Revised Report on
Preliminary Geotechnical Investigation
Proposed Residential Development
1995 Dundas St. East, 3040 and 3044 Universal Drive
Mississauga, Ontario

Prepared For:

Landeal Asset Management Inc.

Project No: 24-070-100-R
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Table of Contents

1. INTRODUCTION

DS Consultants Ltd (DS) was retained by Landeal Asset Management Inc. (the Client) to undertake a preliminary geotechnical investigation for the proposed residential development located at 1995 Dundas Street East, and 3040 and 3044 Universal Drive, Mississauga , Ontario (the Site).

The site is a 0.3-ha parcel of land located on the northwest corner of the intersection of Dundas Street East and Universal and is currently developed with a 1-storey brick commercial building.

It is understood that the existing structure will be demolished to make way for the proposed development which will consist of the construction of two (2) 24 and 25-storey high-rise towers with one (1) level of underground parking (P1). The proposed lowest finished floor elevation (FFE for P1) will be set at about Elev. 115.5 m.

Concurrent with the geotechnical investigation program, a hydrogeological study has been carried out by DS, the results of which will be addressed separately.

The purpose of this geotechnical investigation was to obtain the subsurface conditions at six (6) borehole locations and from the findings at the boreholes provide geotechnical recommendations for the following:

1. Foundations
2. Floor slabs and permanent drainage
3. Excavations and groundwater control
4. Temporary shoring
5. Earth pressures
6. Earthquake considerations

This report is provided on the basis of the terms of reference presented above and, on the assumption, that the design will be in accordance with applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations can cater to the changed design.

The Site investigation and recommendations follow generally accepted practice for geotechnical consultants in Ontario. The format and contents are guided by client specific needs and economics and do not conform to generalized standards for services. Laboratory testing for most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice.

This report has been prepared for Landeal Asset Management Inc. and its architect and designers. Use of this report by third party without DS Consultants Ltd. consent is prohibited.

2. FIELD AND LABORATORY WORK

The field work for this investigation was carried out by DS between April 01 and 04, 2024. A total of six (6) boreholes (BH24-1 to BH24-6, see **Drawing 1** for borehole locations) were drilled/cored to depths ranging from 5.6 to 15.3 m below existing grade. Boreholes were drilled to bedrock surface with solid stem continuous flight auger or mud rotary equipment by a drilling sub-contractor under the direction and supervision of DS personnel. Samples were retrieved at regular intervals with a 50 mm O.D. split-barrel sampler driven with a hammer weighing 624 N and dropping 760 mm in accordance with the Standard Penetration Test (SPT) method. Upon encountering the bedrock surface, shale bedrock was cored from 12.3 to 15.3 m depth in BH24-1 and from 6.4 to 9.3 m depth in BH24-6. The bedrock was cored with HQ-2 double tube wireline equipment providing 63 mm dia. rock core samples. The coring was carried out under the full-time supervision of a representative from DS who identified and described the rock samples, noting and recording the percentages of total and solid rock core recovery, RQD values, fracture index and the percentage and thicknesses of hard layers.

The samples were logged in the field and returned to the DS laboratory for detailed examination by the project engineer and for laboratory testing. In addition to visual examination in the laboratory, all the soil samples were tested for moisture contents and results are presented on the respective borehole logs. Five (5) selected soil samples (BH24-1/SS7, BH24-2/SS8, BH24-3/SS7, BH24-5/SS4 and BH24-6/SS4) were tested for grain size analyses and the gradation curves for the grain size analyses are presented on **Drawing 9**.

Water level observations were made during drilling and in the open boreholes at the completion of the drilling operations. Monitoring wells were installed in Boreholes BH24-1, BH24-3, BH24-5, and BH24-6 to allow for groundwater level monitoring and hydrogeological testing.

The geodetic ground surface elevations at the locations of the boreholes/monitoring wells were established by DS using differential GPS system. It should be noted that the elevations at the as-drilled borehole/well locations were not provided by a professional surveyor and should be considered to be approximate. Contractors performing any work referenced to the borehole elevations should confirm the borehole elevations for their work.

3. SITE AND SUBSURFACE CONDITIONS

The borehole location plan is shown on **Drawing 1**. General notes on sample description are provided on **Drawing 1A**. The subsurface conditions in the boreholes are presented in the individual borehole logs presented on **Drawings 2 to 7**. A generalized sub-surface profile is provided on **Drawing 8**.

3.1 Soil and Bedrock Conditions

Pavement Structure:

All boreholes were drilled from the existing pavement and encountered a pavement structure consisting of approximately 150 to 200 mm of asphaltic concrete overlying 70 to 950 mm of sand and gravel granular fill materials.

It should be noted that the pavement structure was measured from the collar of boreholes and should not be relied on to calculate the pavement structure thicknesses across the site. Asphalt cores and/or Shallow hand-dug test-pits in the close distance should be carried out to further explore the pavement structure makeup/thicknesses, if required.

Fill Materials:

Fill materials consisting of clayey silt to silty clay, sandy silt to silty sand and/or gravelly sand, with varying inclusions of gravel and organics were encountered in all boreholes, except BH24-4. The fill material extended to depths ranging from 0.6 to 3.3 m below existing ground surface. Brick pieces were observed in the fill in BH24-1. A buried asphalt layer was noted below 0.5 m depth and concrete pieces below 0.8 m in BH24-2. The fill had a loose to very dense relative density/firm to hard consistency, as indicated by measured SPT 'N' values ranging from 6 to 56 blows per 300 mm penetration.

Sand and Gravel to Gravelly Sand Deposits:

Cohesionless deposits of sand and gravel to gravelly sand were encountered below the pavement structure in BH24-4 and below the fill materials in the remaining boreholes and extended to depths ranging from 3.0 to 7.5 m below existing ground surface. The deposit was present in a loose to very dense state, with measured SPT 'N' values ranging from 6 to over 50 blows per 300 mm of penetration. The moisture content of the sand and gravel to gravelly sand deposit ranged from 4 to 14 %.

Grain size analyses of three (3) sand and gravel to gravelly sand soil samples (BH24-1/SS7, BH24-5/SS4 and BH24-6/SS4) were conducted, and the results are presented on **Drawing 9**, with the following fractions:

Clay: 4 to 5%
Silt: 11 to 16%
Sand: 38 to 52%
Gravel: 29 to 41%

Sandy Silt to Silty Sand Till Deposits:

Glacial cohesionless sandy silt to silty sand till deposits were encountered below the sand and gravel to gravelly sand deposits in all boreholes and extended to depths ranging from 4.8 to 10.6 m below existing ground surface in all boreholes, i.e., maximum depth explored/auger refusal in BH24-3. A gravelly sand layer was embedded in the till between 7.5 and 9.0 m depths in BH24-4. The deposits contained trace clay and trace to some gravel. Cobbles and boulders were noted in the till during drilling.

SPT 'N' values measured within the cohesionless sandy silt to silty sand till ranged from 26 to over 50 blows per 300 mm of penetration, indicating compact to very dense relative density. The moisture content of the till ranged from 5 to 15 %.

Grain size analyses of two (2) sandy silt to silty sand till soil samples (BH24-2/SS8 and BH24-3/SS7) were conducted, and the results are presented on **Drawings 9**, with the following fractions:

Clay:	7 to 10%
Silt:	18 to 48%
Sand:	39 to 65%
Gravel:	3 to 10%

Sandy Silt Till/Shale Complex:

Below the sandy silt till deposit, sandy silt till/shale complex with a thickness of approximately 400 mm was found overlying shale bedrock in BH24-1. This deposit was found to have generally a very dense relative density, with measured SPT 'N' value of over 50 blows per 300 mm of penetration. This deposit consisted of sandy silt till mixed with highly weathered shale.

Shale Bedrock:

Shale bedrock was found at approximate depths ranging from 4.8 to 11.0 m, corresponding to elevations varying from 108.4 to 114.6 m, as presented in **Table 1** below.

Table 1: Depth and Elevation of Top of Bedrock

Borehole No.	Borehole Elevation	Depth of Shale Bedrock Surface below Existing Ground (m)	Approximate Elevation of Shale Bedrock Surface (m)	Notes
BH24-1	121.3	11.0	110.3	Bedrock was cored from 12.3 to 15.3 m
BH24-2	120.1	9.4	110.7	Bedrock was augered
BH24-3	119.9	-	-	Auger Refusal at 7.7 m depth (Elev. 112.2 m). Possible boulder or bedrock
BH24-4	119.0	10.6	108.4	Bedrock was augered
BH24-5	119.4	4.8	114.6	Bedrock was augered
BH24-6	120.1	6.0	114.1	Bedrock was cored from 6.4 to 9.3 m

Because of the method of drilling and sampling, the surface elevations of the bedrock can be different than indicated on the borehole logs. With augering, the auger may penetrate some of the more weathered shale and the coring may therefore begin below the bedrock surface. Commonly the overburden overlying the shale contains slabs of limestone which would give a false indication of the bedrock level. Similarly, the depth of weathering cannot be determined accurately due to the presence of limestone layers.

Shale bedrock was cored at two (2) borehole locations (BH24-1 and BH24-6). General comments on shale bedrock in Greater Toronto area are presented in **Appendix A**. Photographs of recovered bedrock cores are also presented in **Appendix A.**

<u>Total Core Recovery (TCR):</u>

The total core recovery indicates the total length of rock core recovered, expressed as a percentage of the actual length of the core run. The total core recovery in the coreholes was 84 to 100 %.

<u>Solid Core Recovery (SCR):</u>

The solid core recovery is the total length of solid, full diameter rock core that was recovered, expressed as a percentage of the length of the core run. Solid core recovery ranged from 24 to 75 % and appears to generally improve with depth. The SCR index was generally influenced by the orientations of the fractures. SCR was low when fractures oblique to the borehole axis were intercepted.

<u>Rock Quality Designation (RQD):</u>

The rock quality designation index is obtained by measuring the total length of recovered rock core pieces which are longer than 100 mm and expressing their sum total length as a percentage of the length of the core run. RQD is a function of the frequency of joints, bedding plane partings and fractures in the rock cores. While the use of double tube core barrels provided reasonably good protection of the core during drilling and core retrieval, the fissile nature of the shale greatly influences the RQD values of the rock cores. Consequently, it is believed that the RQD values recorded underestimate the rock quality classification of the laminated fissile shale. The recorded RQD values in the cores ranged from 0 to 21 percent.

<u>Hard Layers:</u>

Based on the visual examination of the rock cores, an attempt was made to identify and record the thickness and percentages of the relatively harder siltstone and limestone layers. The percentage of the "hard layers" per core run ranges between 10 and 17 %. The thickness of these layers varied but was generally less than 230 mm, however, thicker layers to be as much as 750 to 900 mm have been observed at other sites in GTA. The layers are actually lenses and they can vary significantly in thickness over short distance. Encountering such thick layers should be anticipated. It is also common to encounter closely spaced groupings of thin strong limestone/siltstone layers which individually may only be 25 to 50 mm thick but collectively can be 1 m in thickness.

<u>Methane Gas:</u>

Methane gas is expected in the bedrock as indicated in **Appendix A**. Appropriate care and monitoring is essential in all confined bedrock excavations.

3.2 Groundwater Conditions

Monitoring wells were installed in Boreholes BH24-1, BH24-3, BH24-5, and BH24-6 for the long-term groundwater table monitoring and hydrogeological testing. Stabilized groundwater was found in monitoring wells at depths ranging from 2.4 to 5.7 m, corresponding to Elev. 115.6 to 117.0 m, as listed on **Table 2**:

Table 2: Groundwater Levels Observed in Monitoring Wells

Monitoring Well No.	Ground Surface Elevation (m)	Date of Observation	Groundwater Depth (m)	Elevation of Groundwater (m)	Note
BH24-1	121.3	April 22, 2024	5.7	115.6	Well screened in Sand and Gravel to Gravelly Sand and Sandy Silt Till
BH24-3	119.9	April 22, 2024	4.0	115.9	Well screened in Sandy Silty Till
BH24-5	119.4	April 22, 2024	2.4	117.0	Well screened in Sand and Gravel to Gravelly Sand and Sandy Silt Till
BH24-6	120.1	April 22, 2024	3.7	116.4	Well screened in Sand and Gravel to Gravelly Sand, Sandy Silt to Silty Sand Till and Shale Bedrock

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to major weather events.

Therefore, reference is made to the hydrogeology study report prepared by DS Consultants for further details on the extent and the conditions of the groundwater, as well as the recommended groundwater control. Further groundwater level monitoring is recommended.

4. FOUNDATIONS

It is understood that the proposed development which will consist of the construction of two (2) 24 and 25-storey high-rise towers with one (1) level of underground parking (P1). The proposed lowest finished floor elevation (FFE for P1) will be set at about Elev. 115.5 m.

The measured groundwater levels on April 22, 2024, in monitoring wells were at depths ranging from 2.4 to 5.7 m, corresponding to Elev. 115.6 to 117.0 m. As such, it is expected that the P1 basement floor will be below the groundwater level.

Based on the borehole information, shallow foundations (raft foundations and footings) are recommended to support the proposed structures (P1). Depending on the elevation of the P1 FFE relative to the shale bedrock and required bearing capacity for conventional footings/raft foundations, drilled caissons founded in sound bedrock can also be adopted to support the buildings with 1 level of basement, if required.

4.1 Bearing Capacity of Soil and Rock for Footings and Raft Foundations

Based on the borehole information, bearing capacity values of 500 to 700 kPa at SLS (750 to 1000 kPa at ULS) are available for footings and raft foundations founded on the undisturbed very dense native soils (P2). Additionally, bearing capacity values of 2500 kPa at SLS (3750 kPa at ULS) are available for footings and raft foundations founded on weathered shale bedrock at a minimum of 0.3 m below the bedrock surface. For preliminary guidance, the bearing capacity values and the corresponding founding elevations at the borehole locations are summarized on **Table 3**.

Table 3: Bearing Values and Founding Levels of Footings/Rafts on Undisturbed Native Soils or Bedrock

Borehole No.	**Ground Surface Elevation (m)**	**Bearing Capacity at SLS (kPa)**	**Bearing Capacity at ULS (kPa)**	**Minimum Depth below Existing Basement floor** (m)	**Founding Level at or Below Elevation (m)**
BH24-1	121.3	500 700	750 1000	4.0 7.8	117.3 113.5
BH24-2	120.1	500 700	750 1000	3.0 5.5	117.1 114.6
BH24-3	119.9	500 700	750 1000	2.5 6.0	117.4 113.9
BH24-4	119.0	500 700	750 1000	4.5 5.0	114.5 114.0
BH24-5	119.4	500 700 2500	750 1000 3750	2.0 4.1 5.1	117.4 115.3 114.3
BH24-6	120.1	500 700 2500	750 1000 3750	3.5 4.8 6.3	116.6 115.3 113.8

A subgrade reaction modulus of K_t = 20 MPa/m can be used for the design of the raft foundations on the very dense native soils with bearing capacity value of 500 kPa at SLS. A subgrade reaction modulus of K_t = 30 MPa/m can be used for the design of the raft foundations on the very dense native soils with bearing capacity value of 700 kPa at SLS. A subgrade reaction modulus of K_t=200 MPa/m can be used for the design of raft foundations on weathered shale bedrock with bearing capacity value of 2500 kPa at SLS.

4.2 Drilled Caissons

For the proposed buildings with 1 level of basement, depending on the building loads drilled caissons founded in shale bedrock can also be considered to support the proposed structures.

For compression capacity of the caissons in sound bedrock, the bearing capacity can consist of end bearing capacity and skin friction bearing capacity.

The end bearing capacity for the caissons in sound bedrock at minimum 3.0 m below the bedrock surface can be designed for 5.0 MPa at SLS and 7.5 MPa at ULS.

The skin friction bearing capacity can be calculated using skin friction values of 0.5 MPa at SLS and 0.7 MPa at ULS between caisson shaft and sound bedrock. Sound bedrock is considered to be 3.0 m below bedrock surface (Refer to **Table 1** for estimated bedrock surface elevations). The skin friction in the top 3.0 m weathered bedrock and in the soils must be ignored. The skin friction capacity will increase with the caisson socket depth in bedrock. The total bearing capacity (skin friction + end bearing) of the caissons should not exceed 12 MPa at SLS and 17 MPa at ULS.

Due to the presence of hard limestone/siltstone layers in the bedrock, bedrock coring will be required for the installation of the caissons.

For closely spaced caissons, group effect should be considered on the skin friction bearing capacity, using a reduction factor (Beta), Beta = 0.5 + 0.5*X/(2.5B). In the equation, X represents the centre-to-centre distance between adjacent caissons, and B is the diameter of the caissons. If the centre-to-centre distance between the adjacent caissons is equal to or greater than 2.5 times its diameter (2.5B), the group effect on skin friction bearing capacity can be ignored. Group effect on end bearing capacity of caissons can be ignored.

The presence of groundwater table in the overburden soils overlying the shale bedrock will make the construction of the caissons difficult. An oversize liner will be required and must be sealed in the underlying bedrock. Sealing of the liner will be difficult where limestone layers are present at the surface above the shale and coring of the limestone layer will be required to advance the casing. All caisson holes and bases must be inspected by this office on full time basis to ensure that the caisson bases are founded on sound bedrock and free from mud and loose/disturbed materials. The side of the caisson holes in bedrock must be clean and free from mud and other unsuitable materials to ensure that the design skin friction between the bedrock and the caisson concrete can be achieved. The caisson holes and bases can be inspected by down-hole camera. Tremie method will be required if the concrete is poured below water.

4.3 General Notes on Foundations

Additional boreholes with rock coring, will be required to confirm the subsurface soil conditions, depth of sound shale bedrock, recommended bearing resistance/founding elevations for footings and raft foundations. Our report should be updated, and our recommendations should be revised accordingly.

Foundations designed to the specified bearing capacity at the serviceability limit states (SLS) are expected to settle less than 25 mm total and 19 mm differential.

Positive dewatering will be required for the installation of footings/raft foundations below the groundwater table. The groundwater table must be lowered to at least 1.0 m below the excavation base.

Prior to placing concrete, all footing/raft foundation and drilled caisson bases must be inspected by this office to confirm the founding soil conditions and design bearing capacity.

The excavated footing bases in soil must be covered with 50 mm thick mud slab immediately after inspection and cleaning, in order to avoid disturbance of the founding soil due to weathering and construction activity.

Where it is necessary to place footings on soil at different levels, the upper footing must be founded below an imaginary 10 horizontal to 7 vertical line drawn up from the base of the lower footing. The lower footing must be installed first to help minimize the risk of undermining the upper footing.

The shale bedrock weathers rapidly between wetting and drying cycles. In view of this, it is suggested that a lean concrete mat slab be placed immediately after the excavation is complete to keep the shale intact, unless the footings are cast immediately after excavating.

It should be noted that the recommended bearing capacities have been calculated by DS from the available borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of the underground conditions becomes available.

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

5. FLOOR SLAB AND PERMANENT DRAINAGE

The P1 floor slab can be supported on grade, provided any loose/soft/disturbed soils are removed, and approved by the geotechnical engineer.

A moisture barrier consisting of at least 200 mm of 19 mm clear crushed stone should be installed under the floor slab.

A permanent perimeter and subfloor drainage system will be required around the exterior basement walls. The perimeter and underfloor drainages for shoring systems are shown on **Drawings 10 and 11**. Where the exposed subgrade consists of cohesionless (sandy) deposits below the water table, all openings including the subgrade must be entirely covered or wrapped with geotextile filter fabric, typically a Class II non-woven textile with a filtration opening size (F.O.S.) of 50 to 100 μm.

Feasibility studies of permanent underfloor drainage and perimeter drainage were carried out in the hydrogeological investigation, to estimate seepage rates into the permanent drainage systems. If it is not feasible to install permanent underfloor and perimeter drainages, tanked basement structures can be considered.

In case of raft foundation alternative is adopted, then the substructure should be waterproofed.

6. ELEVATOR PITS

The elevator pits may be installed in cohesionless sandy soils below the water table. In this case, drainage systems at the base level of the elevator pits are not recommended, due to the concern of loss of fines. The elevator pits should be designed as a water-tight structures and should be fully waterproofed to avoid any water leaks, due to the presence of wet seams/layers of gravelly sand/sand/silt soils.

7. FROST PROTECTION

All footings exposed to seasonal freezing conditions must have at least 1.2 metres of soil cover for frost protection.

It is also emphasized that underfloor drainage (if possible) and/or an adequate free draining gravel base is required to minimize the risk of floor dampness. Floor dampness could lead to temporary icing and the risk of accidents.

8. EARTH, ROCK, AND WATER PRESSURES

The design of basement walls can incorporate the conventional design in the overburden using the earth pressure coefficient $K_1=0.40$. In the bedrock, the earth pressure coefficient K can be reduced to $K_2=0.25$.

The lateral earth/rock pressure acting at any depth on basement walls can be calculated as follows:

In soil: $p = K_1 (\gamma_1 h_1 + q) + p_w$

In rock: $p = K_2 (\gamma_1 H_1 + q + \gamma_2 h_2) + p_w$

where p = lateral earth and water pressure in kPa acting at depth h_1 or h_2

K_1, K_2 = earth pressure coefficients, K_1=0.40 for overburden soil; K_2=0.25 for bedrock

γ_1 = unit weight of overburden soil, assuming 21 kN/m^3 above the water table and 11 kN/m^3 below the water table

γ_2 = unit weight of rock below water, assuming 13 kN/m^3

h_1 = Depth in overburden soil, below ground surface

H_1 = thickness of soil above rock

h_2 = Depth in rock, below rock surface

q = value of surcharge in kPa

p_w = hydrostatic water pressure.

When the foundation wall is poured against the caisson wall, the foundation wall as well as the caisson wall should be designed for hydrostatic pressure, even though a drainage board is provided between the basement wall and the caisson wall.

9. EXCAVATION AND GROUNDWATER CONTROL

Excavation of the overburden will be relatively straightforward; however, obstructions in the fill and boulders in tills should be expected. Excavation of the shale (if required) can be carried out using the heaviest available single tooth ripper equipment. The limestone beds are frequent and may overlay the shale bedrock surface at some locations. It will be necessary to utilize jackhammer type equipment to “open” the limestone layers for the ripper.

The measured groundwater levels on April 22, 2024, in monitoring wells were at depths ranging from 2.4 to 5.7 m, corresponding to Elev. 115.6 to 117.0 m.

Positive dewatering will be required for excavations in the cohesionless sandy soils (sand and gravel to gravelly sand and sandy silt to silty sand till) below groundwater table. The groundwater table must be lowered to at least 1.0 m below the excavation bases. Groundwater is expected in shale bedrock through the fractures which will also require dewatering.

DS is carrying out a hydrogeological study at the subject Site and more comments regarding the type and extent of groundwater control required will be addressed in the hydrogeology report.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the fill material can be classified as Type 3 Soil above the groundwater table and Type 4 Soil below groundwater table or in perched water. Cohesionless sandy soils (sand and gravel to gravelly sand and sandy silt to silty sand till) can also be classified as Type 3 Soil above groundwater table and as Type 4 Soil below groundwater table or in perched water.

Obstructions in the fill material and boulders in till are anticipated. Provisions must be made in the excavation/shoring contracts for the removal of obstructions in the fill materials, boulders in till and hard layers in the shale bedrock.

The native soils free from topsoil and organics can be used as general construction backfill, provided its moisture content is within 2 percent of the optimum moisture content. Loose lifts of soil, which are to be compacted, should not exceed 200 mm. Depending on the time of construction and weather, some excavated material may be too wet to compact and will require aeration prior to its use.

Imported granular fill, which can be compacted with handheld equipment, should be used in confined areas. The excavated soils are not considered to be free draining. Where free draining backfill is required, imported granular fill such as OPSS Granular B should be used.

It should be noted that the excavated soils are subject to moisture content increase during wet weather which would make these materials too wet for adequate compaction. Stockpiles should be compacted at the surface or be covered with tarpaulins to minimize moisture uptake.

10. TEMPORARY SHORING

The proposed excavations may be supported by a temporary shoring system consisting of timber lagging and soldier piles. A tightly braced caisson wall may be required to support adjacent structures and utilities. Unsupported open cut excavation may be utilized at areas where sufficient space exists. The requirement for caisson wall to support adjacent structures is given on **Drawing 12**.

The shoring system must be designed in accordance with the 4th Edition of the Canadian Foundation Engineering Manual. The surcharge loading from adjacent structures must be considered. The soil parameters estimated to be applicable for this design are as follows:

1) Earth Pressure Coefficient for shoring:

   (a) where movement must be minimal K=0.45

   (b) where minor movement (.002H) can be tolerated K=0.30

   (c) passive earth pressure for soldier piles (unfactored) Kp=4.0 for weathered shale and 5.0 for sound shale

2) For stability check

   $\phi = 31°$

C = 0

$\gamma = 21$ kN/m^3

surcharge is to be determined by shoring contractor.

3) For rock anchors

An allowable bond stress of 600 kPa can be used in sound bedrock for the design of anchors.

However, the suggested values depend on anchor installation methods and grouting procedures. Gravity poured concrete can result in low bond values while pressure grouted anchors will give higher values and produce a more satisfactory anchor.

The soldier piles should be installed in pre-augered holes taken below the deepest adjacent excavation. The holes should be filled with concrete below the excavation level and half bag mix above the base of the excavation. The concrete strength must be specified by the shoring designer. Temporary liners will be required to help prevent the fill from caving during the installation period.

The top anchor must not be placed lower than 3.0 metres below the top of level ground surface. The contractor must decide the anchor capacity and confirm its availability. All anchors must be tested as indicated in the Canadian Foundation Engineering Manual, 4th edition.

Adhesion on the buried caisson shaft or behind the shoring system must be neglected when designing this shoring system.

Movement of the shoring system is inevitable. Vertical movements will result from the vertical load on the soldier piles resulting from the inclined tiebacks and inward horizontal movement results from earth and water pressures. The magnitude of this movement can be controlled by sound construction practices, and it is anticipated that the horizontal movement will be in the range of 0.1 to 0.25% of the shoring height.

To ensure that movements of the shoring are within an acceptable range, monitoring must be carried out. Vertical and horizontal targets on the soldier piles must be located and surveyed before excavation begins. Weekly readings during excavation should show that the movements will be within those predicted; if not, the monitoring results will enable directions to be given to improve the shoring.

11. EARTHQUAKE CONSIDERATIONS

Based on the existing borehole information and according to Table 4.1.8.4.A of OBC 2012, the subject Site for the proposed development with 1 level of underground parking can be classified as “Class C” for seismic response.

12. GENERAL COMMENTS AND LIMITATIONS OF REPORT

DS Consultants Ltd. (DS) should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, DS will assume no responsibility for interpretation of the recommendations in the report. The design recommendations given in this 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.

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to DS at the time of preparation. Unless otherwise agreed in writing by DS, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the Site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

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. DS accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report. We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

DS CONSULTANTS LTD

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Drawings

J:\2024 Projects\24-070-100 Geo-Hydro- 1995 Dundas East Mississauga, ON\Geotechnical\QGIS\1-QGIS\Drawing 1 Borehole Location Plan.qgs Apr-22

Legend

- ⌖ Borehole
- ● Monitoring Well

DS CONSULTANTS LTD.
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Telephone: (905) 264-9393
www.dsconsultants.ca

Project: GEOTECHNICAL INVESTIGATION
1995 Dundas Street East, Mississauga, ON

Title: **BOREHOLE LOCATION PLAN**

N

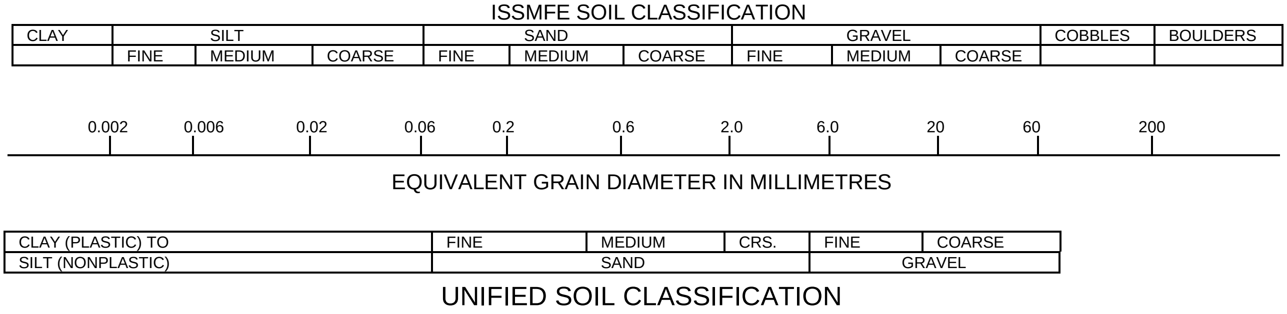
Client: LANDEAL ASSET MANAGEMENT INC.

Size: 8.5 x 11	Approved By:	O.B	Drawn By:	K.T	Date:	April 2024
Rev: 0	Scale:	As Shown	Project No.:	24-070-100	Drawing No.:	1

Image/Map Source: *Google Satellite Image*

Drawing 1A: Notes On Sample Descriptions

1. All sample descriptions included in this report generally follow the Unified Soil Classification. Laboratory grain size analyses provided by DS also follow the same system. Different classification systems may be used by others, such as the system by the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). Please note that, with the exception of those samples where a grain size analysis and/or Atterberg Limits testing have been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.



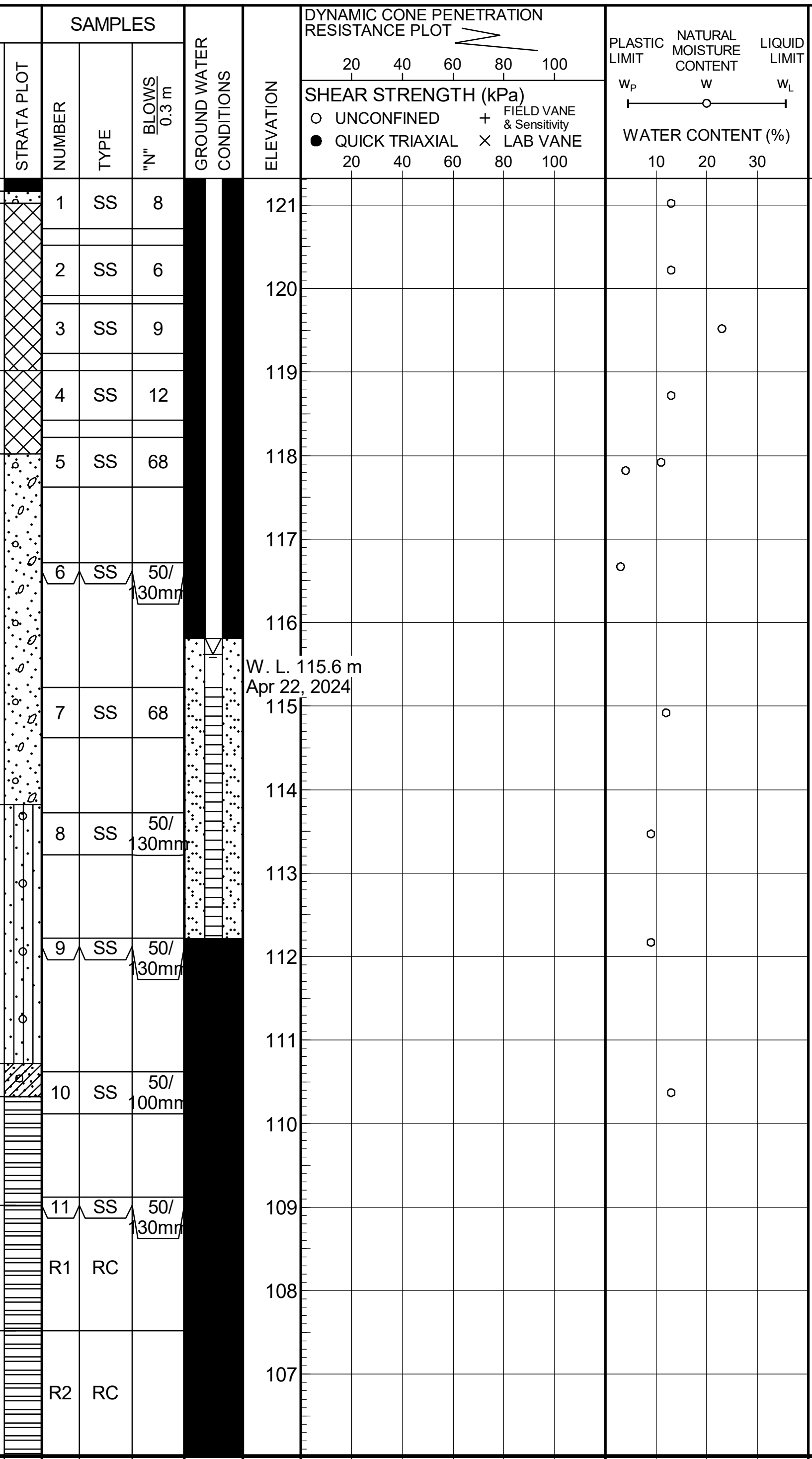
2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional preliminary geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

LOG OF BOREHOLE BH24-1

PROJECT: Geotechnical Investigation	**DRILLING DATA**
CLIENT: Landeal Asset Management Inc	Method: Solid Stem Auger/Mud Rotary
PROJECT LOCATION: 1995 Dundas St. E, Mississauga, ON	Diameter: 150mm REF. NO.: 24-070-100
DATUM: Geodetic	Date: Apr-01-2024 ENCL NO.: 2
BH LOCATION: See Drawing 1 N 4830896.44 E 615318.58	

(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
121.3					
121.1 / 0.2	**ASPHALT:** 150mm	1	SS	8	
121.0 / 0.3	**GRANULAR BASE:** sand and gravel, 150mm				
	FILL: clayey silt to silty clay, trace sand, trace gravel, brown to dark brown, moist, firm to stiff trace brick pieces, organic staining at 1.5m	2 3	SS SS	6 9	
119.0 / 2.3	**FILL:** sandy silt, some clay, some gravel, trace organics, trace brick pieces, brown, moist, compact to very dense	4	SS	12	
118.0 / 3.3	**SAND AND GRAVEL TO GRAVELLY SAND:** trace to some silt, trace clay, trace cobbles/boulders, brown, moist, very dense crushed stone fragments at 4.6m wet at 6.1m	5 6 7	SS SS SS	68 50/130mm 68	41 38 16 5
113.8 / 7.5	**SANDY SILT TILL:** trace clay, trace gravel, occasional cobbles/boulders, grey, moist to very moist, very dense	8 9	SS SS	50/130mm 50/130mm	
110.7 / 10.6	**SANDY SILT TILL/SHALE COMPLEX:** trace clay, trace gravel, grey, moist, very dense	10	SS	50/100mm	
110.3 / 11.0	**SHALE BEDROCK:** Georgian Bay Formation, grey, weathered				
109.0 / 12.3	TCR=100%, SCR=43%, RQD=6% Hard layers=10%, Maximum hard layer thickness=50mm Fragmented zones below 12.5m	11 R1	SS RC	50/130mm	
107.5 / 13.8	TCR=100%, SCR=75%, RQD=21% Hard layers=13%, Maximum hard layer thickness=230mm	R2	RC		
106.0 / 15.3	**END OF BOREHOLE:**				



Notes:
1) 50mm dia. monitoring well installed upon completion.
2) Water Level Readings:

Date: Water Level(mbgl):
April 22, 2024 5.7

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ 8=3% Strain at Failure

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LOG OF BOREHOLE BH24-2

1 OF 1

PROJECT: Geotechnical Investigation
CLIENT: Landeal Asset Management Inc
PROJECT LOCATION: 1995 Dundas St. E, Mississauga, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4830955.04 E 615311.61

DRILLING DATA
Method: Solid Stem Auger/Mud Rotary
Diameter: 150mm
Date: Apr-04-2024

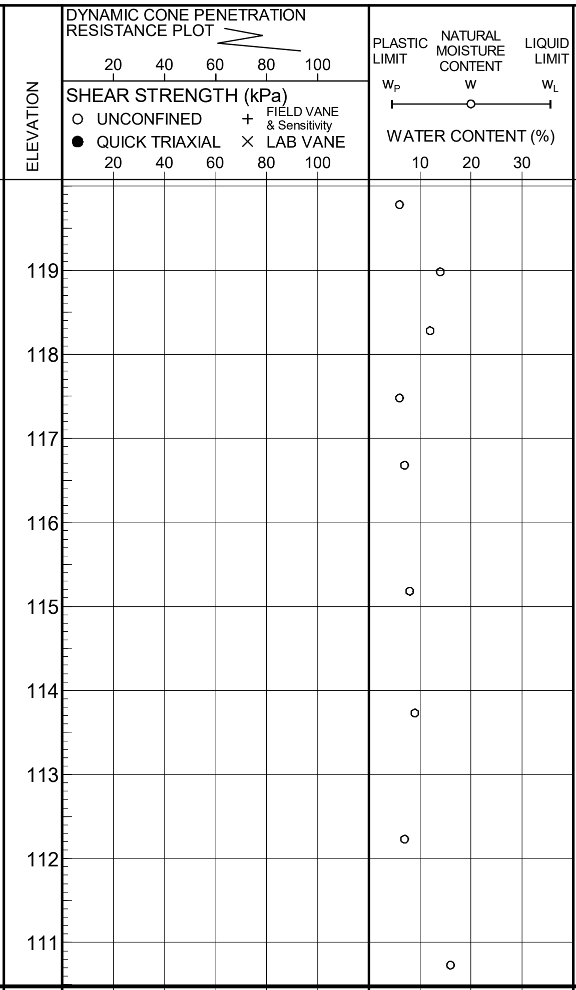
REF. NO.: 24-070-100
ENCL NO.: 3

SOIL PROFILE

(m) ELEV DEPTH	DESCRIPTION
120.1 0.0	**ASPHALT:** 150mm
119.9 0.2	**GRANULAR BASE:** sand and gravel, 150mm
119.8 0.3	**FILL:** silty sand, trace gravel, brown, moist, very dense
119.6 0.5	**ASPHALT:** layer of asphalt
119.3 0.8	**FILL:** silty sand, some clay, trace concrete pieces, brown, moist, compact
118.6 1.5	**FILL:** gravelly sand, trace silt, brown, moist, compact
118.4 1.7	**GRAVELLY SAND TO SAND AND GRAVEL:** trace silt, brown, occasional cobbles/boulders, moist, compact to very dense
115.1 5.0	**SANDY SILT TO SILTY SAND TILL:** trace to some clay, trace to some gravel, occasional cobbles/boulders, brown, moist to wet, very dense grey below 6.1m silty sand pockets below 7.6m
110.7 9.4	**SHALE BEDROCK:** Georgian Bay Formation, grey, weathered
110.5 9.6	**END OF BOREHOLE:** Notes: 1) Water encountered at depth of 4.6m during drilling.

SAMPLES

NUMBER	TYPE	"N" BLOWS/0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	56	
2	SS	28	
3	SS	16	
4	SS	46	
5	SS	53	
6	SS	54	
7	SS	50/ 100mm	
8	SS	50/ 100mm	10 65 18 7
9	SS SS	50/ 100mm	



GROUND WATER CONDITIONS

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m^3)

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
$+^3$, $\times^3$: Numbers refer to Sensitivity
○ ^{8}=3% Strain at Failure

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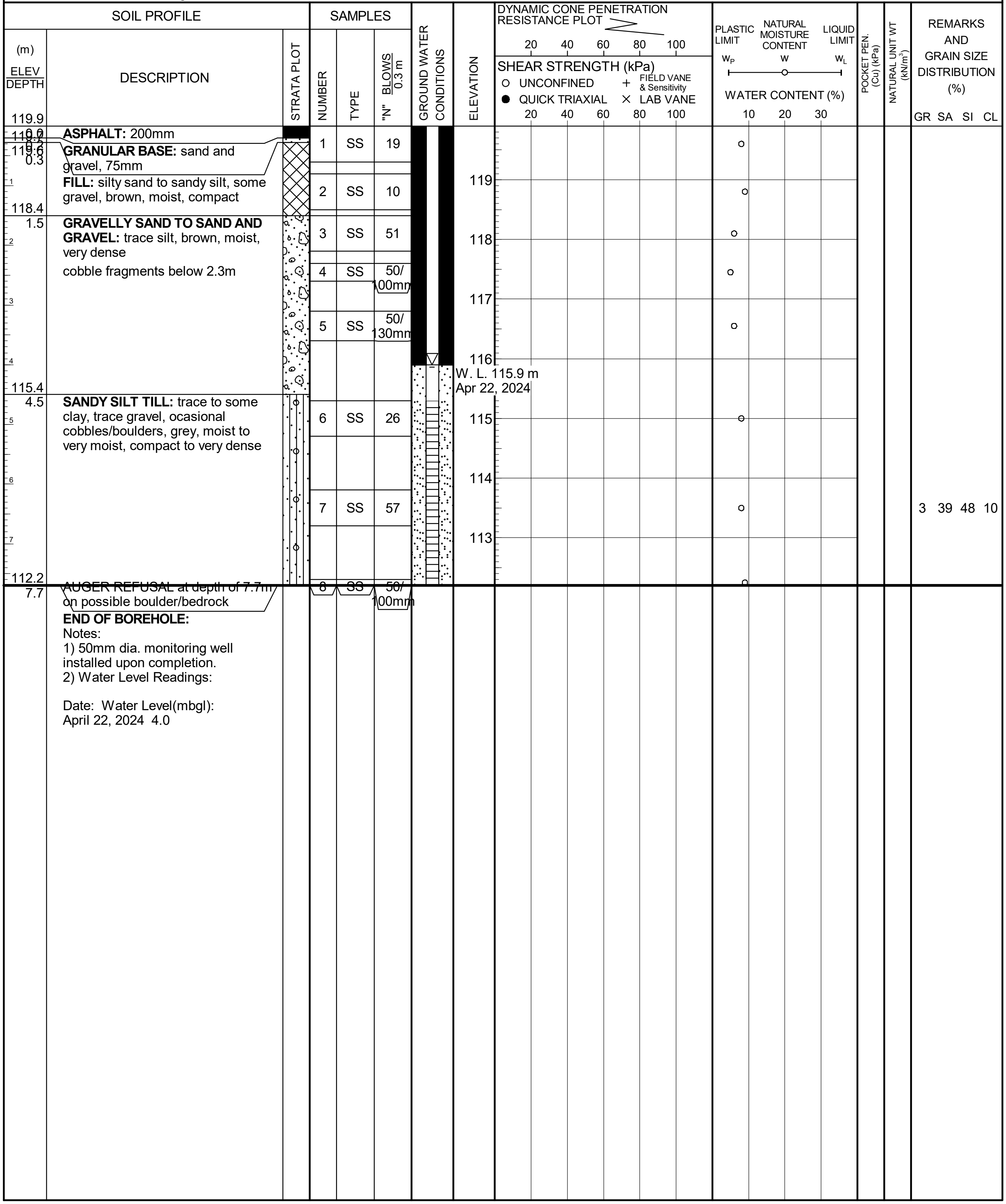
LOG OF BOREHOLE BH24-3

1 OF 1

PROJECT: Geotechnical Investigation
CLIENT: Landeal Asset Management Inc
PROJECT LOCATION: 1995 Dundas St. E, Mississauga, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4830974.51 E 615287.96

DRILLING DATA
Method: Solid Stem Auger/Mud Rotary
Diameter: 150mm
Date: Apr-04-2024

REF. NO.: 24-070-100
ENCL NO.: 4



ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	REMARKS (GR SA SI CL)
119.9					
0.0	**ASPHALT:** 200mm	1	SS	19	
119.7 0.2	**GRANULAR BASE:** sand and gravel, 75mm				
119.6 0.3	**FILL:** silty sand to sandy silt, some gravel, brown, moist, compact	2	SS	10	
118.4 1.5	**GRAVELLY SAND TO SAND AND GRAVEL:** trace silt, brown, moist, very dense cobble fragments below 2.3m	3	SS	51	
		4	SS	50/100mm	
		5	SS	50/130mm	
115.4 4.5	**SANDY SILT TILL:** trace to some clay, trace gravel, ocasional cobbles/boulders, grey, moist to very moist, compact to very dense	6	SS	26	
		7	SS	57	3 39 48 10
112.2 7.7	AUGER REFUSAL at depth of 7.7m on possible boulder/bedrock	8	SS	50/100mm	

END OF BOREHOLE:
Notes:
1) 50mm dia. monitoring well installed upon completion.
2) Water Level Readings:

Date: Water Level(mbgl):
April 22, 2024 4.0

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ 3 =3% Strain at Failure

DS SOIL LOG-2021-FINAL 24-070-100GEO.GPJ DS.GDT 24-5-7

LOG OF BOREHOLE BH24-4

PROJECT: Geotechnical Investigation
CLIENT: Landeal Asset Management Inc
PROJECT LOCATION: 1995 Dundas St. E, Mississauga, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4831004.19 E 615246.84

DRILLING DATA
Method: Solid Stem Auger/Mud Rotary
Diameter: 150mm
Date: Apr-04-2024

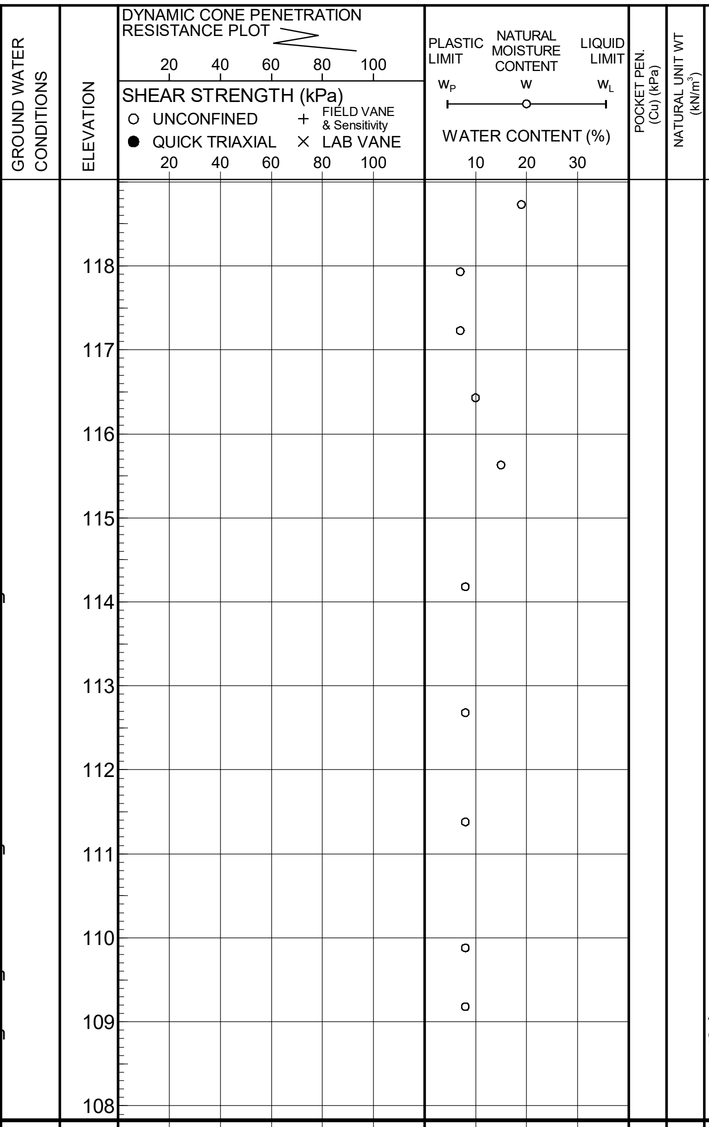
REF. NO.: 24-070-100
ENCL NO.: 5

SOIL PROFILE

(m) ELEV / DEPTH	DESCRIPTION
119.0 0.0	**ASPHALT:** 150mm
118.9 0.2	**GRANULAR BASE:** sand and gravel, 70mm
118.8 0.2	**GRANULAR FILL:** sand and gravel, brown, moist, compact to very dense
117.9 1.1	**SAND AND GRAVEL TO GRAVELLY SAND:** trace silt, cobbles/boulders, brown, moist, dense to very dense wet at 2.3m
116.0 3.0	**SANDY SILT TILL:** trace clay, trace gravel, occasional cobbles/boulders, brown, wet, dense to very dense grey, moist to very moist below 3.3m
111.5 7.5	**GRAVELLY SAND:** trace silt, trace clay, cobbles/boulders, grey, wet, very dense
110.0 9.0	**SANDY SILT TILL:** trace clay, trace gravel, occasional cobbles/boulders, grey, moist to very moist, very dense
108.4 10.6	**SHALE BEDROCK:** Georgian Bay Formation, grey, weathered
107.8 11.2	**END OF BOREHOLE:** Notes: 1) Water encountered at depth of 2.4m during drilling. 2) Borehole was moved 1.0m away due auger refusal at 9.8m due to cobble/boulder at initial location.

SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	11	
2	SS	47	
3	SS	87	
4	SS	42	
5	SS	35	
6	SS	50/ 100mm	
7	SS	50/ 50mm	
8	SS	50/ 130mm	
9	SS	50/ 100mm	
10	SS	200/ 150mm	Auger grinding-Inferred boulders
11	SS	100/ 25mm	
12	SS	110/ 0mm	



GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
$+^3, \times^3$: Numbers refer to Sensitivity
$\circ^{8}$=3% Strain at Failure

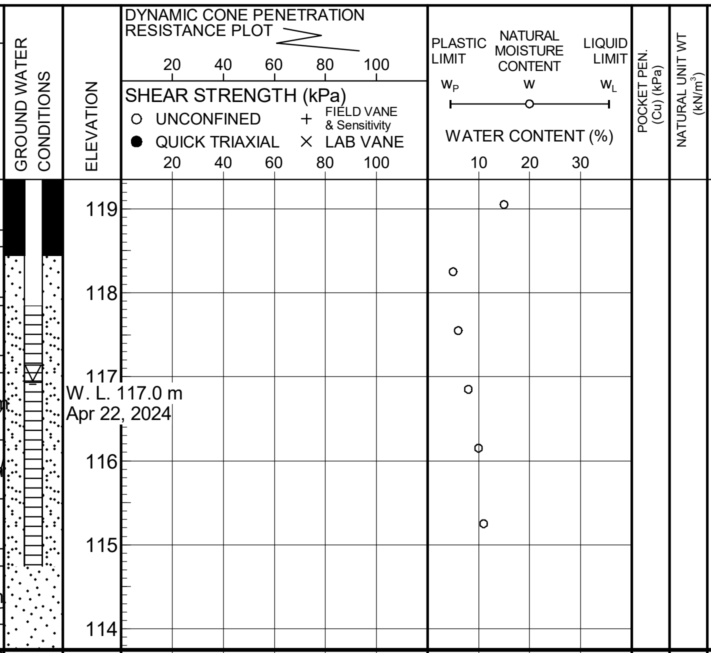
LOG OF BOREHOLE BH24-5

PROJECT: Geotechnical Investigation
CLIENT: Landeal Asset Management Inc
PROJECT LOCATION: 1995 Dundas St. E, Mississauga, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4830976.68 E 615213.51

DRILLING DATA
Method: Solid Stem Auger/Mud Rotary
Diameter: 150mm
Date: Apr-04-2024

REF. NO.: 24-070-100
ENCL NO.: 6

(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	GR	SA	SI	CL
119.4								
119.2 0.2	**ASPHALT:** 150mm	1	SS	20				
119.1 0.2	**GRANULAR BASE:** sand and gravel, 70mm							
118.8 0.6	**FILL:** silty sand, trace gravel, brown, moist, compact	2	SS	42				
	GRAVELLY SAND TO SAND AND GRAVEL: trace to some silt, trace clay, cobbles/boulders, brown, moist, dense to very dense	3	SS	72				
		4	SS	50/ 100mm	29	51	16	4
	wet at 3.1m	5	SS	50/ 75mm				
115.6 3.8	**SANDY SILT TILL:** trace clay, silty clay pockets, occasional cobbles/boulders, shale fragments inclusions, grey, moist, very dense	6	SS	81				
114.6 4.8	**SHALE BEDROCK:** Georgian Bay Formation, grey, weathered	7	SS	50/ 75mm				
113.8 5.6	**END OF BOREHOLE:**	8	SS	50/ 75mm				



Notes:
1) 50mm dia. monitoring well installed upon completion.
2) Water Level Readings:

Date: Water Level(mbgl):
April 22, 2024 2.4

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

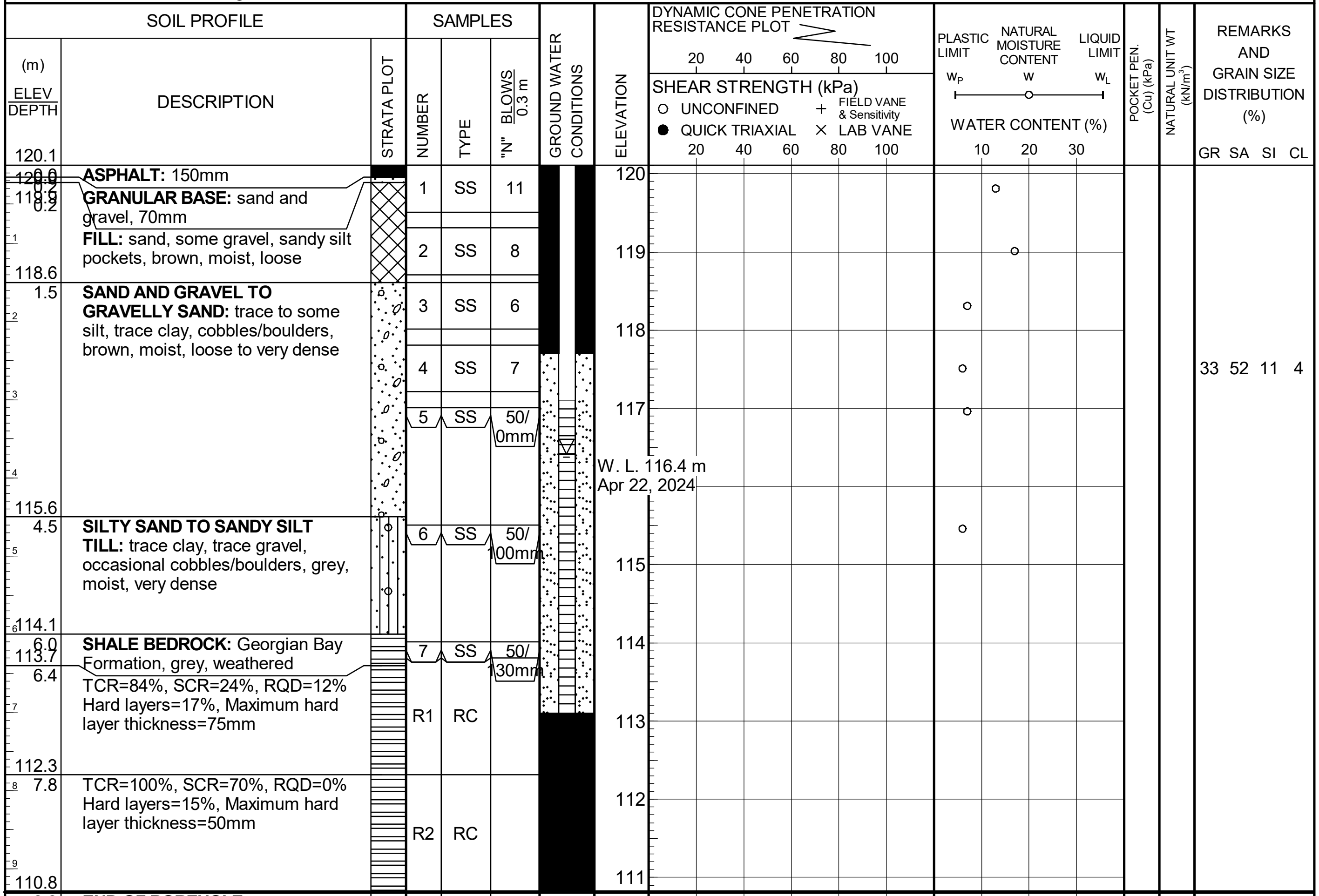
GRAPH NOTES
$+^3$, $\times^3$: Numbers refer to Sensitivity
○ ^{8}=3% Strain at Failure

LOG OF BOREHOLE BH24-6

PROJECT: Geotechnical Investigation
CLIENT: Landeal Asset Management Inc
PROJECT LOCATION: 1995 Dundas St. E, Mississauga, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4830894.61 E 615275.17

DRILLING DATA
Method: Solid Stem Auger/Mud Rotary
Diameter: 150mm
Date: Apr-02-2024

REF. NO.: 24-070-100
ENCL NO.: 7



ELEV (m) / DEPTH	DESCRIPTION
120.1	
120.0 0.2	**ASPHALT:** 150mm
119.9 0.2	**GRANULAR BASE:** sand and gravel, 70mm
	FILL: sand, some gravel, sandy silt pockets, brown, moist, loose
118.6 1.5	**SAND AND GRAVEL TO GRAVELLY SAND:** trace to some silt, trace clay, cobbles/boulders, brown, moist, loose to very dense
115.6 4.5	**SILTY SAND TO SANDY SILT TILL:** trace clay, trace gravel, occasional cobbles/boulders, grey, moist, very dense
114.1 6.0	**SHALE BEDROCK:** Georgian Bay Formation, grey, weathered
113.7 6.4	TCR=84%, SCR=24%, RQD=12% Hard layers=17%, Maximum hard layer thickness=75mm
112.3 7.8	TCR=100%, SCR=70%, RQD=0% Hard layers=15%, Maximum hard layer thickness=50mm
110.8 9.3	**END OF BOREHOLE:**

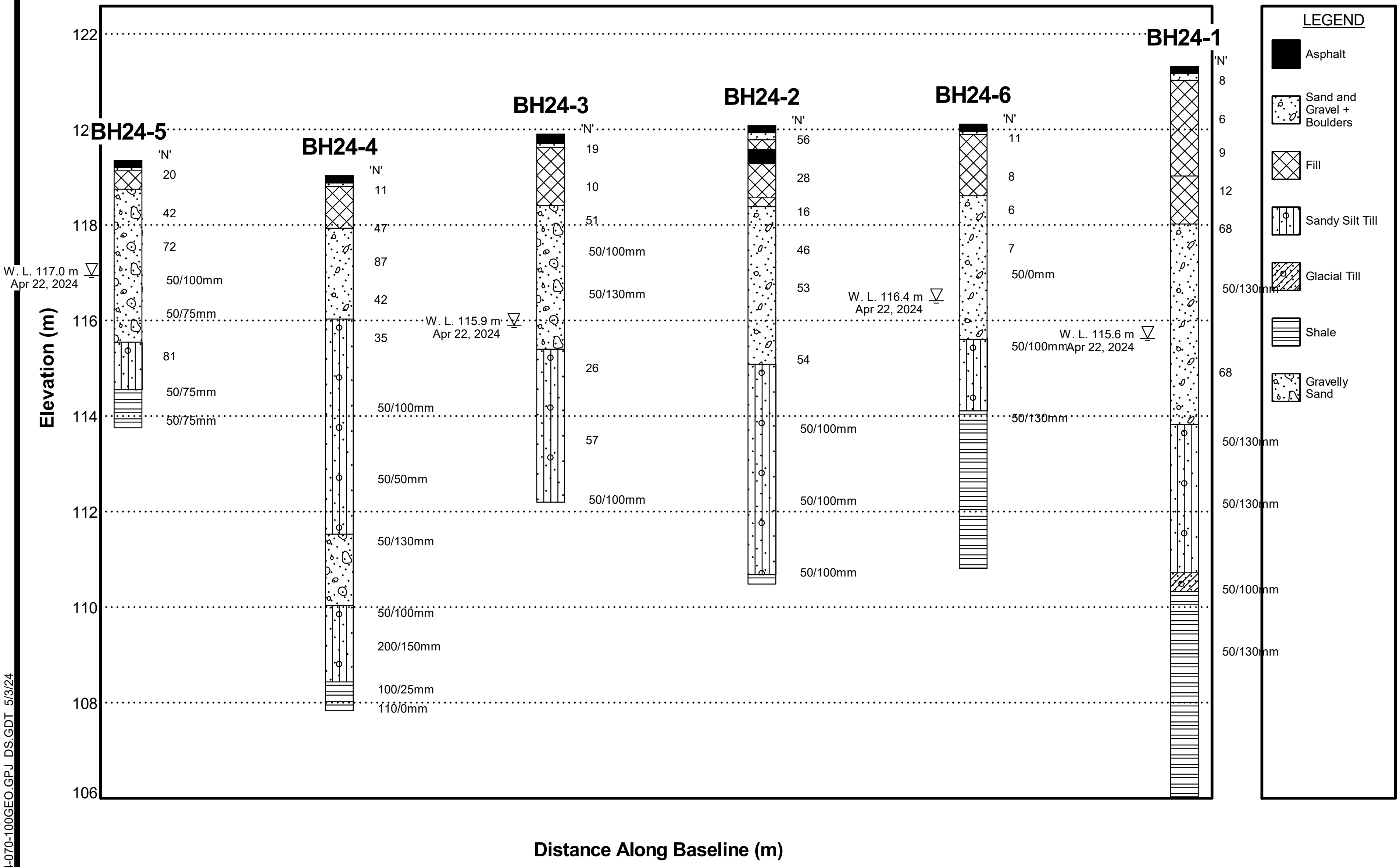
SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	11				
2	SS	8				
3	SS	6				
4	SS	7	33	52	11	4
5	SS	50/ 0mm				
6	SS	50/ 100mm				
7	SS	50/ 130mm				
R1	RC					
R2	RC					

Notes:
1) 50mm dia. monitoring well installed upon completion.
2) Water Level Readings:

Date: Water Level(mbgl):
April 22, 2024 3.7

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity
○ ³=3% Strain at Failure



DS FENCE (M) 24-070-100GEO.GPJ DS.GDT 5/3/24

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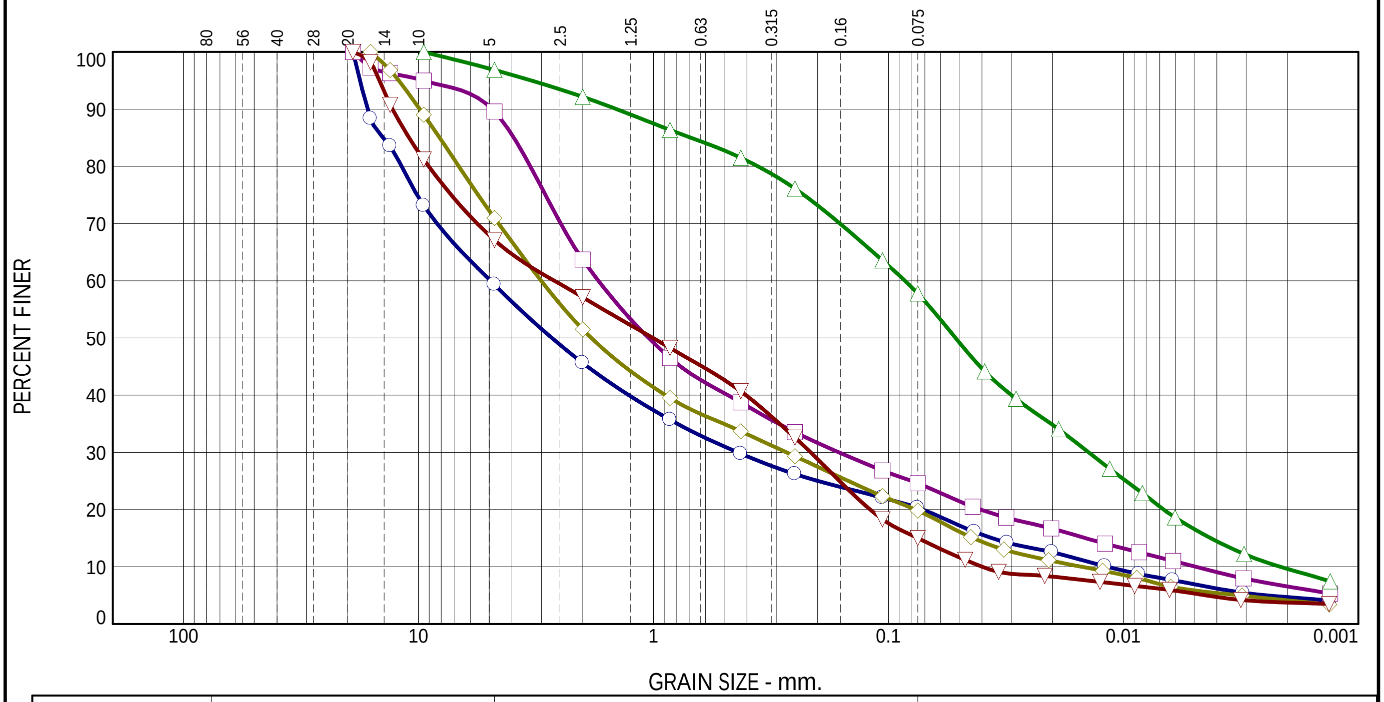
Generalized Sub-surface Profile (NTS)

DRAWING NO.	8
JOB NO.	24-070-100
DATE	May 2, 2024

Particle Size Distribution Report

ASTM D422



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	40.7	13.7	15.8	9.5	15.6	4.7
□	0.0	0.0	10.4	25.9	25.0	14.1	18.0	6.6
△	0.0	0.0	3.2	4.6	10.7	23.7	48.2	9.6
◇	0.0	0.0	29.0	19.5	17.8	13.9	15.7	4.1
▽	0.0	0.0	32.9	10.0	16.4	25.7	11.3	3.7

╳	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			14.1185	4.9351	2.7058	0.4375	0.0361	0.0117	3.32	421.84
□			3.9262	1.7150	1.0485	0.1626	0.0145	0.0049	3.13	348.37
△			0.6932	0.0850	0.0518	0.0141	0.0042	0.0021	1.08	39.56
◇			8.1226	3.0049	1.8374	0.2715	0.0435	0.0150	1.63	200.14
▽			10.9677	2.6422	1.0044	0.2141	0.0747	0.0398	0.44	66.32

Material Description	USCS	AASHTO
○ Sand & gravel, Some silt, trace clay		
□ Silty sand till, trace clay, some gravel		
△ Sandy silt till, trace clay, trace gravel		
◇ Gravelly Sand, Some silt, trace clay		
▽ Gravelly sand, some silt, trace clay		

Project No. 24-070-100 Client: Landeal Asset Management Inc.
Project: Geotechnical Investigation, 1995 Dundas Street East, Mississauga.

○Location: BH24-1 SS7 Sample Number: VM-5199
□Location: BH24-2 SS8 Sample Number: VM-5199
△Location: BH24-3 SS7 Sample Number: VM-5199
◇Location: BH24-5 SS4 Sample Number: VM-5199
▽Location: BH24-6 SS4 Sample Number: VM-5199

Remarks:
○F.M.=3.97
□F.M.=2.88
△F.M.=0.90
◇F.M.=3.50
▽F.M.=3.37

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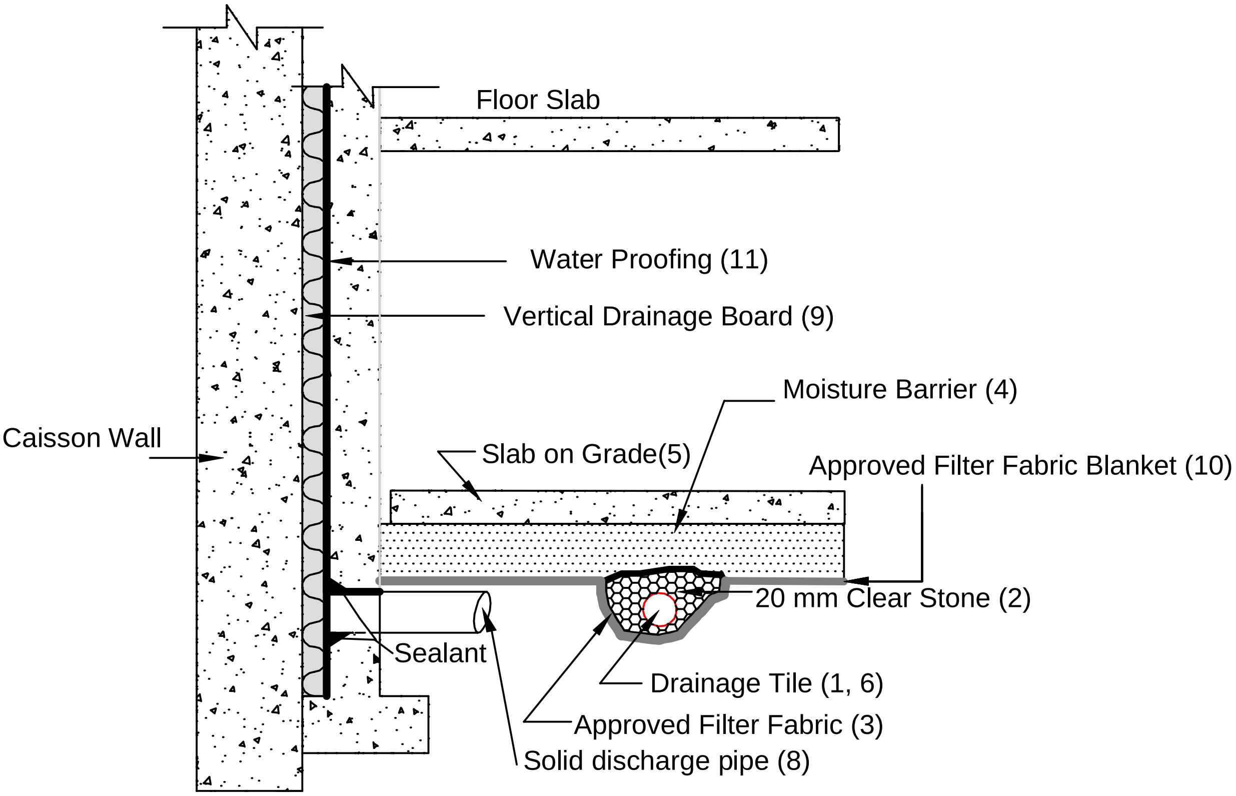
CCIL

Figure: 9

Tested By: Helen/Disha Checked By: S.Kirupa



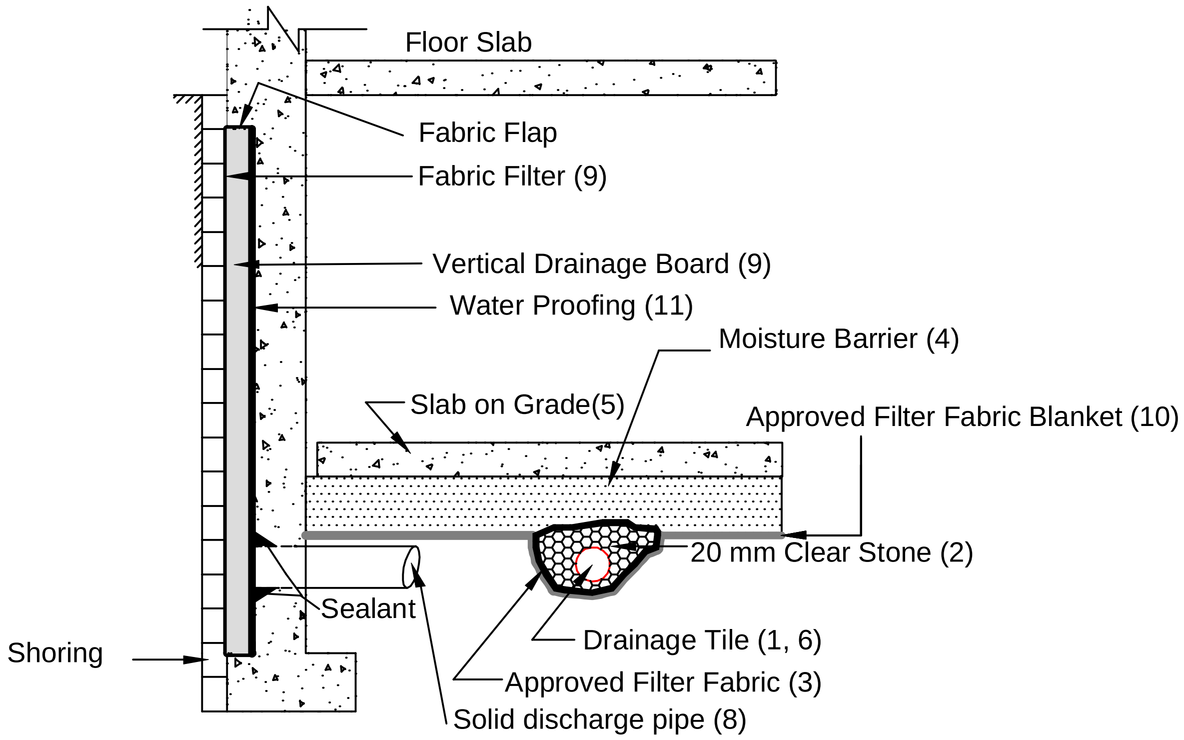
EXTERIOR FOOTING

Notes

1. Drainage tile to consist of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet, spaced between columns.
2. 20 mm (3/4") clear stone - 150 mm (6") top and side of drain. If drain is not on footing, place100 mm (4 inches) of stone below drain .
3. Wrap the clear stone with an approved filter membrane (Terrafix 270R or equivalent).
4. Moisture barrier to be at least 200 mm (8") of compacted clear 20 mm (3/4") stone or equivalent free draining material. A vapour barrier may be required for specialty floors.
5. Slab on grade should not be structurally connected to the wall or footing.
6. Underfloor drain invert to be at least 300 mm (12") below underside of floor slab. Drainage tile placed in parallel rows 6 to 8 m (20 to 25') centers one way. Place drain on 100 mm (4") clear stone with 150 mm (6") of clear stone on top and sides. Enclose stone with filter fabric as noted in (3).
7. Do not connect the underfloor drains to perimeter drains.
8. Solid discharge pipe located at the middle of each bay between the solider piles, approximate spacing 2.5 m, outletting into a solid pipe leading to a sump.
9. Vertical drainage board mira-drain 6000 or eqivalent with filter cloth should be continous from bottom to 1.2 m below exterior finished grade.
10. The entire subgrade to be sealed with approved filter fabric (Terrafix 270R or equivalent) if non-cohesive (sandy) soils below ground water table encountered.
11. The basement walls must be water proofed using bentonite or equivalent water-proofing system.
12. Review the geotechnical report for specific details. Final detail must be approved before system is considered acceptable.

DRAINAGE RECOMMENDATIONS
Shored Basement wall with Underfloor Drainage System

(not to scale)



EXTERIOR FOOTING

Notes

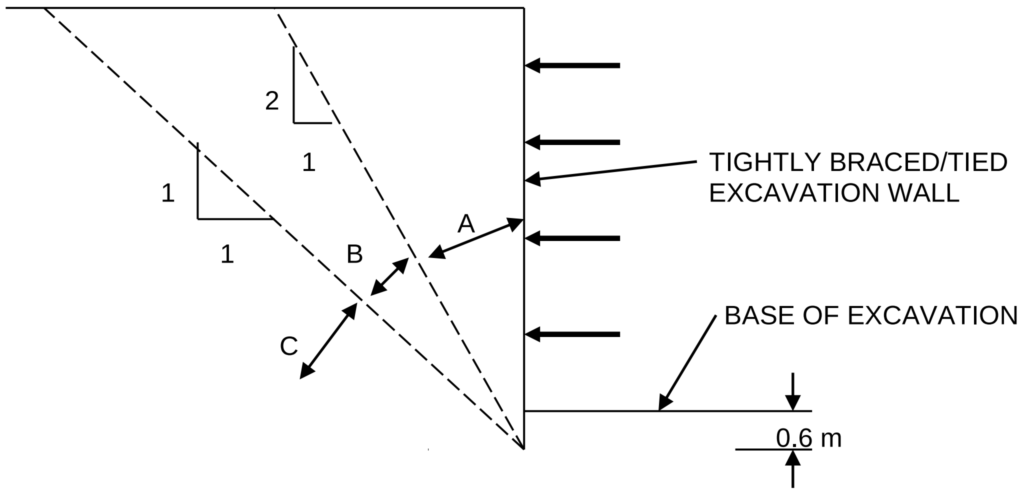
1. Drainage tile to consist of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet, spaced between columns.
2. 20 mm (3/4") clear stone - 150 mm (6") top and side of drain. If drain is not on footing, place100 mm (4 inches) of stone below drain .
3. Wrap the clear stone with an approved filter membrane (Terrafix 270R or equivalent).
4. Moisture barrier to be at least 200 mm (8") of compacted clear 20 mm (3/4") stone or equivalent free draining material. A vapour barrier may be required for specialty floors.
5. Slab on grade should not be structurally connected to the wall or footing.
6. Underfloor drain invert to be at least 300 mm (12") below underside of floor slab. Drainage tile placed in parallel rows 6 to 8 m (20 to 25') centers one way. Place drain on 100 mm (4") clear stone with 150 mm (6") of clear stone on top and sides. Enclose stone with filter fabric as noted in (3).
7. Do not connect the underfloor drains to perimeter drains.
8. Solid discharge pipe located at the middle of each bay between the solider piles, approximate spacing 2.5 m, outletting into a solid pipe leading to a sump.
9. Vertical drainage board with filter cloth should be kept a minium of 1.2 m below exterior finished grade.
10. The entire subgrade to be sealed with approved filter fabric (Terrafix 270R or equivalent) if non-cohesive (sandy) soils below ground water table encountered.
11. The basement walls should be water proofed using bentonite or equivalent water-proofing system.
12. Review the geotechnical report for specific details. Final detail must be approved before system is considered acceptable.

DRAINAGE RECOMMENDATIONS
Shored Basement wall with Underfloor Drainage System

(not to scale)

Guidelines for Underpinning in Soil and Excavation Support

Existing foundations located within Zone A normally require underpinning, especially for heavy structures. For some foundations in Zone A, it may be possible to eliminate underpinning and control foundation movement by tightly braced excavation walls, such as caisson walls.



Zone A Foundations located within this zone normally require underpinning. Horizontal and vertical pressures on the excavation wall of non underpinned foundations must be considered

Zone B Foundations located within this zone normally do not require underpinning. Horizontal and vertical pressures on the excavation wall of non underpinned foundations must be considered

Zone C Underpinning to structures is normally founded in this zone. Lateral pressure from underpinning is not normally considered

(Reference: Figure 26.27 from Canadian Foundation Engineering Manual, 4th Edition)

Appendix A

GENERAL COMMENTS - BEDROCK IN METRO TORONTO AREA
PHOTOGRAPHS OF ROCK CORES

General Comments – Bedrock in Metro Toronto Area

The bedrock that makes spread footings or caissons a popular choice for high-rise foundation support is a shale or shale limestone composition. The highest member, the Queenston Formation, is generally found west of Toronto, while the Georgian Bay Formation underlies most of Metro Toronto, with the Collingwood Formation east of Toronto. The Queenston is, relatively speaking, the weaker of the three formations that are likely to support caissons or footings.

The Georgian Bay as well as the Queenston and Collingwood Formation are of Middle Ordovician Age. It is defined as the rock unit that overlies the bluish grey shales of the Collingwood Formation and is in turn overlain by the red shale of the Queenston Formation. The Georgian Bay Formation consists of bluish and grey shale with interbeds of sandstone, limestone and dolostone. Towards the west where the Georgian Bay formation underlies the Queenston Formation, the limestone content increases significantly and limestone and/or sandstone may comprise as much as 70 to 90 percent of the bedrock. The hard layers are usually less than about 100 to 150 mm thick, but some layers are much thicker. The thicker layers have been observed to be as much as 750 to 900 mm at some sites. The layers are actually lenses and they can vary significantly in thickness over short distances.

The upper portion of the bedrock is commonly weathered for a depth of 600 to 1000 mm and within this weathered zone hard limestone layers or lenses are common. These hard limestone layers can result in contractual problems for augers and can provide misleading bedrock elevations. Where the weathering is more extensive a shale till layer may be found above the bedrock. In the sound bedrock, the limestone, sandstone, dolostone is hard to very hard.

Stress relief features such as folds and faults are common in the bedrock. In these features, the rock is heavily fractured and sheared, and contains layers of shale rubble and clay. Weathering is much deeper than the surrounding rock in these features and often there is a lateral migration of the stress relief features resulting in sound unweathered bedrock overlying fractured and weather bedrock. The stress relief features are usually in the order of 4 to 6 m wide, but the depth can vary from 4 to 5 m to in excess of 10 m. These features occur randomly.

The bedrock contains significant high locked in horizontal stresses. These stresses can impose significant loads on tunnel walls but the slower rate of construction for basements allows for a relaxation of these stresses and they are not normally a problem for basement construction.

Groundwater seepage below the top 1000 mm is generally small, however, at several locations in Toronto and Mississauga large quantities have been encountered.

Bedding joints in the bedrock are very close-to-close, smooth planar in the shale and rough planar in the limestone. Significant vertical jointing is common.

Where the bedrock was cored, a detailed description of the rock core is appended to the borehole log.

Design features related to the bedrock are discussed in other sections of this report, and these general comments must be considered with these comments.

Methane gas exists in the bedrock, normally below the top 1000 mm and more concentrated with depth. Appropriate care and monitoring are essential in all confined bedrock excavations, particularly caissons and tunnels.

24-070-100

BH24-1

R1: ~40’7” ~45’7”
R2: ~45’7” ~50’7”

BH24-6

R1: ~21’2” ~25’11”
R2: ~25’11” ~30’11”