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Geotechnical Investigation
Proposed Residential Development

1167 Plains Road East
Burlington, Ontario
L7S 2K2

Prepared for:

1134/1167 Plains Road East
% Molinaro Group
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EXECUTIVE SUMMARY

SCOPE OF SERVICES

Proposed Development	1134/1167 Plains Road East is intending to develop the site identified as civic address 1167 Plains Road East in Burlington, Ontario for a primarily residential end use. Based on the Proposed Site Plan provided to Landtek, as developed by Graziani + Corazza Architects Inc., the proposed structure is to comprise of a 35-storey tower and a 30-storey tower connected by a two-storey podium. The structure will include a maximum of seven 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 mixed-use development.

SITE DETAILS AND SETTING

Coordinates	595218, 4798120	**Site Area (approx.)**	8,019 m^2 (1.98 acres)
Site Description	It is situated in a developed predominantly commercial area and is bounded by Plains Road East to the east and a Canadian National Railway (CN) railroad to the west. Commercial premises bound the site to the north, while the Queen Elizabeth Way (QEW) highway is situated approximately 40 m south of the site. The site is currently undeveloped, relatively flat-lying, and is covered predominantly by unmaintained grasses, shrubs, and a small number of trees		
Geology	Topsoil was encountered at the ground level. Underlying the topsoil is fill material comprising of either silty clay/clayey silt or silty sand. The fill extends to a maximum depth of 2.3 m. Underlying the fill is a veneer of sand. Native silty clay to clayey silt till was encountered underlying the sand. The till extends to depths of between 14.3 m and 15.4 m below existing ground level. Red shale bedrock was encountered underlying the till.		
Groundwater	During drilling, wet to saturated soil conditions were observed in boreholes at depths of approximately 1.5 m and 2.3 m respectively 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. A deeper groundwater regime was identified at depths greater than 12.1 m below existing ground level.		

ENGINEERING CONSIDERATIONS

Foundations	It is assumed that the proposed structure will include a maximum of seven 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, or elements thereof, 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 generally have a mild sulphate environment and a low chloride concentration. For parking garage decks and ramps, a C-1 exposure class is recommended.
Dewatering	Considerations for groundwater control during construction have been provided under separate cover.

TABLE OF CONTENTS

SECTION PAGE(S)

1.0 INTRODUCTION 1

2.0 SITE SETTING 2
2.1 Site Location and Description 2
2.2 Published Geology 2

3.0 FIELDWORK AND INVESTIGATION METHODOLOGY 4

4.0 SUBSURFACE CONDITIONS 5
4.1 Overview 5
4.2 Organic Soil 5
4.3 Fill Materials 5
4.4 Sand 5
4.5 Silty Clay to Clayey Silt Till 6
4.6 Bedrock 6
4.7 Groundwater 6

5.0 FOUNDATION DESIGN CONSIDERATIONS 8
5.1 Shallow Foundations on Bedrock 8
5.2 Frost Susceptibility 9
5.3 Settlement Considerations 9
5.4 Seismic Design Considerations 9
5.5 Damp Proofing and Waterproofing Considerations 9

6.0 FLOOR SLAB AND PERIMETER DRAINAGE CONSIDERATIONS 11

7.0 EARTH PRESSURE CONSIDERATIONS ON SUBSURFACE WALLS 12

8.0 SOIL CORROSIVITY AND SUBSURFACE CONCRETE 14
8.1 Soil Corrosivity 14
8.2 Concrete Class Considerations 15
8.3 Methods for Specifying Concrete 15

9.0 EXCAVATION AND BACKFILL CONSIDERATIONS 16
9.1 General Excavation Considerations 16
9.2 Construction Dewatering Considerations 17
9.3 General Backfill Considerations 17
9.4 Bedrock Considerations 18

10.0 TEMPRARY SHORING CONSIDERATIONS 19

11.0 SITE SERVICING CONSIDERATIONS 20

12.0 SOIL MANAGEMENT CONSIDERATIONS 21

13.0 DECK PAVEMENT CONSIDERATIONS 22
13.1 Deck Pavement Design Considerations 22
13.2 Pavement Materials 22

14.0 CLOSURE 23

TABLE OF CONTENTS

SECTION PAGE(S)

TABLES

Table 4.1.1 Summary of Boreholes Conditions Encountered 5
Table 4.7.1 Summary of Water Level Measurements 6
Table 5.1.1 Recommended Limit State Foundation Design Values..... 8
Table 8.1.1 Results of Soil Corrosivity Potential..... 14

FIGURES

Figure 2.1.1 Area of Proposed Development 2
Figure 9.1.1 Criteria for Assessing Excavation Shoring Requirements..... 17

APPENDICES

Appendix A Limitations of Report
Appendix B Symbols and Terms Used in the Report
Classification of Soils for Engineering Purposes
Appendix C Drawing 20256A-01 - Borehole Location Plan
Borehole Logs
Appendix D Drawing 20256A-02 - Engineering Commentaries – General Requirements for Drainage to Basement Structures
Drawing 20256A-03 - Engineering Commentaries – General Requirements for Underfloor Drainage Systems
Appendix E Chemical Laboratory Testing Results
Appendix F Rock Core Testing Results

1.0 INTRODUCTION

Landtek Limited (herein "*Landtek*") is pleased to submit this geotechnical investigation report for the proposed residential development at the site identified as civic address 1167 Plains Road East in Burlington, Ontario. Authorization to proceed with the work was received from Mr. Sam DiSanto on behalf of 1134/1167 Plains Road East on August 12th, 2020. The scope of services undertaken for this geotechnical investigation was completed in accordance with proposal reference P20160R, dated May 22nd, 2020.

1134/1167 Plains Road East is intending to develop the site for a primarily residential end use. Based on the Preliminary Site Plan provided to Landtek, as developed by Graziani + Corazza Architects Inc. (herein "*Graziani + Corazza*"), the proposed development is to comprise the following:

- Tower A: a 35-storey tower complex;
- Tower B: a 30-storey tower complex;
- A two-storey podium connecting the two towers; and,
- An access and egress route from Plains Road East leading into the proposed underground parking levels and landscaped areas.

The development is to include seven levels of underground parking, the structural footprint of which is expected to cover the full site 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 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.

This report has been prepared for the Client, the nominated engineers, designers, and project managers pertaining to the proposed residential development at the site identified as civic address 1167 Plains Road East 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 595218, 4798120 (UTM 17T coordinates). The approximate Geodetic elevation of the ground surface at the borehole locations ranges between 97.4 m and 97.8 m.

The site location is shown in Figure 2.1.1 below.

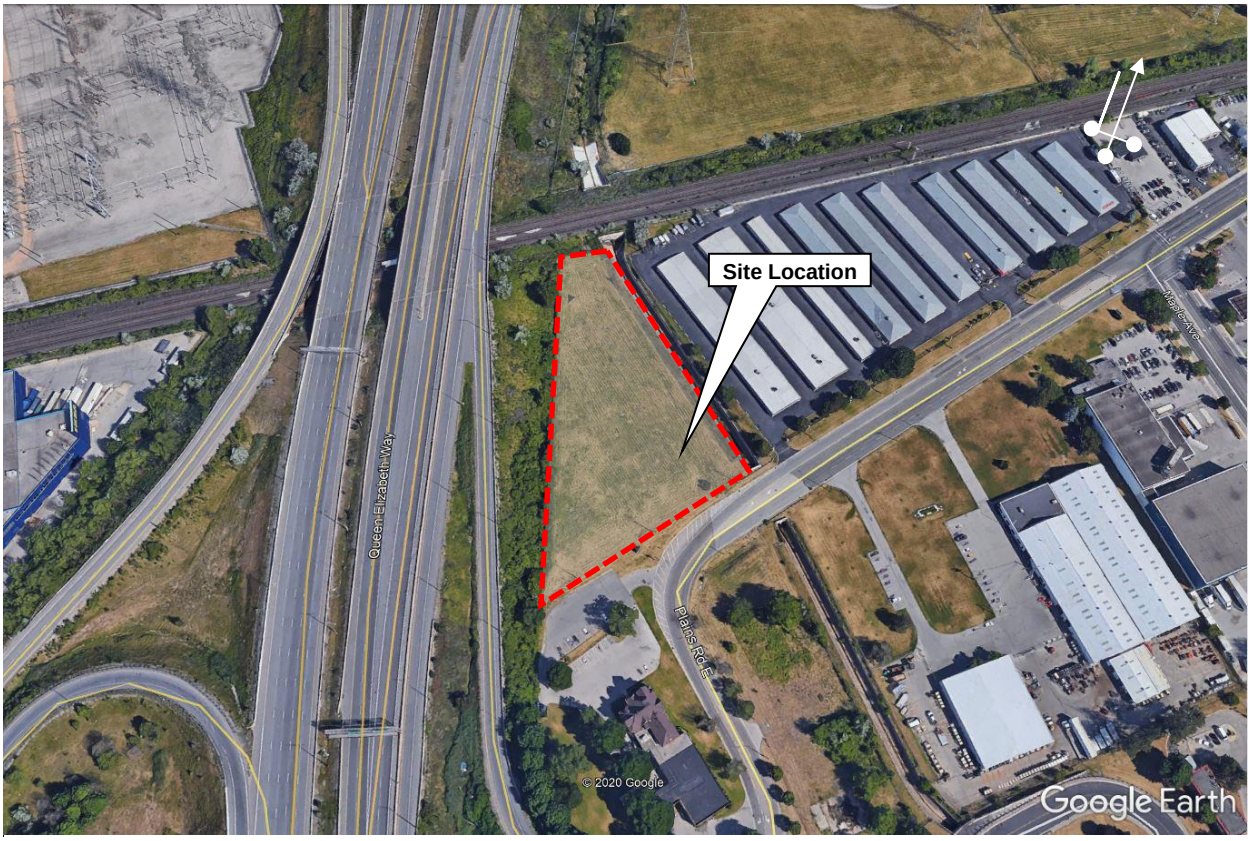


Figure 2.1.1: Area of Proposed Development

The site, identified as "Parcel A" of the Queen Elizabeth + Fairview development area, is approximately 8,019 m^2 (1.98 acres) in plan area. It is situated in a developed predominantly commercial area and is bounded by Plains Road East to the east and a Canadian National Railway (CN) railroad to the west. Commercial premises bound the site to the north, while the Queen Elizabeth Way (QEW) highway is situated approximately 40 m south of the site. The site is currently undeveloped, relatively flat-lying, and is covered predominantly by unmaintained grasses, shrubs, and a small number of trees.

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 14 m to 15 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 1.4 m and overly till deposits. The till deposits extend to a depth of approximately 13.6 m. The bedrock in the area is red shale of the Queenston Formation and is identified as being at depths in the order of 13.6 m to 17.3 m below existing ground level.

3.0 FIELDWORK AND INVESTIGATION METHODOLOGY

Fieldwork undertaken by Landtek at the site, identified as "Parcel A" of the wider Queen Elizabeth + Fairview development area included clearance of underground services, borehole layout, borehole drilling and soil sampling, and field supervision. The geotechnical investigation was undertaken in conjunction with a hydrogeological investigation being conducted by Landtek for the site. Three boreholes (boreholes BH1, BH2 and BH3) were drilled at "Parcel A" between August 26th and September 8th, 2020. The boreholes were logged using those standard symbols and terms defined in Appendix B. The Borehole Location Plan, Drawing 20256A-01, and associated borehole logs are provided in Appendix C.

The boreholes were drilled using a CME-75 track-mounted drilling rig equipped with continuous flight solid and hollow stem augers and were advanced to practical auger refusal on bedrock at depths of approximately 15 m below existing ground level. Standard Penetration Tests (SPT's) and split spoon samples were taken during drilling at selected depths. Boreholes BH2 was then extended into the bedrock using rotary HQ coring methodology to a depth of approximately 23.5 m below existing ground level. Full time supervision of drilling and coring operations was carried out by a representative of Landtek.

Borehole BH2 was completed as a monitoring well to monitor groundwater levels along the proposed anticipated lowest basement excavation and was renamed BH/MW2. 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 logs in Appendix C present the monitoring well installation details and the results of the subsequent groundwater level monitoring visit completed on September 24, 2020.

All soil samples 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. Selected, composite samples were submitted to AGAT Laboratories Ltd. (herein *"AGAT"*) to be analyzed for soil corrosivity to assist with any protective requirements for buried concrete and metal infrastructure.

The rock cores collected were examined to determine their rock classification. Selected rock core samples were subsequently submitted to Geomechanica Inc. (herein "*Geomechanica*") for Unconfined Compressive Strength (UCS) testing. The results of the rock core testing are presented in Appendix F to this report.

Borehole locations were established by Landtek using measurements and offsets relative to existing, permanent site structures and centreline chainages. Approximate ground surface Geodetic elevations at the borehole locations were established by Landtek in reference to the topographical survey for the site, as completed by A.T. McLaren Limited and dated April 6, 2018.

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 sand deposits overlying silty clay to clayey silt till. Red shale bedrock 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 (m) and (Description)		Bedrock Contact
	Topsoil	Fill	Native Soil	
BH1	100 mm	0.1 – 1.5 (silty clay)	1.5 – 3.0 (sand) 3.0 - 15.2 (silty clay to clayey silt till)	15.2 m
BH/MW2	100 mm	0.1 – 1.5 (silty clay)	1.5 – 3.0 (sand) 3.0 - 15.4 (silty clay to clayey silt till)	15.4 m
BH3	100 mm	0.1 – 2.3 (silty sand and clayey silt)	2.3 - 14.3 (silty clay to clayey silt till)	14.3 m

4.2 Organic Soil

An approximately 100 mm thick layer of organic soil was encountered from ground surface in all the boreholes. The organic soil thicknesses may vary across the site, and the thicknesses measured at the borehole locations may not be representative of the topsoil depth throughout the site.

4.3 Fill Materials

Fill materials were encountered in all boreholes underlying the topsoil and extend of depths of between 1.5 m (boreholes BH1 and BH2) and 2.3 m (borehole BH3). The fill materials generally comprised either silty clay to clayey silt or silty sand. The fill materials also include trace fractions of demolition rubble (red brick), gravel, and rootlets.

SPT ‘‘N’’ values ranging from 6 to 29 were reported within the fill materials, indicating their compactness condition to be variable from poor to well, but generally poorly compacted and as expected for uncontrolled or older fill soils.

4.4 Sand

A veneer of native sand was encountered in boreholes BH1 and BH2 underlying the fill and extends to depths of approximately 1.5 m below existing ground level. The sand was observed to be brown in colour and is generally fine to coarse grained. The sand contains some silt and trace fractions of gravel and cobbles.

SPT ‘N’ values ranging from 21 to 49 were reported, indicating the sand to be of a compact to dense condition. Moisture contents are in the order of 8 % to 26 %.

4.5 Silty Clay to Clayey Silt Till

Silty clay to clayey silt till was encountered in all boreholes underlying the sand deposits and extends to depths of between approximately 14.3 m and 15.4 m below existing ground level. The till contains traces of gravel and red shale and is generally grey in colour.

SPT ‘‘N’’ values ranging from 3 to 60 were reported, indicating the till deposits to be of a soft to hard, but generally hard consistency. Moisture contents are in the order of 7 % to 36 %, which are generally representative of a damp to wet silty clay to clayey silt soil.

4.6 Bedrock

Red shale of the Queenston Formation was encountered in all boreholes underlying the till deposits at depths of between approximately 14.3 m and 15.4 m below existing ground level.

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 93 % to 100 % indicating the bedrock to be of an “excellent” quality at depth.

4.7 Groundwater

During drilling, wet to saturated soil conditions were observed in boreholes BH/MW2 and BH3 at depths of approximately 1.5 m and 2.3 m respectively 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. A deeper groundwater regime was identified at depths greater than 12.1 m below existing ground level.

On completion of drilling, borehole BH2 was converted to a monitoring well. Water level readings recorded in the monitoring well on September 24, 2020 are summarized in Table 4.7.1.

Table 4.7.1: Summary of Water Level Measurements

BH ID	Northing	Easting	Ground Elevation	Water Strike Depth (mbgl*)	Well Screen Depth (mbgl*)	Measured Water Level (m)		
						Depth	Elevation	Date
BH1	4798164	595177	97.4 m	-	NA	-	-	-
BH/MW2	4798112	595252	97.8 m	1.5	17.5 - 23.5	4.56	93.23	Sept. 24, 2020
BH3	4798067	595227	97.5 m	2.3	NA	-	-	-

* mbgl meters below ground level

Groundwater conditions, as screened by monitoring wells BH/MW2, indicate a piezometric water level of approximately 4.56 m depth 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, or elements thereof, 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 seven levels of basement. On this basis, it is anticipated that foundations for a structure with seven levels of basement will be seated at depths between approximately 19.0 m and 21.0 m below existing ground level, equating to Geodetic elevations of between approximately 78.5 m and 76.5 m.

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. Such shallowing of foundations is to be limited to only those areas where a minimum of one basement level is to be included.

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

Founding Depth Range	Founding Stratum	Foundation Design Value	
		SLS [1,2]	ULS [3,4]
19.0± m to 21.0± m	Competent Shale Bedrock	-	14 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 silt and clay 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. This given however, consideration should be given to the use of non-frost susceptible materials as backfill for foundation wall excavations and the installation of foundation drainage in order to minimize the risk of adfreezing.

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

The subsurface areas should be damp proofed and comply with the OBC requirements. 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.

It is recommended that all subsurface structures and areas (i.e. basement walls, floor slabs etc.) are appropriately waterproofed below the seasonally high groundwater level established by the hydrogeological investigation being undertaken by Landtek plus the required buffer zone

(nominally 1.0 m to 1.5 m above the stabilized or highest recorded groundwater level). Recommendations regarding waterproofing and long-term dewatering are provided under separate cover.

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 seven levels of basement, groundwater seepage should be anticipated during periods of wet weather. 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 20256A-02 and 20256A-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 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 products) between the rock face and the back of the structural wall; and/or,
- The over-excavation of the bedrock ±0.6 m and backfilling the void space with 19 mm Clear Stone.

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 Soil Corrosivity

Two selected soil samples were submitted to AGAT for analysis of PH, soil conductivity and redox potential, and concentrations of sulphides, sulphates, and chlorides.

The American Water Works Association (AWWA) document, "*Polyethylene Encasement for Ductile-Iron Pipe Systems*" *ANSI/AWWA C105/A21.5-18,* dated December 1, 2018, uses a 10-point scoring method to determine the soil corrosivity potential. For each given soil sample, points were assigned to the different parameters to evaluate their contribution towards the corrosivity of soil. The test results are provided in Appendix E and are summarized in Table 8.1.1.

Table 8.1.1: Results of Soil Corrosivity Potential

Sample ID	Composite Sample Depth	Parameters Analysed	Measured Value	ANSI/AWWA Point Rating	Total ANSI/AWWA Points
BH1-SS4 and SS5	9.0 – 12.5 (m)	Sulphide (%)	0.12	2	12
		pH (pH units)	8.08	0	
		Resistivity (ohm.cm)	1680	8	
		Redox Potential (mV)	111	0	
		Moisture (%)	11.0	2	
BH2-SS4 and SS5	2.3 – 3.5 (m)	Sulphide (%)	0.19	2	9
		pH (pH units)	8.00	0	
		Resistivity (ohm.cm)	2040	5	
		Redox Potential (mV)	171	0	
		Moisture (%)	26	2	

Corrosion protection for buried metallic improvements or steel is recommended, when a score of 10 points or greater is reported. Based on the ANSI/AWWA value of 12 for the overburden soils obtained from boreholes BH1, it is considered that the soils at the site have corrosion potential. Therefore, ductile-iron pipes used at the site will require corrosion protective measures such as cathodic protection. It should be noted that the analytical results only provide an indication of the potential for corrosion.

The contribution of chloride ions to soil corrosivity towards buried metallic improvements or steel structures is very significant. According to the Corrosion Guidelines (Caltrans, January 2015, version 2.1), a site is considered corrosive if, "*chloride concentration is 500 ppm or greater, sulphate concentration is 2000 ppm or greater, or the pH is 5.5 or less*. "

In addition, the Canadian Standards Association (CSA) A23.1-14 "*Concrete materials and methods of concrete construction*", Table 3, "*Additional requirements for concrete subjected to sulphate attack*", states that design requirements for sulphate resistant concrete are only necessary when the water soluble sulphate content of the soil in which the concrete is to be embedded is greater than 0.1 % (1,000 µg/g).

The representative soil samples at the site are reported to contain sulphate and chloride ion concentrations of up to 286 µg/g and 148 µg/g respectively. This given, the test results indicate a negligible degree of sulphate and chloride attack on buried reinforced concrete structures.

8.2 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"*.

Given the mild sulphate and low chloride test results, normal General use (GU) hydraulic cement can be used for the below grade concrete structures. However, it is recommended that the concrete exposure class for the 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.

8.3 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*" while the generally very stiff to hard "*native clayey silt to silty clay 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 about 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.

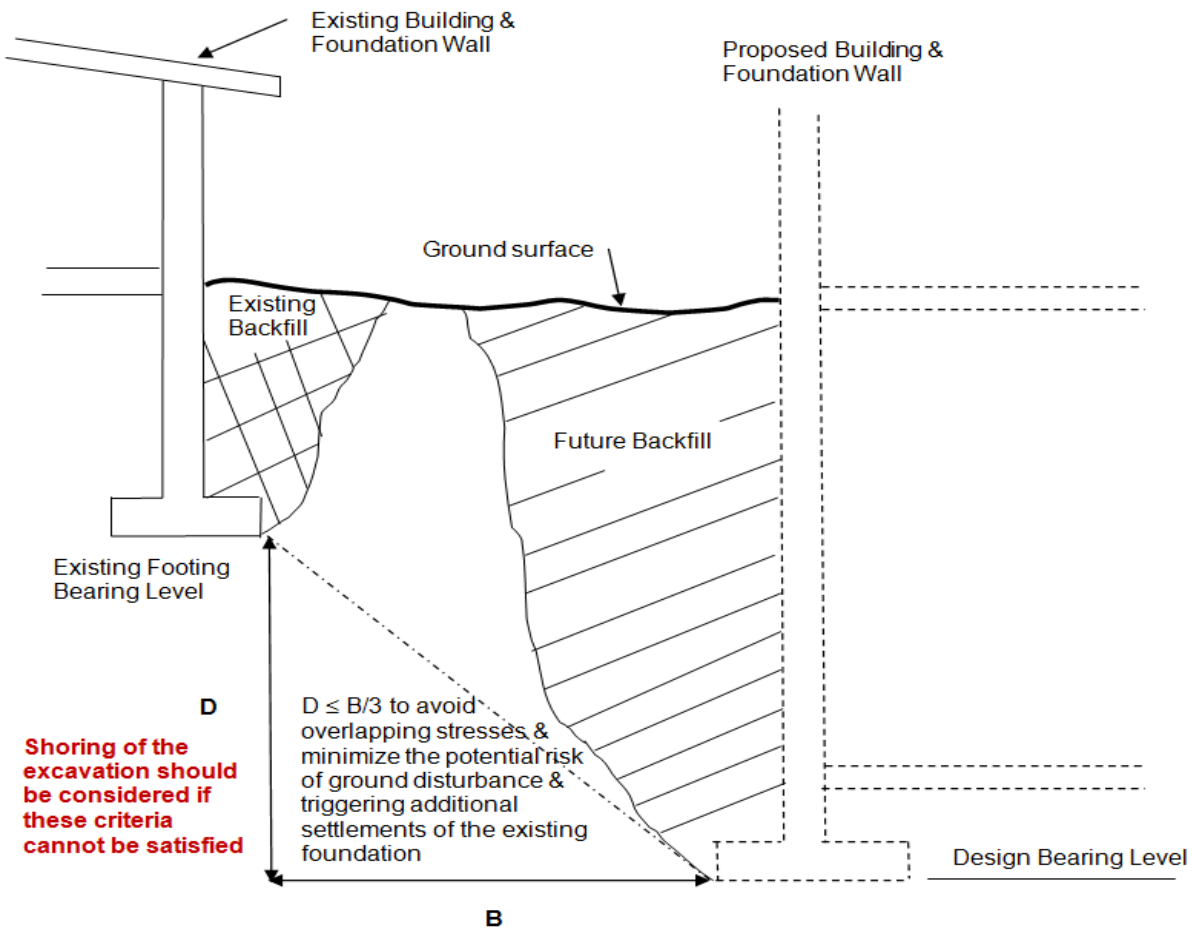


Figure 9.1.1: Criteria for Assessing Excavation Shoring Requirements (Not to Scale)

9.2 Construction Dewatering Considerations

Considerations regarding groundwater control and dewatering requirements during construction have been provided under separate cover.

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 DECK PAVEMENT CONSIDERATIONS

13.1 Deck Pavement Design Considerations

It is understood that the footprint of the proposed development will cover the site area in full. As such, any 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.

13.2 Pavement Materials

Granular Base Course

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.

Hot Mix Asphalt

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

Compaction

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

Isaac Asonya, B.S.Eng.
Author

LICENSED PROFESSIONAL ENGINEER
R. DI CIENZO
Apr. 27/26
PROVINCE OF ONTARIO

James Dann, B.Eng. (Hons)
Manager, Geotechnical Projects

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.

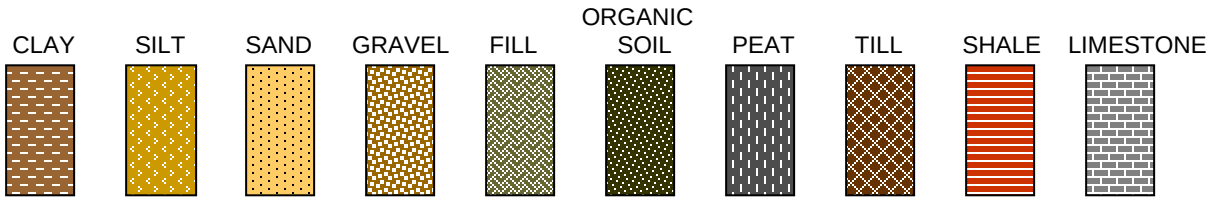
The comments made in this report on potential construction problems and possible remedial methods are intended only for the guidance of the designer. The number of Boreholes may not be sufficient to determine all the factors that may influence construction methods and costs. For example, the thickness 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

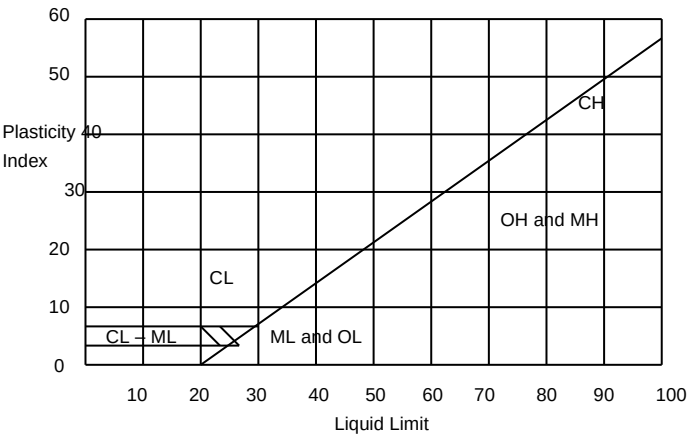
CONSISTENCY OF COHESIVE SOILS

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.

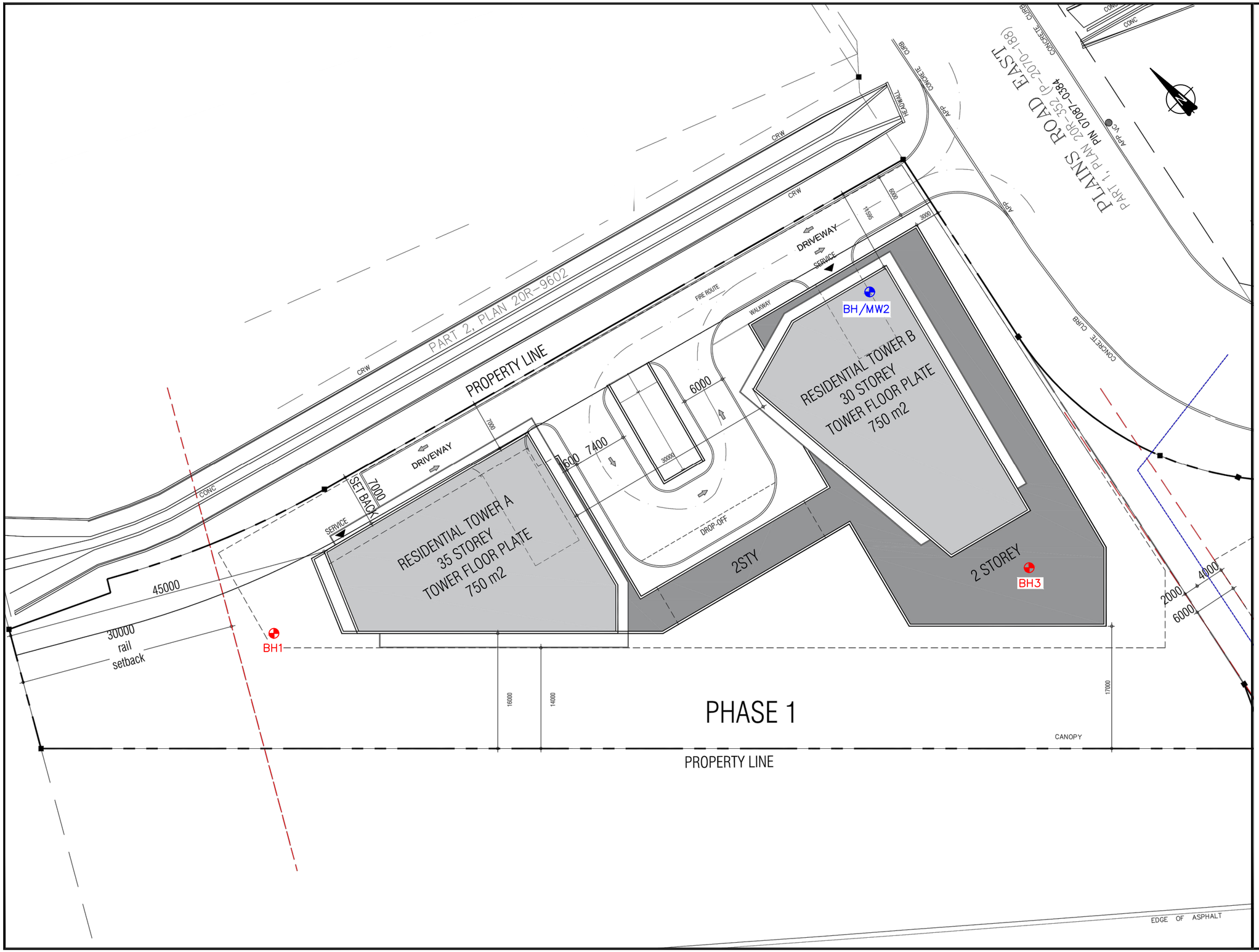
APPENDIX B CONTINUED
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES
ASTM Designation: D 2487 - 69 AND D 2488 – 69
(Unified Soil Classification System)

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 20256A-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 August 27, and September 8, 2020.

Approximate location of borehole completed as a monitoring well and installed by Landtek Limited on August 26, 2020.

Borehole and monitoring well locations are considered approximate.

Base plan provided by Graziani + Corazza Architects Inc.

revisions/ submissions

#	date	description
1	September 16, 2020	Issued for report
2	April 27, 2021	Issued for report

client
1134/1167 Plains Road East

municipality
The Corporation of the City of Burlington

project
Geotechnical Investigation
1167 Plains Road East

sheet
Borehole Location Plan

date: April 27, 2021
drawn: JDC
checked: IA
project #: 20256A
scale: NOT TO SCALE

20256A-01

LANDTEK LIMITED

BH 1

Project No.:	20256A	Drill Date:	September 8, 2020
Project:	Hydrogeological and Geotechnical Investigation	Drill Method:	[ ] solid stem [x] hollow stem [ ] Coring
Location:	1167 Plains Road East, Burlington, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. Depth	Samples No.	Samples Type	SPT "N" Value	Soil Moisture (%)	GWL	Monitor Details	Test Data
Ground Surface		97.4							
±100 mm of topsoil		0.0	1	SS	24	11			
FILL: silty clay, trace gravel, trace red shale, trace iron staining, brown and grey, stiff to very stiff, moist		95.9							
SAND: medium to coarse grained, some silt, brown, compact, moist		1.5 94.4							
CLAYEY SILT TILL: trace gravel, trace red shale fragments, brownish grey, stiff, moist		3.0 91.4	2	SS	10	17			
SILTY CLAY TILL: trace gravel, trace red shale fragments, brownish grey, hard, moist		6.0 87.4	3	SS	45	11			
			4	SS	52	9			
CLAYEY SILT TILL: trace gravel, trace red shale fragments, brownish red, hard, moist		10.0 82.2	5	SS	35	7			
SHALE BEDROCK (Queenston Formation) weak, highly to slightly weathered, red		15.2 81.6	6	SS	50/25mm	7			
BOREHOLE TERMINATED		15.8							

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, -10.5, -11.0, -11.5, -12.0, -12.5, -13.0, -13.5, -14.0, -14.5, -15.0, -15.5, -16.0, -16.5, -17.0

SPT "N" Value: 0, 25, 50, 75, 100

Soil Moisture (%): 0, 25, 50

Notes:
1. On completion, borehole open to 12.2 m
2. Free water not encountered during the drilling process

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

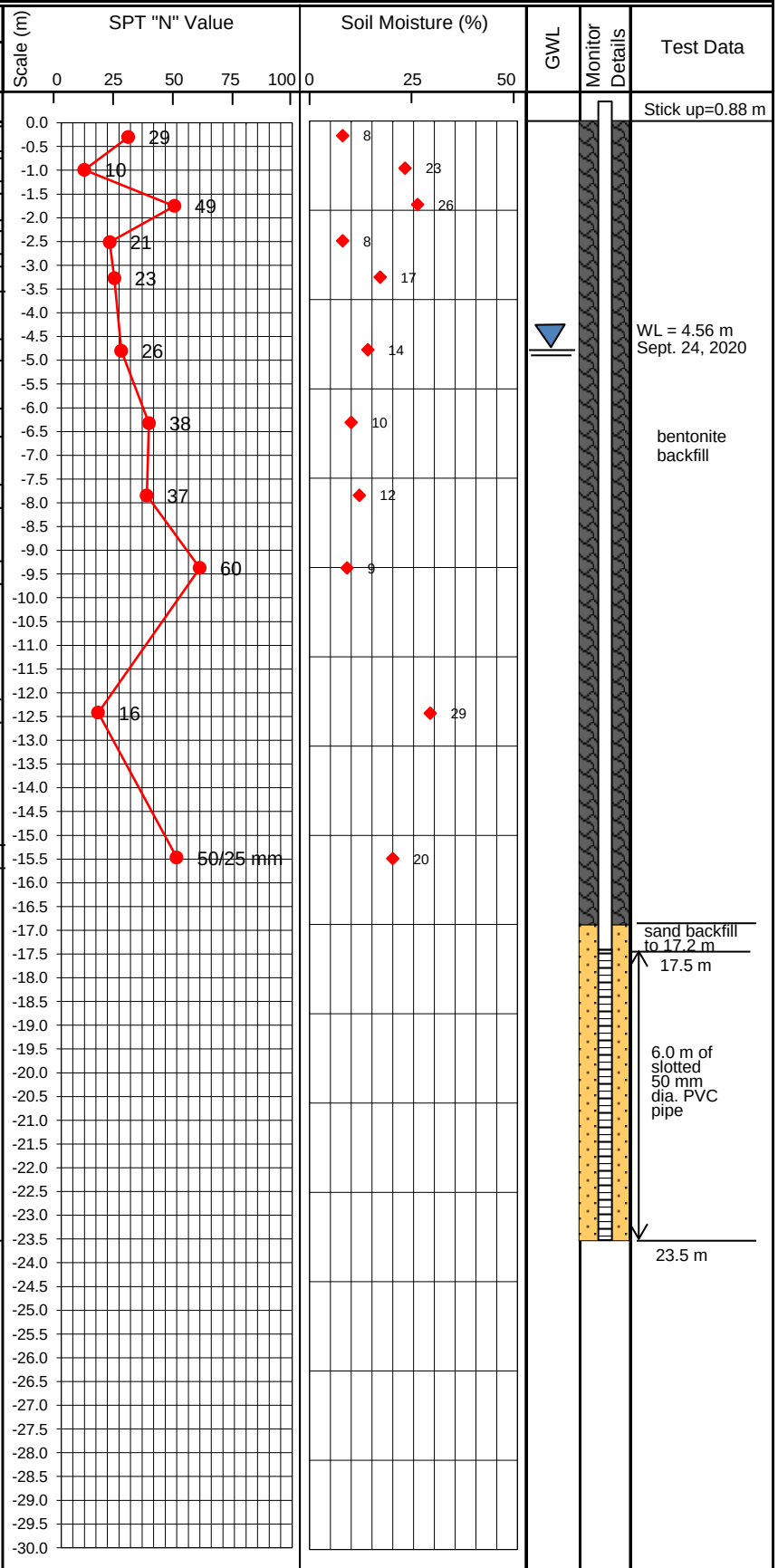
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LANDTEK LIMITED

BH/MW 2

Project No.:	20256A	Drill Date:	August 26, 2020
Project:	Hydrogeological and geotechnical Investigation	Drill Method:	[ ] solid stem [x] hollow stem [x] Coring
Location:	1167 Plains Road East, Burlington, Ontario	Datum:	Geodetic (Approximate)

Material Description	Elev. Depth	Samples No.	Type	Test Data
Ground Surface	97.8			Stick up=0.88 m
±100 mm of topsoil	0.0	1	SS	
FILL: silty clay, black, firm to stiff, moist	96.3	2	SS	
SAND: fine to medium grained, some silt, trace cobble, brown, compact to dense, wet	1.5 / 94.8	3, 4	SS	
SILTY CLAY TILL: trace gravel, trace red shale fragments, brownish grey, very stiff, wet	3.0 / 91.8	5, 6	SS	WL = 4.56 m Sept. 24, 2020
CLAYEY SILT TILL: trace gravel, trace red shale fragments, grey, hard, moist	6.0	7, 8, 9	SS	bentonite backfill
12.1 m to 15.4 m: wet	82.4	10	SS	
SHALE BEDROCK (Queenston Formation) slightly weathered to fresh, red, strong	15.4	11	SS	sand backfill to 17.2 m
Recovery = 100 %; RQD = 93 %	16.5	CORE RUN #1 TO RUN#5	63 mm Dia. Core	17.5 m
fresh, red and grey, strong ∂ = 25.4 kN/m³; UCS strength = 56.8 MPa	17.8			6.0 m of slotted 50 mm dia. PVC pipe
Recovery = 100 %; RQD = 100 %				
fresh, red and grey, strong Recovery = 100 %; RQD = 100 %	21.2			
fresh, red and grey, strong ∂ = 25.4 kN/m³; UCS strength = 71.6 MPa Recovery = 100 %; RQD = 100 %	74.3			
BOREHOLE TERMINATED	23.5			23.5 m



Notes:
1. Wet soils encountered between 1.5 m and 6.0 m & between 12.1 m and 15.4 m
2. Water level reading: WL at 4.56 m depth on September 24, 2020

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

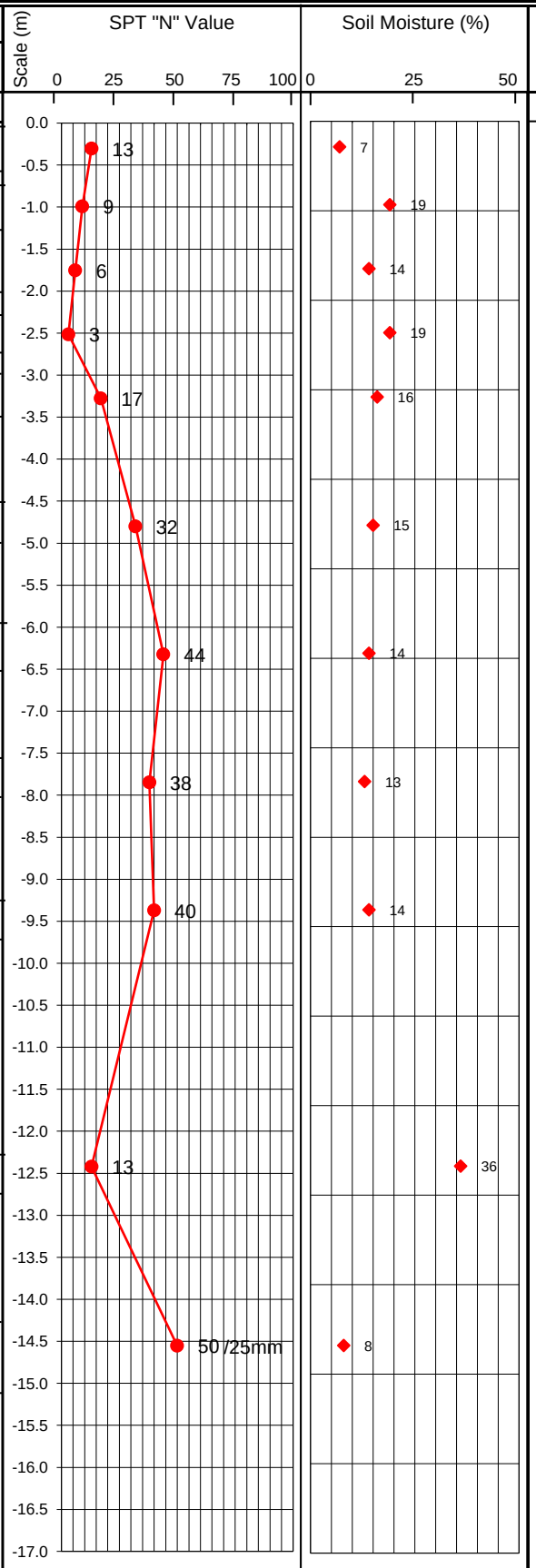
LANDTEK LIMITED
205 Nebo Road, Unit 3
Hamilton, Ontario, Canada, L8W 2E1
Ph: (905) 383-3733 Fax: (905) 383-8433
www.landteklimited.com

LANDTEK LIMITED

BH 3

Project No.:	20256A	Drill Date:	August 27, 2020
Project:	Hydrogeological and Geotechnical Investigation	Drill Method:	[ ] solid stem [x] hollow stem [ ] Coring
Location:	1167 Plains Road East, Burlington, Ontario	Datum:	Geodetic (Approximate)

Material Description	Symbol	Elev. Depth	Samples No.	Samples Type	Test Data
Ground Surface		97.5			
±100 mm of topsoil		0.0	1	SS	
FILL: silty sand, some gravel, trace red brick, trace rootlets, dark brown, loose to compact, moist			2	SS	
		96.0			
FILL: silty sand and clayey silt, trace red brick, trace gravel, dark brown, loose, moist		1.5	3	SS	
		95.2			
SILTY CLAY TILL: trace gravel, trace red shale fragments, greyish brown, very stiff, wet		2.3	4	SS	
			5	SS	
4.5 m to 6.0 m: grey, hard			6	SS	
		91.5			
CLAYEY SILT TILL: trace gravel, trace red shale fragments, grey, hard, wet		6.0	7	SS	
			8	SS	
			9	SS	
			10	SS	
		83.2			
SHALE BEDROCK (Queenston Formation) weak, highly to slightly weathered, red		14.3	11		
		82.5			
BOREHOLE TERMINATED		15.0			



GWL | Monitor Details

Notes:
1. On completion, borehole open to 14.3 m
2. Wet soils encountered below 2.3 m

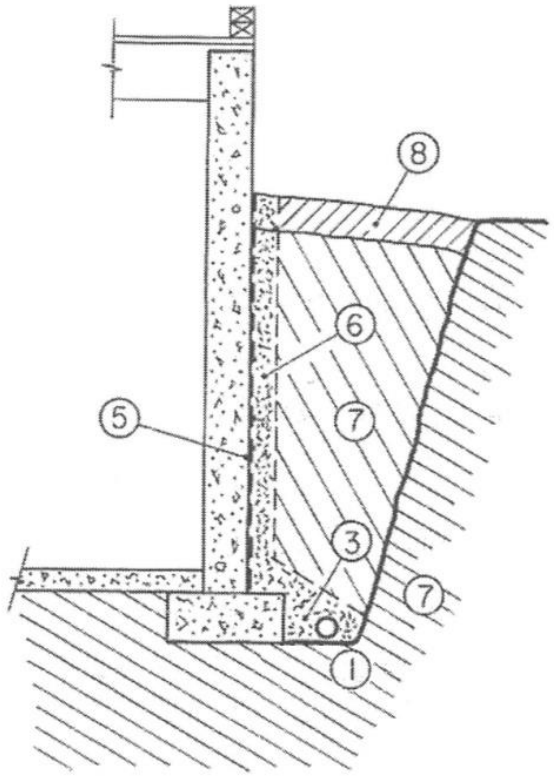
PP = pocket penetrometer TCV = total combustible vapour BRD = bulk relative density
PL = plastic limit LL = liquid limit PI = plasticity index FV = field vane LV = lab vane VS = vane sensitivity

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APPENDIX D

DRAWING 20256A-02 – ENGINEERING COMMENTARIES – GENERAL REQUIREMENTS FOR DRAINAGE TO BASEMENT STRUCTURES

DRAWING 20256A-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.

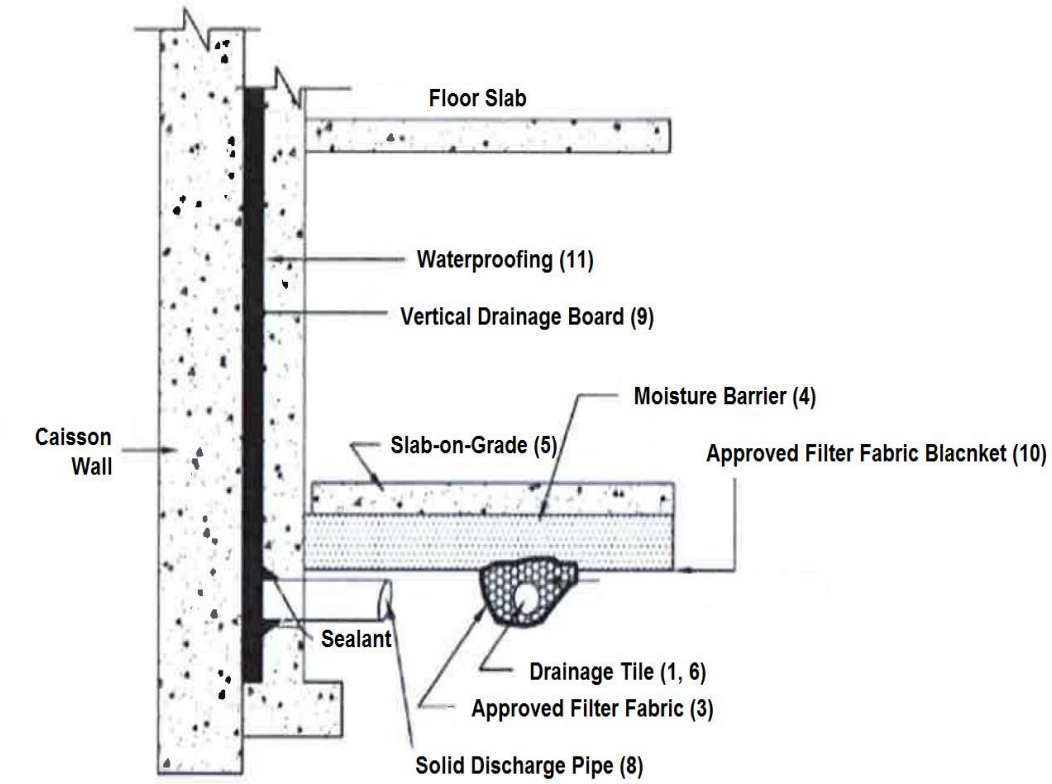
Additional Notes:

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	1134/1167 Plains Road East		
project	1167 Plains Road East, Burlington, Ontario		
project #	20256-A	drawing #	20256A-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	1134/1167 Plains Road East		
project	1167 Plains Road East, Burlington, Ontario		
project #	20256-A	drawing #	20256A-03

APPENDIX E

CHEMICAL LABORATORY TESTING RESULTS

AGAT Laboratories

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Certificate of Analysis

AGAT WORK ORDER: 20T656424

PROJECT: 20256

CLIENT NAME: LANDTEK LTD.

ATTENTION TO: Isaac Asonya

SAMPLING SITE:

SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2020-09-28

DATE REPORTED: 2020-10-06

Parameter	Unit	G / S: A	G / S: B	G / S: C	RDL	BH1-SS4 and SS5 Soil 2020-09-25 17:00 1495059	BH2-SS4 and SS5 Soil 2020-09-25 17:00 1495131	BH4-SS7 and SS8 Soil 2020-09-25 17:00 1495132	BH6-SS8 and SS9 Soil 2020-09-25 17:00 1495133
Chloride (2:1)	µg/g	NA	NA	NA	2	136	66	37	148
Sulphate (2:1)	µg/g				2	259	286	259	244
pH (2:1)	pH Units				NA	8.08	8.00	8.10	8.21
Electrical Conductivity (2:1)	mS/cm	0.57	0.7	0.7	0.005	0.595[A-B]	0.491[<A]	0.423[<A]	0.609[A-B]
Resistivity (2:1) (Calculated)	ohm.cm				1	1680	2040	2360	1640
Redox Potential 1	mV				NA	111	173	131	121
Redox Potential 2	mV				NA	107	171	121	112
Redox Potential 3	mV				NA	117	167	129	115

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard: A Refers to Table 1: Full Depth Background Site Condition Standards - Soil - Residential/Parkland/Institutional/Industrial/Commercial/Community Property Use, B Refers to Table 2: Full Depth Generic Site Condition Standards in a Potable Ground Water Condition - Soil - Residential/Parkland/Institutional Property Use - Medium and Fine Textured Soils, C Refers to Table 3: Full Depth Generic Site Condition Standards in a Non-Potable Ground Water Condition - Soil - Residential/Parkland/Institutional Property Use - Medium and Fine Textured Soils
Guideline values are for general reference only. The guidelines provided may or may not be relevant for the intended use. Refer directly to the applicable standard for regulatory interpretation.

1495059-1495133 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter.
Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results.
Redox potential measurement in soil is quite variable and non reproducible due in part, to the general heterogeneity of a given soil. It is also related to the introduction of increased oxygen into the sample after extraction. The interpretation of soil redox potential should be considered in terms of its general range rather than as an absolute measurement.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By:



Results relate only to the items tested. Results apply to samples as received.

AGAT Laboratories

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
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FAX (905)712-5122
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Exceedance Summary

AGAT WORK ORDER: 20T656424

PROJECT: 20256

CLIENT NAME: LANDTEK LTD.

ATTENTION TO: Isaac Asonya

SAMPLEID	SAMPLE TITLE	GUIDELINE	ANALYSIS PACKAGE	PARAMETER	UNIT	GUIDEVALUE	RESULT
1495059	BH1-SS4 and SS5	ON T1 S RPI/ICC	Corrosivity Package	Electrical Conductivity (2:1)	mS/cm	0.57	0.595
1495133	BH6-SS8 and SS9	ON T1 S RPI/ICC	Corrosivity Package	Electrical Conductivity (2:1)	mS/cm	0.57	0.609

Results relate only to the items tested. Results apply to samples as received.

AGAT Laboratories

5623 McADAM ROAD
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Certificate of Analysis

AGAT WORK ORDER: 20T658800

PROJECT: 20T656424

CLIENT NAME: LANDTEK LTD.

ATTENTION TO: Isaac Asonya

(201-042) Sulfide

DATE SAMPLED: Oct 01, 2020 | DATE RECEIVED: Oct 02, 2020 | DATE REPORTED: Oct 07, 2020 | SAMPLE TYPE: Other

Analyte:	Sulfide
Unit:	%
Sample ID (AGAT ID) RDL:	0.05
BH1-SS4 and SS5-1495059 (1509739)	0.12
BH1-SS4 and SS5-1495059-DUP (1509740)	0.13
BH2-SS4 and SS5-14995131 (1509741)	0.19
BH4-SS7 and SS8-14995132 (1509742)	0.23
BH6-SS8 and SS9-14995133 (1509743)	0.16

Comments: RDL - Reported Detection Limit

Analysis performed at AGAT 5623 McAdam Rd., Mississauga, ON (unless marked by *)

Certified By: Sherin Moussa

Results relate only to the items tested. Results apply to samples as received.

APPENDIX F

ROCK CORE TESTING RESULTS

GEOMECHANICA

Rock Laboratory Testing Results

A report submitted to:

Isaac Asonya
Landtek Limited
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Prepared by:

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October 6, 2020
Project number: 20256A, B

Abstract

This document summarizes the results of rock laboratory testing, including 2 Uniaxial Compressive Strength (UCS) tests and 2 Point Load Tests (PLT). The results of each test type are presented in separate sub-sections herein.

In this document:

1 Uniaxial Compressive Strength Tests 1
2 Point Load Testing 3
Appendices 6

Disclaimer: This report was prepared by Geomechanica Inc. for Landtek Limited. The material herein reflects Geomechanica Inc.'s best judgment given the information available at the time of preparation. Any use which a third party makes of this report, any reliance on or decision to be made based on it, are the responsibility of such third parties. Geomechanica Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

1 Uniaxial Compressive Strength Tests

1.1 Overview

This section summarizes the results of uniaxial compressive strength (UCS) testing. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.10 mm/min (Figure 1). The preparation and testing procedure of each sample included the following:

1. Unwrapping of the core sample, inspecting it for damage, and re-wrapping it in electrical tape to minimize exposure to moisture during subsequent specimen preparation.
2. Diamond cutting of the core sample to obtain cylindrical specimens with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Diamond grinding of the specimen to obtain flat (within ± 0.025 mm) and parallel end faces (within 0.25°).
4. Placing of the specimen into the loading frame, applying a 1 kN axial load, and removing the electrical tape.
5. Axially loading the specimens to rupture while continuously recording axial force and axial deformation to determine the peak strength (UCS).



Figure 1: Forney loading frame setup for uniaxial compression testing.

Using a precision V-block mounted on the magnetic chuck of the surface grinder, test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-19. The side straightness criteria, as checked with a feeler gauge, was met for all samples and the minimum length:diameter criteria was met for all specimens unless noted otherwise in Table 1. Testing of the specimens followed ASTM D7012-14 Method C.

1.2 Results

The results of UCS testing are summarized in Table 1. Additional specimen measurements and test details are provided in the summary spreadsheet that accompanies this report.

Table 1: Summary of Uniaxial Compression test results.

Sample	Depth (m)	Bulk density ρ (g/cm^3)	UCS (MPa)	Lithology	Failure description
20256A-BH2	22.83 - 23.00	0.763	71.6	Queenston Shale	1
20256B-BH6, Run 6	20.07 - 20.39	2.585	38.5	Queenston Shale - greenish-grey reduction zone	2, 3
	Average	1.674	55.1		
	Standard deviation	0.911	16.6		

[1] Partial hourglass failure

[2] Axial splitting failure

[3] Localized crushing near platen

1.3 Specimen photographs

Photographs of the specimen before and after testing are presented in the Appendix of this report.

2 Point Load Testing

2.1 Overview

This section summarizes the results of Point Load Testing (PLT). The tests were performed using a Carver 12-ton hydraulic press with point load test platens and equipped with a 0-5000 psi digital pressure gauge with a peak pressure holding capability (Figure 2). Testing was completed on rock core samples. Axial tests were performed according to ASTM D5731-16.



Figure 2: Point load tester equipped with digital pressure gauge.

2.2 Results

The results of the PLT tests are summarized in Table 2. Note that the load, *P*, in kN was calculated from the measured peak pressure, as:

$$P = p \times A_{ram} \tag{1}$$

where, p is the peak pressure in kPa and A_{ram} is the effective cross-sectional area of the hydraulic ram in square metres. The effective diameter of the ram of the employed tester was 52 mm.

The uncorrected point load strength (I_s) is calculated as:

$$I_s = \frac{P}{{D_e}^2} \tag{2}$$

where, D_e is the equivalent core diameter in mm calculated as:

$${D_e}^2 = D^2 \quad \text{for diameteral tests} \tag{3}$$

$$= \frac{4A}{\pi} \quad \text{for axial tests} \tag{4}$$

where D is the distance between platens in mm and A is the minimum cross sectional area of a plane through the platen contact points. The value of A is given by:

$$A = W \times D \tag{5}$$

where W is the width of the specimen.

The size correction factor (F) is obtained from the expression:

$$F = \left(\frac{D_e}{50}\right)^{0.45} \tag{6}$$

and the size-corrected point load strength ($I_{s(50)}$) for a core with D = 50 mm was calculated as:

$$I_{s50} = F \times I_s. \tag{7}$$

Table 2: Summary of PLT results.

Sample	Depth (m)	Test type A-axial D-diametric	Distance Between Platens, D (mm)	Failure Load P (kN)	Effective Diameter De (mm)	Uncorrected Point Strength, Strength, I_s (MPa)	Size Correction Factor, F	Size-Corrected Point Load Strength, $I_{s(50)}$ (MPa)
20256A-BH2, Run 2	17.04 - 17.20	A [1]	38.50	9.14	55.48	2.97	1.05	3.11
		A [1]	30.00	5.70	48.98	2.37	0.99	2.35
		A [1]	34.00	5.05	52.14	1.86	1.02	1.89
		A [1]	24.50	4.76	44.26	2.43	0.95	2.30
		A [1]	25.00	4.66	44.71	2.33	0.95	2.22
					Axial Mean	2.39		2.37
					Estimated UCS ($24I_{s(50)}$)			56.88 MPa
20256B-BH6, Run 4	15.90 - 16.13	A [1]	50.00	7.07	63.32	1.76	1.11	1.96
		A [1]	39.00	5.86	55.92	1.87	1.05	1.97
		A [1]	32.00	6.46	50.66	2.52	1.01	2.53
		A [2]	30.00	15.02	49.05	6.24	0.99	6.19

Continued on next page

Table 2 – Summary of PLT results. (continued from previous page)

Sample	Depth (m)	Test type A-axial D-diametric	Distance Between Platens, D (mm)	Failure Load P (kN)	Effective Diameter De (mm)	Uncorrected Point Strength, Strength, I_s (MPa)	Size Correction Factor, F	Size-Corrected Point Load Strength, $I_{s(50)}$ (MPa)
		A [1]	30.50	10.67	49.45	4.36	1.00	4.34
		A [1]	22.00	4.16	42.00	2.36	0.92	2.18
				Axial Mean		3.19		3.20
				Estimated UCS ($24I_{s(50)}$)				76.80 MPa

[1] Queenston Formation - red shale

[2] Queenston Formation - red shale with limestone inter-beds

Appendices

Specimen sheets

- 20256A-BH2
- 20256B-BH6, Run 6

GEOMECHANICA

Uniaxial Compression Test

Client	Landtek Limited	**Project**	20256A, B
Sample	20256A-BH2	**Depth**	22.83 - 23.00

Specimen parameters	
Diameter (mm) [a]	63.15
Length (mm) [a]	131.45
Bulk density ρ (g/cm^3)	0.763
UCS (MPa)	71.6
Lithology	Queenston Shale
Failure description [b]	1

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Failure description: [1] Partial hourglass failure;

Prior to testing



After testing



Remarks:

Performed by	BSAT	**Date**	2020-10-05

GEOMECHANICA

Uniaxial Compression Test

Client	Landtek Limited	**Project**	20256A, B
Sample	20256B-BH6, Run 6	**Depth**	20.07 - 20.39

Specimen parameters	
Diameter (mm) [a]	63.16
Length (mm) [a]	131.20
Bulk density ρ (g/cm^3)	2.585
UCS (MPa)	38.5
Lithology	Queenston Shale - greenish-grey reduction zone
Failure description [b]	2, 3

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Failure description: [2] Axial splitting failure; [3] Localized crushing near platen;

Prior to testing



After testing



Remarks:

Performed by	BSAT	**Date**	2020-10-05