- # Springer Pond Developments Inc.

Geotechnical Assessment

Project Name
Proposed Residential Development
Springer Street, Komoka, Ontario

Project Number
LON-00014641-GE

Prepared By:
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Canada

Date Submitted
May 11, 2016

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Date Submitted:
May 11, 2016

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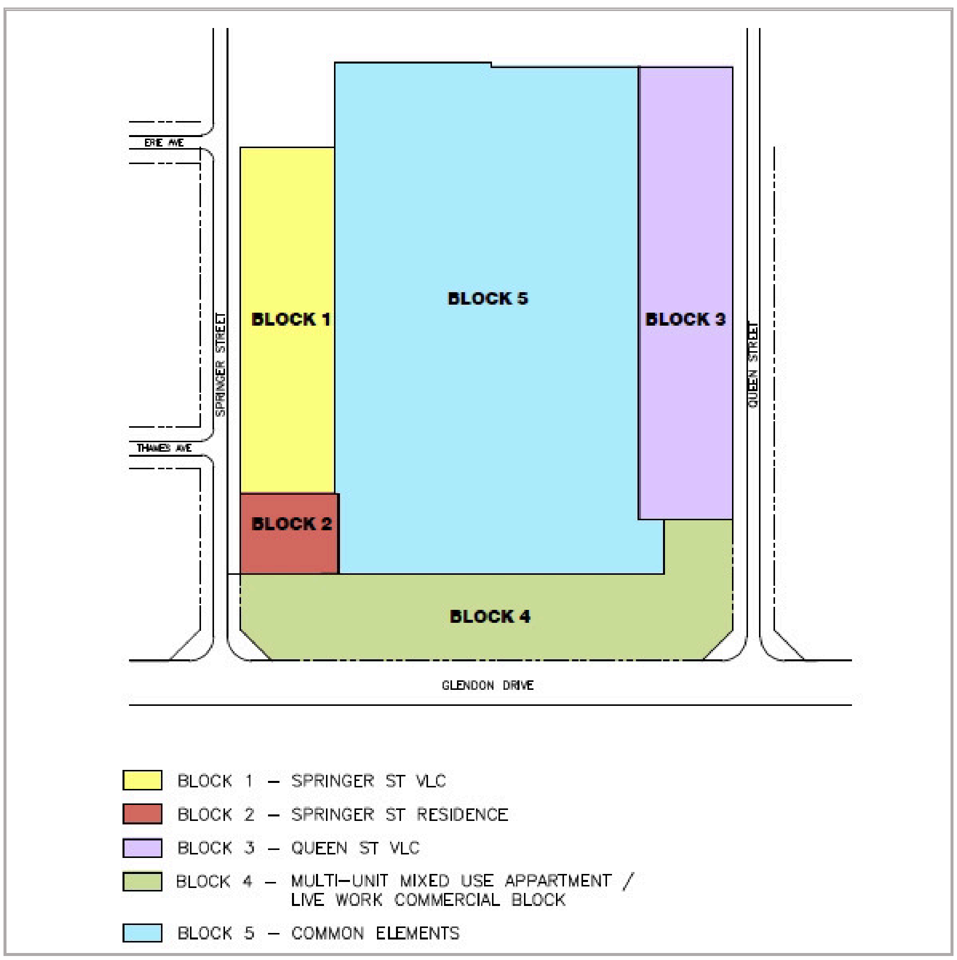
Appendices

1. Introduction

As requested, exp Services Inc. (**exp**) has prepared a Geotechnical Assessment Report for the proposed residential development a geotechnical investigation in conjunction with the proposed residential subdivision to be located at the northeast corner of Springer Street and Glendon Drive, in Komoka, Ontario.

It is understood that the proposed development will be undertaken with a phased approach. Figure 1 (below) denotes the proposed parceling for the overall site. Block 1 which fronts onto Springer Street will comprise of a Vacant Land Condominium Block. Block 2 consists of the existing residence, which is expected to remain. Block 3 which fronts onto Queen Street will comprise of a vacant land condominium block. Block 4 which fronts onto Glendon Drive along the south side of the site is expected to be comprised of a multi-storey (3 to 5 storey) apartment style condominium with retail space in the lower levels. Block 5 will comprise of the existing pond area.

Figure 1: Phased Development Approach



The development in Blocks 1, 3 and 4 is expected to require earthworks to complete some filling which will encroach on the existing pond limits to accommodate future buildings.

This report is intended to provide specific comments regarding Block 1, which is expected to involve the creation of 8 lots fronting onto Springer Street. However, information is also provided from a geotechnical standpoint for the other Blocks which are expected to follow as the overall proposed residential development plan moves forward.

1.1 Terms of Reference

Authorization to proceed with the investigation was received from Mr. Laverne Kirkness of Kirkness Consulting, on behalf of Springer Pond Developments.

The purpose of the Geotechnical Assessment was to examine the details of the proposed development, and the existing geotechnical and environmental work which has been done at the site, to prepare a consolidated Geotechnical Report which presents a summary of the existing soil and groundwater conditions at the Site, and provides geotechnical comments and engineering guidelines for the proposed residential development. The background information used in supporting the preparation of this report is outlined in Section 1.2.

More specifically, this report provides comments on site preparation and earth grading, confirmation of soil bearing capacity and foundation design recommendations, excavations, groundwater control, seismic design considerations and recommendations for site servicing, and pavement design.

This report is provided on the basis 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.

1.2 Background Information

The preparation of this report has relied upon existing work and technical reports prepared by other consultants. A list of the relevant documents which have been reviewed by exp is provided below:

Technical Reports

- Preliminary Geotechnical Information, April 28, 1998, prepared by Golder Associates Ltd.
- Geotechnical Comments, June 19, 2002, prepared by Golder Associates Ltd.
- Uncontrolled Filling Operations, October 4, 2002, prepared by Golder Associates Ltd.
- Geotechnical Investigation, August 29, 2003, prepared by Golder Associates Ltd.

- Geotechnical Comments, January 9, 2004, prepared by Golder Associates Ltd.
- Preliminary Environmental Assessment, April 2, 2004, prepared by LAW Engineering (London) Inc.

In addition to the aforementioned technical reports, exp has also reviewed additional correspondence provided by the client, as it relates to the proposed development of the site. A list of additional documents is provided below for reference.

<u>Development Applications and Correspondence from Approval Authorities</u>

- Application for Consent to Place Fill, August 8, 2003, prepared by PlanCan Associates Inc.
- UTRCA Application for Consent #26-03 (letter), July 26 2004, prepared by Mark Snowsell of Upper Thames River Conservation Authority.
- Memorandum for Filling Activity, January 19, 2004, prepared by Mark Snowsell of Upper Thames River Conservation Authority.
- Applications of Consent B-9/13 and B10/13 (letter), March 14, 2013, prepared by Tracy Annett of Upper Thames River Conservation Authority.
- Email Correspondance from Don Riley, to Jeff Brick at UTRCA, March 21, 2013.
- Proposed Filling and Lot Creation, 45 Springer Street, Komoka (letter), March 25, 2013, prepared by Karen Winfield of Upper Thames River Conservation Authority.

2. Site and Subsurface Conditions

3.1 Site Location

The site is located at the the northeast corner of Springer Street and Glendon Drive, in Komoka, Ontario. In general, the proposed development encompasses a total area of approximately 6.76 hectares.

A key plan showing the location of the site is provided below on Figure 2, for reference.

Figure 2: Key Plan

Source: County of Middlesex digital mapping, available online at http://middlesex.maps.arcgis.com.

3.2 Site Description

The Site is roughly rectangular in shape and is bound by single family residential along the north side (fronting onto Ontario Avenue), by Queen Street to the east, Glendon Drive (also known as Middlesex County Road 14) to the south, and Springer Street to the west, which is in turn bordered by single family residential lands.

As mentioned previously, the site is approximately 6.8 ha in size. Within the central portion of the site, there is a large pond which measures approximately 4.6 ha in size. It is understood that the existing pond is a remnant from aggregate extraction activities which occurred at the site. This is further discussed in Section 3.3 of this report.

Site grades slope up slightly towards the north, with elevations outside of the pond area ranging from about Elevation 239 m near the existing residence in the southwest corner, to Elevation 242 m near the north end of the site, based on a review on MNR topographic mapping.

The site is currently occupied with a single family residence, which is near the southwest corner of the site. A series of select site photographs (taken in May 2016) are presented below for reference.

Photograph 1

Looking west along the south side of the site. Glendon Drive can be seen on the left of the photo.

Photograph 2

Looking east along the south side of the site. Glendon Drive can be seen on the right of the photo.

3.3 Review of Aerial Photographs

A review of historical aerial photographs was carried out for the area around the site. A copy of select photographs dating from 1950, 1971, 1978 and 2016 is provided in Appendix A.

Based on a review of the aerial photographs, the aggregate extraction at the site occurred after 1950. The aerial photograph from this period shows that the subject property is free of significant vegetation, and likely used for agriculture based on surrounding conditions. In addition, the later gravel pits which appear south of Glendon Drive and west of Komoka Road are also not present in 1950.

By 1971, the ponds resulting from gravel extraction on the subject lands, and lands to the south of Glendon Drive and west of Komoka Road are present.

3.4 Review of Aggregate Resource Mapping

A review of the Aggregate Resource Inventory was carried out, given the former aggregate extraction activities that have occurred at the site. A reference for the reviewed document is provided below:

- Aggregate Resources Inventory of Lobo Township, Middlesex County; Ontario Geological Survey, Aggregate Resources Inventory Paper 58, 33 p., 6 tables, 3 maps, scale 1:50 000, 1981.

The report identifies that the sand and gravel deposits of Lobo Township are the product of glacial activity which occurred during the Late Wisconsinan. The two primary resource areas in the village of Komoka are outwash deposits associated with the deltas of glacial Lakes Maumee and Whittlesey.

Based on a review of the mapping, the aggregate deposits in the area of the site are identified as outwash gravel deposits (greater than 35% gravel), with average thicknesses of greater than 6 m. Quality indicators for the deposit indicate the present of silt, as well as oversize particles. These indicators are consistent with outwash deposits which are deposited near the margin of a glacier, resulting in some variability in the texture and composition of the aggregate.

3.5 Summarized Soil Conditions (from Previous Studies)

The detailed stratigraphy encountered in each borehole and the results of routine laboratory tests carried out on representative samples of the subsoils are presented on the attached borehole logs. It must be noted that boundaries of soil indicated on the logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purposes of geotechnical design and should not be interpreted as exact planes of geological change.

The subsurface soil conditions encountered in the test pits and boreholes are summarized below.

3.2.1 Block 1 – fronting onto Springer Street

In April 1998, Golder Associates oversaw the excavation of a series of test pits located within the limits of the area currently denoted as Block 1. The test pits (denoted as Test Pits TP3 to TP7) are provided in Appendix B, for reference.

The test pits encountered 0.8 to 4.3 m of sand and gravel fill material. The fill material was observed to contain silt, trace topsoil, occasional cobbles and occasional concrete pieces. The fill material was in a moist to wet state, with an insitu moisture content recorded between 10 and 18 percent.

The fill material was observed to be overlying natural sand, or sand and gravel soils. The natural deposits were observed to be in a wet state, with insitu moisture contents in the range of 9 to 18 percent.

Groundwater seepage was observed in the open excavation at about 1.8 to 4.0 m depth.

3.2.2 Block 2 – existing residence

In April 1998, Golder Associates oversaw the excavation of a series of test pits at the site, of which, Test Pit TP 2 was located within Block 2 where the existing residence is located. A copy of the test pits log is provided in Appendix B, for reference.

A supplemental Geotechnical investigation was conducted by Golder Associates in 2003, at which time two boreholes were advanced in the area of the residence. Boreholes 1 and 2 are provided in Appendix B for reference.

The surficial soils in Block 2 were generally comprised of 3.0 to 6.0 m of fill material. The fill material ranged in composition from sandy silt to sand and gravel, and was observed to contain topsoil, organic inclusions and pieces of concrete block. Within the boreholes, the fill was verified to have a loose relative density, with Standard Penetration (SPT) N values in the range of 1 to 7 blows per 0.3 m penetration of the split spoon sampler. The fill material was in a moist to wet state, with insitu moisture contents typically ranging from 12 to 22 percent.

The fill material was observed to be overlying natural sand and/or sand and gravel soils with occasional cobbles and boulders. The natural soils were generally found to be in a compact to very dense state, with SPT N-values greater than 36 blows. The sand, and sand and gravel soils were generally in a wet state, with insitu moisture contents recorded in the range of 10 to 20 percent.

Groundwater seepage was observed in the open test pit and boreholes at about 3.5 to 4.0 m depth.

3.2.3 Block 3 – fronting onto Queen Street

At this time, no test pits or boreholes have been recorded in the lands which are identified as Block 3. Based on the review of aerial photographs and the limits of the pond which have been recorded at the site, it is anticipated that fill material, similar to that encountered onsite, is present, overlying the natural sand and/or sand and gravel subgrade soils. The extent and depth of the fill has not been verified at this time.

3.2.4 Block 4 - fronting onto Glendon Drive

In April 1998, Golder Associates oversaw the excavation of a series of test pits at the site, of which, Test Pit TP1 was located within at the west end of Block 4. A copy of the test pits log is provided in Appendix B, for reference.

Test Pit 1 encountered 2.4 m of sand and gravel fill material. The fill material was observed to contain intermittent topsoil inclusions. The fill material was in a moist to wet state, with an insitu moisture content recorded at about 25 percent.

The fill material was observed to be overlying natural silty sand soils. Some black organic staining was noted at the interface between the fill and the silty sand. The sand was in a wet to saturated state, with insitu moisture content tested at 25 percent.

Groundwater seepage was observed in the open excavation at about 2.2 m depth.

3.2.5 Block 5 – Pond Area

Based on a review of the existing reports and correspondence which has been provided for the site, it is understood that the pond depth has a maximum depth of approximately 4 m, and pond side slopes are relatively gentle, with approximate inclinations of 3 horizontal to 1 vertical or less. Based on the test pits on the adjacent lands, the depth of the water is consistent with the fill thicknesses recorded in the test pit logs. Deeper sections of the pond in localized areas may be present in the base of the pond due to variations in extraction methods from the original gravel extraction.

A survey of the pond depths and sediment depths in the pond has not been carried out at this time.

3.3 Shallow Groundwater

In the previous investigations, shallow groundwater seepage was observed in the open test pits and boreholes, at the depths noted in the previous section of the report. In general the water seepage was generally found to be slightly below the water level observed in the pond.

Within Borehole 2 (located in Block 2), the stabilized water level was measured in a standpipe, and was recorded at about 4.3 m below ground surface, at an Elevation of 236.0 m. In comparison to the water level observed in the pond, the water level observed in the standpipe was 2.5 m lower than the pond.

It is 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 high levels in wet seasons. Capillary rise effects should also be anticipated in fine-grained soil deposits.

3.4 Environmental Considerations

In April 2004, LAW Engineering (London) Inc. produced an Environmental Assessment Report with regards to the analytical characteristics of the fill material which was stockpiled onsite at that time, and intended for use to fill in portions of the pond to extend rear yard areas for the proposed blocks.

Six random samples of the fill material were secured by LAW Engineering, and analytical testing for metals, BTEX, and petroleum hydrocarbons was carried out. The test results were compared to the MOE Guideline for Use at Contaminated Sites in Ontario (June 1996, updated September 1998), and no exceedances were detected in the samples. A copy of the Environmental Assessment Report is provided in Appendix C, for reference.

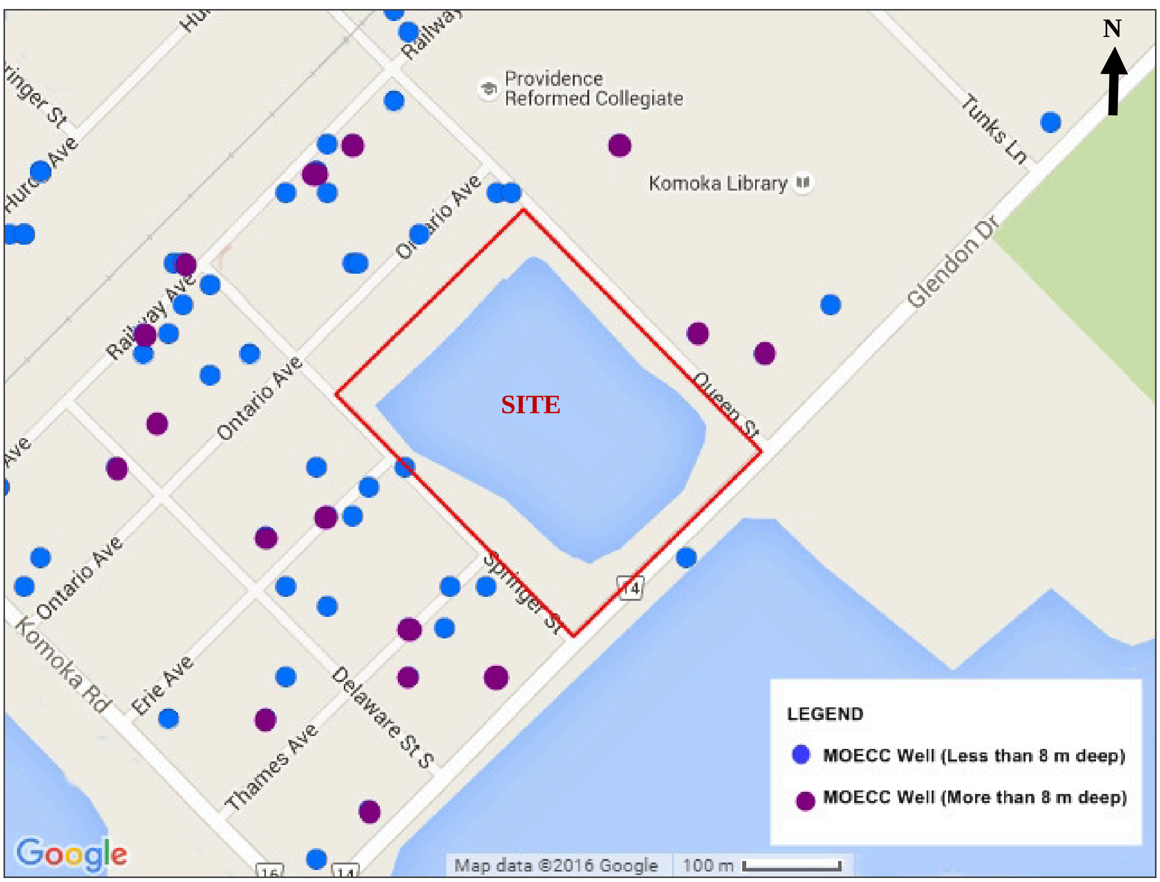
It is understood that the source of the fill material was imported from construction sites in the west end of London. The composition of the imported fill material generally consisted of sandy silt with trace to some gravel. The fill material was deemed to be suitable for use as bulk fill. In this regard, filling in rear yard areas with this material, where settlements could be tolerated was deemed appropriate.

3.5 Review of MOECC Well Records

Information regarding potable wells located within the limits of the site, and a distance of approximately 250 m from the site was examined on a cursory level, to collect information on the potable aquifers which provide source water to existing wells.

An overall plan of the area showing the closest wells, recorded by the Ministry of Environment and Climate Change (www.ontario.ca/environment-and-energy/map-well-records) is shown below on Figure 3.

Figure 3: MOECC Well Record Locations



A number of the wells are present at shallow depths (typically set at 5 to 8 m deep) in the overburden sand or sand and gravel soils which are present in the area. A number of deeper overburden wells are also present, with depths ranging from 8 to 30+ m depth, as denoted in the above figure.

This information confirms that shallow groundwater conditions encountered in the test pit and boreholes which were advanced at the site are consistent with shallow groundwater conditions present in the overall area.

4. Discussion and Recommendations

4.1 General

It is understood that the proposed development will be undertaken with a phased approach. Figure 1 (provided in Section 1 of this report) denotes the proposed parceling for the overall site.

- Block 1 which fronts onto Springer Street will comprise of a Vacant Land Condominium Block.
- Block 2 consists of the existing residence, which is expected to remain.
- Block 3 which fronts onto Queen Street will comprise of a vacant land condominium block.
- Block 4 which fronts onto Glendon Drive along the south side of the site is expected to be comprised of a multi-storey (3 to 5 storey) apartment style condominium with retail space in the lower levels.
- Block 5 will comprise of a portion of the existing pond, remaining in the central part of the site.

The various phases of the proposed development are expected to have full municipal servicing. The following sections of this report provide geotechnical recommendations regarding site preparation, excavations and groundwater control, foundation options, site servicing, seismic design considerations and pavement design.

4.2 Regulatory Approvals

It is understood that consultation at various stages of the proposed development has occurred with the local municipality and the Conservation Authority. In March 2013, the Upper Thames River Conservation Authority (UTRCA) issued a letter identifying the applicable regulations and requirements which are assigned to the Conservation Authority through the Conservation Authorities Act (O.Reg. 157/06). A Sectoon 28 Development, Interference with Wetlands and Alterations to Shorelines and Watercourses permit / approval will be required from UTRCA.

Because of the site of the existing pond (greater than 2 hectares in size), it is understood that the natural hazard policies which are typically associated with Riverine Flooding and Riverine Erosion Hazards also apply to the area around the existing pond. The existing pond is the result of previous aggregate extraction activities at the site, and are therefore considered man-made. In this regard, it is understood that the Conservation Authority will review the proposed development adjacent to the pond to ensure that geotechnical information is provided to support the proposed building locations and foundations. In this regard, this report provides recommendations for site preparation work associated with filling in portions of the pond, and foundation design for proposed buildings in proximity to the pond area.

In addition, this report provides comments on lot drainage; condition, stability and improvements to existing slopes or new slopes created as a result of filling activities, and groundwater control for open excavations. Recommendations are also provided for sediment control measures during construction.

The Conservation Authority will also require a Hydrogeological Assessment to characterize the hydrogeological setting at the site. Since the site has aspects of surface water and shallow groundwater which appear to be inter-related, this work is expected to be carried out and presented under a separate report. In this regard, the following scope of work is suggested (at a minimum):

- Review of existing geologic mapping and available information for the site.

- Review of local MOECC Well Records. A preliminary review of the area within 200 m of the site limits indicates the presence of a large number of wells which are considered to be relatively shallow depth (less than 7 m). Although many of these properties have connection to municipal water service, it is anticipated that some of these wells may still remain, and may be in use. A well survey within 200 m of the site is advised to confirm the presence and type of use of the shallow wells which may be considered potential receptors, susceptible to impact from the proposed development.

- Installation of Monitoring Wells (in accordance with O.Reg. 903) to document stabilized water levels at the site. It is recommended that at least 3 wells be installed in each Block (Blocks 1, 3 and 4). The wells are expected to be used to conduct single well response tests to assess the permeability of the soils where shallow groundwater is present. Where practical, seasonal water level measurements should be recorded to document stabilized groundwater levels and seasonal changes that may occur at the site. Strategically placed wells can also be used for long-term monitoring and water sampling if required.

- Water quality samples of the shallow groundwater should be collected, and compared to water quality samples taken from the surface water at the site. Parameters for water quality testing should include general chemistry (RCAP comprehensive), O.Reg. 153 petroleum hydrocarbons, O.Reg. 153 organic compounds, coliform, E.coli and heterographic plate count.

- The remainder of the fieldwork and associated report should follow the Conservation Ontario Guidelines for Hydrogeological Assessment Submissions. Additional site specific discussion and recommendations required for the report should be confirmed by the Conservation Authority.

Consultation with the Conservation Authority is recommended to confirm the suitability of the recommended scope of work.

4.3 Site Preparation

4.3.1 Existing Wells

Prior to further development at the site, it is anticipated that monitoring wells may be present in the work area. Any monitoring wells which are not maintained for long term groundwater monitoring at the site should be properly decommissioned by a licensed contractor, in accordance with O. Reg. 903. Where wells are being maintained for monitoring purposes, the condition of the well covers should be reviewed to ensure that the wells are suitably protected, and that the integrity of the wells can be maintained.

4.3.2 Filling Activities

In order to accommodate the proposed building areas and rear yard space for the proposed buildings, it is anticipated that some filling activities will be required. There is not a sufficient resource of stockpiled material at the site presently to carry out this filling work. Given the size of the pond and the existing volume of water within the pond area, draining or dewatering the pond area is not considered economically viable. As such, the fill placed in the rear yard and landscaped areas which encroach on the pond should be treated as bulk fill. In this regard, the fill is generally not considered suitable to support buildings or structures, without suitable subgrade enhancements.

Therefore, the following recommendations are provided for filling work which is expected to take place in the future:

- Potential sources of fill material should be reviewed from a geotechnical standpoint. It is recommended that fill materials be comprised of natural mineral soils, with minimal amounts of topsoil and organics. Material containing construction debris or deleterious material is not recommended for use as bulk fill. It is also understood that as part of the UTRCA permit/approval process, the Conservation Authority will require details on the types of materials proposed for use for filling activities within the pond.

- Where possible, sand or sand and gravel soils would be considered prudent for use, to minimize sediment plumes into the pond when fill material is placed. In the event that silty soils are utilized for filling, turbidity/TSS testing is recommended during the course of the work. Turbidity testing post construction may also be considered, to confirm when the water turbidity returns to baseline conditions.

- The geotechnical consultant should review the imported materials to confirm the stable slope configuration for the fill placed below the water level, to ensure that pond slopes are in a stable condition. For example, saturated sand and gravel soils may be considered stable with slope inclinations of 3H:1V, whereas silty soils may require a more gentle slope under fully saturated conditions to provide a comparable level of stability.

- During the fill placement, the water level in the pond should be monitored regularly (weekly) to confirm that there are no significant changes in the water level as work proceeds. A maximum change in water level of 0.3 m is recommended as a target. Where changes exceed that level, the filling operations should be halted and reviewed to identify if the changes are related to the site works.

- Consideration may be given to removing a build up of sediment from the base of the pond, to ensure continued connectivity to the shallow groundwater contained within the natural sand or sand and gravel soils in the area. As indicated previously, the disposal of any excess excavated materials must conform to the MOE Guidelines and requirements. **Exp** can be of assistance if an assessment of the materials is required.

- Where disturbed subgrade soils are exposed in proximity to the pond, sediment control measures (such as robust silt fence, straw bales) should be placed to limit sediment-bearing surface water flows discharging directly to the pond area.

- It is recommended that site grading in each of the blocks be designed to direct surface water from paved areas and site pavements away from the pond. Grading in rear yard and landscaped areas may be designed to allow surface water flows towards the pond into areas where the flows can be controlled.

4.3.2 Subgrade Improvements

Buildings are expected to be founded on deep foundations. A number of alternatives may be considered for the site, and are discussed in Section 4.5. However, it is expected that existing site pavements may be located in areas where fill may be present. As such, subgrade improvements may be required to ensure that the condition of the fill is sufficient to provide adequate support for the site pavements.

Within the pavement areas, and prior to placement of site services, 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. Any fill placed for structural support below site pavements and site servicing should consist of clean (i.e., free of organics and/or debris), compactable, inorganic soils with a moisture content within about 3 percent of optimum, as determined by standard Proctor testing. The structural 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 a minimum of 98 percent Standard Proctor Maximum Dry Density (SPMDD).

Where structural fill is placed to help bridge over soft or wet fill soils, a minimum thickness of 1.0 m of granular material is recommended. In this regard, material meeting OPSS 1010 gradation requirements for Granular B (Type II) aggregate is recommended.

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 used 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 standards for placement and transportation.

4.4 Excavation and Construction Dewatering

4.4.1 General

Side slopes of temporary excavations must conform to Regulation 213/91 of the Occupational Health and Safety Act of Ontario. The fill, natural sand and natural sand and gravel soils at the Site are classified as Type 3 soils above the stabilized water level. Below the stabilized water table, these soils may be expected to behave as Type 4 soils.

It is expected that most excavations will extend through Type 3 soils and therefore, must be cut back at a maximum inclination of 1H:1V from the base of the excavation. Without groundwater control, excavations within the Type 4 soils must be cut back at a maximum inclination of 3H:1V from the base of the excavation.

It should be noted that the presence of concrete block pieces in the existing fill material and cobbles and boulders in natural deposits may influence the progress of excavation and construction.

During excavation for the proposed development, care should be taken to not undermine any existing site services or structures. In the event that soils below existing foundations are disturbed, some method of temporary support or underpinning may be required. **Exp** can provide additional assistance in this regard, if necessary.

4.4.2 Excavation Support

The recommendations for side slopes given in Section 4.4.1 would apply to most of the conventional excavations expected for the proposed development. However, in areas adjacent to existing structures and 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 soils, the set back 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 20.4 kN/m^3 may be assumed;
- h = depth of point of interest in m;
- q = equivalent value of any surcharge on the ground surface in kPa.
- K_a = earth pressure coefficient, assumed to be 0.4

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.

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 them to the engineer for review and comment.

4.4.3 Construction Dewatering

Based on the results of the field investigation, moderate groundwater infiltration should be anticipated within conventional depths for service trench excavations. It is expected that minor groundwater infiltration can likely be accommodated using conventional sump pumping techniques. Where groundwater infiltration persists, more extensive dewatering measures may be required. Exp would be pleased to provide additional comments and recommendations for dewatering these soils, when additional design information is available. It is also recommended that contractors bidding on the work conduct further investigation including test pits to further determine groundwater conditions and how it will affect their work.

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.

It is important to mention that for any projects requiring positive groundwater control with a removal rate in excess of 50,000 litres per day, a Permit to Take Water (PTTW - Groundwater) will be required. This may be required at the Site depending on final invert levels and the persistence of groundwater from the lower sand deposit(s). PTTW applications will need to be approved by the Ministry of Environment according to 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.

4.5 Building Foundations

Due to the presence of fill material within the proposed building foot prints in Block 1, and the expected presence of fill to be placed in Blocks 3 and 4, it is recommended that building designs incorporate the use of deep foundations to ensure that suitable support is provided for the structures. Detailed design will require additional site—specific borehole information to confirm the depth to competent founding soils.

Until this additional information can be provided, the following section of the report provides preliminary comments for deep foundation alternatives which may be considered.

4.5.1 Deep Foundation Alternatives

Driven Piles – Block 1, Preliminary Comments

Pressure treated timber piles may be considered for supporting the proposed building in Block 1. Within Block 1, the existing test pit information confirms that fill is present to depths of 4.3 m below existing grade. Some variation in this overall fill thickness is anticipated between the test pit locations, and since additional fill placement may have occurred since the time when the test pits were excavated. From a preliminary standpoint, the following comments are provided, assuming that dense founding soils are present at depths of about 6.5 to 8.0 m below grade.

Timber piles can be driven through the existing fill material using a drop hammer or small diesel hammer in order to minimize damage to the piles as they are advanced into the ground. The piles should be driven down to the competent subgrade soils.

Timber piles with a minimum tip diameter of 150 mm and a nominal butt diameter of 200 mm driven to the dense sand and gravel (below the fill) will provide a bearing capacity of approximately 500 kN.

A driving criterion in the form of minimum blows per inch of pile penetration at final set will be required prior to pile installation. The above bearing value is based on using a piling hammer with a rated energy of 13,500 joules, and achieving a final set of maximum 5 blows per 25 mm. The final setting criteria for the piles should be determined during the pile driving operation based on the performance of the pile hammer, and correlation using a dynamic pile driving formula (Hiley formula).

The piling operation should be observed and inspected by a Geotechnical consultant. The contractor should survey the top of the piles to confirm that uplift has not occurred during the pile driving operation. Any piles which show evidence of uplift or movement should be re-tapped to the design level.

It must be noted that the presence of cobbles and boulders in the natural subgrade soils may influence the progress of installation of piles.

Driven Piles - Block 3 and 4, Preliminary Comments

For the larger buildings proposed in Block 3 and Block 4, consideration may be given to a more robust driven pipe pile.

Site specific boreholes within the building foundation for structures in Block 3 and 4 are recommended to confirm the depth of competent founding soils, and to determine suitable design depths for pile foundations.

Piles may be set into the dense to very dense sand and gravel soils, or alternatively, may be extended down to greater depths, to provide bearing on the bedrock level. The required bearing capacity for the building will dictate the depth of the piles to be installed.

For preliminary design guidance, it is considered that a 245 mm diameter pipe pile for instance with minimum wall thickness of 9.5 mm driven to practical refusal with a pile driving hammer of rated energy 39 kJ, into the sound bedrock would support a design load of 1000 kN. Larger diameter piles can be used if higher loads are required. For H-piles steel plates can be welded vertically to the lower section of the pile to increase the contact area.

Experience indicates that the maximum stresses developed in driven piles occur during installation. Thus, the wall thickness should be sufficient to prevent damage during installation. It may be economical to allow the piling contractor to select the optimum wall thickness for steel pipe piles with the acceptance by the piling contractor that any piles damaged during installation should be replaced by the contractor at no cost.

The minimum centre-to-centre pile spacing should be 3 pile diameters or greater for driven steel piles, otherwise the piles must be considered as a group and the total capacity may need to be reduced from that determined on the basis of single piles.

Where significant grade changes induce settlement of overburden, down drag force must be considered. The potential maximum down drag is equal to the positive soil - pile skin friction since pile is founded on bedrock. The pile section must be structurally able to handle the capacity and the down drag force.

Pipe piles displace relatively large volumes of soil during their installation. When driven in a group or cluster, they tend to jack up adjacent piles already driven. Consequently, the elevation of these piles should be established immediately on driving and again after all the piles in the group have been installed to determine if heaving has occurred. If so, the piles must be re-driven below the original level to the specified set. Alternatively, all piles in the group should be re-tapped after completion of the group.

A driving criterion in the form of minimum blows per inch of pile penetration at final set will be required prior to pile installation. A maximum blows per inch should also be determined prior to pile installation to reduce the risk of pile damage due to overdriving. The criteria for practical refusal and production driving are a function of the driving equipment and pile dimensions. These criteria to be used can be provided by Trow upon request, using the wave equation analyses (WEAP), once the hammer energies and pile details are known

Full time inspection by a representative of this office will be required during pile driving. It cannot be overemphasized that competent pile driving inspection involving a site inspector in conjunction with and under the direction of a qualified piling engineer is required to minimize the possibility of damage to piles due to over driving, to ensure their proper placement and penetration to firm bearing, to minimize danger of under driving and to maintain adequate records of the installations. For each pile, a complete driving record should be obtained by the inspector and reviewed during pile installation by the designer.

It must be noted that the presence of cobbles and boulders in the natural subgrade soils may influence the progress of installation of piles.

Helical Piers – Preliminary Comments

A deep foundation system such as helical piers (screw piles) may be considered for buildings at the site. This foundation scheme can be considered for areas where underpinning may be necessary. The following section will discuss the underpinning requirements further. The piers can be 'twisted' into the underlying glacial till by portable hydraulic units.

The helical pier comprises one or multi helices on the end of a small diameter solid steel shaft. The steel helices are screwed into the ground to the level of competent bearing soil. Based on the soil and groundwater conditions expected at the site, helical pier systems should be installed through the fill material and into the compact to dense sand or sand and gravel soils. Additional borehole information will be required at specific building locations to confirm the depth of the competent subgrade soils.

The support capacity and installation procedures should conform to the manufacturer's specifications. For a preliminary reference, the following equation may be applied for determining the vertical capacity of a single helical pier installed in sandy, gravelly and silty soils.

$$Q_u = (N_q \gamma' H)\frac{\pi(D^2 - d^2)}{4}$$

Where:

Qu = ultimate compressive load capacity (kN)

Nq = bearing capacity coefficient

H = height of soils above the helix plate (measured from the surface in the case of the upper-most helix and from the bottom of upper helix to the top of the lower helix in the case of multiple helix piles)

D = diameter of the helix

d = diameter of the shaft

γ' = effective soil unit weight

Where multiple helixes are utilized on a screw pier, the bearing capacity can be increased accordingly, and additional calculations are required. To determine the allowable capacities, a suitable factor of safety (at least 2.5) should be applied to the ultimate values. The design and installation of the helical piers should be done by specialist contractors and in accordance with the Canadian Foundation Engineering Manual.

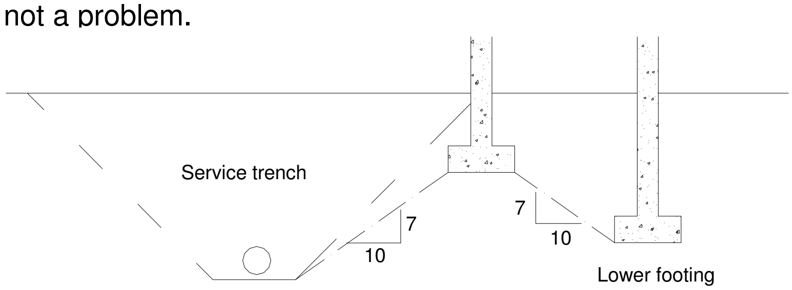
Full time inspection by the geotechnical consultant will be required during pier installation. It cannot be overemphasized that competent pier installation inspection involving a site inspector in conjunction with and under the direction of a qualified pier engineer is required to ensure their proper placement and penetration to firm bearing, to minimize danger of under driving and to maintain adequate records of the installations. For each pier, a complete record should be obtained by the inspector and reviewed during pier installation by the designer.

4.5.2 Conventional Spread and Strip Foundations

It is understood that a commercial building may be located at the west end of Block 4, fronting onto Glendon Drive. A previous test pit in this area has documented the presence of 2.4 m of sand and gravel fill material. Although the remainder of the proposed development in Block 4 is expected to be support on deep foundations, there may be opportunity to partially excavate and re-compact the granular fill material to provide sufficient stability for conventional building foundations for the smaller commercial building. Confirmation of the density of the fill through additional site specific boreholes or dynamic cone penetration tests is recommended. Drawing 1, in Appendix D shows the geometric requirements for engineered fill placement.

Once the subgrade soils are deemed suitable, the commercial building may be supported on conventional spread and strip footings founded directly on the natural mineral soils, or on approved, recompacted engineered fill. An allowable bearing pressure of 145 kPa (3,000 psf) can be used for design below a typical depth of approximately 1.2 m (4 ft) below existing grade throughout the Site. 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.

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 excavations, 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.

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 (i.e., where more specific information becomes available with respect to conditions between test locations 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.

4.6 Basements

Shallow groundwater levels at the site have been recorded at depths of 1.8 to 4.0 m below existing grade. A Hydrogeological Assessment is expected to be conducted for the site, and can confirm the stabilized groundwater level at the site. Monitoring wells installed as part of that Assessment can be used to record changes in water levels as additional fill activities and site preparation work is carried out.

Where possible, basements should be designed such that the basement floor slab is above the seasonal high groundwater level. Where deep foundation alternatives are used to support the buildings, the basement or lowest level is expected to be comprised of a structural slab tied to the foundation units.

A minimum 200 mm (8 inch) thick compacted layer of 19 mm (¾ inch) clear stone should be placed between the exposed subgrade and the floor slab to serve as a moisture barrier.

The installation and requirement of vapour barrier under the slab, where applicable, should conform to the flooring manufacturer's and designer's requirements. Relative humidity and/or moisture emission testing may be required to determine the concrete condition prior to flooring installation. Ongoing liaison from this office is available, upon request.

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 20.4 kN/m^3 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 recommended for any basements constructed 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 2.

4.7 Site Servicing

The subgrade soils beneath the water and sewer pipes (installed at conventional depths) which will service the site are generally expected to consist of recompacted silt/sand fill material or natural mineral soils. No bearing problems area anticipated for flexible or rigid pipes founded on the native deposits or re-compacted approved subgrade soils.

The bedding course may be thickened if portions of the subgrade become wet during excavation. The bedding aggregate should be placed around the pipe to at least 300 mm (12 inch) above the pipe. The bedding aggregate should 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.

Clear stone or crushed stone bedding may be used in the service trenches as bedding below the spring line of the pipe if necessary to assist groundwater control and provide stabilization to the excavation base in wet silty soils. Geotextile should be wrapped around the stone bedding to minimize migration of fines. The potential locations for use of stone bedding should be identified during construction and is expected to vary across the site due to seasonal conditions and variations in the perched groundwater.

A summary of the general recommendations for trench backfill is presented on Drawing 3. 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, much of the excavated natural soils may be used for construction backfill, provided that reasonable care is exercised in handling as discussed previously. In this regard, the material should be within 3 percent of the optimum moisture as determined in the Standard Proctor density test. Stockpiling of material for prolonged periods of time should be avoided. This is particularly important if construction is carried out in wet, adverse weather.

Soils excavated from below the stabilized groundwater table may be too wet for re-use as backfill unless adequate time is allowed for drying, or if the material is blended with approved dry fill; otherwise, it may be stockpiled on the Site for re-use as landscape fill. The use of any imported material is subject to review and approval by the contract administrator and geotechnical consultant.

Disposal of excavated materials off site should conform to current Ministry of Environment guidelines.

4.8 Earthquake Design Considerations

The recommendations for the geotechnical aspects to determine the earthquake loading for design using the OBC 2006 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 2006. Excluding the topsoil, the subsoils expected in the proposed building footprints will generally consist of sand, silty sand, sandy silt till, and clayey silt till. It is anticipated that the proposed structures will be founded on these deposits, below any loose or soft zones.

Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2006 indicated 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 ranged from 3 to 9 m depth. 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 2006. 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 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 zones 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 street classifications, a typical design life of 15 years, and the anticipated subgrade soil conditions.

Recommended Pavement Structure Thickness

Pavement Layer	Compaction Requirements	Light Duty Pavements	Roadways and Heavy Duty Pavements
Asphaltic Concrete	97% Marshall Density	40 mm HL-3 50 mm HL-8	45 mm HL-3 60 mm HL-8
Granular ‘A’ (Base)	100% SPMDD*	100 mm	150 mm
Granular ‘B’ (Subbase)	100% SPMDD*	300 mm	350 mm

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

The recommended pavement structures should be considered for preliminary design purposes only. 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.

Depending on the staging of the subdivision development, and possible areas of concentrated construction access routes, additional granular thicknesses may also be considered. 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' aggregates should be checked for conformance to OPSS 1010 prior to use 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 1150. The asphalt should be placed in accordance with OPSS 310 and compacted to at least 97 percent of the Marshall mix design bulk density.

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 catchbasins. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas. In low areas (at catchbasin locations), subdrains should be installed to intercept excess subsurface moisture and prevent subgrade softening. The locations and extent of subdrainage 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.

4.10 Curbs and Sidewalks

The concrete for the curbs and gutters should be proportioned, mixed placed and cured in accordance with the requirements of OPSS 353, OPSS 1350.

During cold weather, the freshly placed concrete should be covered with insulating blankets to protect against freezing.

The subgrade for sidewalks should consist of undisturbed natural soil or well-compacted fill. A minimum 100 mm thick layer of compacted (minimum 98 percent SPMDD) Granular 'A' should be placed below sidewalk slabs.

5. General Comments

As noted in this report, additional work is required in assessing the Hydrogeological Setting for the site, and identifying potential impacts to sensitive surface water and shallow groundwater receptors in the area.

This geotechnical report provides recommendations for site preparation and pond filling activities which have been proposed for the site, to provide expanded areas to support building construction and landscaped areas around the buildings. Preliminary recommendations are provided for foundation alternatives which may be considered for future buildings. Since additional work is anticipated with pond filling activities and associated site grading work, exp recommends that additional fieldwork (such as supplemental test pits and boreholes) be conducted within specific building areas to confirm the details of the foundation design. However, based on the information which is available at this time, there are reasonable deep foundation options which can be considered for the proposed buildings.

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 preliminary design purposes.

Exp Services Inc. would be pleased to provide geotechnical review for design work associated with the proposed site development, to ensure that this report has been properly interpreted and recommendations have been suitably implemented. Assistance can also be provided to provide scoping recommendations, where additional investigation work is recommended.

We trust that this report is satisfactory to your present requirements and we look forward to assisting you in the completion of this project. Should you have any questions, please contact the undersigned at your convenience.

Appendix A

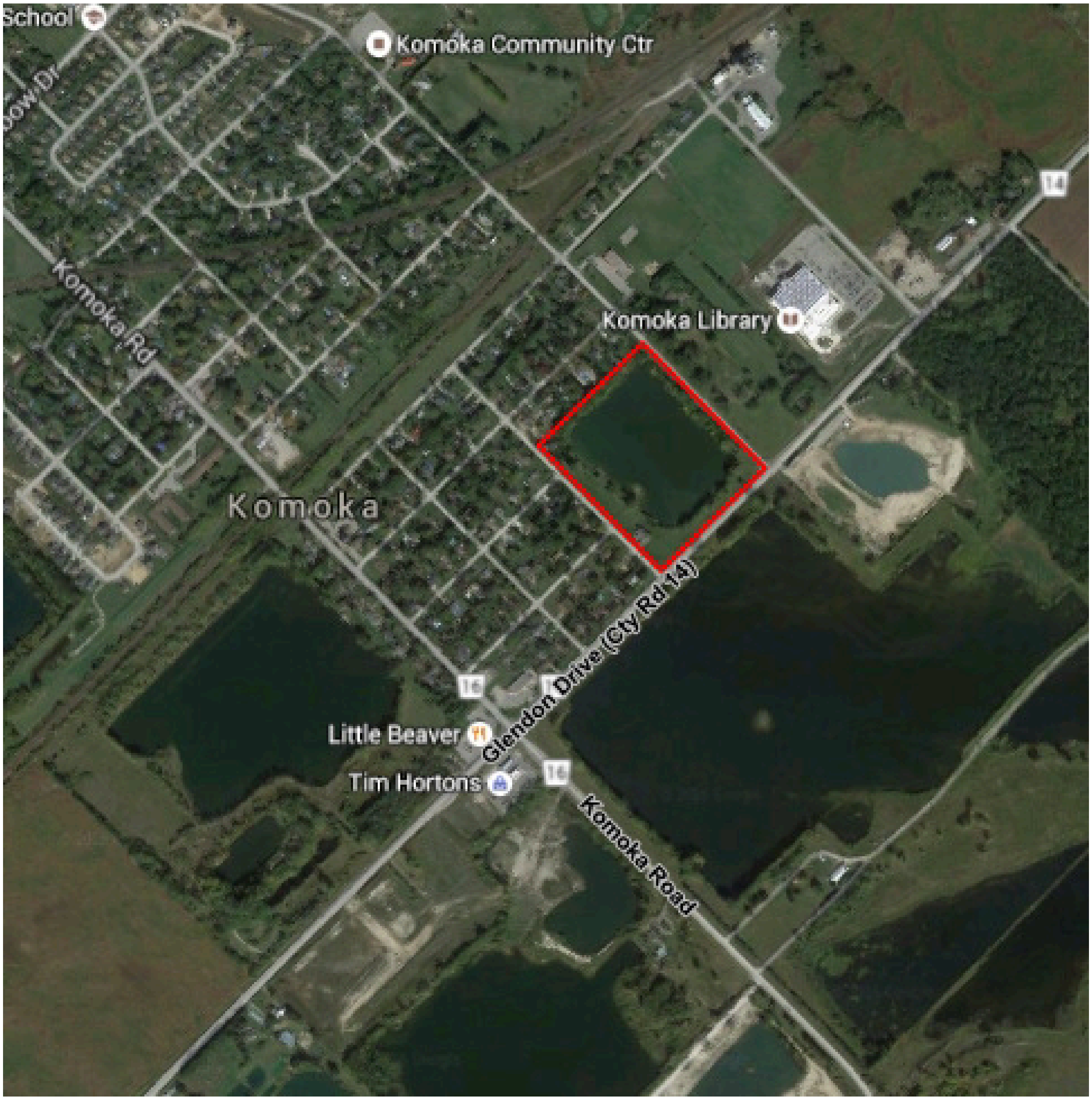
Aerial Photographs

1950 Aerial Photograph

1971 Aerial Photograph

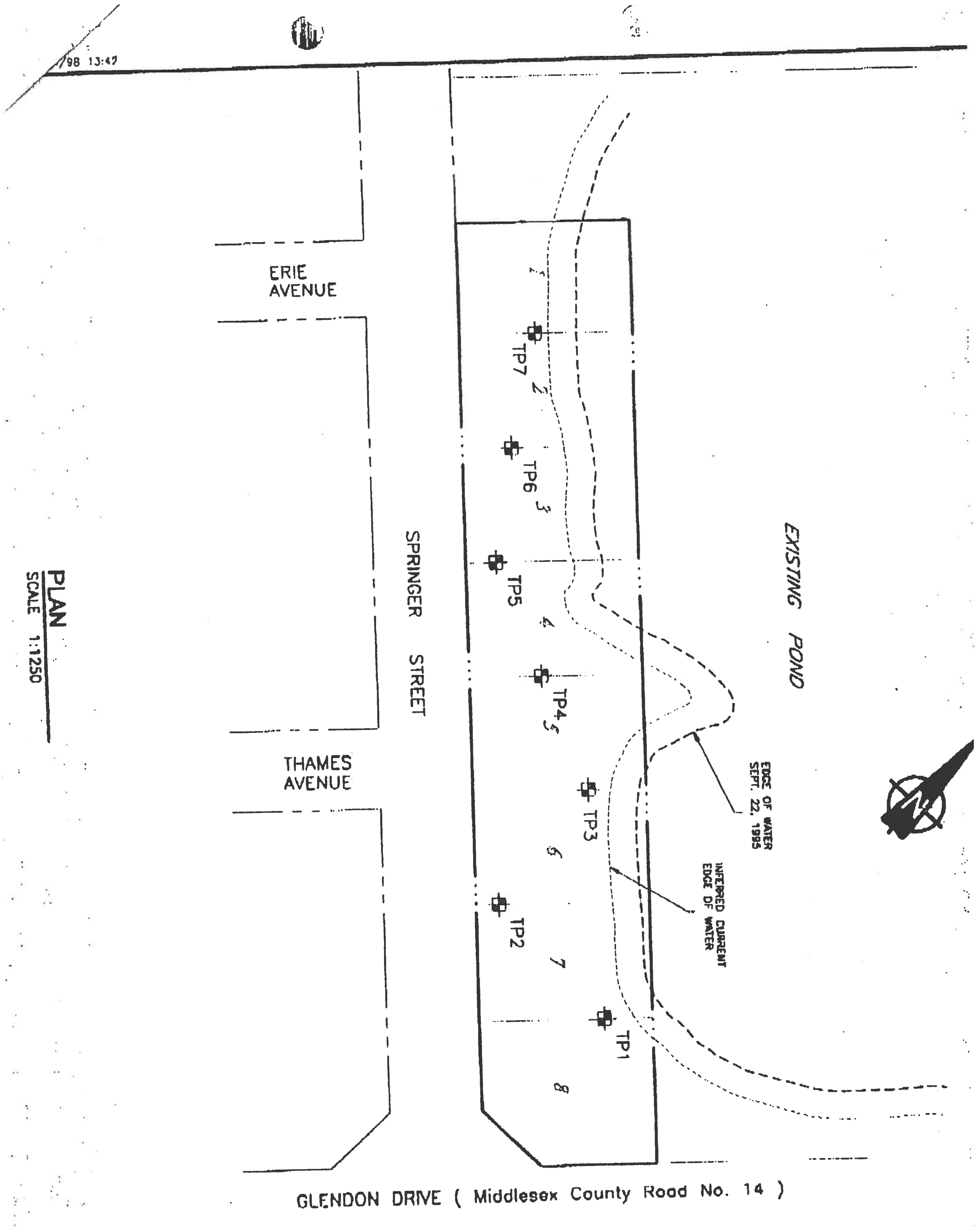
1978 Aerial Photograph

2016 Aerial Photograph



Appendix B

Boreholes and Test Pits (by Others)



ERIE AVENUE
THAMES AVENUE
SPRINGER STREET
EXISTING POND
PLAN
SCALE 1:1250
TP7
TP6
TP5
TP4
TP3
TP2
TP1
EDGE OF WATER SEPT. 22, 1995
INFERRED CURRENT EDGE OF WATER
GLENDON DRIVE (Middlesex County Road No. 14)

DBT-3066

CATION: REFER TO LOCATION PLAN

RECORD OF TEST PIT 1

EXCAVATION DATE: APR. 24, 1998

SHEET 1 OF 1

DATUM: LOCAL

DEPTH SCALE METRES	METHOD / SIZE	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SHEAR STRENGTH Cu, kPa (VANE TEST - +, PENETROMETER - ●)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT (Wp —○W— Wl; 10, 20, 30, 40)	ADDITIONAL LAB. TESTING	GROUND WATER CONDITIONS AND INSTALLATION
0		GROUND SURFACE		97.88 / 0.00						
0 – 2.40	Backhoe Dug; 0.5m x 0.8m x 3.3m	Brown sand some gravel & silt occasional topsoil lumps (FILL)		95.48 / 2.40	1	CS		○		
2.40 – 3.30		Grey SILTY SAND occasional gravel trace to some black organic sediment		94.58 / 3.30	2	CS		○		TEST PIT WET BELOW ELEV. 95.78 DURING EXCAVATION APR. 24, 1998. TEST PIT WALLS CAVING
3.30		END OF TEST PIT								

PRELIMINARY

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: AMH

CHECKED:

RECORD OF TEST PIT 2

…: 981-3066
…CATION: REFER TO LOCATION PLAN

EXCAVATION DATE: APR. 24, 1998

SHEET 1 OF 1
DATUM: LOCAL

DEPTH SCALE METRES	METHOD / SIZE	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SHEAR STRENGTH Cu, kPa (VANE TEST +, PENETROMETER ●)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp — W — Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUND WATER CONDITIONS AND INSTALLATION
0		GROUND SURFACE		97.65 0.00						
1–3	Backhoe Dug 3.5m x 2.3m x 4.3m	Brown sand some gravel occasional concrete blocks (FILL)			1	CS		O		
3				94.65 3.00	2	CS		O		
3–4		Grey SAND AND GRAVEL occasional cobbles & boulders								SEEPAGE INTO TEST PIT AT ELEV. 93.65 DURING EXCAVATION APR. 24, 1998
4		END OF TEST PIT		93.35 4.30						

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: AMH
CHECKED:

981-3068

RECORD OF TEST PIT 3

SHEET 1 OF 1

LOCATION: REFER TO LOCATION PLAN

EXCAVATION DATE: APR. 24, 1998

DATUM: LOCAL

DEPTH SCALE METRES	METHOD / SIZE	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SHEAR STRENGTH Cu, kPa (VANE TEST +, PENETROMETER ●) / HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT (Wp, W, Wl; 10, 20, 30, 40)	ADDITIONAL LAB. TESTING	GROUND WATER CONDITIONS AND INSTALLATION
0		GROUND SURFACE		97.94 / 0.00					
0–3	Backhoe Dug; 1.0m x 7.5m x 3.8m	Brown sand some gravel & silt trace topsoil, occasional pieces of concrete (FILL)			1	CS			TEST PIT WET BELOW ELEV. 95.74 DURING EXCAVATION APR. 24, 1998. TEST PIT WALLS CAVING
3–3.8		Grey SAND some silt trace black organic sediment		94.94 / 3.00	2	CS			
3.8		END OF TEST PIT (Sand & Gravel material at 3.8m depth - Unable to sample due to caving)		94.14 / 3.80					

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AMH

CHECKED:

RECORD OF TEST PIT 4

PROJECT: 981-3066
LOCATION: REFER TO LOCATION PLAN

EXCAVATION DATE: APR. 24, 1998

SHEET 1 OF 1
DATUM: LOCAL

DEPTH SCALE METRES	METHOD / SIZE	DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	SAMPLE TYPE
0	Backhoe Dug, 3.5m x 3.5m x 4.5m	GROUND SURFACE	98.54 (98.54)		
		Brown sand some gravel & silt trace topsoil, occasional cobbles (FILL)	0.00	1	CS
1		Brown SAND occasional gravel	97.74 / 0.80	2	CS
4		Brown / grey SAND AND GRAVEL	94.64 / 3.90	3	CS
		Brown SANDY SILT	94.34 / 4.20	4	CS
		END OF TEST PIT	94.04 / 4.50		

SHEAR STRENGTH Cu, kPa — VANE TEST +, PENETROMETER ●

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT — Wp, W, Wl — 10, 20, 30, 40

ADDITIONAL LAB. TESTING

GROUND WATER CONDITIONS AND INSTALLATION

TE

RECORD OF TEST PIT 5

*1-3086 ... REFER TO LOCATION PLAN

EXCAVATION DATE: APR. 24, 1998

DATUM: LOCAL

DEPTH SCALE METRES	METHOD / SIZE	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SHEAR STRENGTH Cu, kPa (VANE TEST +, PENETROMETER ●) / HYDRAULIC CONDUCTIVITY, k, cm/s	WATER CONTENT, PERCENT (Wp, W, Wl; 10 20 30 40)	ADDITIONAL LAB. TESTING	GROUND WATER CONDITIONS AND INSTALLATION
0		GROUND SURFACE		99.05 / 0.00						
0–4.30	Backhoe Dug; 3.5m x 2.5m x 4.8m	Brown and black sand, some gravel, silt and topsoil (FILL)			1	CS		~27		
4.30–4.80		Brown / grey SAND AND GRAVEL		94.75 / 4.30	2	CS		~25		SEEPAGE INTO TEST PIT AT ELEV. 94.75 DURING EXCAVATION APR. 24, 1998. MINOR CAVING OF TEST PIT WALLS
4.80		END OF TEST PIT		94.25 / 4.80						

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: AMH

CHECKED:

DATA INPUT: WDF

-3066 RECORD OF TEST PIT 6 SHEET 1 OF 1

REFER TO LOCATION PLAN EXCAVA... DATE: APR. 24, 1998 DATUM: LOCAL

DEPTH SCALE METRES	METHOD / SIZE	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	SHEAR STRENGTH Cu, kPa (VANE TEST +, PENETROMETER ●)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp — W — Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUND WATER CONDITIONS AND INSTALLATION
0	Backhoe Dug 3.5m x 1.6m x 4.5m	GROUND SURFACE		99.28 / 0.00						
1–3		Brown sand some gravel and silt trace topsoil (FILL)			1	CS		o		
3.70		Grey SAND AND GRAVEL		95.58 / 3.70	2	CS		o		SEEPAGE INTO TEST PIT AT ELEV. 95.58 DURING EXCAVATION APR. 24, 1998.
4.30		Brown SANDY SILT		94.98 / 4.30	3	CS		o		MINOR CAVING OF TEST PIT WALLS
4.50		END OF TEST PIT		94.78 / 4.50						

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: AMH

CHECKED:

RECORD OF TEST PIT 7

...: 981-3086

...CATION: REFER TO LOCATION PLAN

EXCAVATION DATE: APR. 24, 1998

SHEET 1 OF 1

DATUM: LOCAL

DEPTH SCALE METRES	METHOD / SIZE	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	SHEAR STRENGTH Cu, kPa (VANE TEST +, PENETROMETER ●) / HYDRAULIC CONDUCTIVITY, k, cm/s	WATER CONTENT, PERCENT Wp —O W— Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	GROUND WATER CONDITIONS AND INSTALLATION
0	Backhoe Dig 3.5m x 1.6m x 3.6m	GROUND SURFACE		97.65 0.00						
1		Brown silty sand trace topsoil (FILL)			1	CS		O		
2				95.05 2.60						TEST PIT WET BELOW ELEV. 95.65 DURING EXCAVATION APR. 24, 1998. TEST PIT WALLS CAVING
3		Grey SAND some black organic sediments			2	CS		O		
		END OF TEST PIT		94.05 3.60						
4										
5										
6										
7										
8										
9										

LIST OF SYMBOLS

I. GENERAL

$\pi = 3.1416$
e = base of natural logarithms 2.7183
$\log_e a$ or $\ln a$, natural logarithm of a
$\log_{10} a$ or $\log a$, logarithm of a to base 10

t	time
g	acceleration due to gravity
V	volume
W	weight
m	mass
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress (σ is also used)
τ	shear stress
ε	linear strain
ε_{sy}	shear strain
υ	Poisson's ration (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) *Unit weight*

γ	unit weight of soil (bulk density)
γ_s	unit weight of solid particles
γ_w	unit weight of water
γ_d	unit dry weight of soil (dry density)
γ'	unit weight of submerged soil
G_s	specific gravity of solid particles $G_s = \gamma_s/\gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) *Consistency*

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_S	shrinkage limit
I_L	liquidity index $= (w - w_p)I_p$
I_C	consistency index - $(w_L - w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density $= (e_{max} - e)/(e_{max} - e_{min})$

(c) *Permeability*

h	hydraulic head or potential
q	rate of discharge
v	velocity of flow
i	hydraulic gradient
κ	coefficient of permeability
j	seepage force per unit volume

(d) *Consolidation (one-dimensional)*

m_v	coefficient of volume change $= -\Delta e/(1+e)\Delta\sigma'$
C_c	compression index $= -\Delta e/\Delta\log_{10}\sigma'$
c_v	coefficient of consolidation
T_F	time factor $= c_v t/d^2$ (d, drainage path)
U	degree of consolidation

(e) *Shear strength*

τ_f	shear strength	in terms of effective stress $\tau_f = c' + \sigma'\tan\phi$
c'	effective cohesion intercept	
ϕ'	effective angle of shearing resistance, or friction	
c_u	apparent cohesion*	in terms of total stress $\tau_f = cu + \sigma\tan\phi_u$
ϕ_u	apparent angle of shearing resistance, or friction	
μ	coefficient of friction	
S_t	sensitivity	

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole", on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS	auger sample
CS	chunk sample
DO	drive open
DS	Denison type sample
FS	foil sample
RC	rock core
ST	slotted tube
TO	thin-walled, open
TP	thin-walled, piston
WS	wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 0.3 m (12 in.).

Standard Penetration Resistance, N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 0.3 m (12 in.).

WH sampler advanced by static weight-weight, hammer

PH sampler advanced by hydraulic force

PM sampler advanced by manual force

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

Relative Density	"N" Blows/0.3 m or Blow/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) *Cohesive Soils*

Consistency	"Cu" kPa	psf.
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	over 200	over 4000

IV. SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	combined analysis, sieve and hydrometer[1]
Q	undrained triaxial[2]
R	consolidated undrained triaxial[2]
S	drained triaxial
U	unconfined compression
V	field vane test
Chem	chemical analysis

NOTES:

1. Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.
2. Undrained triaxial tests in which pore pressures are measured are shown as Q or R.

PROJECT: 031-130119
LOCATION: SEE PLAN FIGURE 1
SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 1

BORING DATE: AUG. 13 & 18, 2003

SHEET 1 OF 1
DATUM: GEODETIC
PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

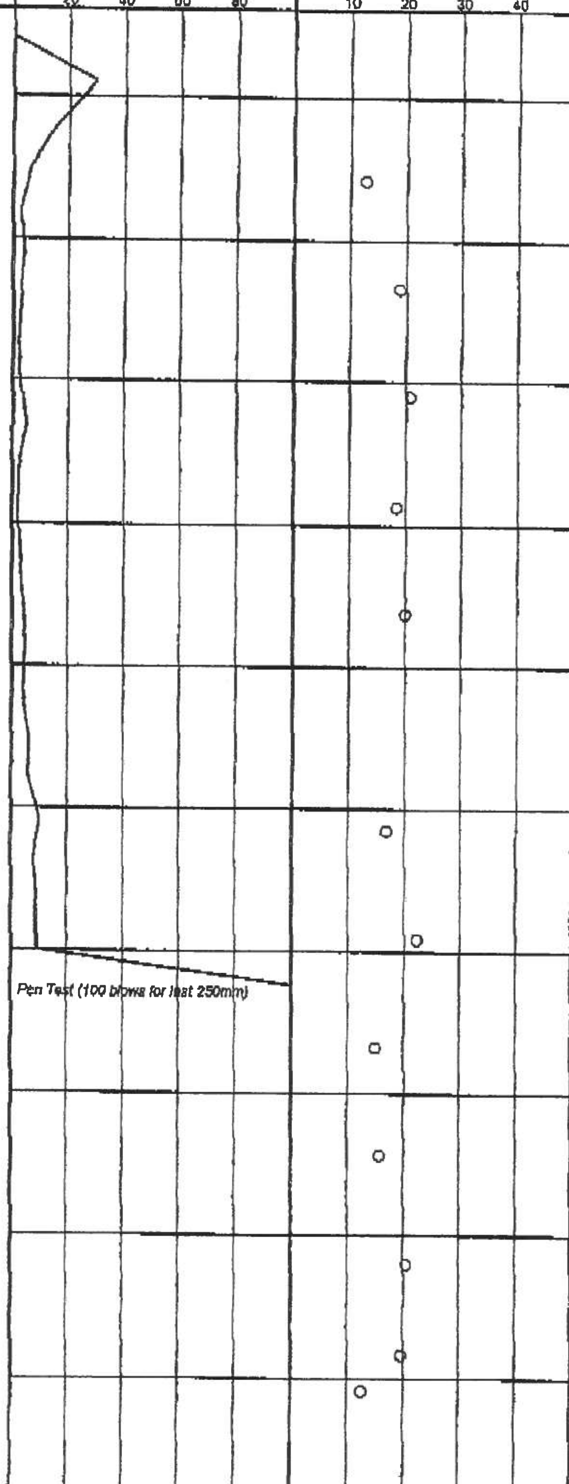
BORING METHOD: POWER AUGER (HOLLOW STEM)

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	240.41 / 0.00
Loose to very loose brown sandy silt to silty fine sand, trace gravel, trace topsoil (FILL)	237.21 / 3.20
Very loose black sandy silt (FILL)	236.90 / 3.51
Loose brown silty fine sand to sandy silt, trace gravel, (FILL)	234.47 / 5.94
Very loose grey layered SILT and fine sand	233.70 / 6.71
Very dense brown and grey SILT, trace sand	233.09 / 7.32
Dense grey SILTY FINE SAND, with silt layers	232.18 / 8.23
Dense grey fine to medium SAND	230.96 / 9.45
Hard grey CLAYEY SILT, trace sand	9.60
END OF BOREHOLE	

SAMPLE NUMBER	TYPE	BLOWS/0.3m
1	50 DO	7
2	50 DO	4
3	50 DO	2
4	50 DO	1
5	50 DO	4
6	50 DO	4
7	50 DO	4
8	50 DO	3
9	50 DO	53
10	50 DO	42
11	50 DO	36
12	50 DO	75

ELEVATION: 240, 239, 238, 237, 236, 235, 234, 233, 232, 231

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat V. + Q - ●; rem V. ⊕ U - ○: 20 40 60 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl: 10 20 30 40



ADDITIONAL LAB. TESTING

INSTALLATION AND GROUNDWATER OBSERVATIONS:
Water level encountered in borehole at elev. 237.36m during drilling Aug. 13, 2003

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: DJM
CHECKED:

PROJECT: 031-130119

LOCATION: SEE PLAN FIGURE 1

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 2

BORING DATE: AUG. 18, 2003

SHEET 1 OF 1

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: POWER AUGER (HOLLOW STEM)

Description	Elev. / Depth (m)	Sample Number	Type	Blows/0.3m
GROUND SURFACE	240.33 / 0.00			
Compact to very loose brown silty fine sand, trace gravel, trace topsoil (FILL)		1	50 DO	10
		2	50 DO	6
		3	50 DO	3
		4	50 DO	1
Very loose dark brown sandy silt, some topsoil (FILL)	236.67 / 3.66	5	50 DO	2
Very loose brown silty fine sand, trace gravel, (FILL)	235.91 / 4.42	6	50 DO	3
Dense grey SAND AND GRAVEL, with cobbles	234.84 / 5.49			
Dense to very dense grey SILTY FINE SAND, with silt layers	234.08 / 6.25	7	50 DO	48
		8	50 DO	64
END OF BOREHOLE	232.25 / 8.08			

Dynamic Penetration Resistance, Blows/0.3m: Pen Test (100 blows for last 200mm)

Installation and Groundwater Observations: Backfill Material; Caved material; Standpipe

Groundwater encountered in borehole at elev. 237.28m during drilling Aug. 18, 2003
Water level in Standpipe at elev. 236.01m Aug. 22, 2003
Standpipe removed Aug. 22, 2003

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: DJM

CHECKED:

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

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

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, S_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

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LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

π	3.1416
ln x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p)/I_p$
I_C	consistency index $= (w_l - w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e)/(e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p/\sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan\delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

Notes:
1. $\tau = c' + \sigma' \tan\phi'$
2. shear strength = (compressive strength)/2

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

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Appendix C

Environmental Assessment (by Others)

LAW Engineering (London) Inc.

69 Bessemer Rd.
Unit #35
London, Ontario
N6E 2V6

Phone: (519)-680-9991
Fax: (519)-680-9993
Email: info@lawengineering.com
Website: lawengineering.com

April 2, 2004
Project: 04017

Brian Snyder and Associates
319 Brock Street
London, Ontario
N6K 2M3

Attention: Mr. Brian Snyder

**Re: Preliminary Environmental Assessment
Proposed Springer Pond Development
Komoka, Ontario**

Dear Sir:

LAW Engineering (London) Inc. is pleased to present our report on the above mentioned project. This work was authorized by Mr. Brian Snyder on January 30, 2004.

Background

The subject site is the Springer Pond Development located in the Village of Komoka, Township of Middlesex Centre, Ontario.

It is proposed to extend the back yards of the proposed building lots backing onto the pond left by an existing abandoned gravel pit bounded by Springer Street, Glendon Drive, Queen Street and Ontario Avenue, in Komoka, Ontario. The fill material proposed for use at this site was obtained from a road cut made during the recent extension of the west end of Oxford Street in the City of London, Ontario, and from foundation excavations of the condominium development at Cadeau and Commissioners Road, Byron Subdivision, in London, Ontario.

According to Mr. Snyder, the fill material obtained was cut from undisturbed soil and the lands were treed prior to the beginning of construction at both borrow sites. Mr. Snyder took the writer to the areas of the road cut and condominiums to observe the borrow locations.

The purpose of this investigation has been to make a preliminary environmental assessment of the proposed material prior to its use on site.

Mr. Snyder stated that the Township of Middlesex Centre has approved the plan to extend the back yards of the building lots and has suggested to Mr. Snyder that some preliminary environmental assessment of the proposed fill materials would be prudent but not required as a condition of his permit.

The purpose of this environmental site assessment has been to determine the environmental condition of the proposed fill materials. The geotechnical aspects of placing and compaction of the proposed fill material is not covered in this report and fall outside of the scope of work provided by the Client.

Environmental Activities

The environmental activities for this project consisted of collecting a total of 6 random samples of the proposed fill materials at the locations as shown on the Sample Location Plan, Enclosure 1. The first series of sampling consisted of taking 2 samples (SA1 and SA2) on January 30, 2004. A second series of sampling consisted of taking 4 additional samples (SA3, SA4, SA1R and SA2R) on February 19, 2004. The second series of sampling was carried out due to the presence of traces of total petroleum hydrocarbons in the heavy oil range. The environmental soil samples were taken by an Environmental Engineer and preserved in accordance with MOE sampling protocol prior to delivery to the chemical laboratory.

An inspection of the sources where the proposed fill material was taken from was also made on February 19, 2004.

The material was previously delivered to the subject site and stockpiled mostly along the west side of the property immediately east of Springer Street, with a smaller stockpile located at the south-east corner of the property. The material is stockpiled in heaps of about 1.2m high, and consists mainly of sandy silt, with a trace of to some embedded gravel.

Chemical Analyses

The soil samples were delivered to PSC Analytical Services for chemical analyses

The initial series of soil samples were analyzed for metals, BTEX and TPH.

The second series of soil samples were analyzed for heavy oil only.

A summary of the chemical analyses results and the Certificates of Analyses are presented in Enclosure 2.

DISCUSSION & RECOMMENDATIONS

General

The fill material being assessed consists of sandy silt with a trace of to some gravel.

It is proposed to use the material for extension of the back yards of the proposed building lots on-site.

Assessment Criteria

The criteria used for this preliminary environmental assessment are formulated based on Section 6 of the Guideline For Use at Contaminated Sites in Ontario, June 1996 (revised appendices September, 1998). The following parameters are used for criteria selection in accordance with Figure 6a of the MOE Cleanup Guidelines:

- the site is not a potentially sensitive site in accordance with Section 6.1 of the MOE Guideline For Use at Contaminated Sites in Ontario;
- the current land use is residential;
- the area of the site is serviced by municipal piped water supply, potable groundwater use is suspected in the area of the subject site;
- the subsoil at this site is classified as coarse textured being the governing soil;

Criteria from Table A was selected for near surface soil in a potable groundwater condition based on the above parameters.

Soil Quality

The results of the chemical analyses and the Certificates of Analyses are appended as Enclosure 2.

The samples analysed met the MOE Table A criteria for the parameters tested.

The initial series of chemical analyses performed on SA1and SA2 revealed the presence of TPH in the heavy oil range of 130 and 126 µg/g, as compared to the Table A maximum allowable concentration of 1,000 µg/g. While these concentrations are approximately one tenth of the allowable concentration, LAW Engineering felt that re-sampling and additional chemcial analyses was warranted to confirm the presence or absence of heavy oil within the proposed fill.

The second series of chemical analyses confirmed the presence of 134 µg/g in sample SA1R. TPH in the heavy oil range was not detected in SA2R, SA3 nor SA4.

Discussion with Cathy Hughes of PSC Analytical Services, the testing laboratory who performed the chemical analyses, indicated that it has been noted in the past that certain naturally occurring organic compounds have been detected in trace amounts as heavy oil in natural, undisturbed soils, producing false positive results.

CONCLUSION

Based on the chemical quality of the soil samples taken, along with a visual examination of the stockpiles and the sources, it is the opinion of LAW that the proposed fill material is considered suitable for use as bulk fill.

QUALIFICATION OF THE ASSESSOR

This assessment was carried out by Mr. Ralph Billings, P. Eng. Mr. Billings is a professional engineer licensed to practice professional engineering in the Province of Ontario. Mr. Billings has been carrying out Phase 1, Phase 2 and Phase 3 Environmental Site Assessments since the late 1980s.

Statement of Qualifications for LAW Engineering (London) Inc. will be provided upon request.

CLOSURE

The Limitations of Report, as presented in the Appendix, forms an integral part of this report.

The American Society of Testing and Materials Standard of Practice notes that no environmental site assessment can wholly eliminate uncertainty regarding the potential for recognized environmental conditions in connection with a property. Performance of a standardized environmental site assessment protocol is intended to reduce, but not eliminate, uncertainty regarding the potential for recognized environmental conditions in connection with the property, given reasonable limits of time and cost.

This report has been prepared for the exclusive use of the Client. The environmental site assessment was conducted in accordance with the verbal and written requests from the Client, and generally accepted assessment practices. No other warranty, expressed or implied, is made.

We trust that the above report is complete within our terms of reference. If there are any questions concerning this matter, please do not hesitate to contact our office.

Yours Very Truly,
LAW Engineering (London) Inc.

Ralph Billings, P. Eng.
Project Engineer

LICENSED PROFESSIONAL ENGINEER
RDB
R. D. BILLINGS
APRIL 2/04
PROVINCE OF ONTARIO

Reviewed by:

Joseph Law, P. Eng.
Principal Engineer

(04017)rb

LIMITATIONS OF REPORT

1. The work performed in this report was carried out in accordance with the Standard Terms of Conditions made part of our contract. The conclusions presented herein are based solely upon the scope of services and time and budgetary limitations described in our contract.

2. The report has been prepared in accordance with generally accepted environmental study and/or engineering practices. No other warranties, either expressed or implied, are made as to the professional services provided under the terms of our contract and included in this report.

3. The services performed and outlined in this report were based, in part, upon visual observations of the site and attendant structures. Our opinion cannot be extended to portions of the site which were unavailable for direct observation, reasonably beyond the control of LAW Engineering (London) Inc..

4. The objective of this report was to assess environmental conditions at the site, within the context of our contract and existing environmental regulations within the applicable jurisdiction. Evaluating compliance of past or future owners with applicable local, provincial and federal government laws and regulations was not included in our contract for services.

5. LAW Engineering (London) Inc. has relied in good faith on information and services provided by others while conducting the record search. We accept no responsibility for any deficiency, misstatements or inaccuracies contained in this report as a result of omission, misinterpretation or fraudulent acts of the services used.

6. It should be noted that the observations and recommendations presented in this report are limited to the actual locations explored. The information presented in terms of the thickness and types of the subsoils encountered, groundwater levels, and chemical testing results, etc., are only applicable to the actual locations explored. Variations may be present between these locations. Should significant variation become apparent during later investigations, it may be necessary to reevaluate the findings of this report.

7. The conclusions of this report are based in part, on the information provided by others. The possibility remains that unexpected environmental conditions may be encountered at the site in locations not specifically investigated. Should such an event occur, LAW Engineering (London) Inc. must be notified in order that we may determine if modifications to our conclusions are necessary.

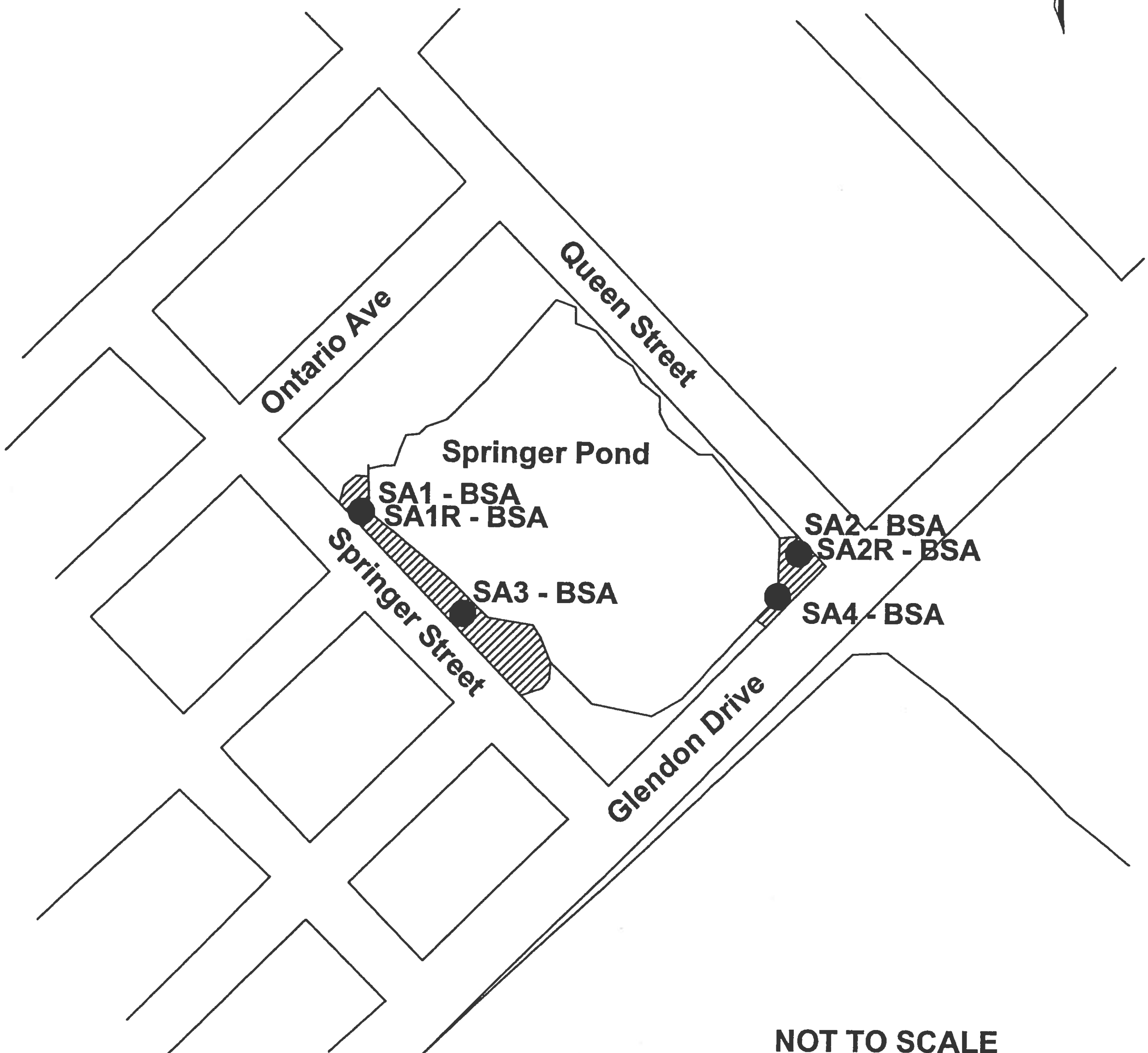
Job No. 04017

Enclosure No. 1

SAMPLE LOCATION PLAN

SPRINER POND DEVELOPMENT

VILLAGE OF KOMOKA, ONTARIO



LEGEND

SA1 - BSA ● SAMPLE LOCATIONS

▨ STOCKPILED PROPOSED FILL

LAW Engineering (London) Inc.

Enclosure 2

Chemical Analysis Summary and Certificates of Analysis

Springer Pond Development

Komoka, Ontario

SUMMARY OF CHEMICAL ANALYSES DATA

Parameters MOE Guideline Metals	SOIL JAN 30 2004 SA1-BSA	SOIL JAN 30 2004 SA2-BSA			**MOE TABLE A CRITERIA SOIL** Residential
Antimony Sb	<0.5	<0.5			13
Arsenic As	0.6	3.3			20
Barium Ba	8	77			750
Beryllium Be	<0.5	0.7			1.2
Boron B. Hot Water Soluble	<0.1	<0.1			1.5
Cadmium Cd	<0.5	<0.5			12
Chromium Cr	5	22			750
Cobalt Co	2	9			40
Copper Cu	5	16			225
Lead Pb	<5	11			200
Mercury Hg	<0.01	0.02			10
Molybdenum Mo	<2	2			40
Nickel Ni	<5	22			150
Selenium Se	<0.5	<0.5			10
Silver Ag	<1	<1			20
Thallium Tl	<1	<1			4.1
Vanadium V	5	20			200
Zinc Zn	16	51			600

Parameters Total Petroleum Hydrocarbons	SOIL JAN 30 2004 SA1-BSA	SOIL JAN 30 2004 SA2-BSA			**MOE TABLE A CRITERIA SOIL** Residential
Benzene	<0.005	<0.005			0.24
Toluene	<0.005	<0.005			2.1
Ethylbenzene	<0.005	<0.005			0.28
m&p zylenes	<0.005	<0.005			25 total zylenes
o-zylene	<0.005	<0.005			25 total zylenes
Styrene	<0.005	<0.005			1.2
Purgable TPH C6 - C10	<0.1	<0.1			Total of C6 to C24 100
TPH C10 - C24	<40	<40			Total of C6 to C24 100
TPH - Hot (Heavy Oil>C25)	130	126			1000

All concentrations are reported in μg/g
Exceedances are bold and underlined.

Project: L04017

Enclosure 2

Springer Pond Development

Komoka, Ontario

SUMMARY OF CHEMICAL ANALYSES DATA

Parameters Total Petroleum Hydrocarbons	SOIL FEB 19 2004 SA1R-BSA	SOIL FEB 19 2004 SA2R-BSA	SOIL FEB 19 2004 SA3-BSA	SOIL FEB 19 2004 SA4-BSA	**MOE TABLE A** **CRITERIA** **SOIL** Residential
TPH - Hot (Heavy Oil>C25)	134	<100	<100	<100	1000

All concentrations are reported $\mu g/g$
Exceedances are bold and underlined.

PSC Analytical Services

Facsimile Cover Sheet

To: MR. RALPH BILLINGS
Company: Law Engineering
Fax No.: 519-680-9993

Confidential and Priviledged

ELECTRONICALLY AUTHORIZED ANALYTICAL REPORT

This transmission contains the Case Narrative and analytical results only. The cover sheet is included as the last page. The complete signed hardcopy report will be mailed.

If you have received this facsimile in error, please notify the sender immediately at (519) 686-7558. Return the facsimile by mail or destroy the copy. It is strictly forbidden for anyone other than the addressee to use, disseminate, distribute or reproduce any portion of this facsimile message.

PSC Analytical Services

CERTIFICATE OF ANALYSIS - SECTION 1

CASE NARRATIVE

Attention: MR. RALPH BILLINGS
Client Name: Law Engineering
Address: 35-69 Bessemer Rd.
London, ON
N6E 2V6
Telephone: 519-680-9991
FAX: 519-680-9993

Laboratory Work Order: 113832

Sample(s) Received on: 20-Feb-2004 Reported on: 23-Feb-2004

Sample Shipment Receipt and Login:

Temperature on receipt was 2.5°C. The maximum allowable temperature is 10°C according to Canadian regulations or guidance documents. Samples submitted to the laboratory soon after sampling are exempt, provided that cooling has been initiated. Cooling is not required for certain situations such as: Waste for classification or specific matrices or tests such as PCB in oil.

There are no other notable comments.

Sample Analysis:

No exceptions were noted during analysis.

General Comments:

None.

PSC Analytical Services

CERTIFICATE OF ANALYSIS - SECTION 2

ANALYTICAL RESULTS

Client:(2965) Law Engineering, London

Reported:23-Feb-2004

Attention:	MR. RALPH BILLINGS	Purchase Order:	
Client Reference:	04017	Date Received:	20-Feb-2004
Work Order:	113832	Sample Type:	Solids

Sample #	Test	Result	Units	EQL	Comment
04-A003878	Sample Description: SA1R-BSA		Date & Time Sampled: 19-Feb-2004 16:00		
	TPH - Hot (Heavy oil>C25)	134.	mg/kg	100	
	Solids, Total	80.3	%	0.1	
04-A003879	Sample Description: SA2R-BSA		Date & Time Sampled: 19-Feb-2004 16:00		
	TPH - Hot (Heavy oil>C25)	< 100	mg/kg	100	
	Solids, Total	77.1	%	0.1	
04-A003880	Sample Description: SA3-BSA		Date & Time Sampled: 19-Feb-2004 16:00		
	TPH - Hot (Heavy oil>C25)	< 100	mg/kg	100	
	Solids, Total	80.7	%	0.1	
04-A003881	Sample Description: SA4-BSA		Date & Time Sampled: 19-Feb-2004 16:00		
	TPH - Hot (Heavy oil>C25)	< 100	mg/kg	100	
	Solids, Total	83.7	%	0.1	

EQL Estimated Quantitation Limit
Refer to the cover page for a list of report contents.

CERTIFICATE OF ANALYSIS

Attention: MR. RALPH BILLINGS
Client Name: Law Engineering
Address: 35-69 Bessemer Rd.
London, ON
N6E 2V6
Telephone: 519-680-9991
FAX: 519-680-9993

Laboratory Work Order: 113832

This Certificate of Analysis is for the following:

Sample Received on: 20-Feb-2004

Reported on: 23-Feb-2004

Client Reference: 04017
Purchase Order:
Quotation No.:

The report contains the following sections:
Section: 1. Case Narrative
2. Analytical Results
3. Methodology Summary
4. Certificate of Quality Control
6. Hold Time Report

Results for solids samples are corrected for moisture and reported as dry weight.

We are proud to be Accredited by: Standard Council of Canada (SCC) / CAEAL to ISO 17025 (#1799)
New York State / NELAP (#11730)
for specific tests

Water samples are discarded 4 weeks after the results have been reported. Solid samples are retained for 3 months. Storage for longer periods requires prior arrangement with the laboratory.

ELECTRONICALLY AUTHORIZED
Signatures to follow on original report.

Reviewed and Authorized by

Kathie Hughes
Project Manager

NOTE: The enclosed results relate only to the sample or item as received by the laboratory.

PSC Analytical Services

Facsimile Cover Sheet

To: MR. RALPH BILLINGS
Company: Law Engineering
Fax No.: 519-680-9993

Confidential and Priviledged

ELECTRONICALLY AUTHORIZED ANALYTICAL REPORT

This transmission contains the Case Narrative and analytical results only. The cover sheet is included as the last page. The complete signed hardcopy report will be mailed.

If you have received this facsimile in error, please notify the sender immediately at (519) 686-7558. Return the facsimile by mail or destroy the copy. It is strictly forbidden for anyone other than the addressee to use, disseminate, distribute or reproduce any portion of this facsimile message.

PSC Analytical Services
921 Leathorne Street, London, Ontario, Canada N5Z 3M7 (519) 686-7558 1-800-268-7396 Fax: (519) 686-6374

PSC Analytical Services

CERTIFICATE OF ANALYSIS - SECTION 1

CASE NARRATIVE

Attention: MR. RALPH BILLINGS
Client Name: Law Engineering
Address: 35-69 Bessemer Rd.
London, ON
N6E 2V6
Telephone: 519-680-9991
FAX: 519-680-9993

Laboratory Work Order: 112781

Sample(s) Received on: 30-Jan-2004 Reported on: 18-Feb-2004

Sample Shipment Receipt and Login:

Temperature on receipt was 1.8°C. The maximum allowable temperature is 10°C according to Canadian regulations or guidance documents. Samples submitted to the laboratory soon after sampling are exempt, provided that cooling has been initiated. Cooling is not required for certain situations such as: Waste for classification or specific matrices or tests such as PCB in oil.

There are no other notable comments.

Sample Analysis:

No exceptions were noted during analysis.

General Comments:

None.

PSC Analytical Services

CERTIFICATE OF ANALYSIS - SECTION 2

ANALYTICAL RESULTS

Client:(2965) Law Engineering, London

Reported:18-Feb-2004

Attention:	MR. RALPH BILLINGS	Purchase Order:	
Client Reference:	BS4A	Date Received:	30-Jan-2004
Work Order:	112781	Sample Type:	Solids

Sample #	Test	Result	Units	EQL	Comment
04-A002171	Sample Description: SA1-BSA		Date & Time Sampled:		
	Solids, Total	84.2	%	0.1	
	Antimony Sb	< 0.5	mg/kg	0.5	
	Arsenic As	0.6	mg/kg	0.5	
	Barium Ba	8.	mg/kg	4	
	Beryllium Be	< 0.5	mg/kg	0.5	
	Boron B,Hot Water Soluble	< 0.1	mg/kg	0.1	
	Cadmium Cd	< 0.5	mg/kg	0.5	
	Chromium Cr	5.	mg/kg	2	
	Cobalt Co	2.	mg/kg	2	
	Copper Cu	5.	mg/kg	2	
	Lead Pb	< 5	mg/kg	5	
	Mercury Hg	< 0.01	mg/kg	0.01	
	Molybdenum Mo	< 2	mg/kg	2	
	Nickel Ni	< 5	mg/kg	5	
	Selenium Se	< 0.5	mg/kg	0.5	
	Silver Ag	< 1	mg/kg	1	
	Thallium Tl	< 1	mg/kg	1	
	Vanadium V	5.	mg/kg	2	
	Zinc Zn	16.	mg/kg	1	

EQL Estimated Quantitation Limit
Refer to the cover page for a list of report contents.

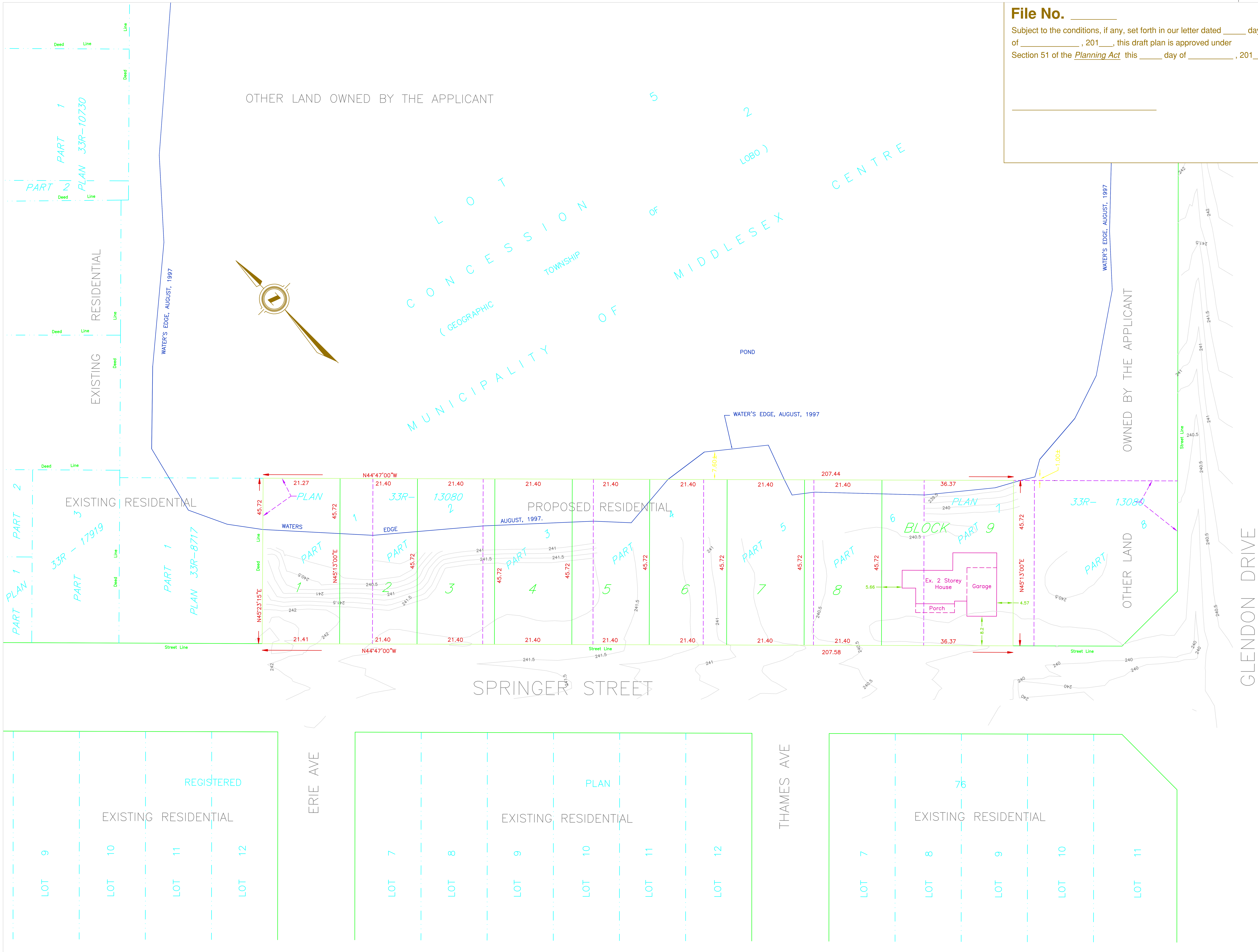
Appendix D

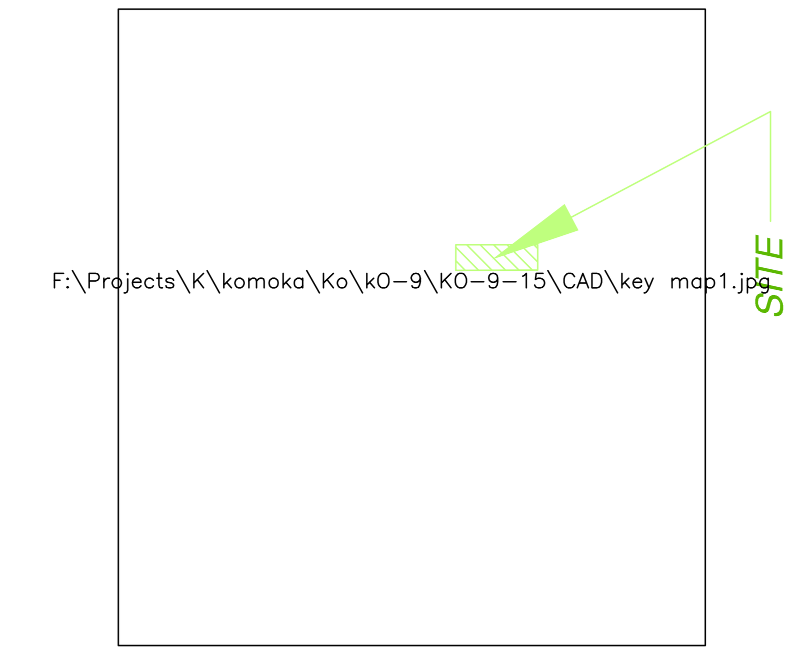
Drawings

File No. _______

Subject to the conditions, if any, set forth in our letter dated _____ day of _____________ , 201___, this draft plan is approved under Section 51 of the *Planning Act* this _____ day of __________ , 201___

____





KEY PLAN
NOT TO SCALE

DRAFT PLAN OF SUBDIVISION

OF PART OF

LOT 5, CONCESSION 2

(GEOGRAPHIC TOWNSHIP OF LOBO)

IN THE

MUNICIPALITY OF MIDDLESEX CENTRE

COUNTY OF MIDDLESEX

SCALE 1:500

0 2 4 6 8 10 20 30 40

SCALE IN METRES

SURVEYOR'S CERTIFICATE:

I CERTIFY THAT THE BOUNDARIES OF THE LANDS TO BE SUBDIVIDED AS SHOWN ON THIS PLAN AND THEIR RELATIONSHIP TO ADJACENT LANDS ARE ACCURATELY AND CORRECTLY SHOWN.

___________ DATE

___________________ BRUCE S. BAKER
ONTARIO LAND SURVEYOR

OWNER'S CERTIFICATE:

I HEREBY AUTHORIZE THE SUBMISSION OF THIS PLAN IN DRAFT FORM.

SPRINGER POND DEVELOPMENTS INC.

___________ DATE

_______________________ BRYAN SNYDER

I HAVE THE AUTHORITY TO BIND THE CORPORATION

ADDITIONAL INFORMATION REQUIRED UNDER SECTION 51 (17) OF THE PLANNING ACT.

(a) AS SHOWN ON PLAN	(g) AS SHOWN ON PLAN
(b) N/A	(h) PIPED WATER
(c) AS SHOWN ON KEY PLAN	(i) SANDY
(d) SINGLE FAMILY RESIDENTIAL	(j) AS SHOWN ON PLAN
(e) AS SHOWN ON PLAN	(k) FULL SERVICES
(f) AS SHOWN ON PLAN	(l) NONE

LAND USE SCHEDULE

SINGLE FAMILY RESIDENTIAL LOTS 1 TO 8 (INCLUSIVE), BLOCK 9	0.95 ha.
ROADS AND RESERVES STREET NAMES BLOCK	N / A
STORM WATER MANAGEMENT BLOCK	N / A
OPEN SPACE BLOCK	N / A
WALKWAY BLOCK	N / A
TOTAL AREA	0.95 ha.

UTM GRID NOTE:

BEARINGS HEREON ARE UTM GRID BEARINGS AND ARE DERIVED FROM HORIZONTAL CONTROL MONUMENTS No. 01019920523 AND No. 01019920524 AND ARE REFERRED TO CENTRAL MERIDIAN 81°00' WEST LONGITUDE, ZONE 17.

U.T.M. CO-ORDINATES (NAD 83 – ORIGINAL)		
STATION	NORTHING	EASTING
SCP 01019920523	4753820.886	463811.570
SCP 01019920524	4754784.003	464812.419
1	4755260.672	464891.687
2	4755137.475	465078.341

DISTANCES SHOWN ON THIS PLAN ARE ADJUSTED GROUND LEVEL DISTANCES AND CAN BE USED TO COMPUTE GRID CO-ORDINATES BY MULTIPLYING THE DISTANCES BY A COMBINED SCALE FACTOR OF 0.999583016.

METRIC:

DISTANCES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

AREAS SHOWN ON THIS PLAN ARE IN HECTARES AND CAN BE CONVERTED TO ACRES BY MULTIPLYING BY 2.471054.

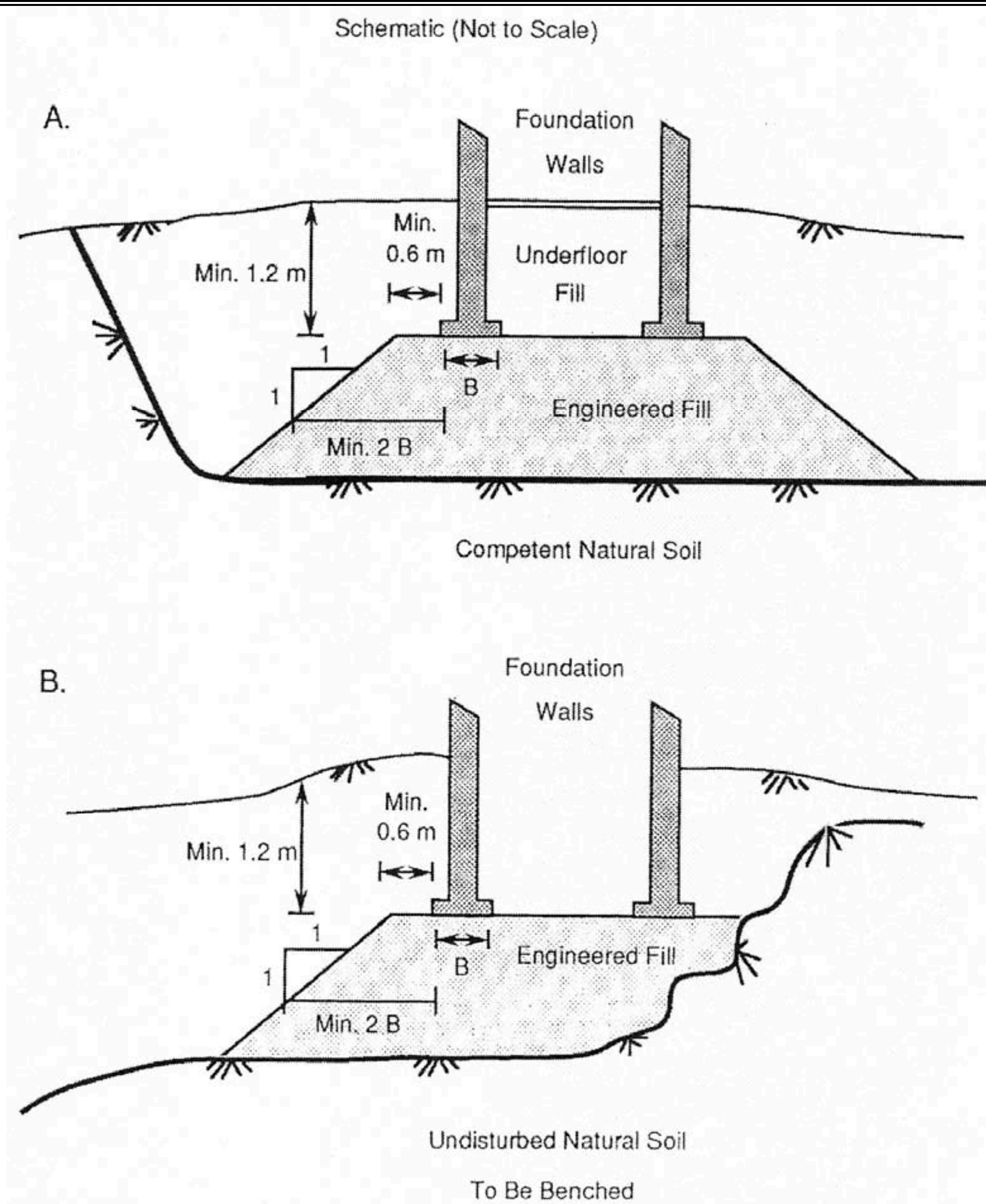
AGM
PLAN • SURVEY • ENGINEER
ARCHIBALD, GRAY & McKAY LTD.
3514 WHITE OAK ROAD, LONDON, ON, N6E 2Z9
PHONE 519-685-5300 FAX 519-685-5303
EMAIL info@agm.on.ca WEB www.agm.on.ca

No.	REVISIONS	DATE	BY
1	.	.	.

DRAWN BY: DH	DIGITAL FILE: ACAD–ko0302dp1.dwg	PLAN No:
CHECKED BY: BAKER	COGO FILE: KO0302c1.cog	**8-L-4827**
Plot date: May 03, 2016	OFFICE FILE: KO–9–15	

C:\Users\swinsonc\Desktop\ACAD–ko0302dp1.dwg

GEOMETRIC REQUIREMENTS FOR FOUNDATIONS ON ENGINEERED FILL



SECTION VIEW

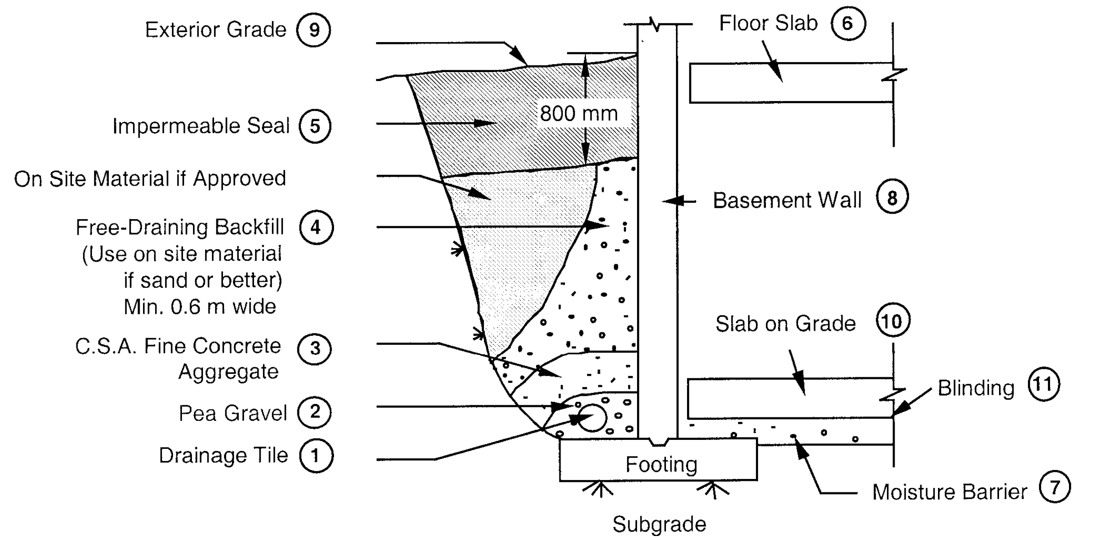
Section A – Typical Section of Slab-on-Grade Building
Section B – Typical Section of Building with Basement

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 native 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. The imported fill should be reviewed to satisfy MOE Requirements.
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.
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 (SLS) of 145 kPa (3,000 psf) may be used for foundations set on engineered fill, 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-00014641-GE).
11. Footing Base inspections are required to verify the suitability of the subgrade soils, at the time of construction. *In situ* density tests may also be required at the footing base level to confirm material density.

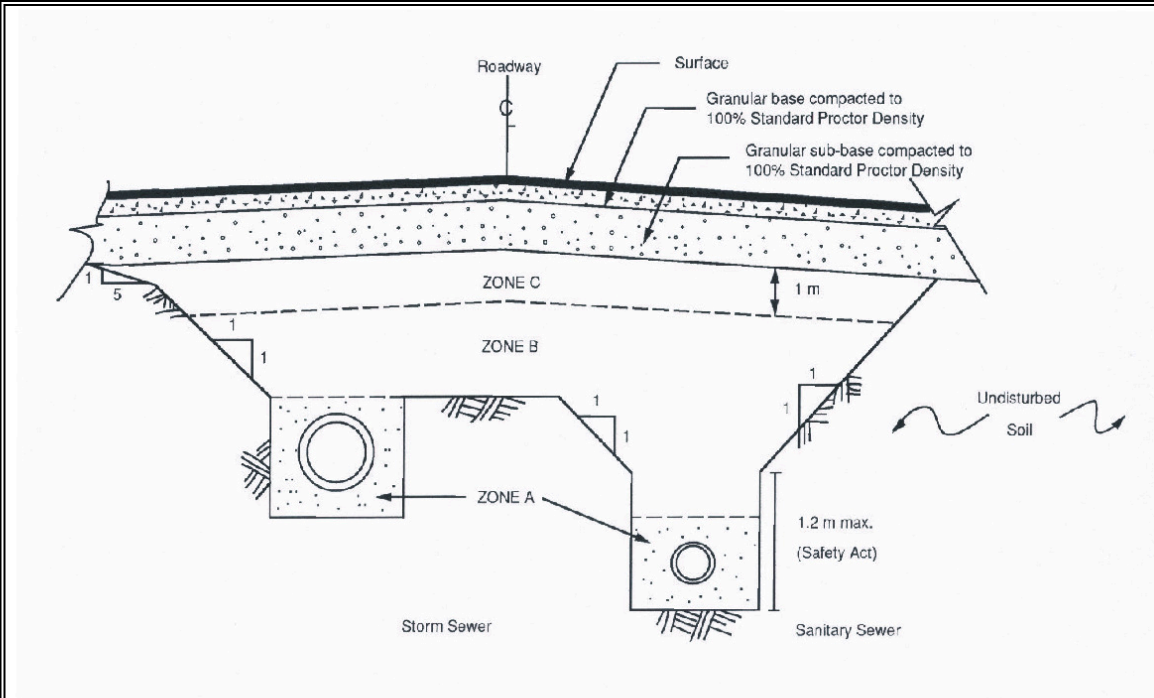
DRAINAGE AND BACKFILL RECOMMENDATIONS
(NOT TO SCALE)



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 floor slab.
2. Pea gravel should be placed within 150 mm (6 in.) of the top and sides of the 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 it is covered by an approved porous geotextile membrane (Terrafix 270R or equivalent).
3. C.S.A. fine concrete aggregate to act as filler material. Minimum 300 mm (12 in.) top and sides of tile drain. This may be replaced by an approved porous geotextile membrane (Terrafix 270R or equivalent).
4. Free-draining backfill, such as Class B pit-run gravel or equivalent compacted to 93 to 95% Standard Proctor density. Do not exceed 95% Standard Proctor density. Do not compact closer than 1.8 m (6 ft) from wall with heavy equipment. Use hand controlled light compaction equipment within 1.8 m (6 ft) of wall. Alternatively, free draining backfill may be replaced with prefabricated wall drains at 2.5 m centres or closer for wet conditions as per OBC.
5. Impermeable backfill seal - compacted native silt, clay, or equivalent. If original soil is free draining, this seal may be omitted.
6. Do not backfill until wall is supported by basement and floor slab or adequate bracing.
7. Moisture barrier to consist of 20 mm (3/4 in.) compacted clear, crushed stone or equivalent free-draining material. Layer to be 200 mm (8 in.) thick.
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. If the 20 mm (3/4 in.) stone requires surface binding, use 6 m (l/4 in.) stone chips.

TYPICAL BACKFILL DETAIL
STORM AND SANITARY SEWER (DOUBLE SERVICE)



SECTION VIEW

NOTES:

ZONE A
Granular bedding satisfying current municipal standards (where applicable) 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).

Appendix E

Limitations and Use of Report

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 borehole results contained in the Report. The number of boreholes 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.