements of footings designed in accordance with the recommendations of this report and with careful attention to construction detail are expected to be less than 25 mm and 20 mm (1 and ¾ inch) respectively.

All footings exposed to seasonal freezing conditions should be protected from frost action by at least 1.2 m (4 ft) of soil cover or equivalent insulation.

It should be noted that the recommended bearing capacities have been calculated by EXP from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, if more specific information becomes available with respect to conditions between boreholes when foundation construction is underway. The interpretation between the boreholes and the recommendations of this report must therefore be checked through field inspections provided by EXP to validate the information for use during the construction stage.

4.4 Slab-on-Grade Construction

Preparation of the subgrade should include the removal of all topsoil and/or deleterious material from the proposed building footprint. The entire floor slab area should then be thoroughly proof-rolled with a heavy roller and examined by a Geotechnical Engineer. Any excessively soft or loose areas should be sub-excavated and replaced with suitable compacted fill. Where the exposed subgrade requires reconstruction to achieve the design elevations, structural fill should be used. It is recommended that structural fill comprises granular material, such as OPSS Granular ‘B’, or approved alternative material. The fill should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98 percent Standard Proctor Maximum Dry Density (SPMDD). For best compaction results, the *in situ* moisture content of the fill should be within about three percent of optimum, as determined by Standard Proctor density testing.

No special underfloor drains are required provided that the exterior grades are lower than the floor slab, and positively sloped away from the slab. It is recommended that an impermeable soil seal such as clay, asphalt or concrete be provided on the surface to minimize water infiltration from the exterior of the building. See **Drawing 6** for Drainage and Backfill recommendations for slab-on-grade construction.

A moisture barrier, consisting of a 200 mm (8 in.) thick, compacted layer of 19 mm (3/4 in.) clear stone, should be then placed between the prepared granular sub-base and the floor slab.

The installation and requirement of a vapor barrier under a concrete slab should conform to the flooring manufacturer’s and designer’s requirements. Moisture emission testing will be required to determine the concrete condition prior to flooring installation. In order to minimize the potential for excess moisture in the floor slab at the time of the flooring installation, a concrete mixture with a low water-to-cement ratio (i.e., 0.45 to 0.55) should be used. Chemical additives may be required at the time of placement to make the concrete workable, and should be used in place of additional water at the point of placement. Ongoing liaison from this office will be required.

For slab on grade design, the modulus of subgrade reaction (k) can be taken as 15 MPa/m for the compacted stone layer over the compacted granular subbase.

The water-to-cement ratio and slump of concrete utilized in the floor slabs should be strictly controlled to minimize shrinkage of the slabs. Adequate joints should be provided in the floor slab to further control cracking. During placement of concrete at the construction site, testing should be performed on the concrete.

4.5 Basement Construction

If the proposed single-family residences are to be constructed with a basement, basement may be constructed using conventional concrete slab-on-grade techniques. The floor subgrade area should be stripped of any fill. The exposed area should be thoroughly proof rolled with a heavy roller and any soft spots detected should be dug out and replaced with compactable fill, following the guidelines set out in Section 4.1.

Care should be taken to protect the subgrade below the floor slabs during construction, by limiting construction traffic on the prepared subgrade soils. In addition, if the exposed subgrade soils are exposed to inclement weather conditions (i.e. rain, snow, freezing conditions), some remedial works may be required to remove wet, soft, or disturbed soils prior to stone and concrete placement.

A moisture barrier, consisting of a 200 mm (8 in.) thick, compacted layer of 19 mm (3/4 in.) clear, crushed stone, should be placed between the prepared subgrade and the floor slab. For design, the modulus of subgrade reaction (k) can be taken as 20 MPa/m for the compacted stone layer over the natural subgrade soils.

The water-to-cement ratio and slump of concrete utilized in the floor slab should be strictly controlled to minimize shrinkage of the slab. Adequate joints should be provided in the floor slab to further control cracking. During placement of concrete at the construction site, testing should be performed on the concrete.

All basement walls should be damp-proofed and must be designed to resist a horizontal earth pressure 'P' at any depth 'h' below the surface as given by the following expression:

$P = K (\gamma h+q)$

where, P = lateral earth pressure in kPa acting at depth h;
γ = natural unit weight, a value of 20.5 kN/m3 may be assumed;
h = depth of point of interest in m;
q = equivalent value of any surcharge on the ground surface in kPa.
K = earth pressure coefficient, assumed to be 0.4

Installation of perimeter drains is required for the basements at the Site. The above expression assumes that the perimeter drainage system prevents the build-up of any hydrostatic pressure behind the wall. Suggestions for permanent perimeter drainage are given on **Drawing 5**. Consideration should be given to the installation of an under-floor drainage system to collect and remove any water buildup beneath the structure.

4.6 Foundation Backfill

In general, the existing natural soils excavated from the foundation area should be suitable for re-use as foundation wall backfill outside of the free-draining zone if the work is carried out during relatively dry weather. The materials to be re-used should be within three percent of optimum moisture for best compaction results. Materials should be stockpiled per their composition; i.e. sandy soils should not be mixed with clayey soils.

If the weather conditions are very wet during construction, then imported granular material such as OPSS Granular 'B' should be used. Site review by the geotechnical consultant may be advised.

The backfill must be brought up evenly on both sides of walls not designed to resist lateral earth pressures.

During construction, the fill surface around the perimeter of structures should be sloped in such a way that the surface runoff water does not accumulate around the structure.

4.7 Site Servicing

The subgrade soils beneath the water and sewer pipes which are anticipated to service the site are generally expected to comprise clayey silt till, silt till, silt or sand. For services constructed on the natural soils or engineered fill, the bedding should conform to OPS Standards. The bedding course may be thickened if portions of the subgrade become wet during excavation. Bedding aggregate should be placed around the pipe to at least 300 mm (12 inch) above the pipe and be compacted to a minimum 95 percent SPMDD.

Water and sewer lines installed outside of heated areas should be provided with a minimum 1.2 m (4 ft.) of soil cover for frost protection.

The bases of excavations which cut into and terminate in competent till, silt or sand are expected to remain stable for the short construction period. For bases terminated in the wet sand or silt layers, localized improvement will be required. Base improvement may also be required if work is carried out in wet weather seasons. The extent of base improvement or stabilization is best determined in the field during construction, with consultation from a Geotechnical Engineer.

To minimize disturbance to the base, pipe laying should be carried out in short sections, with backfilling following closely after laying and no section of trench should be left open overnight.

The trenches above the specified pipe bedding should be backfilled with inorganic on-site soils placed in 300 mm thick lifts and uniformly compacted to at least 95% SPMDD. For trench backfill within 1 meter below the roadway subbase, the fill should be uniformly compacted to at least 98% SPMDD. A program of *in situ* density testing should be set up to ensure that satisfactory levels of compaction are achieved.

Requirements for backfill in service trenches, etc. should also have regard for OPS requirements. A summary of the general recommendations for trench backfill is presented on **Drawings 7** and **8**. A program of *in situ* density testing should be set up to ensure that satisfactory levels of compaction are achieved.

Based on the results of this investigation, the majority of the excavated till, silt or sand material may be used for construction backfill provided that reasonable care is exercised in handling. In this regard, the material should be within 3 percent of the optimum moisture as determined in the Standard Proctor density test, and stockpiling of material for prolonged periods of time should be avoided. This is particularly important if construction is carried out in wet or otherwise adverse weather.

Soils excavated from below the stabilized groundwater table may be too wet for reuse as backfill unless adequate time is allowed for drying, or if the material is blended with approved dry fill; otherwise, it may be stockpiled onsite for reuse as landscape fill.

As noted previously, disposal of excavated materials off site should conform to current MECP guidelines.

4.8 Earthquake Design Considerations

The recommendations for the geotechnical aspects to determine the earthquake loading for design using the OBC 2012 are presented below.

The subsoil and groundwater information at this Site have been examined in relation to Section 4.1.8.4 of the OBC 2012. The subsoils at the Site generally consist of topsoil over fill overlying till, silt, sand and sand and gravel deposits. It is anticipated that the proposed structures will be founded on the natural silt or till deposits, below any loose or soft zones.

Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2012 indicates that to determine the site classification, the average properties in the top 30 m (below the lowest basement level) are to be used. The boreholes advanced at this Site were excavated to a maximum depth of 9.6 m below existing grade. Therefore, the Site Classification recommendation would be based on the available information as well as our interpretation of conditions below the boreholes based on our knowledge of the soil conditions in the area.

Based on the above assumptions, interpretations in combination with the known local geological conditions, the Site Class for the proposed development is “D” as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012. Additional depth drilling may be advised to determine if the soil conditions below the current depth of exploration can support a higher Site Classification.

4.9 Site Pavement Design

Areas to be paved should be stripped of all topsoil, organics and other obviously unsuitable material. The exposed subgrade must then be thoroughly proof-rolled. Any soft areas revealed by this or any other observations must be over-excavated and backfilled with approved material. All fill required to backfill service trenches or to raise the subgrade to design levels must conform to requirements outlined previously. Preferably, the natural inorganic excavated soils should be used to maintain uniform subgrade conditions, provided adequate compaction can be achieved.

Provided the preceding recommendations are followed, the pavement thickness design requirements given in the following table are recommended for the anticipated specified classification and anticipated subgrade conditions.

Table 3 – Recommended Pavement Structure Thicknesses

Pavement Layer	Compaction Requirements	Light Duty (Cars Only)	Heavy Duty (Cars and Trucks)
Asphaltic Concrete	92% MRD[1] or 97% BRD[1]	40 mm HL-3 50 mm HL-8	50 mm HL-3 60 mm HL-8
Granular ‘A’ (Base)	100% SPMDD[1]	150 mm	150 mm
Granular ‘B’ (Base)	100% SPMDD[1]	300 mm	450 mm

*Notes: 1) SPMDD denotes Standard Proctor Maximum Dry Density, MRD denotes Maximum Relative Density, BRD denotes Bulk Relative Density.
2) The subgrade must be compacted to 98% SPMDD.
3) The above recommendations are minimum requirements.

The recommended pavement structure provided in the above table is based on the existing subgrade soil properties determined from visual examination and textural classification of the soil samples. Consequently, the recommended pavement structures should be considered for preliminary design purposes only. Other granular configurations may also be possible provided the granular base equivalency (GBE) thickness is maintained. These recommendations on thickness design are not intended to support heavy and concentrated construction traffic, particularly where only a portion of the pavement section is installed.

If construction is undertaken under adverse weather conditions (i.e., wet or freezing conditions) subgrade preparation and granular sub-base requirements should be reviewed by the Geotechnical Engineer. If the sub-base is set on wet or dilatant silty soils, a geotextile will be required. A woven type geotextile such as Terrafix 200W or equivalent would be suitable for this application.

If only a portion of the pavement will be in place during construction, the granular subbase may have to be thickened. This is best determined in the field during the site servicing stage of construction, prior to road construction.

Samples of both the Granular 'A' and Granular 'B' aggregate should be checked for conformance to OPSS 1010 prior to utilization on Site, and during construction. The Granular 'B' subbase and the Granular 'A' base courses must be compacted to 100 percent SPMDD.

The asphaltic concrete paving materials should conform to the requirements of OPSS MUNI 1150 and City of London standards. The asphalt should be placed in accordance with OPSS 310 and compacted to at least 97 percent of the Marshall mix design bulk relative density or 92% of maximum relative density. A tack coat should be applied between the surface and binder asphalt courses.

Good drainage provisions will optimize pavement performance. The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum grade of two percent) to provide effective surface drainage toward catch basins. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas. In low areas, sub-drains should be installed to intercept excess subsurface moisture and prevent subgrade softening, as shown on **Drawing 9**. This is particularly important in heavier traffic areas at the site entrances. The locations and extent of sub-drainage required within the paved areas should be reviewed by this office in conjunction with the proposed grading.

A program of *in situ* density testing must be carried out to verify that satisfactory levels of compaction are being achieved.

To minimize the effects of differential settlements of service trench fill, it is recommended that wherever practical, placement of binder asphalt be delayed for approximately six months after the granular sub-base is put down. The surface course asphalt should be delayed for a further one year. Prior to the surface asphalt being placed, it is recommended that a pavement evaluation be carried out on the base asphalt to identify repair areas or areas requiring remedial works prior to surface asphalt being placed.

4.10 Curbs and Sidewalks

It is recommended that the concrete for curb and gutter and sidewalks should be proportioned, mixed, placed, and cured in accordance with the requirements of the City of London standards, OPSS 353 and OPSS 1350.

During cold weather, the freshly placed concrete must be covered with insulating blankets to protect against freezing. Three cylinders from each day's pour should be taken for compressive strength testing. Air entrainment, temperature, and slump tests should be made from the same batch of concrete from which test cylinders are made.

The subgrade for the sidewalks should comprise undisturbed natural competent soil or well-compacted fill. A minimum 150 mm thick layer of compacted Granular 'A' type aggregate should be placed beneath the sidewalk slabs. It is recommended that the Granular 'A' be compacted to a minimum 100 percent SPMDD, to provide adequate support for the concrete sidewalk. Construction traffic should be kept off the placed curbs and sidewalks as they are not designed to withstand heavy traffic load.

4.11 Inspection and Testing Requirements

An effective inspection and testing program is an essential part of construction monitoring. The Inspection and Testing Program typically includes the following items:

- Subgrade examination prior to engineered fill placement, footing base evaluation;
- Inspection and Materials testing during engineered fill placement (full-time supervision is recommended) and site servicing works, including soil sampling, laboratory testing (moisture contents and Standard Proctor density test on the pipe bedding, trench backfill and engineered fill material), monitoring of fill placement, and *in situ* density testing;
- Materials testing for concrete curbs and sidewalks.
- Inspection and Materials testing during paved area construction, including subgrade examination of the paved area subgrade soils following site servicing, laboratory testing (grain size analyses and Standard Proctor density tests on the Granular A and B material placed on site roadways), and *in situ* density testing;
- Inspection and Materials testing for base and surface asphalt, including laboratory testing on asphalt sampling to confirm conformance to project specifications and standards.

EXP would be pleased to prepare an inspection and testing work program prior to construction, incorporating the above items.

5. Slope Stability Assessment

The slope that runs through the Site is approximately 8 to 9 m in height and is vegetated with mature trees and shrubbery. On the west side of the Site and slope, there is an open drain at the base of the slope. Minor toe erosion was observed at this location. The Thames River is located approximately 80 m north of the Site. The Site has a minor gradient down from south to north.

Two (2) slope profiles were created from Ontario Digital Terrain (DTM) lidar-derived topographic data (2017). The locations of the cross sections are shown on **Drawing 1**.

Site reconnaissance completed by EXP on August 13th, 2021 consisted of completing MNR Slope Rating Charts at the cross-section locations. The survey of the slopes included detailed observations such as slope vegetation, seepage from slope face, table land drainage, toe erosion and previous landslide activity.

Various points along the top and toe of the slope were surveyed by EXP personnel in May 2024 using a Trimble R12i to establish the top of slope and toe of slope, as requested by the UTRCA.

Slope stability analyses were completed at two locations on the slopes to determine the erosion hazard limits (development setback) for the Site.

Based on the cross-section profiles created from the topographic mapping and the MNR rating charts completed during the Site reconnaissance, two (2) slope sections were analyzed using the computer modelling software Slope/W. These sections were selected to represent the worse-case scenarios. The worse-case scenarios are based on the slope height, inclination, and its existing conditions, and proximity of the watercourse to the toe of the slope.

Examination of factors of safety were carried out and analyzed by computer methods utilizing the Slope/W computer program. Soil strength parameters used in the analyses were based on our observations and experience with similar soil and groundwater conditions and are consistent with typical values in literature sources.

5.1 Stable Slope Geometry

The stability of the existing slopes were investigated for a number of different Factors of Safety (FOS). The various types of failures resulting include shallow, moderate depth and deep rotational failures, occasionally through the entire height of the slope. The analyses were undertaken by computer methods utilizing the Slope/W computer program for select slope profiles.

The soil parameters used were conservative to build in an added safety factor for the analyses. The following table summarizes the parameters for the predominant soils which were used in EXP's evaluation of the stable slope configuration:

Table 4 – Soil Parameters

Soil Type	Unit Weight (kN/m^3)	Cohesion (kPa)	Angle of Internal Friction (°)
Silt	18	0	31
Till			
Sand & Gravel	20.0	0	32

In order to determine an appropriate Erosion Hazard Limit setback from the crest of the slope, a minimum factor of safety of 1.40 was used during the computerized stable slope analyses. The following table from the MNR Technical Guide provides guidance on how to select a minimum factor of safety based on the intended land use above or below the slope.

Table 5 – Design Minimum Factor of Safety

Table 4.3

	LAND-USES	DESIGN MINIMUM FACTOR OF SAFETY
A	**PASSIVE**; no buildings near slope; farm field, bush, forest, timberland, woods, wasteland, badlands, tundra	1.10
B	**LIGHT**; no habitable structures near slope; recreational parks, golf courses, buried small utilities, tile beds, barns, garages, swimming pools, sheds, satellite dishes, dog houses	1.20 to 1.30
C	**ACTIVE**; habitable or occupied structures near slope; residential, commercial, and industrial buildings, retaining walls, storage/warehousing of non-hazardous substances	1.30 to 1.50
D	**INFRASTRUCTURE and PUBLIC USE**; public use structures or buildings (i.e., hospitals, schools, stadiums), cemeteries, bridges, high voltage power transmission lines, towers, storage/warehousing of hazardous materials, waste management areas	1.40 to 1.50

Note: Table obtained from page 60 of MNR Technical Guide – River and Stream Systems: Erosion Hazard Limit (2002).

Two (2) cross sections were reviewed and assessed using computerized stable slope stability analyses for the slope assessment at the site (Cross Section A-A' and Cross Section B-B'). The section evaluated was selected to represent the worst-case-scenario of the slope. The cross-section locations are shown on **Drawing 1.** The information plotted in the cross sections was based on Ontario Digital Terrain (DTM) lidar-derived topographic data (2017) and top and toe of slope were established based on a survey completed by EXP personnel in May 2024.

The failures at the cross section consisted of shallow, moderate and deep failures throughout the depth of the slope. The overall slope stability should be considered for moderate depth and deep failure modes due to mature trees and other vegetation along the slope, reducing the probability of shallow failures occurring. After completing the computerized stable slope analysis on the cross section, the minimum calculated factor of safety (FOS) under the existing conditions for moderate and deep failures are summarized results are provided in the following table.

Table 6 – Summary of Pertinent Slope Stability Analyses

Cross Section Condition	Description of Failure Mode	Computer Factor of Safety
Cross Section A-A'	Existing Slope Condition - Shallow Depth Failure	1.40
	Existing Slope Condition - Moderate Depth Failure	1.41
	Existing Slope Condition - Deep Failure	1.55
	2.1H:1V Stable Slope with 250-Year Flood	1.46

Cross Section Condition	Description of Failure Mode	Computer Factor of Safety
Cross Section B-B'	Existing Slope Condition - Shallow Depth Failure	1.45
	Existing Slope Condition - Moderate Depth Failure	1.49
	Existing Slope Condition - Deep Failure	1.63
	2.1H:1V Stable Slope with 250-Year Flood	1.40

The soil conditions encountered in the boreholes near the assessed areas of the Site typically comprise fill over silt and till deposits.

A slope inclination of 2.1H:1V is considered stable for all cross sections based on the slope analysis carried out. To ensure that a satisfactory factor of safety is applied for the Erosion Hazard Limit along the slopes at the site, the stable slope setback line should be drawn from the toe erosion allowance. The stable slope allowance of 2.0H:1V has been applied based on a conservative evaluation and to exceed the target FOS of 1.40.

5.2 Toe Erosion Component

An open drain was observed near the bottom of the slope in the area of Cross Section B-B' (See **Drawing 3**). The water course was observed to be located within 15 m of the toe of the slope in this area. Minor toe erosion was observed in this area at select locations.

Where detailed slope stability analyses have not been carried out, the Natural Hazards Manual by Ministry of Natural Resources indicates that a minimum toe erosion allowance of 5 m is recommended where natural soils at the toe of the slope comprise cohesive soils such as clay, clayey silts and tills and where evidence of active erosion is present.

Considering the nature of the soils at the base of the slope, expected to comprise till, silt or compact sand and gravel, the observed watercourse width and velocity, and minor active erosion observed, a toe erosion allowance of 5 metres has been assigned to the slope in this area.

The slope along Cross Section A-A' is approximately 80 m from the Thames River, therefore, no toe erosion allowance was applied for this area.

5.3 Emergency Access Allowance

The Erosion Access Allowance as specified in Section 3.4 of the MNR Technical Guide is a distance of 6 m from the top of the stable slope. This allowance is required in order to provide access for repairs to the slope from the top of the slope. EXP recommends that a distance of 6 m for the erosion access allowance be provided on the table land. No permanent structures should be constructed within the 6 m of the erosion access allowance.

5.4 Erosion Hazard Limit

The Erosion Hazard Limit is defined by the sum of the Stable Safe Slope Line plus the Toe Erosion Component plus the Erosion Access Allowance. The table below summarizes the 3 components to the Recommended Development Limit Setback.

Table 7 – Erosion Hazard Limit Components

Cross Section	Toe Erosion Allowance (m)	Applied Stable Slope Allowance (From Top of Existing Slope, m)	Emergency Access Allowance (m)	Applied Erosion Hazard Limit (From Top of Existing Slope, m)
	0.0	5.1	6.0	11.1
Cross Section B-B'	5.0	4.8	6.0	10.8

The Toe Erosion Allowance, Stable Slope Setback and Erosion Hazard Limit are shown on **Drawings 1** to **3**. The footprint of the proposed building should not encroach on the Erosion Hazard Limit.

5.5 Additional Comments

The site should be graded such that surface water is directed away from the slope. No water from the table land should be out-letted down the slope.

Water from downspouts and perimeter weeping tile etc. should be collected in a controlled manner and directed away from the slope.

Spoils from any excavation should be removed from the site. Excavated soils should not be placed over the table land near the crest of slope unless the soil is placed as engineered structural fill. No net surcharge should be placed on the slope.

During construction, stockpiles of materials, supplies and construction debris should be located away from the slope crest. Additional loading from stockpiled materials should be avoided in proximity to the slope crest.

Debris littering the slope should be removed and vegetation on the slope should be maintained.

Any bare spot or cracks observed at the slope should be revegetated.

Drawings 1 to **3** show the location of the Toe Erosion Allowance, Stable Slope Setback and Erosion Hazard Limit.

Any structural component should be founded on competent soil below a line drawn from the toe of the slope at 3H:1V and below the frost depth as per the Ontario Building Code.

A regular maintenance program should be implemented such as tree preservation, grading, and drainage control.

6. General Comments

The information presented in this report is based on a limited investigation designed to provide information to support an assessment of the current geotechnical conditions within the subject property. The conclusions and recommendations presented in this report reflect site conditions existing at the time of the investigation. Consequently, during the future development of the property, conditions not observed during this investigation may become apparent. Should this occur, EXP Services Inc. should be contacted to assess the situation, and the need for additional testing and reporting. EXP has qualified personnel to provide assistance in regards to any future geotechnical and environmental issues related to this property.

Our undertaking at EXP, therefore, is to perform our work within limits prescribed by our clients, with the usual thoroughness and competence of the engineering profession.

The comments given in this report are intended only for the guidance of design engineers. The number of test holes required to determine the localized underground conditions between test holes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

EXP Services Inc. should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not afforded the privilege of making this review, EXP Services Inc. will assume no responsibility for interpretation of the recommendations in this report.

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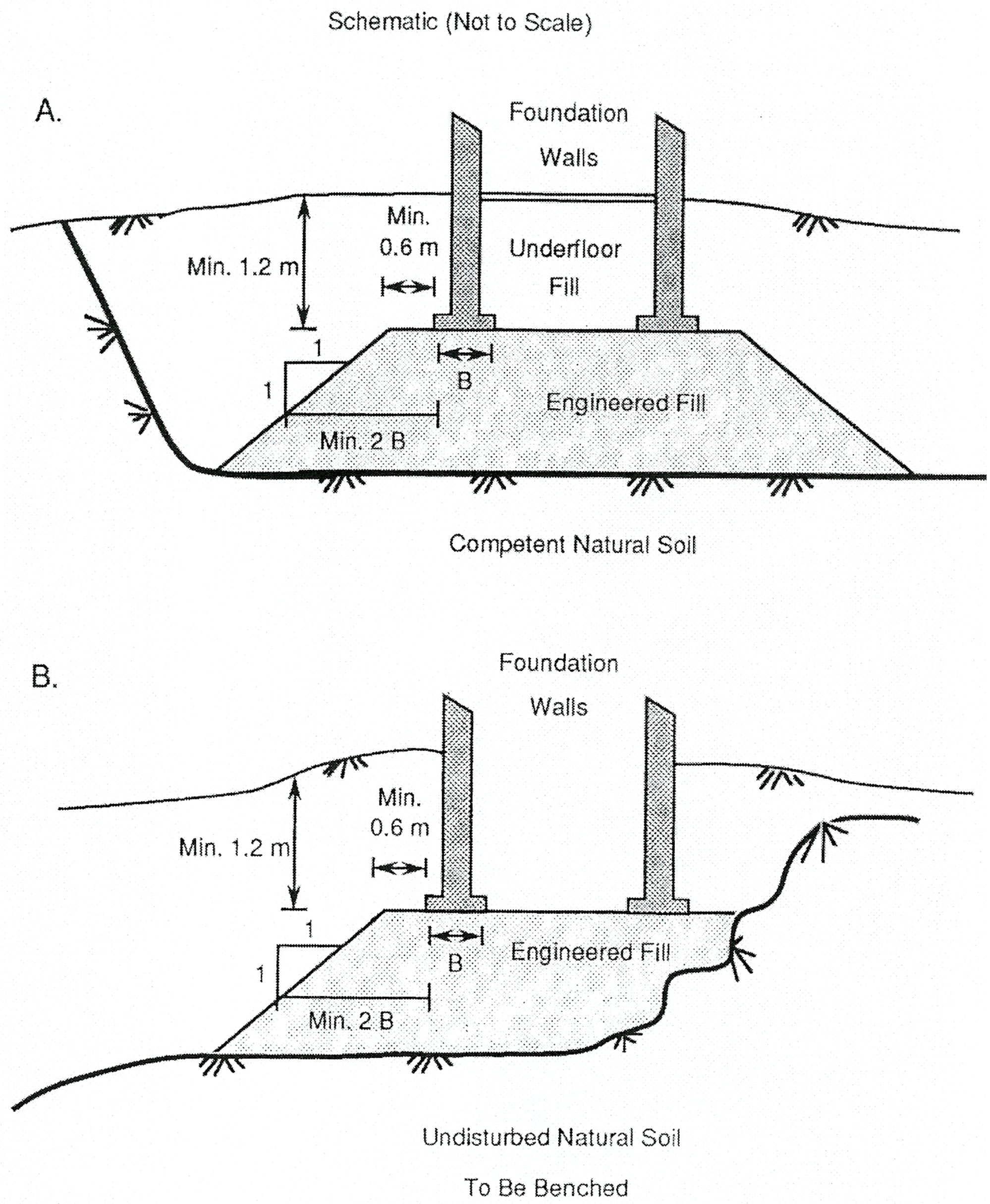
We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

Drawings

DRAWING 4 – GEOMETRIC REQUIREMENTS FOR FOUNDATIONS ON ENGINEERED FILL



SECTION VIEW

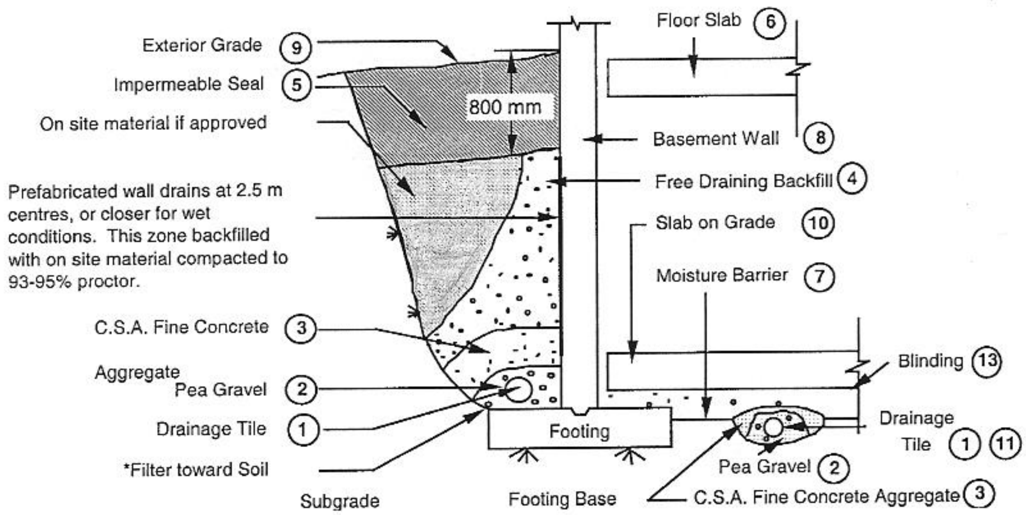
Section A – Typical Section of Slab-on-Grade Building
Section B – Typical Section of Building with Basement

Refer to Detailed Notes on following page.

NOTES FOR ENGINEERED FILL PLACMENT:

1. The area must be stripped of all topsoil contaminated fill material, and other unsuitable soils, and proof rolled. Soft spots must be dug out. The stripped natural subgrade must be examined and approved by an EXP Engineer prior to placement of engineered fill.
2. In areas where engineered fill is placed on a slope, the fill should be benched into the approved subgrade soils. EXP would be pleased to provide additional comments and recommendations in this regard, if required.
3. All excavations must be carried out in accordance with the Occupational Health and Safety Regulation of Ontario (Construction Projects - O.Reg. 213.91)
4. Material used for engineered fill must be free of topsoil, organics, frost and frozen material, and otherwise unsuitable or compressible soils, as determined by a Geotechnical Engineer. Any material proposed for use as engineered fill must be examined and approved by EXP, prior to use onsite. Clean compactable granular fill is preferred.
5. Approved engineered fill should be placed in maximum 300 mm thick lifts, and uniformly compacted to 100% Standard Proctor dry density throughout. For best compaction results, engineered fill should be within 3 percent of its optimum moisture content, as determined by the Standard Proctor density test. Imported fill should satisfy the MECP regulations and requirements.
6. Full time geotechnical monitoring, inspection and *in situ* density (compaction) testing by EXP is required during placement of the engineered fill.
7. Site grades should be maintained during area grading activities to promote drainage, and to minimize ponding of surface water on the engineered fill mat. Rutting by construction equipment should be kept to a minimum, where possible. Additional work to ensure suitability of engineered fill may be required if fill is placed in extreme (hot/cold) weather.
8. The fill must be placed such that the specified geometry is achieved. Refer to sketches (previous page) for minimum requirements. Proper environmental protection will be required, such as providing frost penetration during construction, and after the completion of the engineered fill mat.
9. An allowable bearing pressure of 145 kPa (3000 psf) may be used provided that all conditions outlined above, and in the Geotechnical Report are adhered to.
10. These guidelines are to be read in conjunction with the attached Geotechnical Report. (EXP Project No. LON-21013388-A0)
11. For foundations set on engineered fill, footing enhancement and/or concrete reinforcing steel placement is recommended. The footing geometry and extent of concrete reinforcing steel will depend on site specific conditions. In general, consideration may be given to having a minimum strip footing width of 500 mm (20 inches), containing nominal steel reinforcement. Alternatively, concrete reinforcement may be recommended in the top and bottom of the foundation wall strip. The final footing geometry and extent of reinforcement is best determined in the field, by a Geotechnical Engineer.

DRAWING 5 – BACKFILL AND BASEMENT DRAINAGE DETAIL (NOT TO SCALE)

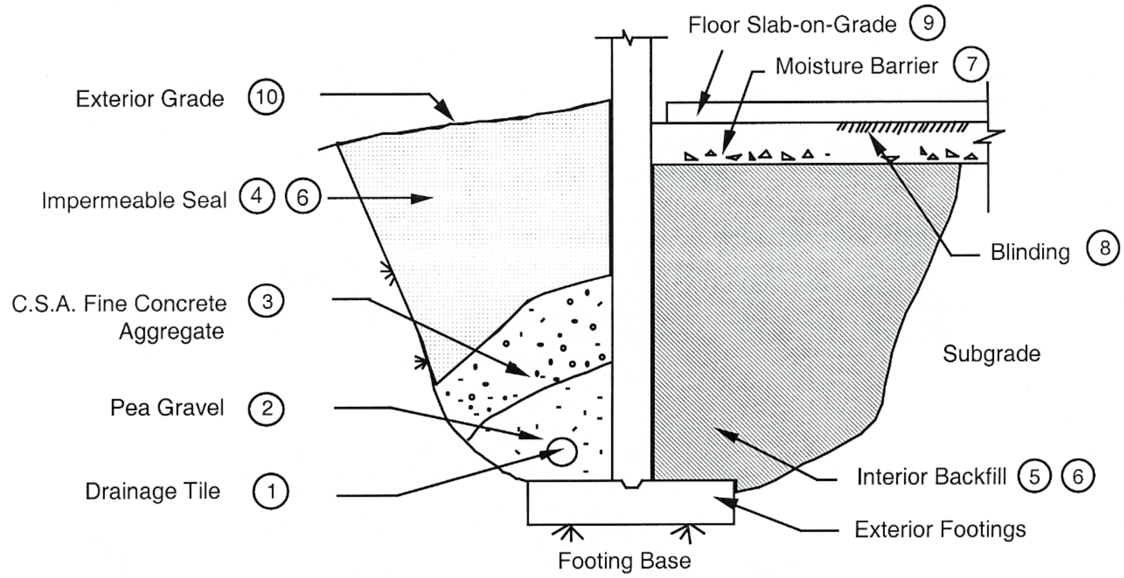


NOTES:

1. Drainage tile to consist of 100 mm (4 in.) diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet. Invert to be minimum of 150 mm (6 in.) below underside of floor slab.
2. Pea gravel 150 mm (6 in.) top and sides of drain. If drain is not on footing, place 100 mm (4 in.) of pea gravel below drain. 20 mm (3/4 in.) clear stone may be used provided if it is covered by an approved porous geotextile fabric membrane (Terrafix 270R or equivalent).
3. C.S.A. fine concrete aggregate to act as filter material. Minimum 300 mm (12 in.) top and side of drain. This may be replaced by an approved porous geotextile membrane (Terrafix 270R or equivalent).
4. Free-draining backfill - OPSS Granular B or equivalent compacted to 93 to 95 (maximum) percent Standard Proctor density. Do not compact closer than l.8 m (6 ft) from wall with heavy equipment. Use hand controlled light compaction equipment within 1.8 m (6 ft) of wall.
5. Impermeable backfill seal of compacted clay, clayey silt or equivalent. If original soil is free-draining, seal may be omitted.
6. Do not backfill until wall is supported by basement and floor slabs or adequate bracing.
7. Moisture barrier to consist of compacted 20 mm (3/4 in.) clear, crushed stone or equivalent free-draining material. Layer to be 200 mm (8 in.) minimum thickness.
8. Basement walls to be damp-proofed.
9. Exterior grade to slope away from wall.
10. Slab on grade should not be structurally connected to wall or footing.
11. Underfloor drain invert to be at least 300 mm (12 in.) below underside of floor slab. Drainage tile placed in parallel rows 6 to 8 m (20 to 25 ft.) centres one way. Place drain on 100 mm (4 in.) of pea gravel with 150 mm (6 in.) of pea gravel top and sides. CSA fine concrete aggregate to be provided as filter material or an approved porous geotextile membrane (as in 2 above) may be used.
12. Do not connect the underfloor drains to perimeter drains.
13. If the 20 mm (3/4 in.) clear stone requires surface binding, use 6 mm (1/4 in.) clear stone chips.

Note: a) Underfloor drainage can be deleted where not required (see report).
b) Free draining backfill, item 4 may be replaced by wall drains, as indicated, if more economical.

DRAWING 6 – DRAINAGE AND BACKFILL RECOMMENDATIONS (NOT TO SCALE)

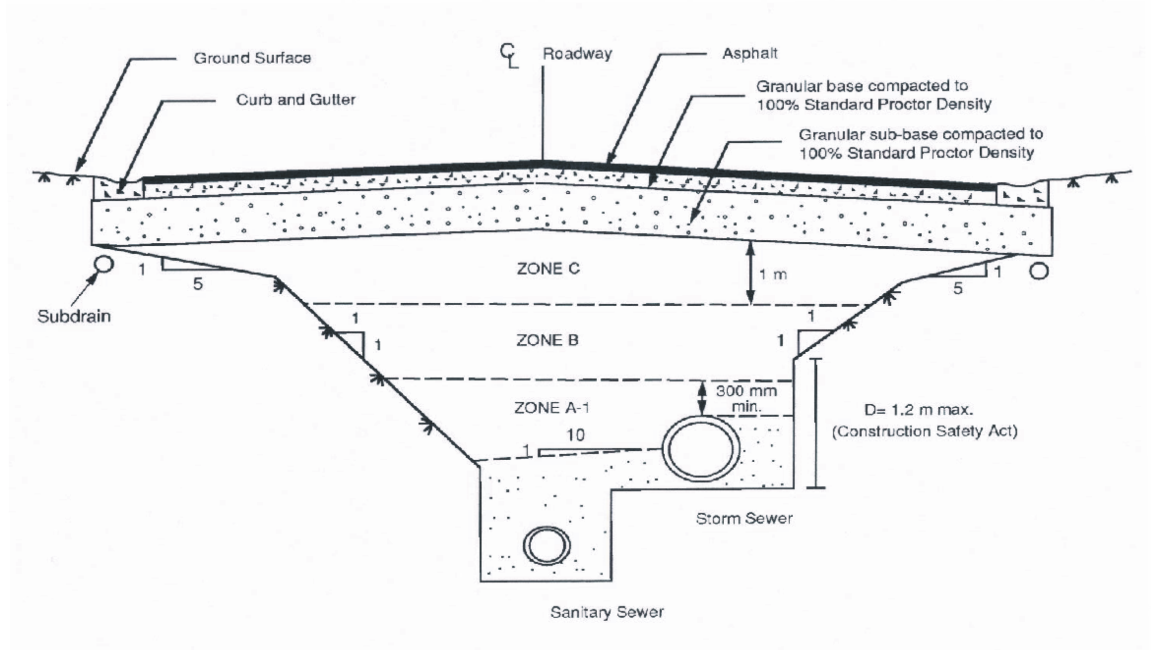


NOTES:

1. Drainage tile to consist of 100 mm (4 in.) diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet. Invert to be minimum of 150 mm (6 in.) below underside of interior floor slab.
2. Pea gravel 150 mm (6 in.) top and sides of drain. If drain is not on footing, place 100 mm (4 in.) of pea gravel below drain. 20 mm (3/4 in.) clear stone may be used provided if it is covered by an approved porous geotextile fabric membrane (Terrafix 270R or equivalent).
3. C.S.A. fine concrete aggregate to act as filter material. Minimum 300 mm (12 in.) top and side of drain. This may be replaced by an approved porous geotextile membrane (Terrafix 270R or equivalent).
4. Impermeable backfill seal of compacted clay, clayey silt or equivalent. If original soil is free-draining, seal may be omitted. Compact backfill to 95 percent Standard Proctor Maximum Dry Density.
5. The interior fill may be any clean, inorganic soil which may be compacted to at least 95 percent Standard Proctor density in this confined space.
6. Do not use heavy compaction equipment within 450 mm (18 in.) of the wall. Do not fill or compact within 1.8 m (6 ft) of wall unless fill is placed on both sides simultaneously.
7. Moisture barrier to be at least 200 mm (8 in.) of compacted 20 mm (3/4 in.) clear, crushed stone or equivalent free-draining material.
8. If the 20 mm (3/4 in.) clear stone requires surface binding, use 60 mm (1/4 in.) clear stone chips.
9. Slab on grade should not be structurally connected to wall or footing.
10. Exterior grade to slope away from building.

This system is not normally required if the floor is at least 300 mm (1 ft.) above exterior grade.

DRAWING 7 – TYPICAL BACKFILL DETAIL
STORM AND SANITARY SEWER (COMMON TRENCH)



SECTION VIEW

NOTES:

ZONE A
Granular bedding satisfying current OPS Standards compacted to 95% Standard Proctor maximum dry density.

ZONE A-l
To be compacted to 95% Standard Proctor maximum dry density.

ZONE B
To be compacted to 95% Standard Proctor maximum dry density.

ZONE C
To be compacted to 98% Standard Proctor maximum dry density.

The excavations shown above are for Type 1 or 2 soils. Where excavations extend through Type 3 soils, the side walls should be sloped back at a maximum inclination of 1 horizontal to 1 vertical from the base (Reference O.Reg 219/31).

DRAWING 8 – TRENCH BACKFILL REQUIREMENTS

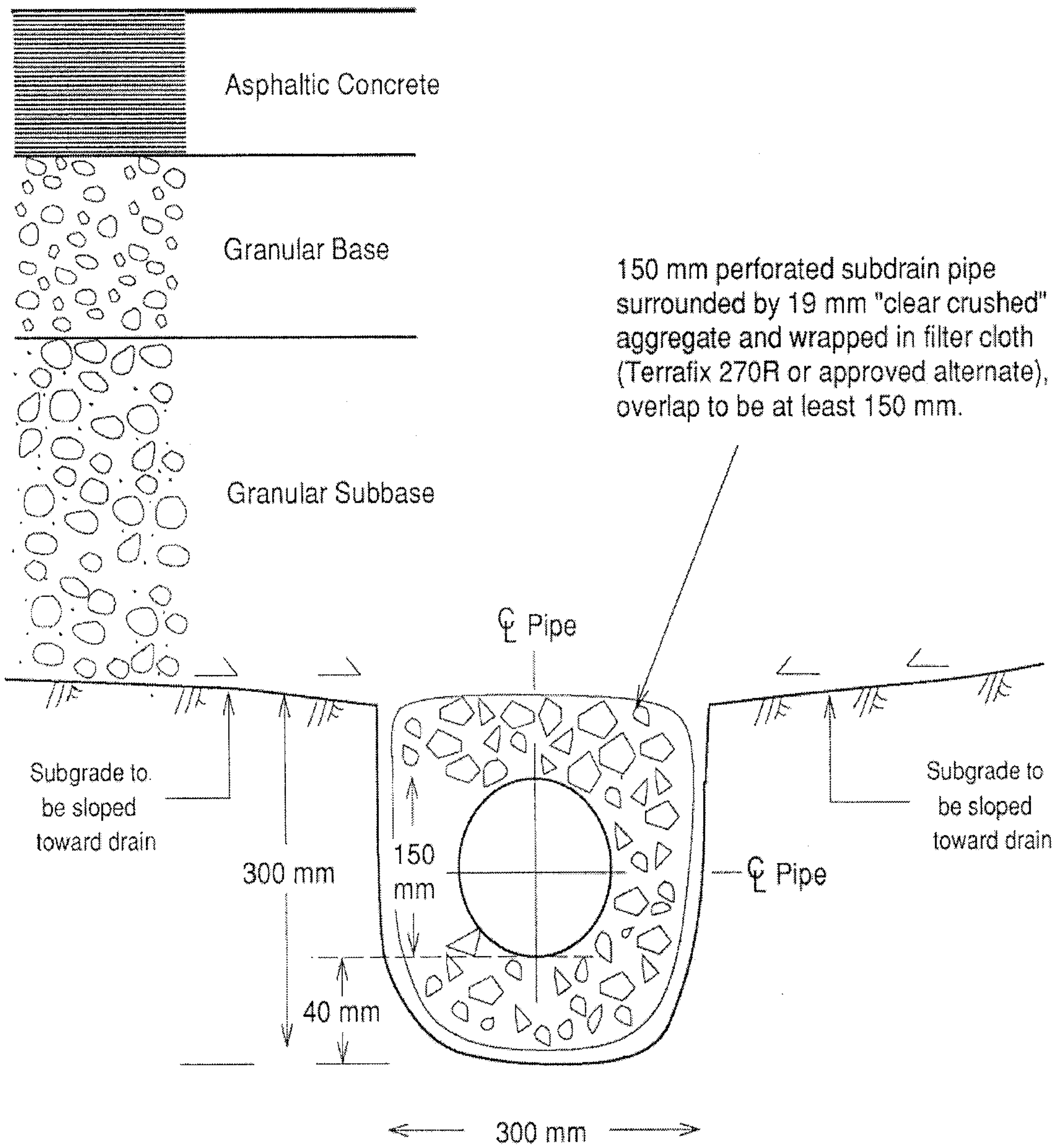
Requirements for backfill in service trenches, etc. should conform to current OPSS requirements. A summary of the general recommendations for trench backfill is presented on **Drawing 7**.

The bedding materials for the services designated as Zone A on the attached drawings should consist of approved granular material satisfying the current OPSS minimum standards and specifications. (Class B bedding should provide adequate support for the pipes). These materials should be uniformly compacted to 95 percent of standard Proctor dry density. Some problems may be encountered in maintaining alignment when bedding pipes in wet sandy soil. If Granular ‘A’ or other sandy material is used for bedding, they may become ‘spongy’ when saturated. If significant amounts of clear stone are used to stabilize the base, a geotextile should be incorporated to avoid problems with migration of fine grained materials and differential settlement under the pipes as the groundwater rises after backfilling. For minor local use of crushed stone without a geotextile filter, a graded HL3 stone is preferable.

The backfill in Zone B will consist of the native material. This material should be placed in loose lifts not exceeding 300 mm (12 inches) and be uniformly compacted to 95 percent of the standard Proctor maximum dry density. Material wetter than 5 percent above optimum must be allowed to dry sufficiently or should be discarded or used in landscaped areas.

The upper 1 meter of the general backfill (i.e. Zone C) should be placed in loose lifts not exceeding 300 mm (12 inches) and be uniformly compacted to at least 98 percent of the standard Proctor maximum dry density. To achieve satisfactory compaction, the fill material should be within 3 percent of standard Proctor optimum moisture content at placement.

DRAWING 9 – PAVEMENT SUBDRAIN DETAIL



NOTES:

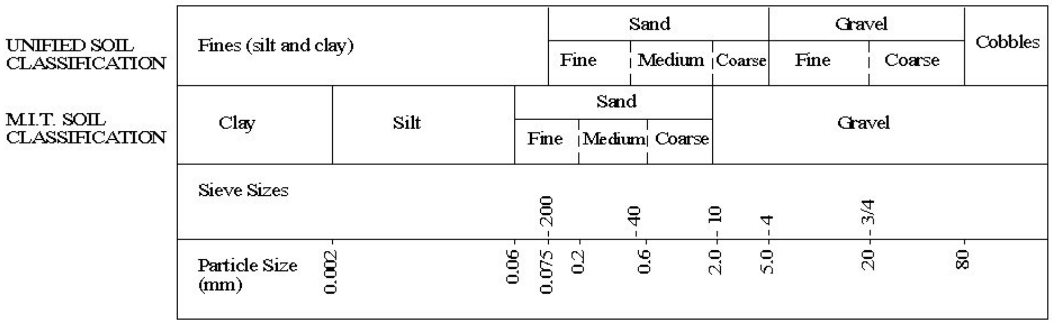
1. All dimensions in millimetres.
2. All sub drains to be set on at least 1% grade draining to a positive outlet.
3. Subgrade soil conditions should be verified onsite, during subgrade preparation works, following site servicing installations.

Scale: NTS

Appendix A – Borehole Logs

NOTES ON SAMPLE DESCRIPTIONS

1. All descriptions included in this report follow the 'modified' Massachusetts Institute of Technology (M.I.T.) soil classification system. The laboratory grain-size analysis also follows this classification system. Others may designate the Unified Classification System as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain size analysis has been carried out, all samples are classified visually and the accuracy of the visual examination is not sufficient to differentiate between the classification systems or exact grain sizing. The M.I.T. system has been modified and the EXP classification includes a designation for cobbles above the 75 mm size and boulders above the 200 mm size.



2. **Fill**: Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description therefore, may not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces or subsurface basements, floors, tanks, even though none of these obstructions may have been encountered in the borehole. Despite the use of borehole, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. The fill at this site has been monitored for the presence of methane gas and the results are recorded on the borehole logs. The monitoring process neither indicates the volume of gas that can be potentially generated or pinpoints the source of the gas. These readings are to advise of a potential or existing problem (if they exist) and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic waste that renders the material unacceptable for deposition in any but designated land fill sites; unless specifically stated, the fill on the site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common, but not detectable using conventional geotechnical procedures.

3. **Glacial Till**: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process, the till must be considered heterogeneous in composition and as such, may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (75 to 200 mm in diameter) or boulders (greater than 200 mm diameter) and therefore, contractors may encounter them during excavation, even if they are not indicated on the borehole logs. It should be appreciated that normal sampling equipment can not differentiate the size or type of obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited area; therefore, caution is essential when dealing with sensitive excavations or dewatering programs in till material.

Appendix B – Site Photos

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Photo 1 – Drain at Cross Section B-B’

Photo 2 – Cross Section B-B’ Slope

Photo 3 – Top of Slope Cross Section A-A’

Photo 4 – Cross Section A-A' Slope

Photo 5 – Access Path Slope West of Cross Section A-A’

Appendix C – Slope Stability Analyses

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Appendix D – MNR Slope Rating Charts

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Appendix E – Inspection and Testing Schedule

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INSPECTION & TESTING SCHEDULE

The following program outlines suggested minimum testing requirements during backfilling of service trenches and construction of pavements. In adverse weather conditions (wet/freezing), increased testing will be required. The testing frequencies are general requirements and may be adjusted at the discretion of the engineer based on test results and prevailing construction conditions.

I TRENCH BACKFILL

ZONE A
- one *in situ* density test per 100 cubic meters or 50 linear metres of trench whichever is less
- one laboratory grain size and Proctor density test per 50 density tests or 4000 cubic metres or on change of material (source, visual)

ZONE A1
- one *in situ* density test per 75 cubic metres of material or 25 linear metres of each lift of fill
- one laboratory grain size and Proctor density test per each 50 density tests or 4000 cubic metres of material placed or as directed by the engineer

ZONES B & C
- one *in situ* density test per 150 cubic metres of material or 50 linear metres or each lift whichever is less
- one laboratory grain size and Proctor density test per 50 density tests or 4000 cubic metres of material placed or as directed by the engineer

II PAVEMENT MATERIALS

GRANULAR SUBBASE
- one *in situ* density test per 50 linear metres of road
- one laboratory grain size and standard Proctor test per 50 density tests or 4000 cubic metres or each change of material (visual, source), as determined by the engineer

GRANULAR BASE
- one *in situ* density test per 50 linear metres of road
- one laboratory grain size and Proctor per 50 density tests or 8000 cubic metres or change in material (visual, source), as determined by the engineer
- Benkelman beam testing at 10 metre intervals per lane, after final grading and compaction. Asphaltic concrete should not be placed until rebound criteria have been satisfied.

ASPHALTIC CONCRETE
- one *in situ* density test per 25 linear metres of roadway
- one complete Marshall Compliance test including stability flow, etc. for each mix type to check mix acceptability. One extraction and gradation test per each day of paving to be compared to job mix formula

NOTES: Where testing indicates inadequate compaction, additional fill should not be placed until the area is recompacted and retested at the discretion of the engineer.

Appendix F – Limitations and Use of Report

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LIMITATIONS AND USE OF REPORT

BASIS OF REPORT

This report ("Report") is based on site conditions known or inferred by the geotechnical investigation undertaken as of the date of the Report. Should changes occur which potentially impact the geotechnical condition of the site, or if construction is implemented more than one year following the date of the Report, the recommendations of EXP may require re-evaluation.

The Report is provided solely for the guidance of design engineers and on the assumption that the design will be in accordance with applicable codes and standards. Any changes in the design features which potentially impact the geotechnical analyses or issues concerning the geotechnical aspects of applicable codes and standards will necessitate a review of the design by EXP. Additional field work and reporting may also be required.

Where applicable, recommended field services are the minimum necessary to ascertain that construction is being carried out in general conformity with building code guidelines, generally accepted practices and EXP's recommendations. Any reduction in the level of services recommended will result in EXP providing qualified opinions regarding the adequacy of the work. EXP can assist design professionals or contractors retained by the Client to review applicable plans, drawings, and specifications as they relate to the Report or to conduct field reviews during construction.

Contractors contemplating work on the site are responsible for conducting an independent investigation and interpretation of the test pit results contained in the Report. The number of test pits necessary to determine the localized underground conditions as they impact construction costs, techniques, sequencing, equipment and scheduling may be greater than those carried out for the purpose of the Report.

Classification and identification of soils, rocks, geological units, contaminant materials, building envelopment assessments, and engineering estimates are based on investigations performed in accordance with the standard of care set out below and require the exercise of judgment. As a result, even comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations or building envelope descriptions involve an inherent risk that some conditions will not be detected. All documents or records summarizing investigations are based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated. Some conditions are subject to change over time. The Report presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, these should be disclosed to EXP to allow for additional or special investigations to be undertaken not otherwise within the scope of investigation conducted for the purpose of the Report.

RELIANCE ON INFORMATION PROVIDED

The evaluation and conclusions contained in the Report are based on conditions in evidence at the time of site inspections and information provided to EXP by the Client and others. The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose as communicated by the Client. EXP has relied in good faith upon such representations, information and instructions and accepts no responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of any misstatements, omissions, misrepresentation or fraudulent acts of persons providing information. Unless specifically stated otherwise, the applicability and reliability of the findings, recommendations, suggestions or opinions expressed in the Report are only valid to the extent that there has been no material alteration to or variation from any of the information provided to EXP.

STANDARD OF CARE

The Report has been prepared in a manner consistent with the degree of care and skill exercised by engineering consultants currently practicing under similar circumstances and locale. No other warranty, expressed or implied, is made. Unless specifically stated otherwise, the Report does not contain environmental consulting advice.

COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment form part of the Report. This material includes, but is not limited to, the terms of reference given to EXP by its client (“Client”), communications between EXP and the Client, other reports, proposals or documents prepared by EXP for the Client in connection with the site described in the Report. In order to properly understand the suggestions, recommendations and opinions expressed in the Report, reference must be made to the Report in its entirety. EXP is not responsible for use by any party of portions of the Report.

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