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Geotechnical Investigation
StationWest Phase 2 Residential Development

1120 Cooke Boulevard
Burlington, Ontario
L7T 4A8

Prepared for:

Adi Developments (Masonry) Inc.
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EXECUTIVE SUMMARY

SCOPE OF SERVICES

Proposed Development	Adi Developments (Masonry) Inc. is intending to develop the site identified as civic address 1120 Cooke Boulevard in Burlington, Ontario as a high-rise tower complex. Based on the plan and profile drawings provided to Landtek, as developed by Core Architects, the proposed development is to comprise of two, 36-storey towers and a 26-storey tower connected by a four-storey podium. The structure will include a maximum of five levels of basement parking. The footprint of the proposed structure is to cover the full site area.
Report Deliverables	The Geotechnical Investigation is required to determine the subsurface conditions underlying the site and to provide design and construction recommendations for the proposed new development.

SITE DETAILS AND SETTING

Coordinates	592800, 479596	**Site Area (approx.)**	9,900 m^2 (0.99 hectares)

Site Description	The site is part of a former sand and gravel pit. It is situated in a developed, residential area and is bounded by Cooke Boulevard to the east, Masonry Court to the south and Waterdown Road to the west. A flood management pond associated with the StationWest Phase 1 development bounds the site to the north. The site is undeveloped, relatively flat-lying, and is covered predominantly by bare soils and rough vegetation, with surface stripping operations reported to have already been completed in part across the site area.
Geology	Sporadic topsoil cover was encountered at the ground level. Fill material comprising of sand and gravel was encountered extending to a maximum depth of approximately 2.1 m. Underlying the fill are sand, silty sand and silt deposits extending to depths between approximately 3.0 m and 4.3 m. Clayey silt till was encountered underlying these deposits, extending to a proven depth of approximately 9.2 m below existing ground level. Red shale bedrock was encountered underlying the till deposits.
Groundwater	During drilling, wet to saturated soil conditions were observed in all boreholes at depths between approximately 3.0 m and 6.0 m to 8.0 m below existing ground level. These conditions are interpreted as the level at which groundwater was encountered (i.e. groundwater 'strike') and indicate that perched waters may be encountered at shallow depths. One groundwater monitoring visit was completed on August 6, 2014, and groundwater was measured at a level of approximately 1.2 m below ground level.

ENGINEERING CONSIDERATIONS

Foundations	It is understood that the proposed structures will include a maximum of five levels of basement. Based on the ground conditions observed at the borehole locations, it is considered by Landtek that bearing conditions to support the proposed residential development on concrete footings can be provided by the shale bedrock underlying the site.
Settlements	The SLS condition will not govern foundation design in bedrock as the stress required to induce the typical 25 mm settlement criteria at the SLS is anticipated to exceed the ULS. As such. settlements for foundations seated within bedrock are to be deemed negligible (i.e. less than 15 mm).
Earthquake Considerations	Based on the soil conditions encountered, and in accordance with Table 4.1.8.4.A. of the current Ontario Building Code (herein "*OBC*"), the site is considered to be a 'C' Site Class.

CONSTRUCTION CONSIDERATIONS

Excavations	The native soils to be encountered during excavation at the site are expected, in general, to behave as *"Type 2" and "Type 3"* materials according to the OHSA classification in Part III. Type 2 materials are characteristic of the generally very stiff clayey silt till deposits. The fill material encountered on site is considered Type 3 soil. The "*shale bedrock*" encountered at depth has strength properties that exceed a Type 1 soil.
Subsurface Concrete	The native soils are expected to have a mild sulphate environment and a low chloride concentration. For subsurface concrete structures, parking garage decks and ramps, a C-1 exposure class is recommended.
Dewatering	Considerations for groundwater control during construction are to be provided under separate cover.

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1.0 INTRODUCTION

Landtek Limited (herein "*Landtek*") is pleased to submit this geotechnical investigation report for the proposed StationWest Phase 2 residential development at the site identified as civic address 1120 Cooke Boulevard in Burlington, Ontario. Authorization to proceed with the work was received from Mr. Shane Cooney on behalf of Adi Developments (Masonry) Inc. on March 16th, 2021. The scope of services undertaken for this geotechnical investigation was completed in accordance with proposal reference P21030, dated February 25th, 2021.

It is understood that Adi Developments (Masonry) Inc. is intending to develop the site for a primarily residential end use. Based on the plan and profile drawings provided to Landtek, as developed by Core Architects, the proposed development is to comprise the following:

- Building A: a 36-storey tower complex;
- Building B: a 26-storey tower complex;
- Building C: a 36-storey tower complex;
- A four-storey podium connecting Buildings B and C;
- Five levels of basement parking across the full site area, with the P5 level basement floor at a Geodetic elevation of approximately 89.7 m (approximately 15.2 m depth); and,
- An access and egress route from Cooke Boulevard to the east of the site leading to a deck pavement and landscaping area.

It is assumed that no significant grade changes are proposed and that the proposed development will also include full site servicing.

The primary objectives of this investigation report are:

- To confirm the subsurface soil and groundwater conditions for foundation design and construction;
- Provide design and construction recommendations with regards to building foundations, floor slabs, pavement structures, and subsurface drainage and utilities; and,
- Assess the characteristics of the soils to be excavated and their suitability for reuse on site.

It is understood that a Hydrogeological Assessment is to be completed under separate cover.

This report has been prepared for Adi Developments (Masonry) Inc., their nominated engineers, designers, and project managers pertaining to the proposed residential development at the site identified as civic address 1120 Cooke Boulevard in Burlington, Ontario. Further dissemination of this report is not permitted without Landtek's prior written approval. Further details of the limitations of this report are presented in Appendix A.

2.0 SITE SETTING

2.1 Site Location and Description

The site is located in Burlington, Ontario, and is centered at approximate grid reference 592800, 4795965 (UTM 17T coordinates). The approximate Geodetic elevation of the ground surface at the borehole locations ranges between approximately 102.0 m and 103.5 m.

The site location is shown in Figure 2.1.1 below.

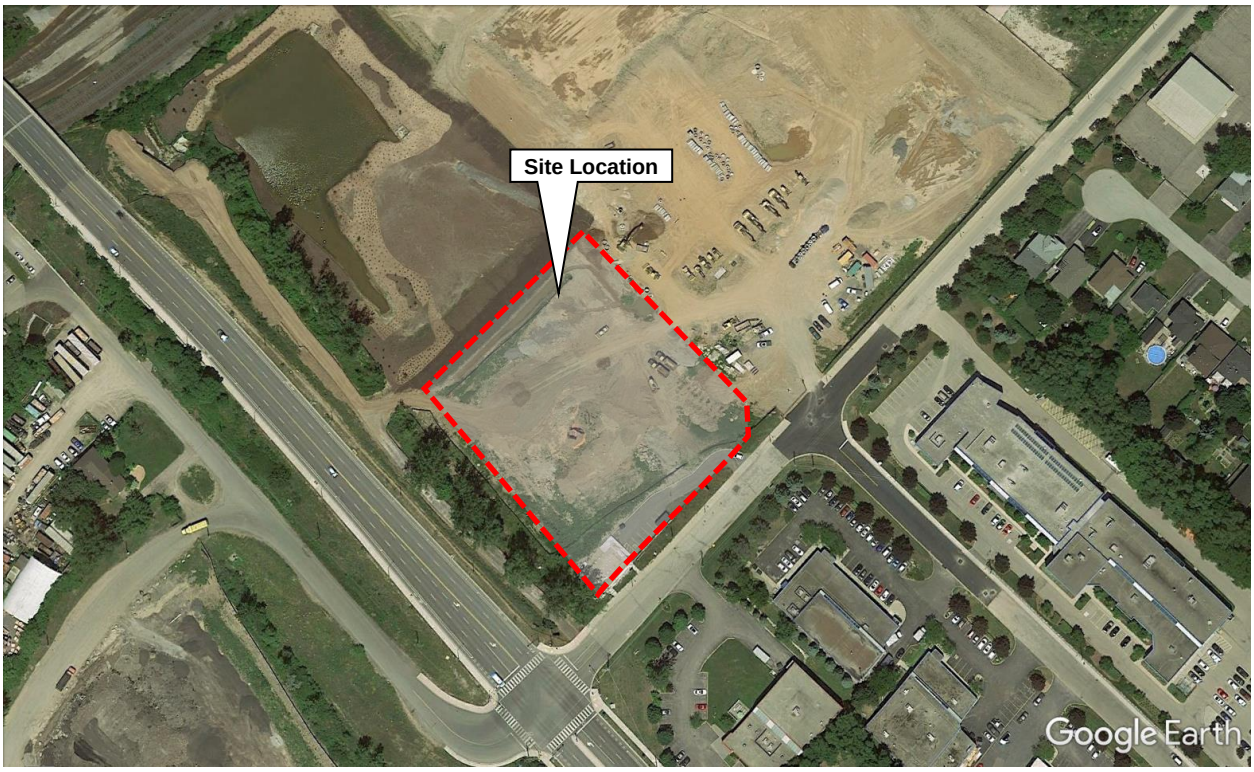


Figure 2.1.1: Area of Proposed Development

The site, identified as "Phase 2" of the StationWest (Masonry Court) development area, is approximately 9,900 m^2 (0.99 hectares) in plan area, and is part of a former sand and gravel pit. It is situated in a developed, residential area and is bounded by Cooke Boulevard to the east, Masonry Court to the south and Waterdown Road to the west. A flood management pond associated with the StationWest Phase 1 development bounds the site to the north. The site is undeveloped, relatively flat-lying, and is covered predominantly by bare soils and rough vegetation, with surface stripping operations reported to have already been completed in part across the site area.

2.2 Published Geology

Based on previous geotechnical experience for the area and a review of the existing geological publications for the site area, Ontario Geological Survey (OGS) Map 2509: "*Quaternary Geology of the Hamilton Area*", the native subsurface soil conditions in the area of the site are anticipated to consist of lacustrine and outwash sand overlying glacial till deposits.

According to the OGS Map M2336 "*Paleozoic Geology of the Hamilton Area*", the superficial geology is underlain by red shale bedrock of the Queenston Formation. Based on the information provided by the OGS Map M2034 "*Bedrock Topography of the Hamilton Area*", bedrock is inferred to underly the site at a depth of approximately 10 m to 12 m below ground level.

Information provided by historical borehole records from within a 200 m radius from the site, and held by the OGS, generally confirms the anticipated geological conditions beneath the site. Based on the data from records for Borehole ID 890915, located approximately 300 m south of the site, the superficial soil profile confirms the presence of silty sand deposits to a depth of approximately 3.4 m and overly till deposits. The till deposits extend to a depth of approximately 10.6 m. The bedrock in the area is red shale of the Queenston Formation.

3.0 FIELDWORK AND INVESTIGATION METHODOLOGY

Fieldwork undertaken by Landtek at the site was completed in two phases, with the first phase completed in 2014 as part of a wider site investigation, and the second phase in 2016. The scope of works included clearance of underground services, borehole layout, borehole drilling and soil sampling, rock coring and sampling and field supervision. The boreholes were logged using those standard symbols and terms defined in Appendix B. The Borehole Location Plan, Drawing 21086-01, and associated borehole logs are provided in Appendix C.

A total of four boreholes were drilled within the Phase 2 development area, as follows:

- Two boreholes (boreholes BH7 and BH8) were drilled in May 2014 using a CME-75 track-mounted drilling rig equipped with continuous flight, solid augers and were advanced to depths of approximately 6.6 m below existing ground level; and,
- Two boreholes (boreholes BH1 and BH2) were drilled in July 2016 using a CME-75 track-mounted drilling rig equipped with continuous flight, hollow stem augers and were advanced to practical auger refusal on bedrock. Both boreholes were then extended into the bedrock using rotary HQ coring methodology to achieve target depths of approximately 18.5 m below existing ground level.

Standard Penetration Tests (SPT's) and split spoon samples were taken at selected depths in all soil profiles during drilling. Full time supervision of drilling and coring operations was carried out by a representative of Landtek.

Borehole BH8 was completed as a monitoring well to monitor groundwater levels where encountered. The monitoring well consisted of new 50 mm poly-vinyl chloride (PVC) screen with No.10 slots threaded onto a matching riser. The screens and risers were pre-threaded including o-ring seals such that no glues or solvents were used to connect the pipe sections. The annular space between the PVC well and the borehole was backfilled to approximately 0.3 m above the top of the screen section with sand pack, and then with bentonite to existing ground level. A J-Plug lockable air-tight cap was installed on the riser. The borehole log in Appendix C present the monitoring well installation details and the results of the subsequent groundwater level monitoring visit completed on August 6, 2014.

All soil samples and rock cores were transported to the Landtek's in-house, Canadian Council of Independent Laboratories (CCIL) certified laboratory and visually examined to determine their textural classification. Moisture content testing was carried out on all samples. The rock cores collected were examined to determine their rock classification. Selected rock core samples were subsequently submitted for Unconfined Compressive Strength (UCS) testing.

Borehole locations were established by Landtek using measurements and offsets relative to existing, permanent site structures and centreline chainages. Ground surface Geodetic elevations at the borehole locations were established by Landtek in reference to the topographical survey for the site, as completed by H. Gelbloom Surveying Ltd., project reference 14-125, dated November 2014.

4.0 SUBSURFACE CONDITIONS

4.1 Overview

The borehole information is generally consistent with the geological data identified in Sections 2.2, with the predominant native soils comprising of a veneer of silty and sandy deposits overlying clayey silt till. Bedrock comprising red shale was encountered underlying the till deposits.

The detailed borehole logs are presented in Appendix C, with the subsurface conditions encountered summarized in the following Table 4.1.1. Subsurface conditions encountered by the boreholes are discussed further in the sections following.

Table 4.1.1: Summary of Borehole Conditions Encountered

BH ID	Surface Material	Superficial Soil Depth and (Description)		Bedrock Contact
	Topsoil	Fill	Native Soil	
BH7	100 mm	0.1 m – 0.9 m (sand and gravel)	0.9 m – 3.0 m (sand and silty sand) 3.0 m – 5.5 m (silt) 5.5 m – 6.6 m (clayey silt till)	-
BH8	100 mm	0.1 m – 2.1 m (sand and gravel)	2.1 m – 3.3 m (sand) 3.3 m – 4.3 m (silt) 4.3 m – 6.6 m (clayey silt till)	-
BH1	-	0.0 m – 1.5 m (sand and gravel)	1.5 m – 3.0 m (silty sand) 3.0 m – 9.2 m (clayey silt till)	9.2 m
BH2	-	0.0 m – 1.5 m (sand and gravel)	1.5 m – 3.0 m (silty sand) 3.0 m – 9.2 m (clayey silt till)	9.2 m

4.2 Organic Soil

An approximately 100 mm thick layer of organic soil was encountered from ground surface in boreholes BH7 and BH8.

The presence and thicknesses of the organic soil are noted to vary across the site, and the thicknesses measured at the borehole locations may not be representative of the overall topsoil volume present at the site.

4.3 Fill Materials

Fill materials were encountered in all boreholes extending of depths of between approximately 0.9 m (borehole BH7) and 2.1 m (borehole BH8). The fill materials generally comprised of sand and gravel with silt.

SPT ‘‘N’’ values ranging from 2 to 13 were reported within the fill materials, indicating their compactness condition to be variable from poor to moderate, but generally poorly compacted and as expected for uncontrolled or older fill soils.

4.4 Sand and Silty Sand

Sand and silty sand deposits were encountered in all boreholes extending to depths between approximately 3.0 m (borehole BH1) and 5.5 m (borehole BH7) below existing ground level. The

sand was observed to be brown in colour and is generally fine to coarse grained. The sand and silty sand deposits include trace to some fractions of gravel.

SPT 'N' values ranging from 5 to 24 were reported, indicating the sand and silty sand deposits to be of a loose to compact condition. Moisture contents are in the order of 14 % to 21 %, which are indicative of a damp to wet soil where sand is a primary constituent.

4.5 Silt

Silt deposits were encountered in boreholes BH7 and BH8 underlying the sand and silty sand deposits. The silt is generally reddish brown in colour, has a fractured structure and includes traces of fine sand.

SPT values of 8 and 11 were reported, indicating the silt to be in a loose to compact condition. Moisture contents of 18 % and 24 % are indicative of a damp to wet soil where silt is a primary constituent.

4.6 Clayey Silt Till

Clayey silt till was encountered in all boreholes underlying the sand and silt deposits and extends to depths of approximately 9.2 m below existing ground level. The till contains traces of gravel and red shale and is generally brown and grey in colour.

SPT ''N'' values ranging from 8 to 14 were reported, indicating the till deposits to be of a firm to stiff, but generally stiff consistency. Moisture contents are in the order of 13 % to 20 %, which are generally representative of a moist to very moist soils with clay and silt as primary constituents.

Grain size analysis undertaken during the 2014 investigation classified the till as an ML soil (Unified Soil Classification System) which relates to inorganic silts and silty or clayey fine sands, and clayey silts with a generally low plasticity.

4.7 Bedrock

Red shale of the Queenston Formation was encountered in all boreholes underlying the till deposits at depths of approximately 9.2 m (boreholes BH1 and BH2) below existing ground level, equating to Geodetic elevations between approximately 93.5 m (borehole BH1) 93.9 m (borehole BH2).

The shale is generally weak and highly to slightly weathered on initial contact, being recovered and behaving as a "*residual soil*" and becomes more competent with depth. Strength testing performed on intact rock core specimens indicate rock strength of the competent shale in the order of 56.8 MPa to 71.8 MPa, indicating that the bedrock can be classified as having a strength grade of R4, and being generally "strong" according to the Canadian Foundation Engineering Manual.

Though not encountered during this investigation, it should be noted that the layered nature of the shale bedrock is such that it may contain discontinuous green and red siltstone and occasional limestone seams that tend to be much sounder and harder than the shale strata. The Rock Quality Designation (RQD) values of the competent shale bedrock were in the order of 60 % to 75 % indicating the bedrock to be of an "good" quality at depth.

4.8 Groundwater

During drilling, wet to saturated soil conditions were observed in all boreholes at depths between approximately 3.0 m and 6.0 m to 8.0 m below existing ground level. These conditions are interpreted as the level at which groundwater was encountered (i.e. groundwater strike) and indicate that perched waters may be encountered at shallow depths within the silt and sand deposits overlying the more impermeable silty clay till.

One groundwater monitoring visit was completed on August 6, 2014, and groundwater was measured at a level of approximately 1.2 m below ground level.

It should be noted that groundwater conditions and surface water flow conditions are expected to vary according to the time of the year and seasonal precipitation levels. Water seepage is also expected from soil fissures above the water table.

5.0 FOUNDATION DESIGN CONSIDERATIONS

5.1 Shallow Foundations on Bedrock

Based on the ground conditions observed at the borehole locations, it is considered by Landtek that bearing conditions to support the proposed residential development on concrete footings can be provided by the shale bedrock underlying the site. It is understood that all elements of the proposed development will include a maximum of five levels of basement. On this basis, it is anticipated that foundations for a structure with five levels of basement will be seated at depths between approximately 15.0 m below existing ground level, equating to the Geodetic elevation of approximately 89.7 m provided to Landtek by Adi Developments (Masonry) Inc.

Table 5.1.1 summarizes the recommended geotechnical reactions at the Ultimate Limit State (herein "*ULS*") for the shale bedrock. Where foundations are seated in bedrock, the SLS State condition will not govern their design as the stress required to induce the typical 25 mm settlement criteria at the SLS is anticipated to exceed the ULS. It should be noted that the design parameters have been determined by Landtek for the design stage only.

In accordance with the Ontario Building Code (herein "*OBC*"), 9.12.2.2 (5), and based on local experience, the shallowing of exterior and interior footings to 0.9 m and 0.6 m depth below the basement finished floor level respectively, may be adopted for the proposed development.

Where the bearing levels of the footings are at different design elevations, the footing base levels should be stepped along a line of 7V:10H, drawn upwards from the lowest footing, to avoid overlapping stresses.

Subsurface conditions can vary over relatively short distances and the subsurface conditions revealed at the test locations may not be representative of subsurface conditions across the site. Therefore, a Geotechnical Engineer should be engaged during construction to examine the exposed sub-soil quality and condition, and confirm the subsurface conditions are consistent with design assumptions. This is in compliance with field review requirements in the National Building Code, Volume 1, Clause 4.2.2.3.

Table 5.1.1: Recommended Limit State Foundation Design Values

Approximate Founding Levels		Founding Stratum	Foundation Design Value	
Depth Range	Geodetic Elevation		SLS [1,2]	ULS [3,4]
±15.0 m to ±18.0 m	±88.6 m to ±84.7 m	Competent Shale Bedrock	-	10 MPa

Notes:
1. The National Building Code general safety criterion for the serviceability limit states is: SLS resistance ≥ effect of service loads.
2. Recommended SLS bearing values conform to Estimated Values based on soil types given in Tables K-8 and K-9 of the National Building Codes User's Guide.
3. The ULS resistance factor for shallow foundations is 0.5, as given in Table K-1 of the National Building Code User's Guide.
4. The National Building Code general safety criterion for the ultimate limit states is: factored ULS resistance ≥ effect of factored loads.

5.2 Frost Susceptibility

The fine-grained soils encountered at shallow depths across the site are considered sensitive to water and frost, and their physical and mechanical properties are dependent on in-situ moisture

content. As such, the founding soils at the site are considered to have a moderate to high frost susceptibility, being classified as Frost Group "*F4*" (Table 13.1 of the "*Canadian Foundation Engineering Manual*", 4th Edition). However, the identified depths for foundations, as given in Section 5.1 are considered to be below the maximum extents of influence from frost penetration in the Burlington area.

Should any re-grading be required as part of the proposed development and adjacent to the new structures, it will be important to ensure that the associated exterior footings will have a minimum of 1.2 m of soil cover, or equivalent suitable insulation, for frost protection.

Concerns regarding frost protection to footings are more directed towards those seated within soils. Foundations in the shale bedrock are generally deemed exempt from any frost protection requirements.

5.3 Settlement Considerations

Based on the outline information provided for the nature of the proposed development of the site, it is anticipated that the loads to be applied to the ground by any such structure will be generally moderate intensity. As such, associated settlements are not expected to be large. Therefore, the general limiting of the total settlement to 25 mm and the differential settlement to 19 mm by the recommended geotechnical reaction at the SLS is considered appropriate.

The SLS condition will not govern foundation design in bedrock as the stress required to induce the typical 25 mm settlement criteria at the SLS is anticipated to exceed the ULS. As such. settlements for foundations seated within bedrock are to be deemed negligible (i.e. less than 15 mm).

5.4 Seismic Design Considerations

Based on the soil conditions encountered, and in accordance with Table 4.1.8.4.A. of the current Ontario Building Code (herein "*OBC*"), the site is considered to be a 'C' Site Class. The acceleration and velocity-based site coefficients, F_a and F_v, should be determined from Tables 4.1.8.4.B. and 4.1.8.4.C. respectively of the OBC for the above recommended Site Class. The seismic design data given in Table 1.2 of Supplementary Standard SB-1 in Volume 2 of the OBC, for selected Municipal locations, should be used to complete the seismic analysis.

5.5 Damp Proofing and Waterproofing Considerations

All subsurface structures (i.e. basement walls, floor slabs etc.) should be damp proofed where above the seasonally high groundwater level established by the proposed Hydrogeological Assessment plus the required buffer zone (nominally 1.0 m to 1.5 m above the stabilized or highest recorded groundwater level). As a minimum, it is recommended that the damp proofing system include a Delta Drainage Board or MiraDrain 2000 series product, or an approved alternative, along with an asphalt-based spray-on wall coating and should comply with the OBC requirements.

It is recommended that all subsurface structures and areas are appropriately waterproofed where below the seasonally high groundwater level established by the proposed Hydrogeological Assessment plus the required buffer zone (nominally 1.0 m to 1.5 m above the stabilized or highest recorded groundwater level).

6.0 FLOOR SLAB AND PERIMETER DRAINAGE CONSIDERATIONS

Based on the borehole soil conditions and preliminary design information provided to Landtek, it should be possible to construct the lowest (i.e. basement) floor slab level using slab-on-grade methods. The subgrade support condition is anticipated to be shale bedrock, which should provide competent conditions for placing the vapour barrier material. However, after the subgrade has been prepared to the underfloor design elevation it is recommended that the area be assessed by Landtek to determine if there is a need for any remedial work.

It is recommended that a minimum 200 mm layer of clear, 19 mm crushed quarried stone be used as the vapour barrier under the floor slab. The vapour barrier stone should meet the requirements of Ontario Provincial Standard Specifications (herein "*OPSS*") 1004 for 19 mm Type II clear stone. If a graded crushed stone is substituted for clear stone, the material should be limited to a maximum of 5 % fines (passing the 0.075 mm sieve). The floor slab thickness should meet the specifications of the project based on anticipated floor loadings.

The finished exterior ground surface should be sloped away from the buildings at a grade in the order of 2 %.

The concrete properties should meet the requirements of OPSS 1350. Contraction and isolation jointing practices should be in accordance with current Portland Cement Association recommendations, as given in the engineering bulletin "*Concrete Floors on Ground*", second edition, by R. E. Spears, and W. C. Panarese.

The design of concrete slabs may be made on the basis of a value of modulus of subgrade reaction of 200 MPa/m for the shale bedrock.

Perimeter drainage should be provided around all subsurface floor areas where water may accumulate. Underfloor drains may be required depending on excavation and groundwater seepage conditions. It is anticipated that, based on the assumed maximum of five levels of basement, groundwater seepage should be anticipated. This given, it is recommended that underfloor drains be provided at all deep foundation locations. The drainage system should comply with the OBC and associated amendments. Further details pertaining to perimeter and underfloor drainage systems are provided in Drawings 21086-02 and 21086-03 respectively, in Appendix D.

After the subgrade has been prepared to the underfloor design elevation it is recommended that the area be proof-rolled with a loaded tandem axle dump truck to delineate if there are soft or unstable ground conditions that require repair. This operation should be completed before the underfloor vapour barrier granular material is placed.

7.0 EARTH PRESSURE CONSIDERATIONS ON SUBSURFACE WALLS

The earth pressure, p, acting on subsurface walls at any depth, h, in metres below the ground surface assumes an equivalent triangular fluid pressure distribution and may be calculated using the expression below. It is assumed that granular material is used as backfill. Allowances for pressure due to compaction operations should be included in the earth pressure determinations and a value of 12 kPa is applicable for a vibratory compactor and granular material.

If the structure retaining soil can move slightly, the active earth pressure case can be used in determining the lateral earth pressure. For restrained structures and no yielding an "at rest" earth pressure condition should be used. The determination of the earth pressures should be based on the following expression:

$$P_1 = K (\delta h + q)$$

where:

P_1 = the pressure in kPa acting against any subsurface wall at depth, h, in metres (feet) below the ground surface;

K = the at rest earth pressure coefficient considered appropriate for subsurface walls; OPSS 1010 Granular B Type 1 (pit-run sand and gravel) material has an effective angle of friction estimated to be 32° with a corresponding at rest earth pressure coefficient, K_o, of 0.45; and,

δ = the moist bulk unit weight of the retained backfill; 21.5 kN/m^3.

and,

q = the value for any adjacent surcharge in kPa, which may be acting close to the wall; and,

h = the depth, in m, at which the pressure is calculated

For subsurface walls below the shallowest reported water level elevation of approximately 91.3 m, the pressure distribution on the wall should include the hydrostatic pressure. The determination of hydrostatic pressure should be based on the following expression:

$$P_2 = \delta_w h_w$$

where:

P_2 = hydrostatic pressure;

δ_w = unit weight of water; 9.8 kN/m^3;

h_w = depth of wall, below reported water level.

Granular B backfill should meet OPSS 1010 Type I or Type II material specifications. The granular fill should be compacted to a minimum of 97 % of the material's Standard Proctor Maximum Dry Density (herein "*SPMDD*"), or to the levels and backfilling procedures specified.

Given the presence of shale bedrock beneath the site, the following parameters should be applied for the bedrock when considering lateral pressures on subsurface walls:

- Internal angle of friction (ϕ) should be taken as 28°; and,
- Bulk unit weight (γ) should be taken as 25.4 kN/m^3.

In designing a subsurface wall within bedrock, a uniform pressure distribution is assumed and is consistent with the maximum earth pressure calculated for the wall where in soil. However, below the weathered bedrock zone, the design does not accommodate for lateral rock swell.

Lateral rock swell will be of concern as fresh bedrock will be exposed in the excavation faces and will be more prone to swell than the shallower, more weathered bedrock zone. No real rate of swell can be put on the bedrock as it is very variable.

There are various approaches to overcome rock swell in the shorter term for design and excavation. There is a generally accepted industry practice which assumes that bedrock exposed for a period of more than 120 days will have swelled such that no significant stresses will be exerted upon foundation walls. Alternatively, a number of methods of mitigation are generally accepted to include:

- The limiting of bedrock exposure to no more than 7 days;
- The application of a mud-mat/shotcrete onto the exposed rock face once excavated;
- The installation of a layer of compressible fill material (e.g. Ethafoam Plank or GeoFoam products) between the rock face and the back of the structural wall; and,
- The over-excavation of the bedrock ±0.6 m and backfilling the void space with 19 mm Clear Stone or compressible fill.

It should be noted that the variability of the Queenston Formation shales means that, without site-specific testing, no definitive time at which point the swell becomes negligible can be assigned.

8.0 SOIL CORROSIVITY AND SUBSURFACE CONCRETE

8.1 Concrete Class Considerations

The requirements for subsurface concrete subject to a sulphate and chloride environment are presented in Canadian Standards Association specification, CSA A23.1-14 *"Concrete Materials and Methods of Concrete Construction, Tables 1-4"*.

Experience in the area indicates that the native soils generally have a mild sulphate environment and are not aggressive to concrete (CSA criteria of less than 0.2 % water soluble sulphate in the soils). Therefore, normal General use (GU) hydraulic cement can be used for the below grade concrete structures. It is recommended that the concrete exposure class for the subsurface concrete structures, parking garage decks and ramps be C-1 and the concrete have the following minimum properties:

- minimum 56-day compressive strength: 35 MPa;
- maximum water to cement ratio: 0.40;
- chloride ion penetrability requirement: < 1500 coulombs (within 91 days)
- cementing materials: GU (general use hydraulic cement) or GUb (blended general use)
- air content: as per CSA A23.1-14 Table 4, air content category 1 (freeze-thaw environment)

The concrete should be placed without segregation and should be consolidated to achieve a uniform dense mass.

8.2 Methods for Specifying Concrete

Alternative methods of specifying concrete for a project are outlined in CSA A23.1-14 and allow for "*Performance*" or "*Prescription*" based methods. Each method attaches different levels of responsibility to the owner, the contractor, and the concrete supplier. The pros and cons of each method should be examined prior to completion of the specifications for the project.

9.0 EXCAVATION AND BACKFILL CONSIDERATIONS

9.1 General Excavation Considerations

All temporary excavations and unbraced side slopes in the soils should conform to standards set out in the Occupational Health and Safety Act, Ontario Regulation 213/91 "*Construction Projects*" (herein "*OHSA*"). The subsurface soils to be encountered during excavation at the site are expected to behave as "*Type 3"* and "*Type 2*" materials according to the OHSA classification in Part III. Type 3 soils are characteristic of the "*fill material*" and "*native silt and sand deposits*" while the generally stiff "*native clayey silt till"* is considered a Type 2 soil. The "*shale bedrock*" encountered at depth has strength properties that exceed a Type 1 soil.

It should be possible to excavate the overburden soils using a hydraulic backhoe. Moist Type 3 and Type 2 soils are expected to be stable for short construction periods at slopes of approximately 45° to the horizontal (i.e. 1V:1H).

A backhoe equipped with a hydraulic breaker and/or a bucket with rock-ripping 'tiger teeth' may be required in the shale strata and particularly where seams and bands of harder strata (such as siltstone or limestone) are encountered within the shale. Significant ground vibrations resulting from excavation works are not anticipated other than those associated with normal construction activities. The shale is expected to remain relatively stable at near vertical slopes for short periods of time. It is recommended that the shale excavation slopes be scaled of loose rock pieces and overhang and cut back to approximately 10V:1H.

Consideration should be given to existing service trenches and backfill that may be present directly behind cut slopes within the native soils that may appear to be stable on first excavation. In these circumstances, slopes can suddenly slough or collapse due to the effects of the adjacent backfill. Consequently, for excavation conditions that cannot satisfy the OHSA requirements for unbraced 1H:1V side slopes, a trench box system should be used, or temporary shoring should be installed to maintain safe working conditions. This may be more applicable to service trench excavations, though may also apply to basement excavations etc., particularly where in close proximity to new road pavements or associated infrastructure.

It should be noted that the design of a temporary shoring system, should one be required, is the responsibility of the Contractor. Therefore, a specialist shoring contractor should be consulted to provide the most appropriate shoring type method and associated installation procedures. In any event, the shoring design should be based on the procedures outlined in the latest edition of the "*Canadian Foundation Engineering Manual*". It is also recommended that lateral and vertical movement of the shoring system be monitored during construction to ensure that movements are within the acceptable range.

Excavations for new foundations should satisfy the criteria given in the example shown in Figure 9.1.1 to avoid overlapping stresses and minimize the risk of undermining existing adjacent structures, including utilities, and/or triggering additional settlements of the existing structures due to soil disturbance.

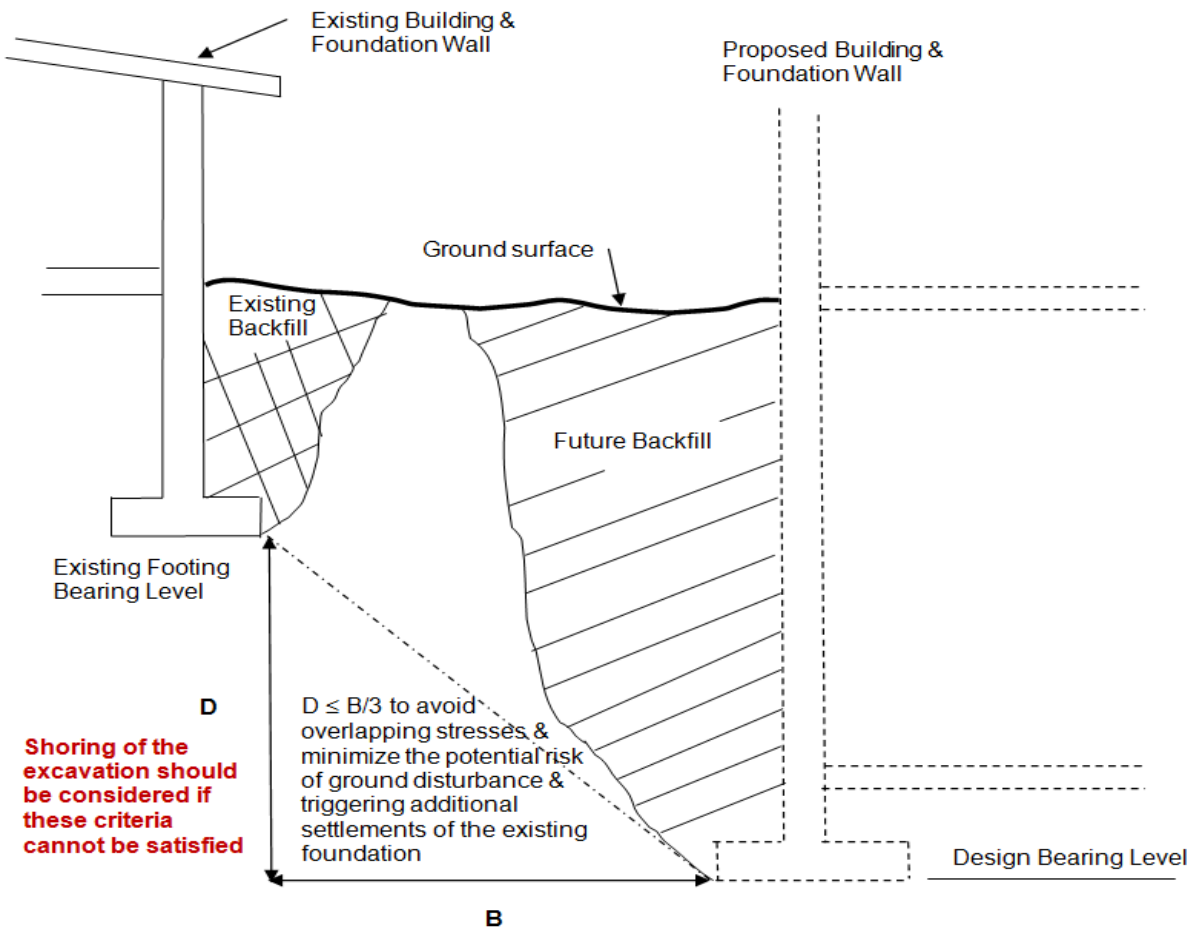


Figure 9.1.1: Criteria for Assessing Excavation Shoring Requirements (Not to Scale)

9.2 Construction Dewatering Considerations

Considerations regarding groundwater control and dewatering requirements during construction are to be provided by the proposed Hydrogeological Assessment for the development.

9.3 General Backfill Considerations

Backfill next to foundation walls and in service trenches should be selected to be compactable in narrow trench conditions. The on-site clayey silt to silty clay till and weathered shale are expected to be reusable as trench backfill and backfill around the proposed structures on the site. Any variation in the moisture contents of the soils encountered may require selective separation of material to avoid the use of wet soil. Experience with shale indicates that long term settlements can occur unless quality control procedures are in place to ensure the shale particle sizes are broken down to minus 200 mm material, the moisture content of the material is maintained at near optimum levels, and the material is placed in maximum lift thicknesses of 300 mm.

Site servicing trench backfill should be uniformly compacted to a density that minimizes the risk of long-term settlements. It is recommended that the target compaction specification for trench backfill be 97 % SPMDD with no individual test below 95 % SPMDD.

During inclement weather the native soils may become too wet to achieve satisfactory compaction. If construction is proposed for late in the year, a reduced level of trench compaction with a higher risk of future settlements is to be anticipated, and it is recommended that provisional contract quantities be established for the supply and placement of imported granular fill under such circumstances. The imported granular should meet the requirements of OPSS 1010 for Granular B Type I material as a minimum requirement.

9.4 Bedrock Considerations

Competent shale of the Queenston Formation has the characteristics of becoming soft or degraded after excavation and subsequent exposure to the elements, the results of which would be basal heaving and compression from rock squeezing along excavation side walls. As such, these effects should be minimized during construction, and requires a well-planned construction program to ensure that the exposure of the shale bedrock is kept to a minimum, as identified in Section 7.0.

Methane gas is known to be present within the fracture networks of the Queenston Formation shale, normally below the top 1 m and becoming more concentrated with depth. As such, the potential could exist for the development of an explosive or oxygen-depleted air environment. Therefore, Landtek recommends that the appropriate air space monitoring is undertaken within all confined excavations, particularly those located close to or within bedrock, as defined by the OHSA.

10.0 TEMPORARY SHORING CONSIDERATIONS

The installation of temporary shoring is also recommended to maintain safe working conditions and eliminate the possibility of loss of ground and damage to nearby structures and buried utilities on the adjacent road allowances during excavation for the basement construction.

The requirement and application of shoring to support excavation side slopes will be dependent on the required excavation depth and the proximity of existing or newly constructed infrastructure adjacent to the excavation.

The preferred method of shoring will consist of a concrete caisson wall. This type of system is expected to provide the additional benefit of sealing the excavation from water penetration and loss of soil fines into the open excavation. Soldier piles and timber lagging may be considered as an option for a shoring system, though this type of system may require measures to prevent the loss of soil between the spaces of lagging boards where a wet or flowing soil layer may be present. Consideration may be also given to the application of shotcrete where groundwater is encountered and/or where Queenston Formation shale bedrock is exposed in the excavation faces.

The shoring methods may provide lateral restraining force through the use of rakers or tieback anchors. Tieback anchors provide additional advantage since they do not protrude into the excavations as rakers would. However, the use of tieback anchors is also dependent upon whether permission is needed or whether it is physically possible to extend the anchors to the required distance into neighbouring properties.

It should be noted that the design of any temporary shoring system is the responsibility of the Contractor. Therefore, a specialist shoring contractor should be consulted to provide the most appropriate shoring type method and associated installation procedures. In any event, the shoring design should be based on the procedures outlined in the latest edition of the Canadian Foundation Engineering Manual. It is also recommended that lateral and vertical movement of the shoring system be monitored during construction to ensure that movements are within the acceptable range.

11.0 SITE SERVICING CONSIDERATIONS

There is no indication that special pipe bedding materials or procedures are required for the installation of services. All bedding cover and backfill materials should be selected in accordance with OPSS 1010 Aggregates – Base, Subbase, Select Subgrade, and Backfill Material.

The pipes should be placed with a minimum bedding thickness in conformance of OPSD 802.010, 802.013 and 802.014 for flexible pipe and OPSD 802.030, 031, 032, 033 and 034 for rigid pipes. The type of bedding shall be selected to suit the applicable pipe strength and site conditions.

Bedding material shall be placed in layers not exceeding 300 mm in thickness, loose measurement, and compacted to 95 % of the SPMDD before a subsequent layer is placed. Site servicing trench backfill should be uniformly compacted to a density that minimizes the risk of long-term settlements. Bedding on each side of the pipe shall be completed simultaneously. At no time shall the levels on each side differ by more than the 300 mm uncompacted layer. The remainder of the trench should be backfilled as per the requirements defined in Section 9.0.

It is assumed all services will have a minimum of 1.2 m of soil cover for frost protection. For services installed at shallower depths, suitable insulation for frost protection is recommended.

12.0 SOIL MANAGEMENT CONSIDERATIONS

Construction for the proposed development may involve cut and fill operations. From a geotechnical perspective, and in order to optimize the use of the on-site soils, a Soil Management Plan should be established. The plan objective should be to achieve a self-sustainable development with respect to excavated materials and control the placement of organic soils so that there is negligible impact on the settlement performance of the compacted fill material.

The soil management criteria should be as follows:

- Surface vegetation, topsoil and organic soils should not be placed within the proposed roadways, below finished subgrade level for pavement construction or building limits. These materials should be placed in landscaped areas where settlements are not critical;

- Excavated soils for structural fill in pavement areas and building floor slab areas, which does not have topsoil or organic matter and are compactable with moisture contents within 2 % to 3 % of the optimum value, should be placed and compacted to a target density of 97 % of the SPMDD with no individual test result below 95 % SPMDD; if engineered fill is required to support building foundations, the engineered fill should be placed and compacted in lifts to a target density of 100 % SPMDD with no individual tests below 98 % SPMDD; the soil should be placed in a loose lift thickness not exceeding 250 mm and should be compacted using a large (10 ton or larger) pad-foot type roller with vibratory capability; if engineered fill to support building foundations is being considered it is recommended that a pre-construction meeting be scheduled to review the proposed fill materials, fill placement and compaction procedures, and the testing and inspection requirements;

- Soils to be placed in landscaped areas where settlements are not critical should receive nominal compaction effort in order to achieve at least 90 % of the SPMDD; and,

- Prior to the placement of underfloor granular fill, the exposed subgrade soil should be inspected and proof-rolled using a loaded tandem axle truck and traversing the exposed subgrade for full coverage; the proof-rolling should be monitored by a geotechnical representative of this office to delineate any soft areas which may require repair.

13.0 PAVEMENT DESIGN CONSIDERATIONS

13.1 At-Grade Pavement Design Considerations

The proposed residential development includes for an access road pavement and limited parking areas. Recommended pavement structure layer thicknesses are provided in Table 12.1.1. Site specific development requirements set out by the Corporation of the City of Burlington (herein "*City of Burlington*") may override the recommendations of this report.

The recommended pavement design section takes into account the accepted design practice that the total pavement structure thickness should meet or exceed one-half the anticipated depth of frost penetration for the geographical area (i.e. 1.2 m) or as close as practicable.

Table 12.1.1: Recommended Pavement Structure Layer Thicknesses

Pavement Layer	Access and Fire Routes	Light Duty Parking Areas
Surface Course Asphalt OPSS HL 3	40 mm	40 mm
Binder Course Asphalt OPSS HL 8	60 mm	50 mm
Granular Base OPSS Granular A	150 mm	150 mm
Granular Subbase OPSS Granular B, Type II	350 mm	250 mm
Total Thickness	600 mm	490 mm

Notes:
1. If construction proceeds late in the year (i.e. November and December), the design thickness of pavement granular materials may have to be increased to address potential problems with subgrade instability and facilitate construction vehicle and truck access.

13.2 Pavement Construction Considerations

The overall performance of the pavement structure will greatly depend upon the support provided by the developed subgrade. A number of factors should be considered at the construction stages to ensure that an acceptable subgrade condition is developed and maintained:

- Sub-drains should be installed and should be 100 mm diameter perforated plastic pipe, with outfalls to catch basins at a continuous and uniform grade. The sub-drains should conform to OPSD 216.01;
- Any soft areas of notable deflection to the subgrade should be sub-excavated and replaced with a suitable backfill material approved by a qualified geotechnical engineer and compacted to 98 % of its SPMDD;
- The subgrade should be properly shaped, crowned and then proof-rolled under the full-time observation of a geotechnical representative of this office to delineate any soft areas which may require repair before placing the granular materials; and,
- Surface water should not be allowed to pond on the surface of or adjacent to the outside edges of any developed subgrade.

Should the pavements proposed for the development be constructed as a two-stage paving operation it will important to ensure that the following is undertaken to develop the surface of the binder course being used as a "*temporary*" surface during the construction phase:

- The surface is thoroughly cleaned and power washed to remove all residual contaminants;
- All deficiencies are corrected to meet the required design specifications; and,
- A suitable tack coat is appropriately applied immediately prior to the placement of the upper asphaltic concrete course(s).

Such preparatory works are to be completed in accordance with the appropriate OPSS, as required.

13.3 Subgrade Considerations

The subgrade conditions and bearing strength may be variable along the road section and some subgrade repairs should be anticipated. It is therefore recommended that, prior to the placement of pavement granular fill, the exposed subgrade soil should be inspected and proof-rolled using a loaded tandem axle truck to traverse the exposed subgrade and provide for full coverage.

The proof-rolling should be monitored by a geotechnical representative of this office to delineate any soft areas which may require repair. Repairs should be undertaken to avoid creating "*bathtub*" conditions in the subgrade within the pavement structure.

13.4 Deck Pavement Design Considerations

It is understood that the footprint of the proposed development will cover the site area in its majority. As such, most of the proposed pavement structures are anticipated to be deck structures rather than standalone pavement structures.

Such deck pavements should comprise a minimum 50 mm cover of OPSS HL 3 asphalt or minimum 80 mm cover of interlocking concrete pavers. The bedding or grading material to be placed between the concrete deck and the asphalt pavement surface or interlocking concrete pavers should comprise either blinding sand or OPSS Granular A material, depending on the thickness of the layer required.

13.5 Pavement Materials

<u>Granular Base Course</u>

If the option with granular base material is used, the granular base course material should meet OPSS Granular "A" specifications. Quarried 20 mm limestone crushed to Granular "A" gradation specifications is recommended.

<u>Hot Mix Asphalt</u>

The surface course asphalt should meet current specifications for HL 3, as prescribed by the City of Burlington or, alternatively, OPSS 1150.

<u>Compaction</u>

Granular base course fill material should be compacted to 100 % of the material's SPMDD.

14.0 CLOSURE

The Limitations of Report, as stated in Appendix A, are an integral part of this report.

Soil samples will be retained and stored by Landtek for a period of three months after the report is issued. The samples will be disposed of at the end of the three-month period unless a written request from the Client to extend the storage period is received.

We trust this report will be of assistance with the design and construction of the proposed development. Should you have any questions, please do not hesitate to contact our office.

Yours sincerely,

LANDTEK LIMITED



James Dann, B.Eng. (Hons).
Author

Ralph Di Cienzo, P. Eng.
Consulting Engineer

APPENDIX A
LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the borehole locations. Subsurface and ground water conditions between and beyond the Boreholes may be different from those encountered at the borehole locations, and conditions may become apparent during construction that could not be detected or anticipated at the time of the geotechnical investigation. It is recommended practice that Landtek be retained during construction to confirm that the subsurface conditions throughout the site are consistent with the conditions encountered in the Boreholes.

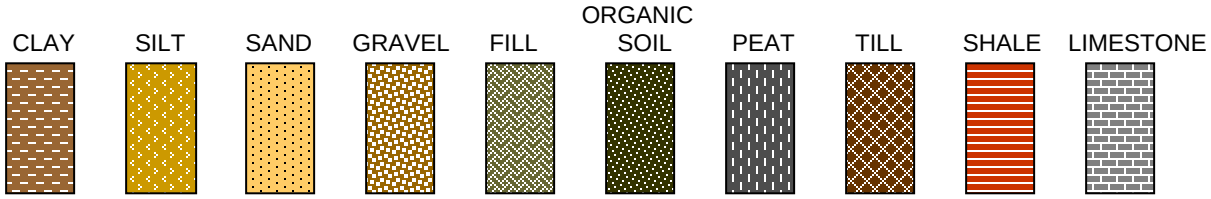
The comments made in this report on potential construction problems and possible remedial methods are intended only for the guidance of the designer. The number of Boreholes may not be sufficient to determine all the factors that may influence construction methods and costs. For example, the thickness and quality of surficial topsoil or fill layers may vary markedly and unpredictably. Additionally, bedrock contact depths throughout the site may vary significantly from what was encountered at the exact borehole locations. Contractors bidding on the project, or undertaking construction on the site should make their own interpretation of the factual borehole information, and establish their own conclusions as to how the subsurface conditions may affect their work.

The survey elevations in the report were obtained by Landtek Limited or others, and are strictly for use by Landtek in the preparation of the geotechnical report. The elevations should not be used by any other parties for any other purpose.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Landtek Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this report.

This report does not reflect environmental issues or concerns related to the property unless otherwise stated in the report. The design recommendations given in the report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, it is recommended that Landtek Limited be retained during the final design stage to verify that the design is consistent with the report recommendations, and that the assumptions made in the report are still valid.

APPENDIX B
SYMBOLS AND TERMS USED IN THE REPORT



RELATIVE PROPORTIONS

Term	Range
Trace	0 - 5%
A Little	5 – 15%
Some	15 – 30%
With	30 – 50%

CLASSIFICATION BY PARTICLE SIZE

Boulder ---------- > 200 mm
Cobble ---------- 80 mm – 200 mm
Gravel -
Coarse ---------- 19 mm – 80 mm
Fine ---------- 4.75 mm – 19 mm
Sand -
Coarse ---------- 4.75 mm – 2 mm
Medium -------- 2 mm – 0.425 mm
Fine ---------- 0.425 mm – 0.75 mm
Silt ---------- 0.075 mm – 0.002 mm
Clay ---------- < 0.002 mm

DENSITY OF NON-COHESIVE SOILS

Descriptive Term	Relative Density	Standard Penetration Test
Very Loose	0 – 15%	0 – 4 Blows Per 300 mm Penetration
Loose	15 – 35%	4 – 10 Blows Per 300 mm Penetration
Compact	35 – 65%	10 – 30 Blows Per 300 mm Penetration
Dense	65 – 85%	30 – 50 Blows Per 300 mm Penetration
Very Dense	85 – 100%	Over 50 Blows Per 300 mm Penetration

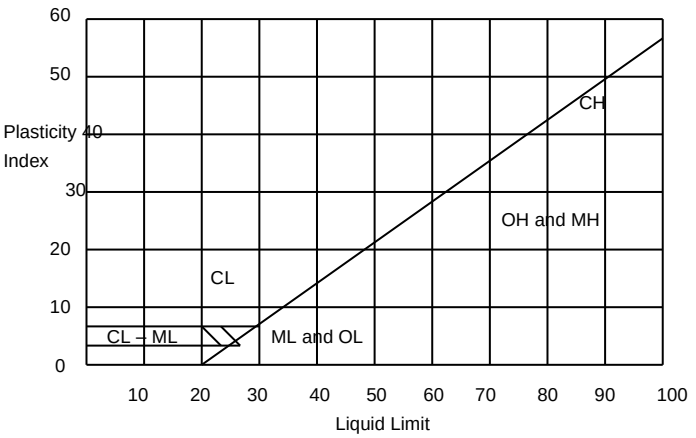
CONSISTENCY OF COHESIVE SOILS

Descriptive Term	Undrained Shear Strength kPa (psf)	N Value Standard Penetration Test	Remarks
Very Soft	< 12 (< 250)	< 2	Can penetrate with fist
Soft	12 – 25 (250 – 500)	2 – 4	Can indent with fist
Firm	25 – 50 (500 –1000)	4 – 8	Can penetrate with thumb
Stiff	50 – 100 (1000 – 2000)	8 – 15	Can indent with thumb
Very Stiff	100 – 200 (2000 – 4000)	15 – 30	Can indent with thumb-nail
Hard	> 200 (> 4000)	> 30	Can indent with thumb-nail

Notes: 1. Relative density determined by standard laboratory tests.
2. N value – blows/300 mm penetration of a 623 N (140 Lb.) hammer falling 760 mm (30 in.) on a 50 mm O.D. split spoon soil sampler. The split spoon sampler is driven 450 mm (18 in.) or 610 mm (24 in.). The “N” value is the Standard Penetration Test (SPT) value and is normally taken as the number of blows to advance the sampler the last 300 mm.

APPENDIX B CONTINUED
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES
ASTM Designation: D 2487 - 69 AND D 2488 – 69
(Unified Soil Classification System)

Major Divisions			Group Symbols	Typical Names	Classification Criteria		
Coarse-grained soils More than 50% retained on No. 200 sieve *	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	Classification on basis of percentage of fines Less than 5% pass No. 200 sieve . . . . . GW, GP, SW, SP More than 12% pass No. 200 sieve . . . . . GM, GC, SM, SC 5 to 12% pass No.200 sieve . . . . . Borderline classifications requiring use of dual symbols	C_u=D60/D10 greater than 4; C_z = $(D30)^2$/(D10xD60) between 1 and 3	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines		Not meeting both criteria for GW	
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
	Sands More than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines		C_u=D60/D10 greater than 6; C_z = $(D30)^2$/ (D10xD60) between 1 and 3	
			SP	Poorly graded sands and gravelly sands, little or no fines		Not meeting both criteria for SW	
		Sands with fines	SM	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
Fine-grained soils 50% or more passes No. 200 sieve *	Silts and clays Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands	Plasticity Chart For classification of fine-grained soils and fine fraction of coarse-grained soils. Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols. Equation of A-line: PI=0.73 (LL-20)		
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silts			
			OL	Organic silts and organic silts of low plasticity			
	Silts and clays Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity			
	Highly organic soils		Pt	Peat, much and other highly organic soils	* Based on the material passing the 3 in. (76mm) sieve.		



APPENDIX C

DRAWING 21086-01 – BOREHOLE LOCATION PLAN
BOREHOLE LOGS

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project location



Key plan provided by Google Earth.

Approximate location of boreholes as drilled by Landtek Limited on July 12th, 2016.

Approximate location of boreholes as drilled by Landtek Limited on May 9th, 2014.

Borehole and monitoring well locations are considered approximate.

Base plan provided by H. Gelbloom Surveying Ltd.

revisions/ submissions

#	date	description
1	April 15, 2021	Issued for report

client
Adi Developments (Masonry) Inc.

municipality
The Corporation of the City of Burlington

project
Geotechnical Investigation
StationWest Phase 2, 1120 Cooke Boulevard

sheet
Borehole Location Plan

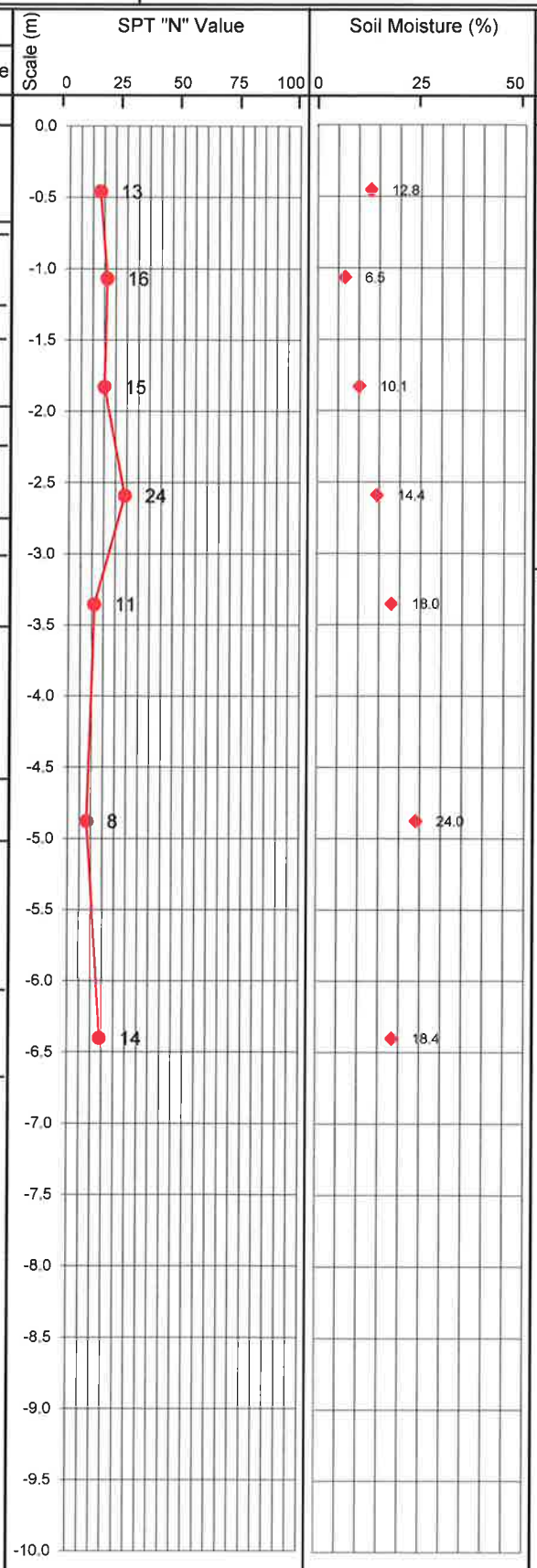
date: April 15, 2021
drawn: JDC
checked: JD
project #: 21086
scale: NOT TO SCALE

21086-01

LANDTEK LIMITED — LOG OF BOREHOLE NO. 7

Project No.:	14123	Drill Date:	May 9, 2014
Project:	Phase Two Environmental Site Assessment	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	101 Masonry Court, Burlington, Ontario	Datum:	Geodetic

Material Description	Symbol	Elev. / Depth	Samples No.	Samples Type	TCV Readings
Ground Surface		103.8			
±75 mm of topsoil		0.0			
FILL mixture of sand, gravel, some silt, moist, brown (Compact)		102.9	1	SS	5 ppm
SAND fine to medium, well graded, moist, brown (Compact)		0.9	2	SS	5 ppm
			3	SS	0 ppm
		101.7			
SILTY SAND AND GRAVEL coarse, trace rootlets, very moist, oxidized brown, (Compact)		2.1	4	SS	0 ppm
		100.8			
SILT fractured structure, clayey with clay seams, fine sand, very moist to wet, reddish-brown (Loose to Compact)		3.0	5	SS	WL = 3.1 m May 9/2014 0 ppm
			6	SS	0 ppm
		98.3			
TILL clayey silt with gravel, red shale pieces, very moist, reddish-brown, (Compact)		5.5	7	SS	0 ppm
		97.2			
BOREHOLE TERMINATED		6.6			



Notes: 1. On completion, borehole open to 6.1 m and water level at 3.1 m.

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

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LANDTEK LIMITED

LOG OF BOREHOLE NO. 8

Project No.:	14123	Drill Date:	May 9, 2014
Project:	Phase Two Environmental Site Assessment	Drill Method:	[ ] solid stem [x] hollow stem [ ] vibratory
Location:	101 Masonry Court, Burlington, Ontario	Datum:	Geodetic

Material Description	Symbol	Elev. / Depth	Samples No.	Samples Type	SPT "N" Value
Ground Surface		103.3			
±75 mm of topsoil		0.0			
FILL mixture of sand, gravel, and silt, moist, brown (Loose to Very Loose)			1	SS	5
			2	SS	2
			3	SS	14
		101.2			
SAND medium to coarse, some silt and gravel sizes, very moist to wet, oxidized brown (Dense)		2.1	4	SS	42
		100.0	5	SS	14
SILT fractured, fine sand, very moist, reddish-brown, (Compact)		3.3			
		99.0			
TILL mixture of clay and silt, some gravel sizes, shale fragments, very moist to wet, reddish-brown (Compact)		4.3	6	SS	10
			7	SS	14
		96.7			
BOREHOLE TERMINATED		6.6			

Scale (m): 0.0, -0.5, -1.0, -1.5, -2.0, -2.5, -3.0, -3.5, -4.0, -4.5, -5.0, -5.5, -6.0, -6.5, -7.0, -7.5, -8.0, -8.5, -9.0, -9.5, -10.0

Soil Moisture (%): 12.0, 17.0, 14.2, 15.0, 21.1, 20.0, 20.6

GWL

TCV Readings:
- 0 ppm
- 5 ppm
- WL = 1.2 m August 6/2014
- 25 ppm
- 10 ppm
- sand backfill to 2.7 m
- 3.0 m 0 ppm
- 3.0 m of slotted 50 mm dia. PVC pipe
- 0 ppm
- WL = 6.0 m May 9/2014
- 6.1 m 0 ppm

Notes:
1. On completion, borehole open to 6.1 m and water level at 6.0 m.
2. Water level measured at 1.2 mbg on Aug.6, 2014.

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

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BH 1

Project No.:	21086	Drill Date:	July 12, 2016
Project:	Geotechnical Investigation	Drill Method:	[ ] solid stem [x] hollow stem [x] Coring
Location:	StationWest Phase 2, 1120 Cooke Blvd., Burlington	Datum:	Geodetic

Material Description	Symbol	Elev. Depth	Samples No.	Samples Type	Scale (m)	SPT "N" Value	Soil Moisture (%)	GWL	Monitor Details	Test Data
Ground Surface		102.7								
FILL: sand and gravel, trace silt, brown, damp		0.0	1	SS		9	11			
		1.5								
SILTY SAND: clayey, trace gravel, trace sand seams, loose, damp to wet		101.2	2	SS		5	20			
		3.0	3	SS		5	20			
CLAYEY SILT TILL: trace to some sand, trace gravel, red shale fragments, brown becoming grey, firm to stiff, moist		99.7	4	SS		10	13			
			5	SS		8	21			
			6	SS		9	21			
			7	SS		10	20			
… wet below approximately 8.0 m depth			8	SS		14	20			
		9.2	9	SS		87/250 mm	10			
BEDROCK shale, highly weathered to weathered, weak, red		93.5								
Recovery = 26 %; RQD = 25 % highly weathered, weak, red		1	10	SS		50/50 mm	8			
Recovery = 98 %; RQD = 35 % weathered, weak, red and grey		2	CORE RUN #1 TO RUN#6	63 mm Dia. Core						
Recovery = 93 %; RQD = 55 % weathered, red and grey, moderately strong		3								
Recovery = 95 %; RQD = 65 % slightly weathered, red and grey, strong ∂ = 25.4 kN/m³; UCS strength = 56.8 MPa		4								
Recovery = 95 %; RQD = 55 % slightly weathered, red and grey, strong		5								
Recovery = 92 %; RQD = 35 % fresh, red and grey, strong ∂ = 25.4 kN/m³; UCS strength = 71.6 MPa		6								
		84.5								
END OF BOREHOLE AT TARGET DEPTH		18.6								

Notes:
1. Wet soils encountered between approximately 2.5 m and 3.0 m, and at 8.0 m.
2. Borehole open on completion.

PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

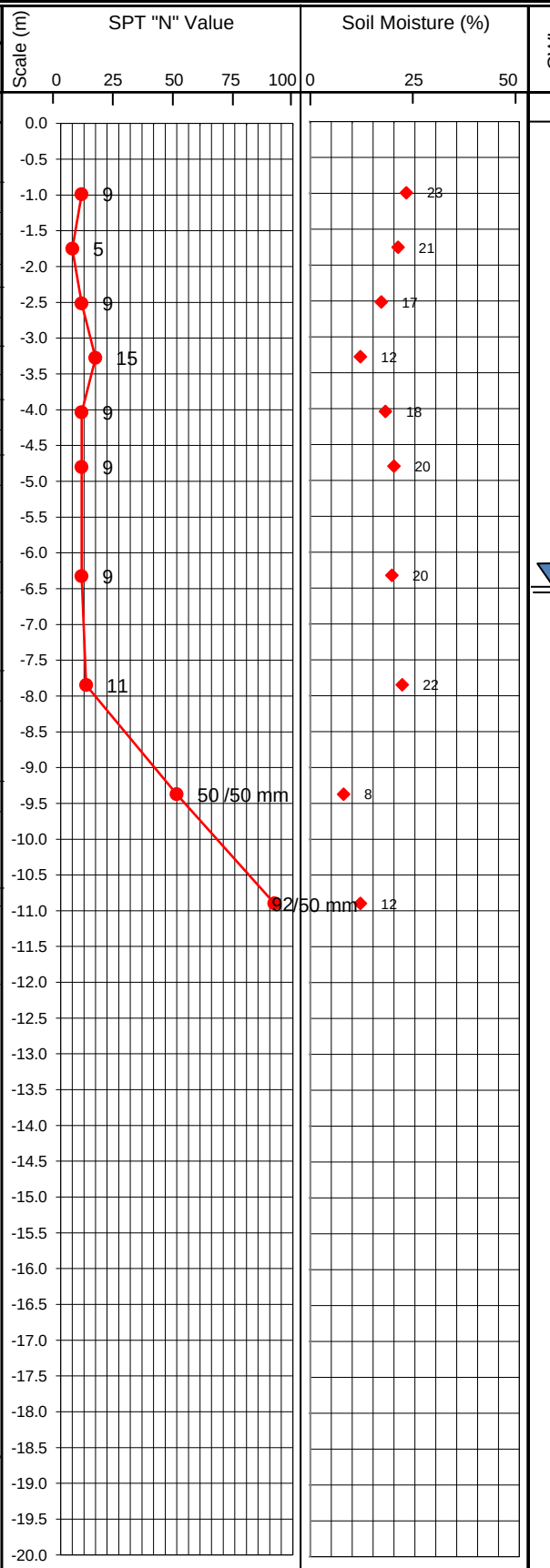
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BH 2

Project No.:	21086	Drill Date:	July 12, 2016
Project:	Geotechnical Investigation	Drill Method:	[ ] solid stem [x] hollow stem [x] Coring
Location:	StationWest Phase 2, 1120 Cooke Blvd., Burlington	Datum:	Geodetic

Material Description	Symbol	Elev. Depth	Samples No.	Samples Type
Ground Surface		103.1		
FILL: sand and gravel, trace silt, brown, damp		0.0	1	SS
		1.5	2	SS
SILTY SAND: clayey, trace gravel, trace sand seams, loose, damp to wet		101.6	3	SS
		3.0	4	SS
CLAYEY SILT TILL: trace to some sand, trace gravel, red shale fragments, brown becoming grey, stiff, moist		100.1	5	SS
			6	SS
… wet below approximately 6.5 m depth			7	SS
… sand and gravel seams below approximately 7.0 m depth			8	SS
		9.2	9	SS
BEDROCK shale, completely to highly weathered, very weak, red		93.9	10	SS
highly weathered, weak, red		1	CORE RUN #1 TO RUN#5	63 mm Dia. Core
Recovery = 93 %; RQD = 38 % weathered, red and grey, moderately strong		2		
Recovery = 99 %; RQD = 65 % slightly weathered, red and grey, strong		3		
Recovery = 99 %; RQD = 75 % slightly weathered, red and grey, strong		4		
Recovery = 99 %; RQD = 60 % fresh, red and grey, strong		5		
		84.5		
END OF BOREHOLE AT TARGET DEPTH		18.6		



GWL | Monitor Details | Test Data

Notes:
1. Wet soils encountered between approximately 2.5 m and 3.0 m, and at 8.0 m.
2. Borehole open on completion.

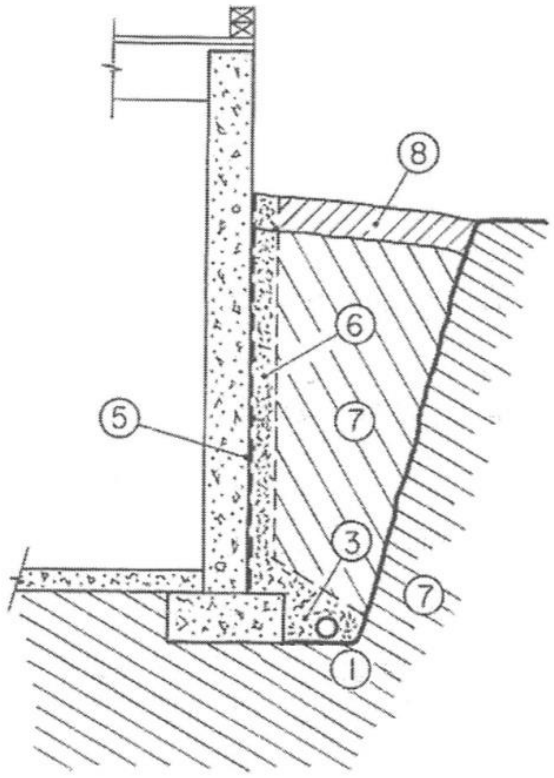
PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

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APPENDIX D

DRAWING 21086-02 – ENGINEERING COMMENTARIES – GENERAL REQUIREMENTS FOR DRAINAGE TO BASEMENT STRUCTURES

DRAWING 21086-03 – ENGINEERING COMMENTARIES – GENERAL REQUIREMENTS FOR UNDERFLOOR DRAINAGE SYSTEMS



① 100 mm, perforated or slotted pipe placed below the upper level of the floor slab.;
③ Filter material that is compatible with the grain size characteristics of the fine grained foundation and backfill soils, as well as with the perforations of the pipe;
④ Filter material continuously or intermittently placed next to the foundation wall to intercept water draining from window wells, down exterior walls and from low areas near the building;
⑤ Damp-proofing on wall – optional depending on the quality of the concrete wall;
⑥ Optional use of sheet drain, or synthetic fire blanket, next to the foundation wall to replace the soil filter according to ④;
⑦ Foundation and backfill soils, which may contain fine grained and erosion-susceptible materials;
⑧ "*Topping off*" material is to be graded such that it slopes outwards to lead surface water away from the building. It is usually desirable to use low permeability topsoil to reduce the risk of overloading the drainage pipe.

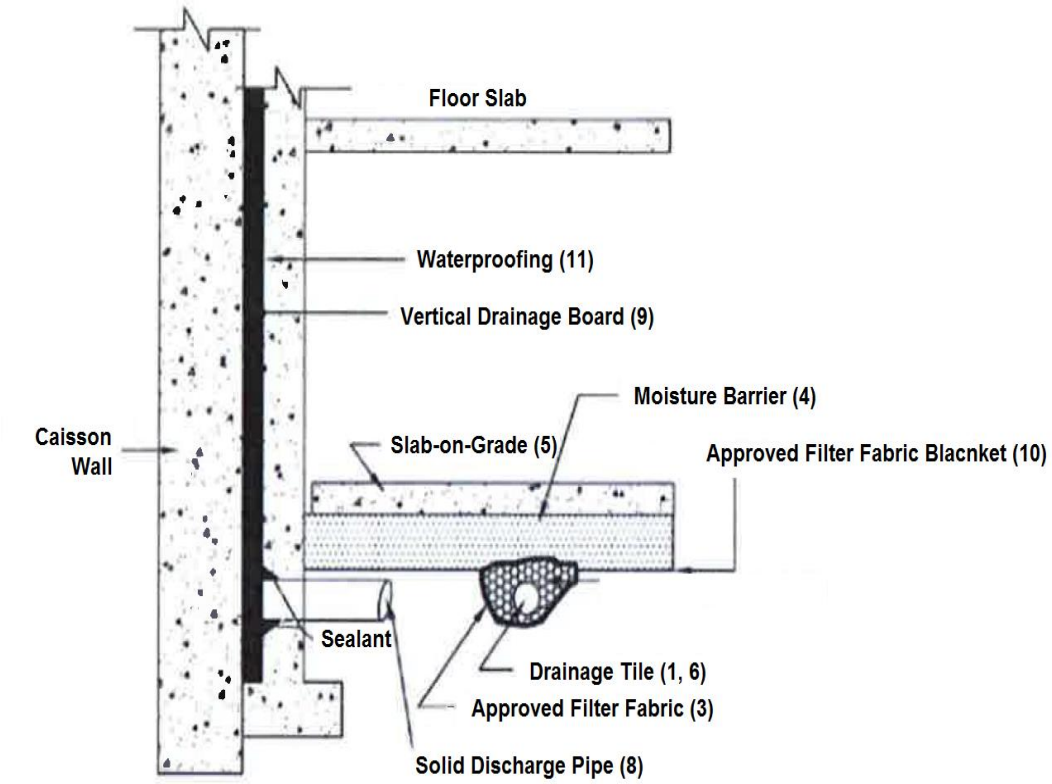
Based on Figure 12.1, Canadian Foundation Engineers Manual, Fourth Edition, 2006.

Additional Notes:

1. The perforated or slotted drainage pipe is to lead to a positive drainage sump or outlet. The invert of the pipe is to be a minimum of 150 mm below the underside of the proposed floor slab.
2. Backfill materials to the interior of the foundation walls may be clean, organic-free soils that can be compacted to the specified density within in a confined space.
3. Heavy, vibratory compaction equipment should not be used within 450 mm of the foundation wall. Fill is not to be placed or compacted within 1.8 m of the wall unless fill is being placed simultaneously on both sides of the wall.
4. The moisture barrier beneath the floor slab is to comprise at least 200 mm of compacted19mm clear stone or an equivalent free-draining material.
5. Should the 19 mm clear stone require surface blinding then 6mm stone chips are to be used.
6. The slab on grade should not be structurally connected to the foundation wall or footing.

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General Requirements for Drainage to Basement Structures			
client	Adi Developments (Masonry) Inc.		
project	StationWest Phase 2, 1120 Cooke Blvd, Burlington		
project #	21086	drawing #	21086-02



Notes:

1. Drainage tile to consist of 100 mm diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet, spaced between columns;
2. 19 mm clear stone – 150 mm top and side of drain. If the drain is not on the footing then place 100 mm of 19 mm clear stone below the drain;
3. Wrap the clear stone with an approved filter fabric (e.g. Terrafix 270R or equivalent);
4. Moisture barrier to be at least 200 mm of compacted, 19 mm clear stone or equivalent (and approved), free-draining material. A vapour barrier may be required for specialty floor coverings;
5. Typically, the slab-on-grade is not structurally connected to the wall or footing. However, if it is connected to the walls it should be designed accordingly;
6. Underfloor drain invert to be at least 300 mm below underside of floor slab. Drainage tile should be placed in parallel rows 6 m to 8 m centres one way. Place drains on 100 mm of 19 mm clear stone and 150 mm of 19 mm clear stone on top and sides. Enclose clear stone with filter fabric as prescribed in Note (3);
7. Do not connect underfloor drainage to perimeter drainage. The two systems are to remain separate.
8. Locate solid discharge at the middle of each bay between soldier piles;
9. Vertical drainage board (e.g. MiraDrain 6000 or equivalent) with filter cloth should be continuous from bottom to 1.2 m below exterior finished grade;
10. The entire subgrade is to be sealed with an approved filter fabric as in Note (3) where non-cohesive (silty/sandy/granular) soils are encountered below the groundwater table;
11. The basement walls must be waterproofed using bentonite or an equivalent waterproofing system;
12. The Geotechnical Report should be reviewed for site-specific details. Final detail must be approved before system is considered acceptable.

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General Requirements for Underfloor Drainage Systems			
client	Adi Developments (Masonry) Inc.		
project	StationWest Phase 2, 1120 Cooke Blvd, Burlington		
project #	21086	drawing #	21086-03