REPORT ON
Preliminary Geotechnical Investigation
Proposed Condominium Development
3400 Dufferin Street and 8 Jane Osler Boulevard
Toronto, Ontario

PREPARED FOR:

Dufferin-401 Properties Limited

PREPARED BY:

DS Consultants Ltd.

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DRAWINGS

1. INTRODUCTION

DS Consultants Ltd. (DS) was retained by Dufferin-401 Properties Limited (the Client) to undertake a preliminary geotechnical investigation for the proposed condominium development located at 3400 Dufferin Street and 8 Jane Osler Boulevard in the City of Toronto, Ontario.

The subject site is presently occupied by a large building (Car Dealership) with surrounding pavements which will be demolished to make way for the proposed new development. Based on the provided information, it is understood that the proposed development will include the construction of two (2), 29-storey buildings and a 9-storey building, and each building supported over its own 2-level underground parking structure (P2). The design grades (elevations) were not provided at the time of preparation of this report.

Concurrent with this geotechnical investigation program, a hydrogeological study and Phase One and Two Environmental Site Assessments (ESAs) have been carried out by DS, and the results of which will be addressed separately.

The purpose of this geotechnical investigation was to obtain the subsurface conditions at five (5) borehole locations and from the findings at the boreholes provide preliminary 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

It is recommended that additional boreholes be carried out for the final design of the proposed buildings.

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 Dufferin-401 Properties Limited and designers and architects. Use of this report by third party without DS consent is prohibited.

2. FIELD AND LABORATORY WORK

A total of five (5) boreholes (BH22-1 through BH21-5, see **Drawing 1** for borehole locations) were drilled by DS to depths ranging from 18.0 m to 30.8 m below existing ground surface.

These boreholes were drilled with hollow stem continuous flight auger and mud rotary drill equipment by a drilling sub-contractor under the direction and supervision of DS Consultants Ltd. personnel. Samples of the soil encountered in the boreholes 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. The samples were logged in the field and returned to the DS Consultants Ltd. laboratory for detailed examination by the project engineer and for laboratory testing.

Prior to drilling operations, all underground utilities were cleared at the borehole locations by representatives of the public and private public utilities company working with personnel from DS.

In addition to visual examination in the laboratory, all soil samples were tested for water content. Ten (10) selected soil samples were subjected to the grain size analysis and the respective gradation curves are provided in **Drawing 7**. Six (6) selected soil sample were subject Atterberg Limits testing and the results are provided in **Drawing 8**.

The elevation surveying of the borehole locations was undertaken by DS personnel, using the differential GPS unit. It should be noted that the elevations at the as-drilled borehole locations were not provided by a professional surveyor and should be considered approximate. Contractors performing any work referenced to the borehole elevations should confirm the borehole elevations for their work.

3. 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 on the individual borehole logs presented 0n **Drawings 2 to 6**.

3.1 SOIL CONDITIONS

Asphaltic Concrete and Granular Material:

Asphaltic concrete pavement was encountered surficially in all boreholes. The thickness of the asphaltic concrete ranged from about 75 to 130 mm.

Granular material consisting of sand and gravel was encountered below asphaltic concrete in all boreholes, and the thickness of granular material ranged from about 230 to 300 mm. The compactness of granular material was loose to compact (generally compact), as indicated by the measured SPT 'N' values of 7 to 22 blows per 300 mm penetration.

Fill Material:

Fill materials consisting of silty clay to clayey silt with trace to some organics and trace gravel were encountered below the pavement structure in all boreholes. The fill material extended to depths of approximately 1.6 to 2.3 m below existing grade. The consistency of the silty clay to clayey silt fill was firm to stiff, as indicated by a measured SPT ‘N’ values of 5 to 9 blows per 300 mm penetration. The moisture content of the fill layer ranged from 14 to 18%.

Silty Clay to Clayey Silt (Till):

Below the fill material, silty clay to clayey silt glacial (till) deposits were encountered in all boreholes and extended to depths ranging from 12.2 to 16.8 m below existing grade. At BH22-1, a thin wet deposit of silt with trace clay, trace to some sand and trace gravel interrupts the silty clay to clayey silt till between 12.2 and 13.7 m depth below existing grade. SPT ‘N’ value of 31 blows per 300 mm of penetration indicates a dense compactness of the deposit. The silty clay to clayey silt (till) deposits had a stiff to hard consistency, with measured SPT ‘N’ values ranging from 9 to 64 blows per 300mm penetration. The silty clay to clayey silt till was generally very stiff to hard below about 1.5 m depth below existing grade, however, at BH22-3, a weaker stiff zone with SPT ‘N’ value of 9 blows per 300 mm of penetration was encountered at a depth of about 15.2 m below existing grade. The moisture content of the silty clay to clayey silt till soil ranged from 10 to 28%.

Grain size analyses of seven (7) silty clay to clayey silt (till) samples (BH22-1/SS7, BH22-1/SS12, BH22-2/SS4, BH22-2/SS12, BH22-3/SS5, BH22-3/SS7 and BH22-5/SS8) were conducted and the results are presented on **Drawing 7**, with the following fractions:

Clay: 17 to 19%

Silt: 42 to 50%

Sand: 29 to 37%

Gravel: 1 to 7%

Atterberg Limits tests of six (6) silty clay to clayey silt (till) samples (BH22-1/SS7, BH22-2/SS4, BH22-2/SS12, BH22-3/SS5, BH22-3/SS7 and BH22-5/SS8) were conducted. The results are shown in **Drawing 8** and on the borehole log and are summarized as follows:

Liquid limit (W_L):	18 to 22%
Plastic limit (W_P):	12 to 14%
Plasticity index (PI):	6 to 9

Sandy Silt (Till):

A sandy silt glacial (till) deposit was encountered at below the silty clay to clayey silt till at varying depths in all boreholes and extended to depths ranging from 18.0 to 19.9 m below existing grade, i.e., depth of investigation in BH22-2, BH22-3, and BH22-5. A very thin (approx. 400 mm thick) layer of silty clay till was encountered below the sandy silt till in BH22-4 and extended to the depth of investigation, i.e., depth of 18.4 m below existing grade. The sandy silt till deposit was in a compact to very dense (generally very dense) state as indicated by measured SPT ‘N’ values ranging from 16 to over 50 blows per 300mm penetration. The moisture content of the sandy silt till soil ranged from 9 to 14%.

Grain size analyses of two (2) sandy silt (till) samples (BH22-3/SS14 and BH22-4/SS13) were conducted and the results are presented in **Drawings 7**, with the following fractions:

Clay: 9 to 11%

Silt: 49 to 66%

Sand: 24 to 37%

Gravel: 1 to 3%

Upper Sand:

Underlying the sandy silt till in BH22-1, a cohesionless sand deposit with some silt, trace clay and trace gravel was encountered and extended to a depth of 19.8 m below existing grade. The sand was found in a very dense compactness, with measured SPT ‘N’ value of over 50 blows per 300mm penetration. The moisture content of the sand soil was in the order of 10%.

Lower Silt:

Underlying the cohesionless deposit in BH22-1, a silt deposit with some clay, trace sand and trace gravel was encountered and extended to a depth of 22.9 m below existing grade. The silt was found in a very dense compactness, with measured SPT ‘N’ value of over 50 blows per 300mm penetration. The moisture content of the silt soil was in the order of 15%.

Clayey Silt:

A cohesive clayey silt deposit with trace to some sand, silt seams and trace gravel was encountered below the silt in BH22-1 and extended to a depth of 25.9 m below existing grade. The clayey silt had a hard consistency, with measured SPT ‘N’ values of 48 to over 50 blows per 300mm penetration. The moisture content of the clayey silt soil was in the order of 19 to 22%.

Lower Sand to Silty Sand:

A lower cohesionless deposit of sand to silty sand with inclusions of clay and gravel was encountered below the clayey silt in BH22-1 and extended to the depth of investigation, i.e., depth of 30.8 m below existing grade. The sand to silty sand was found in a very dense compactness, with measured SPT ‘N’ value of over 50 blows per 300mm penetration. The moisture content of the sand to silty sand soil ranged from 10 to 17%.

3.2 GROUNDWATER CONDITIONS

Monitoring wells were installed in all boreholes (BH22-1 to BH22-5) for the long-term groundwater table monitoring and hydrogeological testing. Stabilized groundwater was found in monitoring wells at depths ranging from 1.3 to 10.4 m, corresponding to Elev. 180.1 to 188.9 m, as listed on **Table 1**:

Table 1: Groundwater Levels Observed in DS Monitoring Wells

BH No.	Ground Surface Elevation (m)	Date of Drilling	Date of Observation	Depth of Groundwater (m)	Elevation of Groundwater (m)
BH22-1	190.5	June 15, 2022	July 6, 2022	10.41	180.1
BH22-2	190.2	June 14, 2022	July 6, 2022	1.70	188.5
BH22-3	190.2	June 22, 2022	July 6, 2022	1.30	188.9
BH22-4	189.7	June 15, 2022	July 6, 2022	2.4	187.3
BH22-5	190.1	June 17, 2022	July 6, 2022	5.5	184.6

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 monitoring should be carried out to confirm the groundwater conditions.

4. DISCUSSION AND GEOTECHNICAL RECOMMENDATIONS

Based on the borehole information, geotechnical discussion and recommendations for the proposed development are presented as follows. In this report, the soil and groundwater conditions are interpreted as relevant to the design and construction of the proposed buildings. Comments relating to construction are intended for the guidance of the design engineer to establish constructability.

The construction methods described in this report must not be considered as being specifications or direct recommendations to the contractors, or as being the only suitable methods. Prospective contractors should evaluate all of the factual information, obtain additional subsurface information as they might deem necessary and should select their construction methods, sequencing and equipment based on their own experience in similar ground conditions. The readers of this report are also reminded that the conditions are known only at the borehole locations and conditions may vary significantly in-between.

4.1 FOUNDATIONS

Based on the design information provided, the proposed development will include the construction of two (2) 29-storey buildings and a 9-storey building, and each building supported over its own 2-level underground parking structure (P2). Finished floor elevation at the ground floor or the lowest basement floor (P2) were not known to us at the time of preparation of this report. It is assumed that the P2 basement floor will be set at about 6 to 8 m± below ground. Footings will be founded 1 to 2 m below P2 level. The measured groundwater levels on July 6, 2022, in monitoring wells were at depths ranging from 1.3 to 10.4 m, corresponding to Elev. 180.1 to 188.9 m. Therefore, the P2 basement floor will be below the groundwater level.

Based on the borehole information, shallow foundations (raft foundations and footings) and CFA piles are recommended to support the proposed structures. Conventional drilled caissons are considered to have constructability issues due to the presence of water bearing sand, silt and silty sand soils and associated base caving/upheaving problems.

4.1.1. Bearing Capacity of Soil for Footings and Raft Foundations

Based on borehole information, raft foundations and conventional footings founded on the undisturbed native soils at or below Elev. 184.5 m can be designed for bearing capacity values of 300 to 400 kPa at SLS and 450 to 600 kPa at ULS.

The bearing capacity values and the corresponding founding elevations at the borehole locations are summarized on **Table 2**.

Table 2: Bearing Values and Founding Levels of Footings/Rafts on Undisturbed Native Soils

Borehole No.	Borehole Elevation in Existing Basement (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)
BH22-1	190.5	400	600	6.0	184.5
BH22-2	190.2	400	600	6.0	184.2
BH22-3	190.2	300 200	450 300	6.1 14.2	184.1 176.0
BH22-4	189.7	400	600	6.0	183.7
BH22-5	190.1	400	600	6.0	184.1

It should be noted that a weaker zone of the clayey silt till was encountered below Elev. 176 m in BH22-3, therefore, a reduced bearing value of 200 kPa SLS/300 kPa ULS is given below Elev. 176.0 m for BH22-3 in **Table 2**. During the design stage, finite element analysis can be conducted to ensure that the underlying soil is not overstressed.

A subgrade reaction modulus of kt = 10 MPa/m in BH22-3 and kt = 13 MPa/m in the other borehole areas can be used for the design of the raft foundations.

It should be noted that, based on City of Toronto's Foundation Drainage Policy ("FDP"; November 1, 2021), long-term Discharge of Foundation Drainage that contains any groundwater will not be permitted to the City's storm or combined sewer system. As such, the proposed building should be designed as a 'bathtub' or a tanked structure, with a raft foundation at the base.

4.1.2. CFA Piles

In the areas where the raft pressure exceeds the available bearing capacity values of 300 to 400 kPa at SLS and 450 to 600 kPa at ULS, a combined raft-CFA pile foundation system can be adopted. The raft foundation can be designed for 400 kPa at SLS (600 kPa at ULS), and the additional design loads exceeding 300 to 400 kPa at SLS (450 to 600 kPa at ULS) can be supported by CFA piles. CFA piles of 600 mm in diameter and minimum 27 m below existing ground (i.e., pile tip/toe at or below Elevation 163.0 m) can be designed for bearing capacity values of 2000 kN/pile at SLS and

CFA piles are cast-in-place concrete piles which are formed by drilling using hollow augers to the target depth then pumping concrete through the augers as they are retracted. A conventional rebar cage is pushed into the concrete after forming the pile.

The bearing resistance of the piles will highly depend on the quality and procedure of the pile installation, as well as on the diameter and depth of the piles. The bearing capacity values (2000 kN/pile at SLS and 2700 kN/pile at ULS) and the required pile diameter and depth are recommended in **Section 4.1.1**. The actual bearing resistance and required length of the piles must be determined by field load tests, prior to the installation of the production piles. Minimum 2 field load tests (pre-production tests) are recommended to confirm the availability of the assumed bearing values. The test piles must be loaded to at least 1.67 times the design loads at ULS, i.e., to 4510 kN/pile. Depending on the load test results, deeper/longer piles may be required to achieve the design bearing resistances.

The bearing resistances of CFA piles will be highly dependent on the contractor's experience, the quality and procedure of the pile installation, and the skills of the installation operator(s). The CFA contractor must review the borehole information and evaluate bearing capacity of the piles based on their experience. The quality and the design bearing resistance of the piles must be ensured by the CFA contractor. A specialty contractor should be retained to design and install the CFA piles based on the performance specification and design bearing resistances.

The glacial till deposits should be expected to contain cobbles and boulders. Appropriate equipment and procedures will be required to penetrate obstructions (cobbles and boulders) that are encountered during drilling for CFA piles.

Prior to the pile construction, the contractor should submit the details of the installation plan, load test program, installation procedure, automated monitoring system and control parameters, grout/concrete mix design, and reinforcement installation etc. for the review by the structural engineer and the geotechnical engineer.

In order to avoid group effect on the bearing capacity of the piles, the horizontal centre-to-centre spacing of adjacent CFA piles should be at least 3 times its diameter.

All pile installation must be inspected by this office. In addition to the pre-production pile load tests, post-installation integrity tests and/or verification load tests are also required. Piles that have installation records out of specification or that otherwise appear abnormal can be selected for integrity tests (i.e., sonic echo tests) or verification load tests to determine if they should be accepted or rejected.

4.1.3. Other Comments on Foundations

Raft foundations/footings and CFA piles designed to the specified bearing capacity values at SLS are expected to settle less than 30 mm total and 25 mm differential.

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

All foundations must be inspected by this office. The excavated footing/raft bases 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 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.

It should be noted that the recommended bearing resistances have been calculated by DS from the borehole information for the 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 to validate the information for use during the construction stage.

4.2 FLOOR SLAB AND PERMANENT DRAINAGE

With 2 levels of basement, the feasibility of adopting permanent underfloor drainage and perimeter drainage must be investigated from the hydrogeological study. If it is not possible to adopt the permanent drainage system for the buildings, then the proposed buildings should be designed as a water-tight tank like structure.

With a raft foundation to be adopted, a moisture barrier should be installed under the floor slab and above the raft foundation. Underfloor drainage should be installed in the clear stone to collect any leakage from the raft foundation.

4.3 ELEVATOR PITS AND SUMP PITS

Elevator pits and sump pits in the P2 basement may be installed in water-bearing cohesionless (sandy/silty) deposits. In this case, drainage systems at the base level of the pits are not recommended, due to the concern of loss of fines. The pits can be designed as water-tight structures, and water pressure on the pit walls and the slab should be considered, assuming the water table at about 0.3 m below the adjacent basement floor.

4.4 FROST PROTECTION

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

There is no official rule governing the required founding depth for footings below unheated basement floors. Certainly, it will not be greater than the 1.2 m required in Southern Ontario for exterior footings. Un-monitored experience indicates that a shallower depth ranging from 0.82 to 0.9 m for interior column footings and 0.4 m for wall footings has been successful where 2 or more basement levels apply. The 0.82 m depth is believed to be close to the minimum structural requirement for interior column footings. Adjacent to air shafts and entrance and exit doors, a footing depth of 1.2 m below floor level is required or, alternatively, insulation protection must be provided.

It is also emphasized that underfloor drainage 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.

4.5 EARTH AND WATER PRESSURES

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

In soils above the groundwater table ($z < d_w$):

$$p = K(\gamma z + q)$$

In soils below the groundwater table ($z \geq d_w$):

$$p = K\{\gamma d_w + \gamma_1 (z - d_w) + q\} + p_w$$

$$\text{In which, } p_w = \gamma_w (z - d_w)$$

where p = lateral earth and water pressure in kPa acting at a depth of z below ground surface

K = earth pressure coefficient
$K = 0.40$ for basement walls

γ = unit weight of soil above groundwater table, assuming $\gamma = 21$ kN/m^3

γ_1 = submerged unit weight of soil below groundwater table, assuming $\gamma_1 = 11$ kN/m^3

γ_w = unit weight of water, assuming $\gamma_w = 9.8$ kN/m^3

z = depth below ground surface to point of interest, in metres

d_w = depth of groundwater table below ground surface, in metres

q = value of surcharge in kPa

p_w = hydrostatic water pressure in kPa

When the basement wall is poured against the shoring caisson wall, the basement wall as well as the shoring caisson wall should be designed for hydrostatic pressure. For tanked basements, the basement walls and base slabs must be designed to resist hydrostatic pressure. The groundwater levels measured in the monitoring wells are provided in **Table 1**.

4.6 EARTHQUAKE CONSIDERATIONS

Based on the borehole information and according to Table 4.1.8.4.A of OBC 2012, the subject site for the proposed buildings with two levels of basement can be classified as Class ‘C’ for seismic site response.

4.7 EXCAVATION AND GROUNDWATER CONTROL

Excavations can be carried out with heavy hydraulic backhoe. The measured groundwater levels on July 6, 2022, in monitoring wells were at depths ranging from 1.3 to 10.4 m, corresponding to Elev. 180.1 to 188.9 m. Excavations for the proposed buildings with 2 levels of basement will extend below the groundwater table. Positive dewatering will be required prior to any excavations below the groundwater. The groundwater table must be lowered to at least 1.0 m below the deepest excavation base. A contractor specializing in dewatering should be retained to design the dewatering systems.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the fill materials, firm to stiff clayey soil, and cohesionless (sandy silt (till), sand, silty sand, and silt) deposits can be classified as Type 3 Soil above the groundwater table and as Type 4 Soil below the groundwater table. Very stiff to hard clayey silt and silty clay to clayey silt (till) deposits can be classified as Type 2 Soil above the groundwater table and as Type 3 Soil below the groundwater table.

It should be noted that the till is a non-sorted sediment and therefore may contain boulders. Provisions must be made in the excavation contract for the removal of possible boulders in the till or obstructions in the fill material.

The select inorganic native soils can be re-used as general construction backfill, provided its water content is within two percent of its optimum water content. Loose lifts of soil, which are to be compacted, should not exceed 200 mm.

Imported granular fill, which can be compacted with hand held equipment, should be used in confined areas. Underfloor fill should be compacted to at least 98 percent of Standard Proctor Maximum Dry Density (SPMDD).

The excavated soils are not considered to be free draining. Where free draining backfill is required, imported granular fill 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.

4.8 TEMPORARY SHORING

Given the site setting and adjacent land use, it is anticipated that 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. The requirement for caisson walls to support adjacent structures is given in **Drawings 9.**

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

1) Earth Pressure Coefficients for shoring:
    (a) where movement must be minimal, K=0.45

(b) where minor movement (0.2% shoring height) can be tolerated K=0.30

(c) passive earth pressure for soldier piles (unfactored) Kp= 3.3 for very stiff to hard or dense to very dense native soils

2) For stability check
ϕ= 32°
c= 0
$\gamma = 21$ kN/m^3
surcharge is to be determined by shoring contractor.

3) For earth anchors

Allowable bond value of 75 kPa is suggested for post grouted anchors in the very stiff to hard or dense to very dense soil deposits. However, these suggested bond values are preliminary since the contractor's installation methods and grouting procedures will determine the actual soil to concrete bond value. Hence, the contractor must decide on a capacity and confirm its availability by field load testing. All anchors must be tested as indicated in the Foundation Manual, 4th Edition.

The soldier piles should be installed in pre-augered holes taken below the deepest 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 sandy deposits from caving during the installation period. Positive measures may be required to prevent the loss of soil through the spaces between the lagging boards. This could probably be achieved by placing well-graded sand and gravel behind the lagging boards or by installing a geotextile filter cloth.

Soil anchors will be required to support the shoring. The anchors must be of a length that meets the Canadian Foundation Manual recommendations. It is important to note that the minimum length lies beyond the 45 - ϕ/2 + .15H line drawn from the base of the soldier pile and the overall stability of the system must be checked at each anchor level.

The top anchor must not be placed lower than 3.0 metres below the top of level ground surface. Anchors will require casing when penetrating through wet sand and silt layers.

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 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.

5. 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.

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 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.

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.

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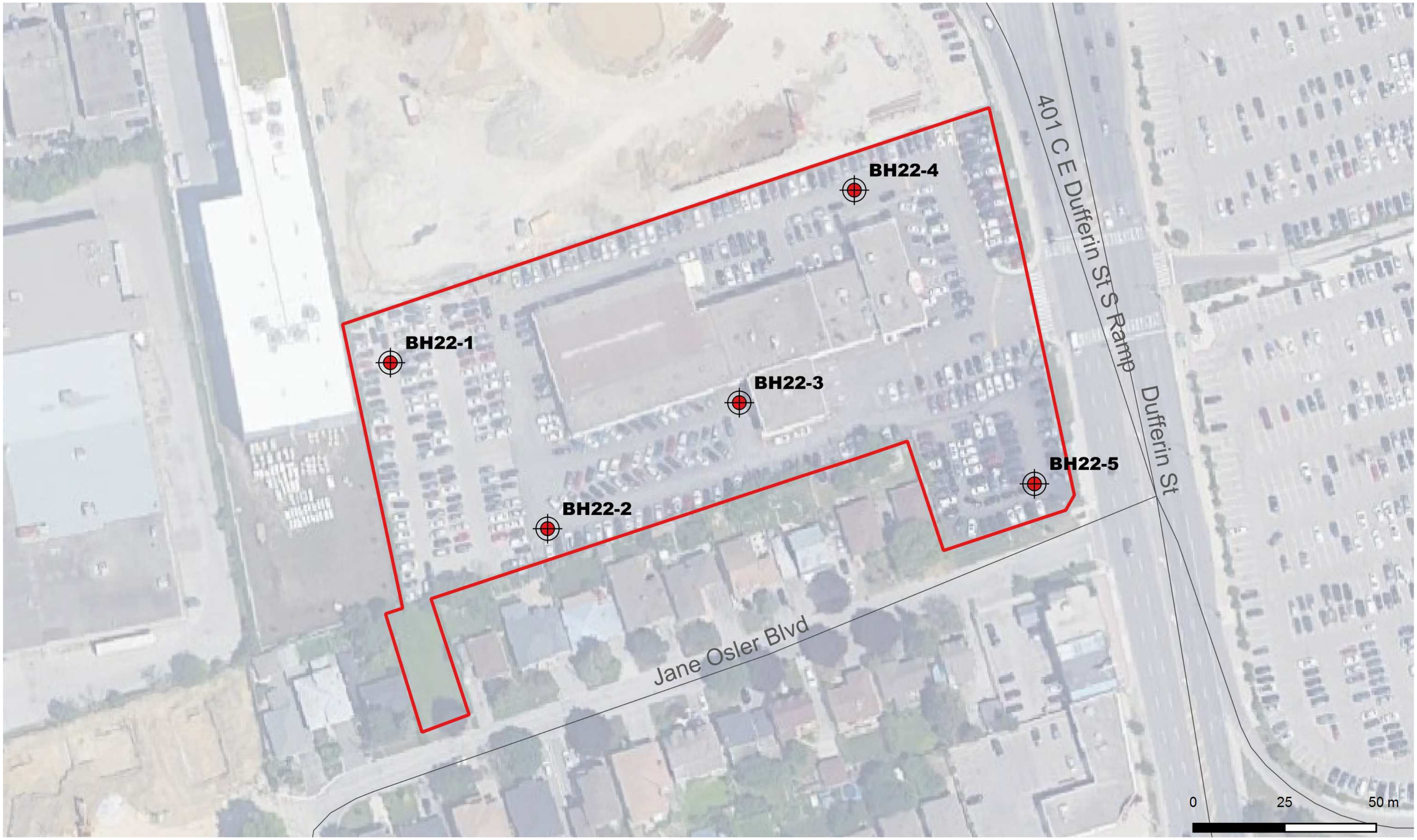
Drawings

C:\0Sharon\22-217-100 Hydro, 3400 Dufferin Street, Toronto\1-QGIS\Geo\Drawing 1 – Borehole and Monitoring Well Locations.qgs Jul-29 15:14



Legend

🟥 Approx Property Boundary

⊕ Monitoring Well

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www.dsconsultants.ca

Project: GEOTECHNICAL INVESTIGATION
3400 Dufferin St & 8 Jane Osler Blvd., Toronto, ON

N

Title: **BOREHOLE AND MONITORING WELL LOCATIONS**

Client:
DUFFERIN-401 PROPERTIES LIMITED

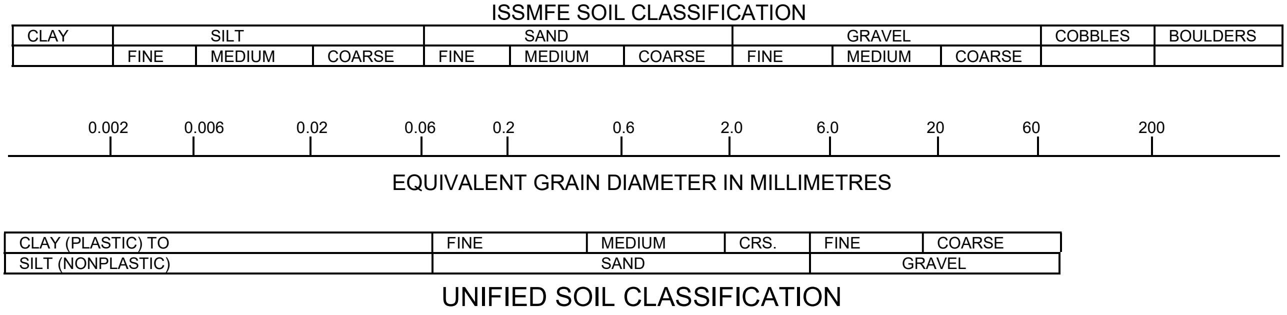
Size: 8.5 x 11
Rev: 0

Approved By: L.M
Drawn By: S.Y
Date: July 2022
Scale: As Shown
Project No.: 22-217-100
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 BH22-1

PROJECT: Geotechnical Investigation
CLIENT: Dufferin-401 Properties Limited
PROJECT LOCATION: 3400 Dufferin St. & 8 Jane Osler Boulevard,Toronto, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4842495.09 E 624063.95

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200mm
Date: Jun-15-2022

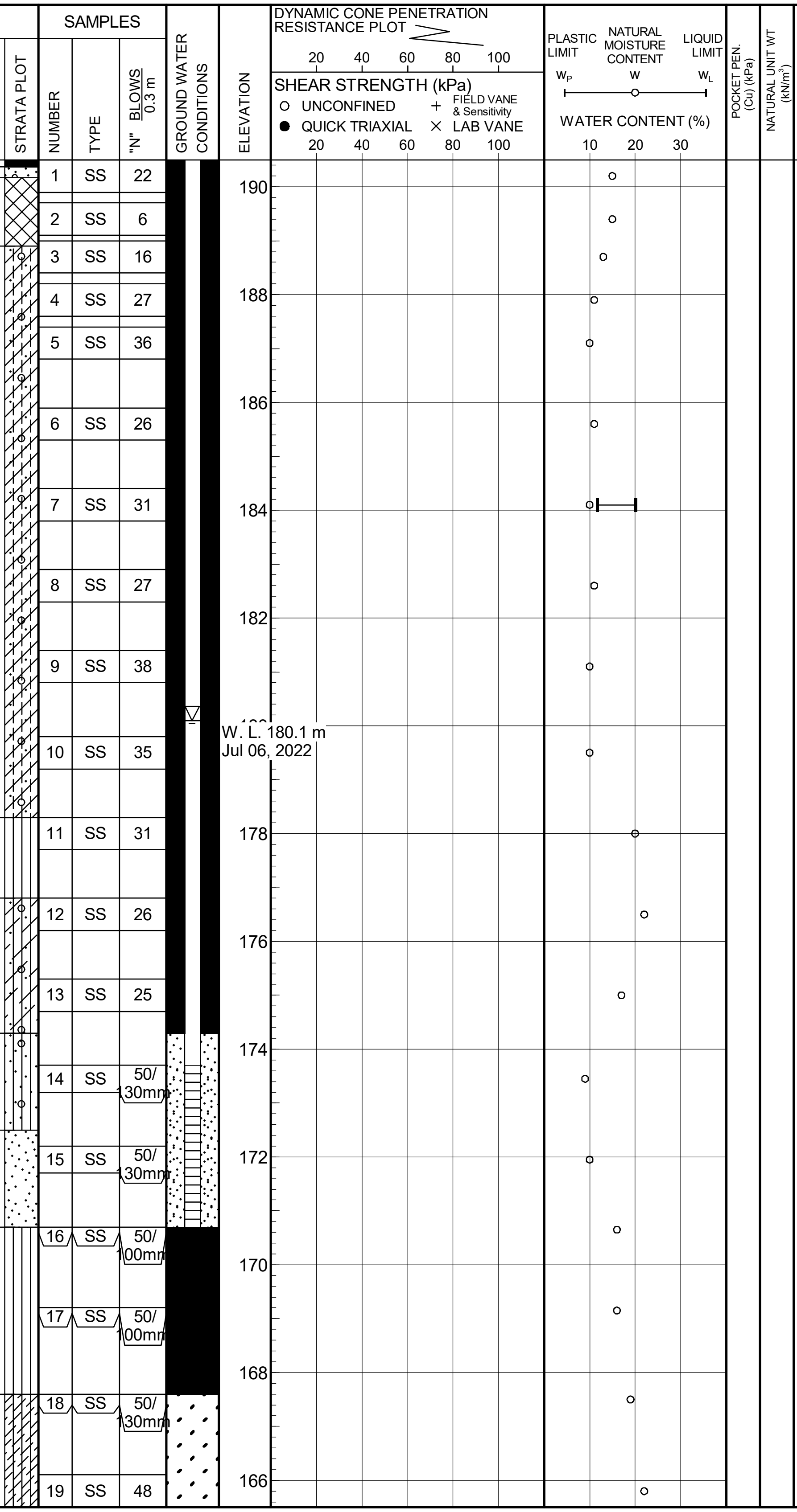
REF. NO.: 22-217-100
ENCL NO.: 2

SOIL PROFILE

(m) ELEV / DEPTH	DESCRIPTION
190.5	
190.4 0.1	**ASPHALT:** 130mm
190.2 0.3	**GRANULAR BASE:** sand and gravel mixed with crusher limestone, 300mm
188.9 1.6	**FILL:** silty clay, some organics, trace gravel, grey to brown, moist, firm
	SILTY CLAY TILL: sandy, trace gravel, brown, moist, very stiff to hard grey below 7.6m
178.3 12.2	**SILT:** trace clay, trace to some sand, trace gravel, grey, wet, dense
176.8 13.7	**CLAYEY SILT TILL:** sandy, trace gravel, grey, moist, very stiff
174.3 16.2	**SANDY SILT TILL:** trace to some clay, trace gravel, grey, moist, very dense
172.5 18.0	**SAND:** some silt, trace clay, trace gravel, grey, moist, very dense
170.7 19.8	**SILT:** some clay, trace sand, trace gravel, grey, moist, very dense
167.6 22.9	**CLAYEY SILT:** trace to some sand, silt seams, trace gravel, grey, moist, hard

SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	22	
2	SS	6	
3	SS	16	
4	SS	27	
5	SS	36	Switched to Mud Rotary at 3.1m
6	SS	26	
7	SS	31	3 30 50 17
8	SS	27	
9	SS	38	
10	SS	35	
11	SS	31	
12	SS	26	4 37 42 17
13	SS	25	
14	SS	50/ 130mm	
15	SS	50/ 130mm	4 82 10 4
16	SS	50/ 100mm	
17	SS	50/ 100mm	
18	SS	50/ 130mm	
19	SS	48	



Continued Next Page

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

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

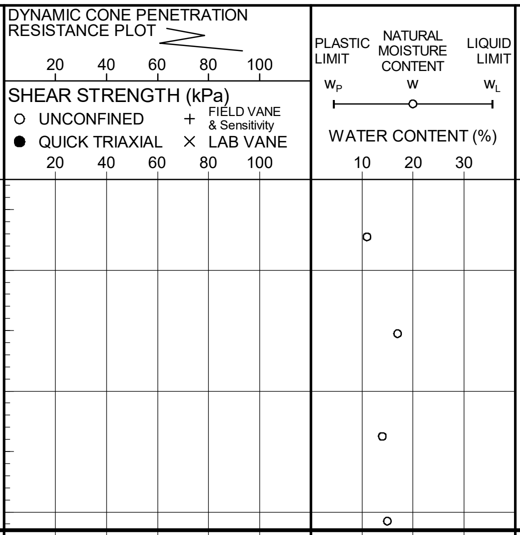
LOG OF BOREHOLE BH22-1

PROJECT: Geotechnical Investigation
CLIENT: Dufferin-401 Properties Limited
PROJECT LOCATION: 3400 Dufferin St. & 8 Jane Osler Boulevard,Toronto, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4842495.09 E 624063.95

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200mm
Date: Jun-15-2022

REF. NO.: 22-217-100
ENCL NO.: 2

(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	ELEVATION
164.6					
25.9	**SAND:** trace clay, trace silt, grey, wet, very dense	20	SS	50/ 130mm	164
163.1 27.4	**SILTY SAND:** some clay, trace gravel, grey, wet, very dense	21	SS	50/ 130mm	162
161.5 29.0	**SAND:** trace to some silt, trace clay, trace gravel, grey, wet, very dense	22	SS	50/ 130mm	
159.7		23	SS	50/ 130mm	160
30.8	**END OF BOREHOLE:** Notes: 1) 50mm dia. monitoring well installed upon completion. 2) Water Level Readings: Date: Water Level(mbgl): July 6, 2022 10.41				



POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (kN/m^3)
REMARKS AND GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

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

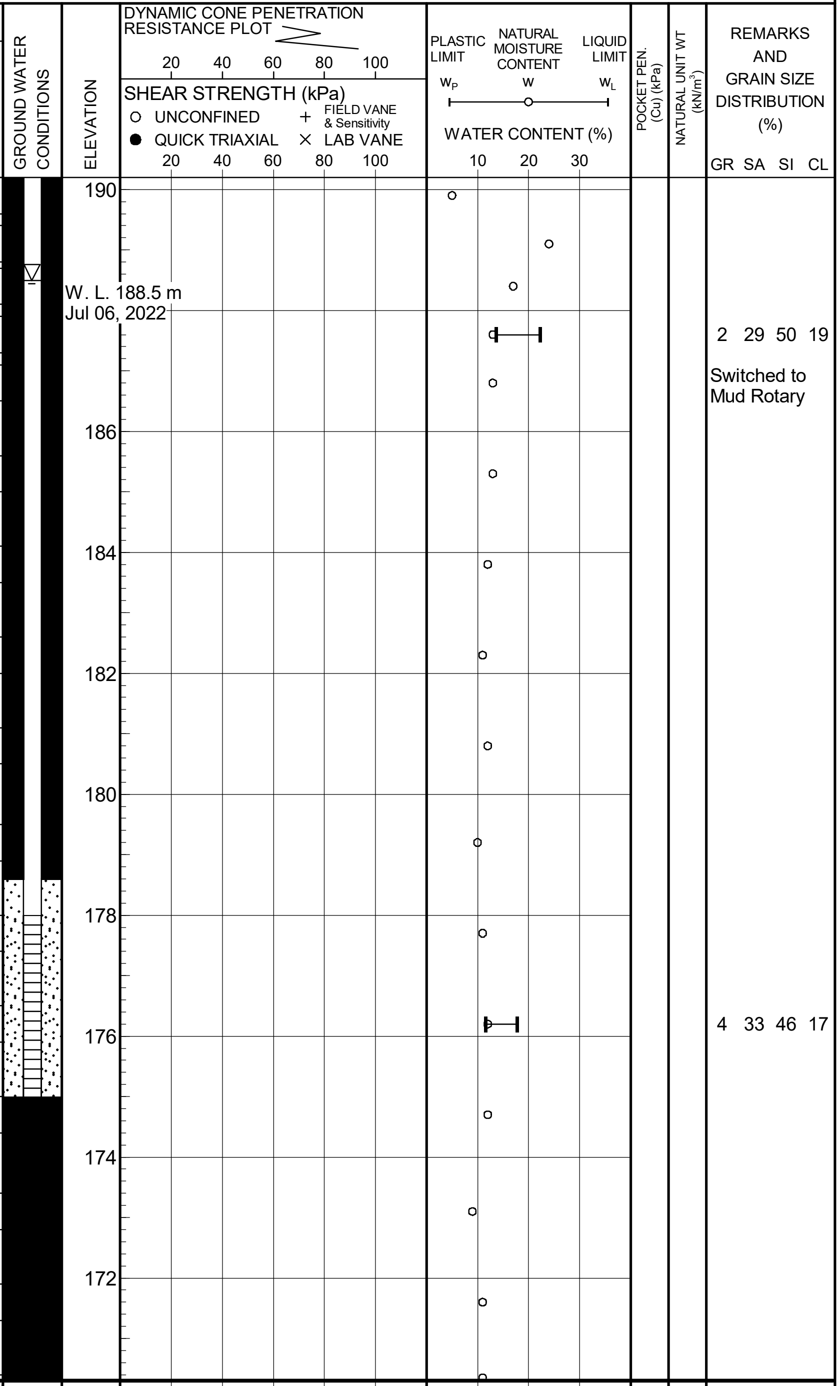
GRAPH NOTES
$+^3, \times^3$: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

LOG OF BOREHOLE BH22-2

PROJECT: Geotechnical Investigation
CLIENT: Dufferin-401 Properties Limited
PROJECT LOCATION: 3400 Dufferin St. & 8 Jane Osler Boulevard,Toronto, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4842449.73 E 624106.89

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200mm
Date: Jun-14-2022

REF. NO.: 22-217-100
ENCL NO.: 3



ELEV (m) / DEPTH	DESCRIPTION
190.2	
190.1 0.1	**ASPHALT:** 75mm
189.9 0.4	**GRANULAR BASE:** sand and gravel, crusher limestone, 280mm
	FILL: silty clay, trace organics, trace gravel, grey to brown, moist, stiff
188.4 1.8	**SILTY CLAY TILL:** sandy, trace gravel, brown, moist, stiff to hard grey below 6.1m
176.5 13.7	**CLAYEY SILT TILL:** sandy, trace gravel, grey, moist, very stiff
173.4 16.8	**SANDY SILT TILL:** trace to some clay, trace gravel, occasional cobble, grey, moist, very dense dense @18.3m
170.3 19.9	**END OF BOREHOLE:** Notes: 1) 50mm dia. monitoring well installed upon completion. 2) Water Level Readings: Date: Water Level(mbgl): July 6, 2022 1.7

NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	12	
2	SS	9	
3	SS	9	
4	SS	19	2 29 50 19
5	SS	21	Switched to Mud Rotary
6	SS	38	
7	SS	23	
8	SS	22	
9	SS	26	
10	SS	34	
11	SS	26	
12	SS	24	4 33 46 17
13	SS	24	
14	SS	71	
15	SS	35	
16	SS	50/ 75mm	

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

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

LOG OF BOREHOLE BH22-3

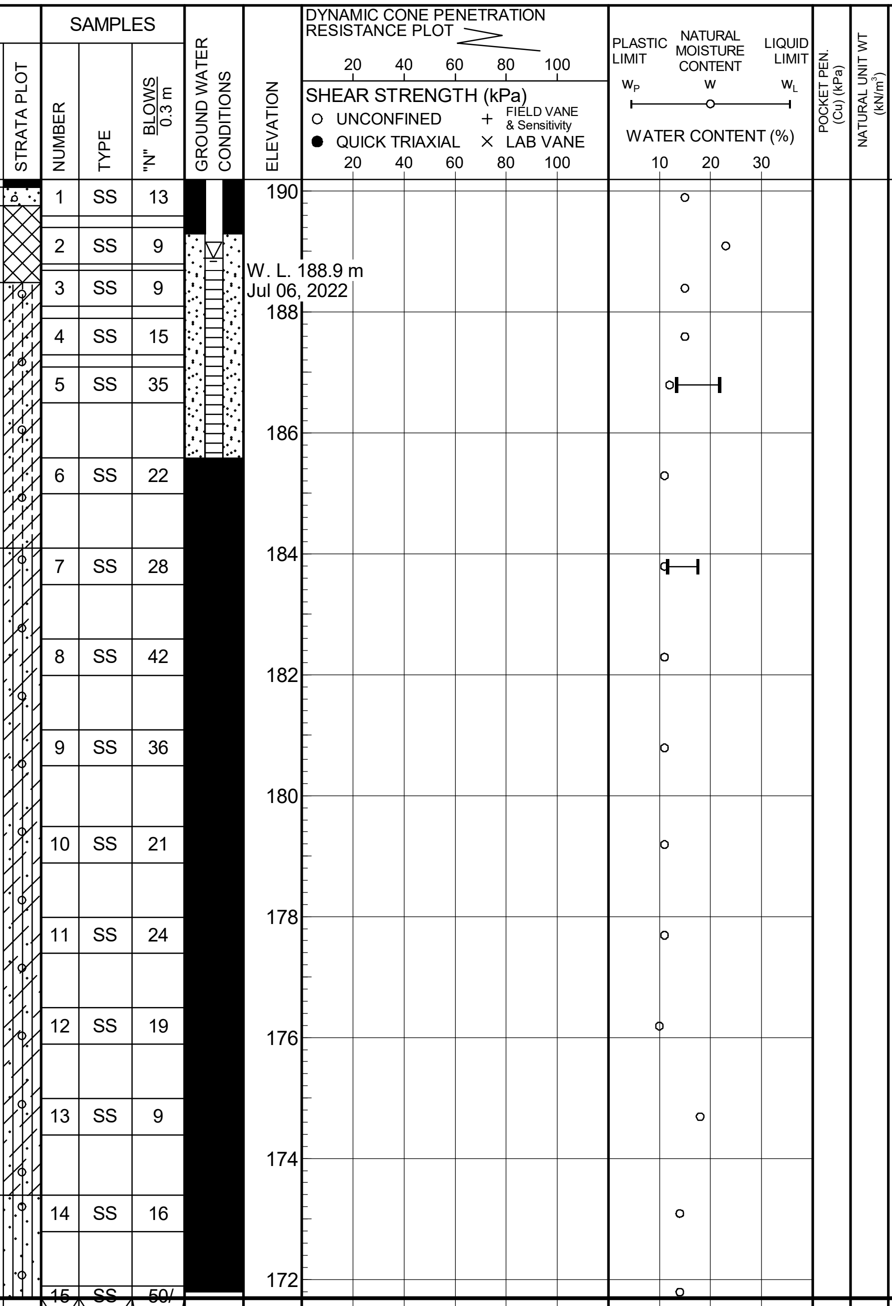
PROJECT: Geotechnical Investigation	**DRILLING DATA**
CLIENT: Dufferin-401 Properties Limited	Method: Hollow Stem Auger
PROJECT LOCATION: 3400 Dufferin St. & 8 Jane Osler Boulevard,Toronto, ON	Diameter: 200mm
DATUM: Geodetic	Date: Jun-22-2022
BH LOCATION: See Drawing 1 N 4842484.16 E 624159.33	

REF. NO.: 22-217-100
ENCL NO.: 4

SOIL PROFILE		SAMPLES			REMARKS AND GRAIN SIZE DISTRIBUTION (%)			
(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS / 0.3 m	GR	SA	SI	CL
190.2								
190.0 0.0 190.1 0.1	**ASPHALT:** 130mm	1	SS	13				
189.8 0.4	**GRANULAR BASE:** sand and gravel, 300mm							
	FILL: silty clay, trace organics, trace gravel, grey to brown, moist, stiff	2	SS	9				
188.5 1.7	**SILTY CLAY TILL:** sandy, trace gravel, occasional cobble, brown, moist, stiff to hard	3	SS	9				
		4	SS	15				
		5	SS	35	1	31	49	19
	grey below 4.6m	6	SS	22				
184.1 6.1	**CLAYEY SILT TILL:** sandy, trace gravel, grey, moist, stiff to hard	7	SS	28	7	30	45	18
		8	SS	42				
		9	SS	36				
		10	SS	21				
		11	SS	24				
		12	SS	19				
	stiff at 15.2m	13	SS	9				
173.4 16.8	**SANDY SILT TILL:** trace to some clay, trace gravel, grey, wet, compact to very dense	14	SS	16	3	37	49	11
171.7 18.5	**END OF BOREHOLE:**	15	SS	50/ 75mm				

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

Date: Water Level(mbgl):
July 6, 2022 1.3



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

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

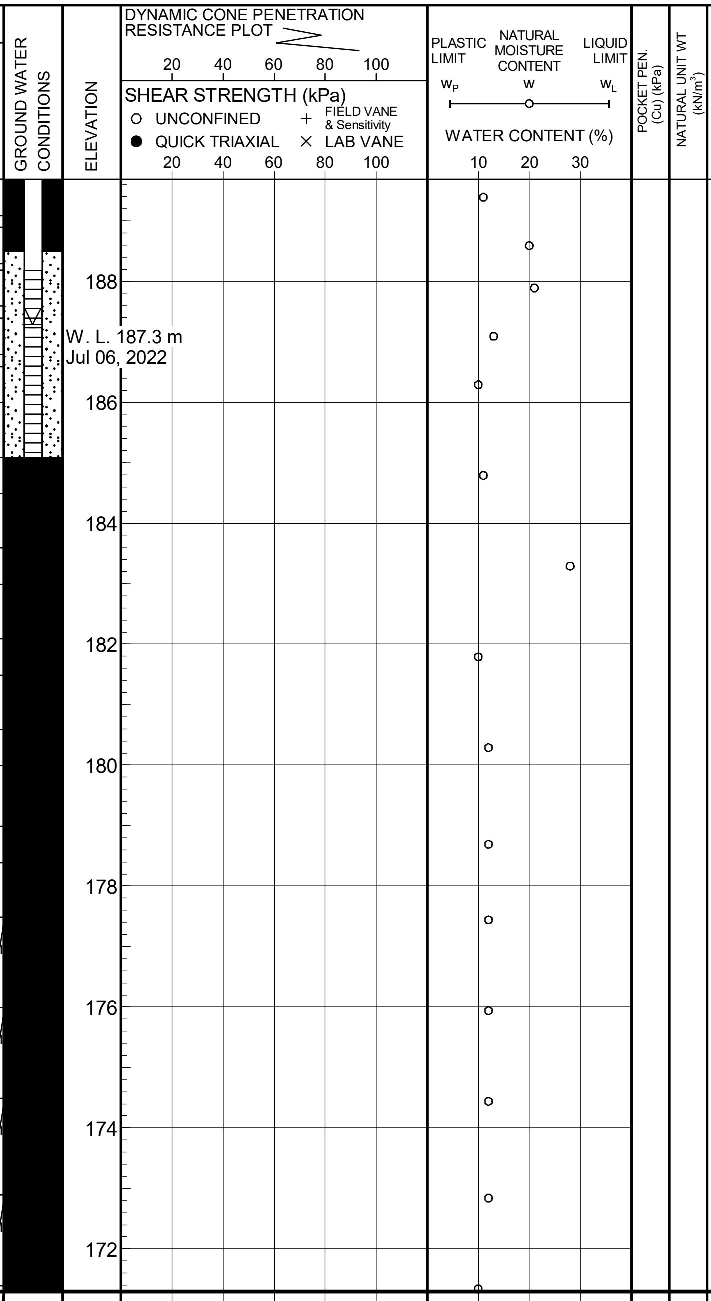
LOG OF BOREHOLE BH22-4

PROJECT: Geotechnical Investigation
CLIENT: Dufferin-401 Properties Limited
PROJECT LOCATION: 3400 Dufferin St. & 8 Jane Osler Boulevard,Toronto, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4842542.23 E 624190.76

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200mm
Date: Jun-15-2022

REF. NO.: 22-217-100
ENCL NO.: 5

(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m	GR	SA	SI	CL
189.7 0.0	**ASPHALT:** 130mm	1	SS	7				
189.6 0.1	**GRANULAR BASE:** sand and gravel, 230mm							
189.3 0.4	**FILL:** silty clay, trace organics, trace gravel, brown, moist, firm	2	SS	5				
		3	SS	5				
187.4 2.3	**SILTY CLAY TILL:** sandy, trace gravel, occasional cobble, brown, moist, very stiff to hard	4	SS	26				
		5	SS	31				
	grey below 4.6m	6	SS	31				
		7	SS	64				
		8	SS	33				
		9	SS	33				
		10	SS	24				
177.5 12.2	**SANDY SILT TILL:** trace to some clay, trace gravel, grey, very moist to wet, very dense	11	SS	50/ 50mm				
		12	SS	50/ 50mm				
		13	SS	50/ 75mm	1	24	66	9
		14	SS	50/ 75mm				
171.7 18.0	**SILTY CLAY TILL:** sandy, trace gravel, occasional cobble, grey, moist, hard	15	SS	50/ 130mm				
171.3 18.4	**END OF BOREHOLE:**							



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

Date: Water Level(mbgl):
July 6, 2022 2.4

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

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

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Geotechnical ◆ Environmental ◆ Materials ◆ Hydrogeology

LOG OF BOREHOLE BH22-5

1 OF 1

PROJECT: Geotechnical Investigation
CLIENT: Dufferin-401 Properties Limited
PROJECT LOCATION: 3400 Dufferin St. & 8 Jane Osler Boulevard,Toronto, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1 N 4842461.99 E 624239.96

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 200mm
Date: Jun-17-2022

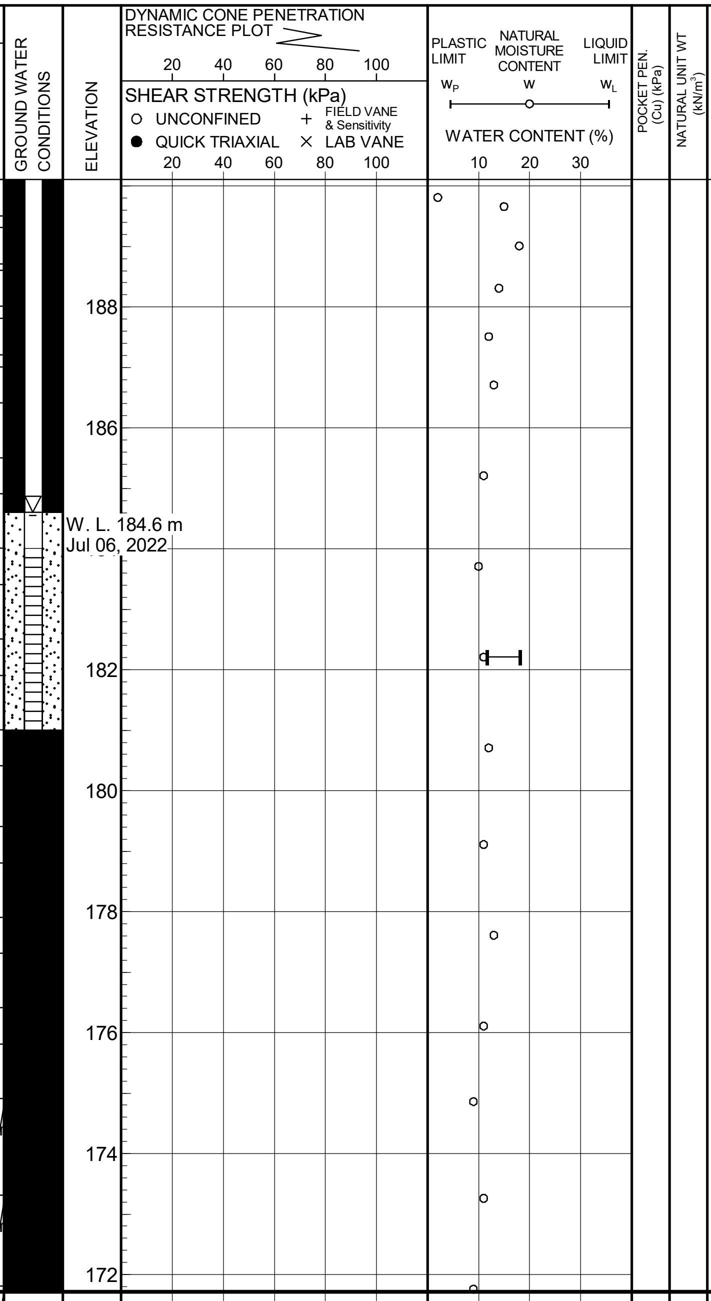
REF. NO.: 22-217-100
ENCL NO.: 6

SOIL PROFILE

(m) ELEV DEPTH	DESCRIPTION
190.1	
190.0 0.1	**ASPHALT:** 75mm
189.8 0.3	**GRANULAR BASE:** sand and gravel, 250mm
	FILL: clayey silt to silty clay, trace organics, trace gravel, grey to brown, moist, firm
187.8 2.3	**SILTY CLAY TILL:** sandy, trace gravel, occasional cobble, brown, moist, very stiff to hard grey below 6.1m
182.5 7.6	**CLAYEY SILT TILL:** sandy, trace gavel, grey, moist, hard very stiff, frequent wet sand seams at 12.2m
174.9 15.2	**SANDY SILT TILL:** trace to some clay, trace gravel, occasional cobble, grey, moist, very dense
171.7 18.4	**END OF BOREHOLE:** Notes: 1) 50mm dia. monitoring well installed upon completion. 2) Water Level Readings: Date: Water Level(mbgl): July 6, 2022 5.5

SAMPLES

NUMBER	TYPE	"N" BLOWS/0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	10	
2	SS	7	
3	SS	7	
4	SS	16	
5	SS	18	
6	SS	48	
7	SS	33	
8	SS	36	2 33 47 18
9	SS	32	
10	SS	37	
11	SS	26	
12	SS	35	
13	SS	50/100mm	
14	SS	50/130mm	
15	SS	50/130mm	

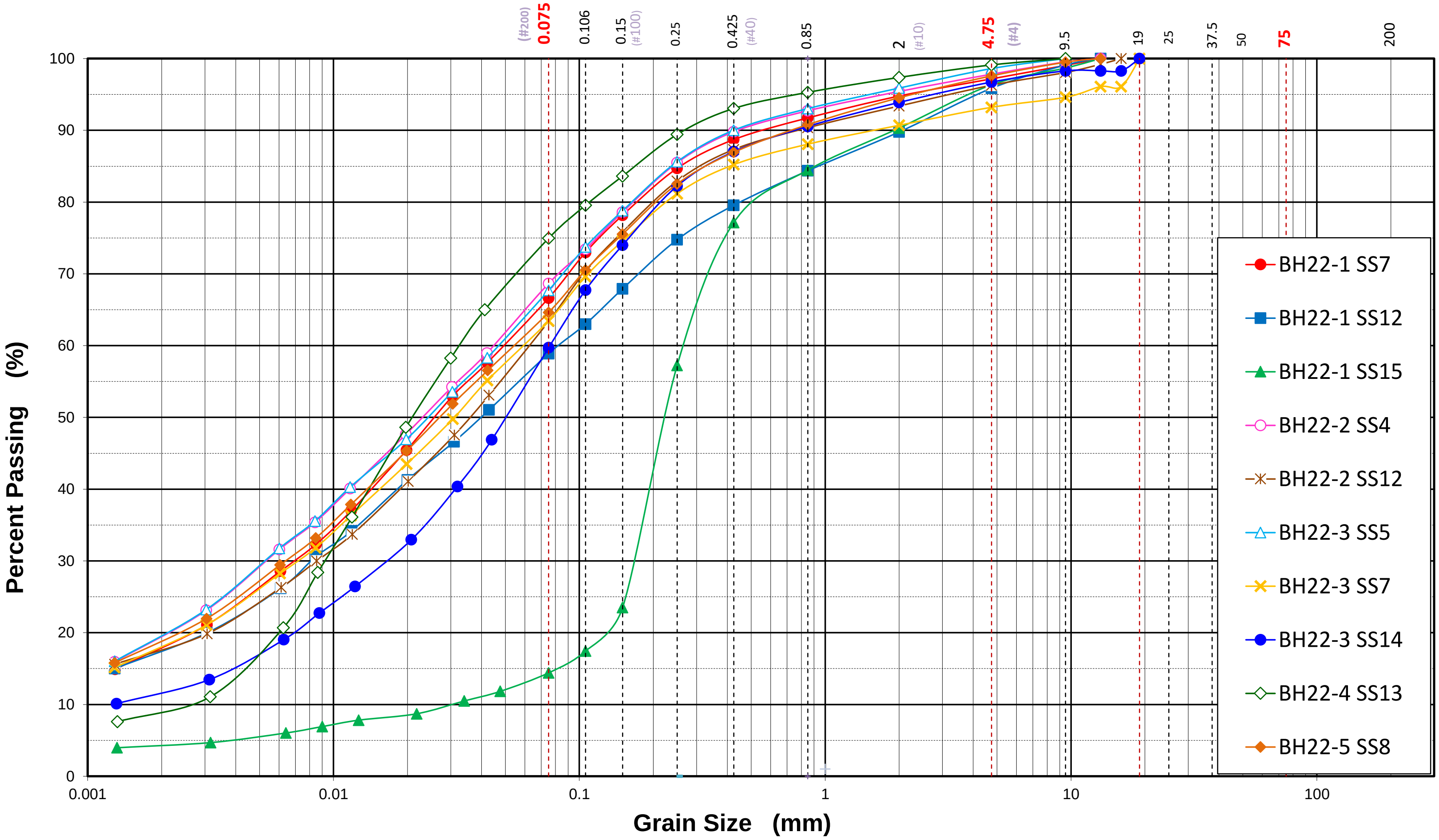


GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

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

DS SOIL LOG-2021-FINAL 22-217-100 GEO COPY.GPJ DS.GDT 22-8-2

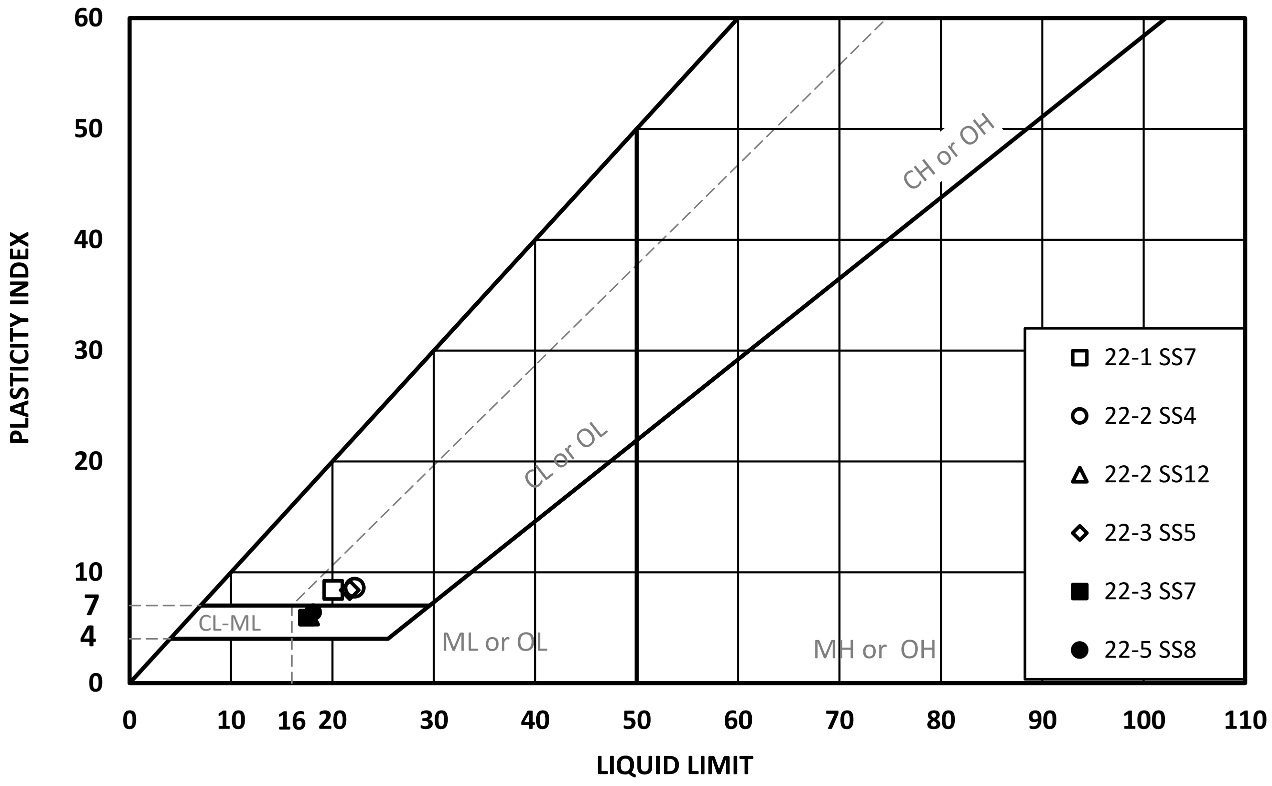
Particle Size Distribution (ASTM-D421/D422)



Silt and Clay		**Sand**			**Gravel**		**Cobble +**
Clay	**Silt**	**Fine**	**Medium**	**Coarse**	**Fine**	**Coarse**	

DS CONSULTANTS LTD. 6221 Highway 7, Unit 16 Vaughan, Ontario, L4H 0K8 Telephone: (905) 264-9393 www.dsconsultants.ca	Project	**Geotechnical Investigation**	Project No	**22-217-100**
	Location	**3400 Dufferin St. & 8 Jane Osler Blvd.,Toronto, ON**	Date	**Jun-28-2022**
	Client	**Dufferin-401 Properties Limited**	Figure No	7

Atterberg Test (ASTM D-4318)



Code	Sample ID	Sample No.		Moisture Contant (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	**USCS Symbol**
1	□	22-1	SS7	10	20.1	11.7	8.4	CL
2	○	22-2	SS4	13	22.2	13.6	8.6	CL
3	△	22-2	SS12	12	17.7	11.6	6.1	CL-ML
4	◇	22-3	SS5	12	21.7	13.3	8.4	CL
5	■	22-3	SS7	11	17.5	11.6	5.9	CL-ML
6	●	22-5	SS8	11	18.1	11.7	6.4	CL-ML

DS CONSULTANTS LTD.
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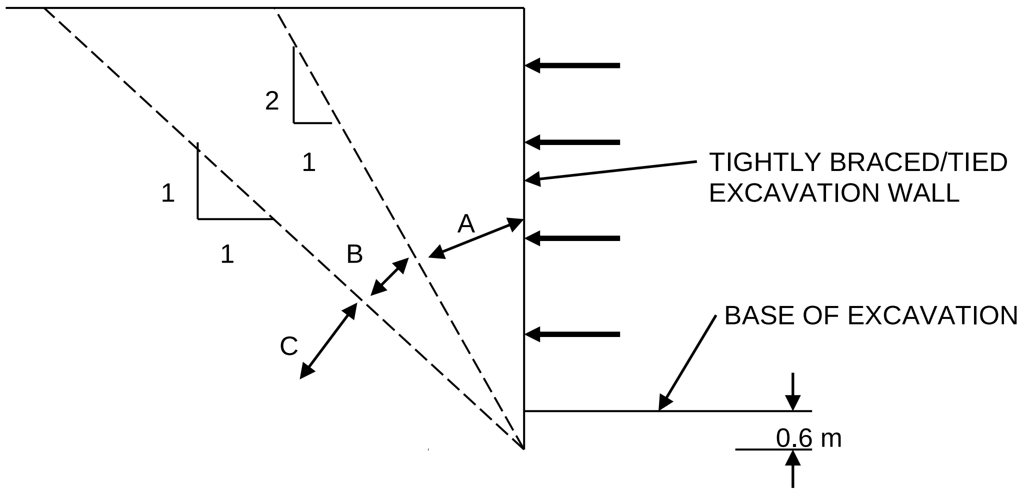
Project	**Geotechnical Investigation**	Project No	**22-217-100**
Location	**3400 Dufferin St./8 Jane Osler Blvd., Toronto, ON**	Date	**Jun-28-2022**
Client	**Dufferin-401 Properties Limited**	Figure No	8

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)