



**REPORT ON
GEOTECHNICAL INVESTIGATION
WEST OF CAMBRIAN ROAD AND GREENBANK ROAD
BARRHAVEN, ONTARIO**

**REPORT NO.: 4817-18-G-LPL-A
REPORT DATE: NOVEMBER 13, 2018**

**PREPARED FOR
LOBLAW REAL ESTATE
1 PRESIDENT'S CHOICE CIRCLE
MISSISSAUGA, ONTARIO
L6Y 5S5**

110 KONRAD CRESCENT, UNIT 16, MARKHAM, ONTARIO L3R 9X2
TEL.: 905-940-8509 FAX: 905-940-8192



Table of Contents

1.0 INTRODUCTION.....	1
2.0 SITE CONDITION.....	1
3.0 INVESTIGATION PROCEDURE.....	2
4.0 SUMMARISED SUBSURFACE CONDITIONS.....	3
4.1 Surface Course.....	3
4.2 Fill.....	3
4.3 Silty Clay / Clayey Silt.....	4
4.4 Sand and Gravel.....	5
4.5 Bedrock.....	6
4.6 Groundwater.....	6
5.0 RECOMMENDATIONS.....	7
5.1 Site Preparation.....	7
5.2 Foundation Design.....	8
5.3 Floor Slab Construction.....	10
5.4 Earthquake Consideration.....	11
5.5 Excavation and Backfilling.....	11
5.6 Pavement Construction.....	12
5.7 Other Engineering Concerns	13
6.0 GENERAL STATEMENT OF LIMITATION.....	14

DRAWINGS

Borehole Location Plan

Drawing No. 1

Logs of Boreholes (BH-1 to BH-12)

Drawing Nos. 2 to 13

FIGURES

Gradation Curves

Figure No.: 1

Atterberg Limits

Figure No.: 2

Consolidation Tests

Figure Nos. 3 & 4

Detail For Subdrain At CB's & DBMH's

Figure No.: 5

Concrete Ramp/Sidewalk Sections

Figure No.: 6

Appendix A

Guidelines of Engineered Fill

1.0 INTRODUCTION

Toronto Inspection Ltd. (TIL) was retained by Loblaw Real Estate to conduct a Geotechnical Investigation at two parcels of land, located at the northwest and southwest corners of Cambrian Road and Greenbank Road in Barrhaven, Ontario (hereinafter described as “the Site”) for a proposed commercial development.

Based on the Preliminary Concept Site Plans, Drawing #: 1314C for the north parcel and Drawing #: 1313D for the south parcel, prepared by Turner Fleischer Architects Inc., dated April 19, 2018, the development at both parcels will consist of Retails A & B with the associated driveway and parking lots.

The purpose of the geotechnical investigation was to determine the subsoil and groundwater conditions within the two parcels and, based on the information obtained at the current boreholes, provide our recommendations of the design and construction of the structures. In particular, geotechnical data was to be provided for:

- General founding conditions
- Foundation design bearing pressures
- Construction recommendations
- Excavation recommendations

This report is provided on the basis of the above terms of reference and on an assumption that the design of structures will be in accordance with the applicable building codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, our office should be consulted to review the design and to confirm the recommendations and comments provided in the report.

2.0 SITE CONDITION

The two open parcels of land of the subject Site are located on the north side and south side of Cambrian Road, along the west side of the proposed Greenbank Road in Barrhaven, Ontario. The north parcel is approximately 1.36 ha in area and approximately square in shape, and the south parcel is approximately 1.51 ha in area and rectangle in shape.

At the time of the investigation, both parcels of land of the Site were vacant with vegetation and bushes at the ground surface at the east portion, and gravel covered surface along the east boundary and at the west portion, indicating some earth work might have been carried out within the Site. Some water ponding at the low lying area was observed at the time of the investigation.

Within the north parcel, a drainage ditch ran across the land in a north-south direction, along the mid-east portion. Piles of mixed soil, rocks, debris and concrete rubble were observed along the south part of the east boundary, with some debris piled at the mid-west portion. With the south parcel, a drainage ditch ran across the land in a north-south direction, along the middle portion. Remnants of construction debris consisting of asphalt, insulation and concrete were observed along the north part of the east boundary.

The site gradient were slightly undulating, generally slightly dropping towards the south and west at the north parcel, and towards the north and west at the south parcel.

3.0 INVESTIGATION PROCEDURE

The field work for the investigation was carried out, at different stages, on April 27, 30, May 1, 2, August 16, 17, September 20, 24 and 25, 2018. A total of twelve sampled boreholes, BH-1 to BH-12, were carried out at the Site. Boreholes BH-1 to BH-6 were drilled within the north parcel and Boreholes BH-7 to BH-12 drilled within the south parcel, as shown on the Borehole Location Plan, Drawing No. 1.

Due to the very wet and unfavourable ground condition at the proposed borehole locations, only Boreholes BH-2 to BH-8 could be drilled during the period of April to May 2, 2018 and the remaining Boreholes BH-1 and BH-9 to BH-12 were drilled in August and September, 2018. The sampled boreholes generally extended to depths of 7.6m to 10.7m below the existing ground level at Boreholes BH-2 to BH-10 and BH-12. In addition to obtaining samples within the very soft clayey silt / silty clay deposits at the shallow boreholes, dynamic cone penetration tests were carried out below the sampling depths at these boreholes to estimate the extent of the very soft deposits. The dynamic cone penetration testing extended to depths of 19.0m to 24.7m from grade. The sampled boreholes BH-1 and BH-11 extended to depths of 24.5m to 24.8m from grade. Field shear vane tests were carried out in the cohesive soil strata to estimate the undrained shear strength of the soil. Relatively undisturbed soil samples were also collected from the very soft silty clay / clayey silt deposits using thin walled Shelby tubes for later laboratory testing. Bedrock samples were cored at BH-1 location for a depth of approximately 3.1m (10ft) to confirm the rock quality. The core sampling at BH-1 was terminated at a depth of 27.6m from grade.

The boreholes were advanced using a track mounted grill rig, equipped with continuous flight hollow stem augers, sampling rods and a drop hammer, supplied by a specialist drilling contractor; Boreholes BH-1 and BH-11 were advanced using mud drilling. Soil samples were

taken generally at 0.76m intervals to depths of 3.0m below the existing ground level. Below the depths, the sampling frequency was increased to 1.5m. The samples were obtained using a split spoon sampler in conjunction with Standard Penetration Tests using a driving energy of 475 joules (350 ft-lbs). The soil samples were identified and logged in the field and were carefully bagged for later visual identification and the determination of moisture content.

Groundwater observations were made in the boreholes during and upon the completion of drilling. Boreholes BH-1, BH-7, BH-8 and BH-11 were completed as monitoring wells for determination of groundwater conditions.

The borehole locations, established in the field by our field personnel, are shown on the appended Borehole Location Plan (Drawing No.1). The ground elevations at the borehole locations were obtained from Plan of Survey with Topography of Part of Lots 10 and 11, Concession 3 (Rideau Front), City of Ottawa, prepared by Speight, Van Nostrand & Gibson Limited, dated October 4, 2018, provided to **TIL** office by the client.

4.0 SUMMARISED SUBSURFACE CONDITIONS

Reference is made to the appended Borehole Location Plan (Drawing No. 1) and Logs of Borehole sheets (Drawing Nos. 2 to 13) for details of field work, including soil classification, inferred stratigraphy, groundwater observations during and on completion of borehole drilling.

The boreholes revealed that the subsoil, below the surficial topsoil or gravel, the overburden over the bedrock consisted of fill material, overlying silty clay / clayey silt, and sand and gravel deposits. Brief descriptions of the subsoil, at the borehole locations, were as follows:

4.1 Surface Course

Topsoil, approximately 25mm to 150mm in thickness, was contacted at the ground surface, at BH-1, BH-3, and BH-8 to BH-11 locations. Gravel, approximately 25mm to 75mm in thickness, was contacted at the remaining locations.

4.2 Fill

Underlying the topsoil and gravel at the borehole locations, a layer of fill was contacted at depths of 0.025m to 0.15m from grade. The fill consisted of a mixture of silty sand, sandy silt, clayey silt, trace to some gravel, occasional shale pieces, with minor or occasional minor rootlets, topsoil, organics or wood pieces. The fill extended to depths of 1.5m to 3.7m from grade.

Based on the soil quality and the Standard Penetration N-values, in the range of 2 to 36 blows per 0.3m penetration, it appears that the fill might have been placed and compacted under some supervision at BH-2, BH-4 BH-7 and BH-8 locations, where are along the east boundary and could be part of site preparation of the proposed Greenbank Road. The fill at the remaining borehole locations was generally in very loose to loose or very soft to soft states.

The in-situ moisture content of the soil samples retrieved from the fill ranged from 8% to 33%, indicating moist to wet conditions, normally wet at the lower portion of the fill.

4.3 Silty Clay / Clayey Silt

Underlying the fill at all borehole locations, at depths of 1.5m to 3.7m from grade, silty clay / clayey silt deposits were contacted. The silty clay / clayey silt deposits, of low to medium plasticity, consisted of a mixture of silt and clay, trace to some sand, with seams of thin layers of sand or silty sand, and contained occasional gravel, cobble or boulder at the deeper portion of the deposits at BH-1 and BH-11 locations. Dark grey organic spots were evident within the deposits.

Boreholes sampling at BH-2 to BH-10 and BH-12 were terminated in the silty clay / clayey silt deposits at depths of 7.6m to 10.7m from grade. The silty clay / clayey silt deposits at Boreholes BH-1 and BH-11 extended to depths of 21.0m from grade.

Based on the Standard Penetration N-values, in the range of 1 to 8 blows per 0.3m penetration, the consistency of the silty clay / clayey silt deposit was very soft to stiff, generally very soft.

The in-situ moisture content of the soil samples retrieved from the deposits ranged from 8% to 54%, indicating moist to wet conditions, generally wet conditions. The in-situ moisture content less than 10% at the lower portion of the deposits at BH-1 location was due to the presence of silty sand and gravel layers.

Dynamic cone penetration tests were carried out at Boreholes BH-2 to BH-10 and BH-12 locations, below depths of 7.6m to 10.7m from grade and extended to depths of 19.0m to 24.7m from grade. Based on the dynamic cone test results, the very soft to soft silty clay deposit extended to depths of 15.0m to 19.5m from grade, where there was a gradual increase in the blow counts.

The in-situ undrained shear strength of the silty clay / clayey silt deposits, at depths of approximately 3.0m to 10.1m from grade, was estimated using a field shear vane. The shear vane test results were in the range of 13 kPa to 25 kPa with sensitivity in the range of 1.5 to 14.0. The shear vane test results are plotted on the appended borehole logs.

Grain size analyses were carried out on selected soil samples, obtained from Borehole BH-2 (SS6, at a depth of 4.6m), and BH-7 (SS6, at a depth of 4.6m), using both mechanical sieves and hydrometer. The grain size distributions are shown on the appended Figure No. 1.

Atterberg Limits test results, carried out on the representative samples, obtained from BH-2 (SS6, at a depth of 4.6m), and BH-7 (SS6, at a depth of 4.6m), indicated that the deposits can be classified as lean clay of low to medium plasticity. The test results are shown on the appended Figure No. 2.

The Atterberg Limits Tests indicated that for the samples tested, the in-situ moisture contents are higher than the liquid limits.

Relatively undisturbed soil samples were obtained from Boreholes BH-2 & BH-11, at depths of 6.1m to 6.7m & 3.8m to 4.4m from grade, respectively, using thin walled Shelby tubes for the determination of bulk unit weight and consolidation test in our laboratory. The unit weight of the clay deposit was estimated to be 17.5 kN/m³. The consolidation test results are shown on Figure Nos. 3 & 4. The results indicated that the soil samples are normally consolidated comparing to the existing effective overburden stresses at the test levels.

Occasional boulders were encountered at BH-1 location between depths of 17.4m and 20.4m from grade. Obtaining core samples between these depths indicated boulders ranged in thicknesses from 400mm and 300mm.

4.4 Sand and Gravel

Underlying the silty clay / clayey silt deposits at BH-1 and BH-11 locations, at depths of 21.0m from grade, a sand and gravel deposit was contacted. The sand and gravel deposit contained occasional cobbles or boulders, thin layers of silt or fine to medium sand, and rock pieces at the bottom of the deposit.

Borehole BH-11 was terminated in the sand and gravel deposit at a depth of 24.8m from grade. The sand and gravel deposit at Borehole BH-1 extended to a depth of 24.5m from grade.

Based on the Standard Penetration N-values, in the range of 40 to more than 100 blows per 0.3m penetration, the relative density of the sand and gravel deposit was dense to very dense.

The in-situ moisture content of the soil samples retrieved from this deposit ranged from 8% to 20%, indicating wet conditions.

A boulder was encountered at BH-1 location at a depth of 21.6m from grade and was proved by coring, approximately 200mm in thickness.

4.5 Bedrock

A bedrock was encountered underlying the sand and gravel deposit at BH-1 location, at a depth of 24.5m from grade.

Approximately 3.1m (10ft) long NQ rock cores were obtained at BH-1 location, below the depth of 24.5m from grade to determine the quality of the bedrock. An inspection of the rock cores indicated that the rock quality, within the cored depths, was very poor, based on RQD values of 0% and Recovery of 98%. The rock cores at BH-1 location was terminated at a depth of 27.6m from grade.

4.6 Groundwater

Free water and/or wet cave-in was recorded in all the open boreholes, at depths of 1.8m to 5.5m from grade, during and upon completion of drilling and sampling, except at BH-1 and BH-11 locations where mud drilling was used and the free water could not be properly measured.

On November 5, 2018, the groundwater levels, measured in the monitoring wells, installed at BH-1, BH-7, BH-8 and BH-11 locations, were at depths of 0.51m, 1.10m, 1.28m and 1.18m from grade, respectively.

Based on the field observation and the moisture content profile of the subsoil samples, it is our opinion that there is a continuous groundwater table across the Site, below depths of 0.5m to 1.3m from grade, in the wet and saturated silty clay / clayey silt deposits, including perched water at the lower portion of the fill.

5.0 RECOMMENDATIONS

Based on the Preliminary Concept Site Plans, Drawing #: 1314C for the north parcel and Drawing #: 1313D for the south parcel, prepared by Turner Fleischer Architects Inc., dated April 19, 2018, the development at both parcels will consist of Retails A & B with the associated driveway and parking lots.

We understand that each of the proposed retail buildings will consist of one single storey structure without a basement. We understand that the building pregrade at both parcels may be at elevation 93.5m, with the Finished Floor Elevation (FFE) at elevation 93.8m.

Our recommendations on the design and construction for the proposed development, based on the subsoil and groundwater data, encountered at the borehole locations, are as follows:

5.1 Site Preparation

The soil description and depth of fill shown on the Borehole Logs are specific depths at the borehole locations only. The thickness of topsoil and the depth of the fill at locations beyond the boreholes may be thicker or deeper. We recommend that the contractor bidding for the job should determine the depths of deleterious material by test pits and allow for removal of any deleterious fill and material, with high moisture and/or organic content, during the site preparation for site grading.

To achieve uniform subgrade conditions, we recommend that after removal of the topsoil, unsuitable surficial soil and the deleterious fill material, the exposed subgrade should be proof-rolled, after it has been reviewed by a soils engineer from our office. Any soft areas, identified during proof-rolling, should be sub-excavated and replaced with organic free soil, compacted to a minimum of 98% of its SPMDD.

Depending on the final grades, the Site may have to be regraded for the proposed development. If cut and fill operation is proposed, the on-site excavated fill, to be used for site grading, should be organic free and maintained at or close to its optimum moisture content during placement and compaction. The new fill should be compacted in lifts of 200mm to at least 98% of its Standard Proctor maximum dry density (SPMDD).

To support the building footings, the new fill within the buildings should be placed and compacted in lifts of 200mm to 100% of SPMDD in accordance with the engineered fill guidelines, after the existing topsoil and fill is completely removed within the proposed building envelopes. The guideline for placement and compaction of engineered fill is provided in Appendix A.

Compressible topsoil and the fill material, containing relatively high organic content, will not be suitable for reuse in areas where future settlement cannot be tolerated. This material will have to be disposed off-site or reused in landscaped areas, subject to approval by the landscape architect.

5.2 Foundation Design

With the placement of engineered fill, replacing the topsoil and the existing fill material, and the presence of the underlying very soft silty clay / clayey silt deposits, the total and differential settlement of the buildings, founded on spread / strip foundations, could be considerably more than normally anticipated limits of 25mm and 20mm, respectively. Since the extent of the very soft to soft deposits, at the borehole locations, extending to depths of 15.0m to 19.5m from grade, and the actual loads imposed by the proposed buildings are not known, the magnitude of the total settlement cannot be accurately determined. In this respect, it is our opinion that the most cost effective option for the subject Site is to preload the proposed building areas to a minimum of 150% of the anticipated loads.

The surcharge, after preparing the building pads to the finish floor elevations, can consist of loose fill and allowed to stay on site, until the primary consolidation of the native soils has been completed. Prior to placement of surcharge fill, settlement plates should be installed on the prepared building pads and the settlement of the building pad, over a period of time, documented by *Toronto Inspection Ltd.* to determine when the primary consolidation settlement has been completed.

The native subsoils at the Site consist of silty clay to clayey silt deposits. In order to speed up the consolidation of these deposits, we recommend that sand wicks should be installed, at approximately 3m centres. It is our opinion, that with the installation of the sand wicks, the primary consolidation could take up to 2 years.

After it has been determined that the primary consolidation is completed, the fill surcharge can be removed and the proposed buildings can be supported on conventional spread / strip footings, founded on the engineered fill or in the native very soft silty clay / clayey silt deposits.

Conventional spread/strip footings of the proposed retail buildings A & B, at north and south parcels, founded on the engineered fill, at or above depths of 1.5m below the proposed finished floor elevations, can be designed using the following bearing pressures:

- 75 kPa at the Serviceability Limit State (SLS)
- 120 kPa at the factored Ultimate Limit State (ULS)

Conventional spread/strip footings of the proposed retail buildings A & B, at north and south parcels, founded on the engineered fill and native very soft silty clay / clayey silt deposits, at or below depths of 1.5m below the proposed finished floor elevations, can be designed using the following bearing pressures:

- 50 kPa at the Serviceability Limit State (SLS)
- 75 kPa at the factored Ultimate Limit State (ULS)

For strip footings placed in the engineered fill, we recommend that all perimeter footings should be reinforced continuously with at least 2-15M steel bars. This reinforcement will bridge any loose pockets of fill, if any, under the footings.

Provided that the Site has been preloaded, as recommended above, and primary settlement completed under the surcharge loads, the total and differential settlement of footings, founded on the engineered fill / native soil strata and designed for the above recommended bearing pressures at the serviceability limit state, will not exceed 25mm and 20mm, respectively.

The native silty clay / clayey silt deposits at the Site is very sensitive to moisture changes and disturbance by the construction traffic.

Concrete for the footings should be poured immediately after the bearing areas have been excavated, cleaned, inspected and approved by a qualified geotechnical engineer from our office. If the base of the footings will be left open for some time, a 100mm mud slab of lean mix concrete should be poured on the footing bases immediately after inspection.

Alternatively, if surcharge is not feasible for the proposed buildings at the Site, deep foundations, cast in place drilled 0.6m diameter concrete piles, CFA piles, founded at least 1m into the bedrock, at or below depths of 25.5m from grade at BH-1 and BH-11 locations, can be used to support the proposed buildings, designed for an axial load of 1000 kN. The minimum spacing of the CFA piles should be 3 times the diameter of piles, centre to centre. It will however, be necessary to confirm this load capacity by a full scale load test.

Geopiers, taken through the very soft silty clay / clayey silt and placed on the bedrock, can also be considered to support the proposed buildings. The Geopiers should be designed and installed by a specialist in the trade.

All perimeter footings or any footings, which may be exposed to freezing conditions, should be placed below the frost penetration depth of 1.5m below the outside grade or provided with an equivalent thermal protection.

We recommend that the site grading and the foundation plans of the proposed development should be reviewed by this office to confirm the bearing strata and the recommended design bearing pressures for the new structures.

It should be noted that the above recommendations for foundations have been analysed by *Toronto Inspection Ltd.* from the subsoil information obtained at the borehole locations. The bearing material, the interpretation between the boreholes and the recommendations of this report must be checked through field inspection provided by *Toronto Inspection Ltd.* to validate the information for use during the construction stage.

5.3 Floor Slab Construction

The floor slab can be designed and constructed as a conventional slab-on-grade method.

Following site preparation in the building areas, the exposed building subgrade should be proof rolled under the supervision of a geotechnical technician. All compressible, loose, or weak spots observed during proof-rolling should be removed from the subgrade and replaced with an approved granular material, compacted in 200mm lifts to 98% Standard Proctor Maximum Dry Density (SPMDD).

However, some long term settlement will be anticipated if the building areas are not preloaded. The amount of settlement depends on the amount of filling for site grading. An alternative is to use a combined light weight fill (Geofoam, cellular concrete or equivalent) and traditional soil to fill the building areas.

A bedding consisting of at least 150 mm of Granular A (OPSS Form 1010) or 20mm crusher run limestone, is recommended as a moisture barrier. The bedding should be compacted to at least 100% SPMDD.

5.4 Earthquake Consideration

The Ontario Building Code requires that all buildings be designed to resist earthquake forces. In accordance with Table 4.1.8.4.A of the Ontario Building Code. The site classification for the Seismic Site Response is Class E (Soft soil).

The acceleration and velocity based site coefficients, F_a and F_v , should conform to Tables 4.1.8.4.B and 4.1.8.4.C. These values should be reviewed by the Structural Engineer.

5.5 Excavation and Backfilling

All excavations should comply with the Ontario Occupational Health and Safety Act. Any excavation should be sloped back to a safe angle of less than 45° . a flatter slope will be required for excavation in saturated soils.

The in-situ moisture contents of the native silty clay / clayey silt deposits were much higher than their optimum moisture contents. The use of the on-site excavated materials, for backfilling the foundations at exterior and service trenches, should be controlled properly on site by the soils technicians. If the seasonal condition is not favourable for drying the silty clay / clayey silt, the reuse of the on-site material will be limited to areas where future settlement will be of little consequence.

Groundwater seepage can be anticipated in excavations of 1.8m and deeper from grade. Due to the very fine grained subsoils of low permeability, the rate of water seepage is expected to be relatively low and, in our opinion, can be handled by conventional pumping from sumps.

Bedding for the underground services, within the clayey silt deposit, should consist of OPSS Granular A or its equivalent. If the subsoil at bottom of trench consists of saturated silty clay, the bedding in the service trenches should consist of 20mm clear limestone and a geotextile filter fabric should be used to separate the clear stone bedding from the base and the sides of the excavation. The geotextile filter fabric must surround the clear stone bedding completely, to prevent internal erosion (loss of fines) from the surrounding soils.

Backfill around catch basins, manholes, and narrow trenches, should consist of imported granular material. The backfill should be compacted using a smaller vibratory equipment.

5.6 Pavement Construction

Since the silty clay / clayey silt material is considered to be highly frost susceptible, the following pavement design is based on an assumption that the subgrade soils for the parking lot and driveways will consist of organic free silty clay / clayey silt material.

	Heavy Duty Driveway	Light Duty Parking
Asphaltic Concrete: OPSS HL3	40 mm	65 mm
OPSS HL8	60 mm	-
Base - OPSS Granular 'A' or 20mm crusher-run	150 mm	150 mm
Sub-base - OPSS Granular 'B' or 50mm crusher-run	450 mm	300 mm

The pavement design is based on pavement construction being carried out during the drier time of the year and that the subgrade is stable, not heaving under construction traffic. If the subgrade is wet and unstable, additional thickness of sub-base material may be required.

Following site grading, the subgrade of the entire pavement should be proof-rolled using a heavy vibratory roller. Any soft spots revealed by the proof rolling should be subexcavated and replaced with an approved dry material and compacted to at least 98% of its SPMDD.

The granular base and sub-base materials should be placed in thin lifts and compacted to a minimum 100% of its SPMDD. Asphaltic concrete should be placed and compacted to at least 96% Marshall density.

Provision should be made for the water to drain out of and not collect in the granular base courses for the pavement to function properly. Perforated subdrains should be provided, extending to a distance of 3m in all directions of catch basins, (see Figure No. 5) and continuously in locations where a drop in the subgrade elevation is relevant, such as beside the ramp or concrete sidewalk (see Figure No. 6). The subdrains should be at least 800mm below the road pavement level, and installed on a positive gradient to allow for a free flow of water. The backfill above the drains should comprise of free draining Granular B or its equivalent and should be continuous with the granular subbase of the pavement.

Catch basins and manholes should be backfilled with OPSS Granular B material. The catch basins should be perforated just above the drain level and the weep holes should

be screened with a filtered fabric. This will help the pavement structure as well as alleviate the differential movement of the catch basins or the manholes due to the frost action.

5.7 Other Engineering Concerns

The Atterberg Limits Test results indicated that for the samples tested, the in-situ moisture content is higher than the liquid limits and loss of moisture from the silty clay deposit could result in shrinkage of the soil and subsequent settlements. It is, therefore, recommended that all sub-surface storm and sewer pipes should have an impervious collars, extending from the base of the service trenches to above the groundwater table, to prevent movement of water through the granular base and the backfill around the service pipes.

The minimum distance of any trees, planted on or close to the Site, must be at least two times the anticipated maximum estimated height of the trees. This will prevent / minimise the loss of groundwater from under the structures by the root system of the trees.

6.0 GENERAL STATEMENT OF LIMITATION

The comments and recommendations presented in this report are based on the subsoil and ground water conditions encountered at the borehole locations, indicated in the borehole location plan, and are intended for the guidance of the design engineer. Although we consider this report to be representative of the existing subsurface conditions at the subject property, the soil and the ground water conditions between and beyond the borehole locations may differ from those encountered at the time of our investigation and may become apparent during construction. Any contractor bidding on, or undertaking the works, should decide on their own investigation and interpretations of the groundwater and the soil conditions between the borehole locations.

Any use and / or the interpretation of the data presented in this report, and any decisions made on it by the third party are responsibility of the third parties. The responsibility of *Toronto Inspection Ltd.* is limited to the accurate interpretation of the soil and ground water conditions prevailing in the locations investigated and accepts no responsibility for the loss of time and damages, if any, suffered by the third party as a result of decisions or actions based on this report.

Any legal actions arising directly or indirectly from this work and/or *Toronto Inspection Ltd.*'s performance of the services shall be filed no longer than two years from the date of *Toronto Inspection Ltd.*'s substantial completion of the services. *Toronto Inspection Ltd.* shall not be responsible to the client for lost revenues, loss of profits, cost of content, claims of customers, or other special indirect, consequential, or punitive damages.

To the fullest extent permitted by law, the client's maximum aggregate recovery against *Toronto Inspection Ltd.*, its directors, employees, sub-contractors, and representatives, for any and all claims by clients for all causes including, but not limited to, claims of breach of contract, breach of warranty and/or negligence, shall be the amount of the fee paid to *Toronto Inspection Ltd.* for its professional services rendered under the agreement with respect to the particular site which is the subject of the claim by the client.

Yours very truly,
TORONTO INSPECTION LTD.



David S. Wang, P. Eng.
Senior Engineer

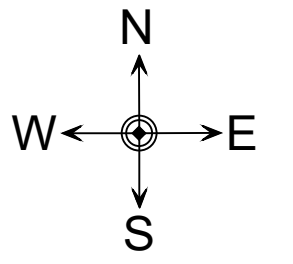


Upkar S. Sappal, P. Eng.
Principal Engineer



Toronto Inspection Ltd.

DRAWINGS
Borehole Location Plan
Logs of Boreholes



LEGEND :

- Approximate Site Boundary
- ⊕/⊗ Borehole / Monitoring Well Location

NOT TO SCALE

TorontoInspection
GEO-ENVIRONMENTAL CONSULTANTS
 110 Konrad Crescent, Unit 16, Markham, Ontario L3R 9X2
 Tel: 905-940 8509 Fax: 905-940 8192
 Email : TIL@torontoinspection.com

TITLE:		Borehole / Monitoring Well Location Plan	
LOCATION:		West of Cambria Road and Greenbank Road, Barrhaven, Ontario	
PROJECT NO.	4817-18-G-LPL-A	DATE :	September 2018
		DRAWING NO.	1

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: West of Cambrian Road and Greenbank Road, Barrhaven, Ontario

Date Drilled: 9/20/18

Auger Sample

Headspace Reading (ppm)

Drill Type: Track Mounted Drill Rig

SPT (N) Value

Natural Moisture

Datum: Geodetic

Dynamic Cone Test

Plastic and Liquid Limit

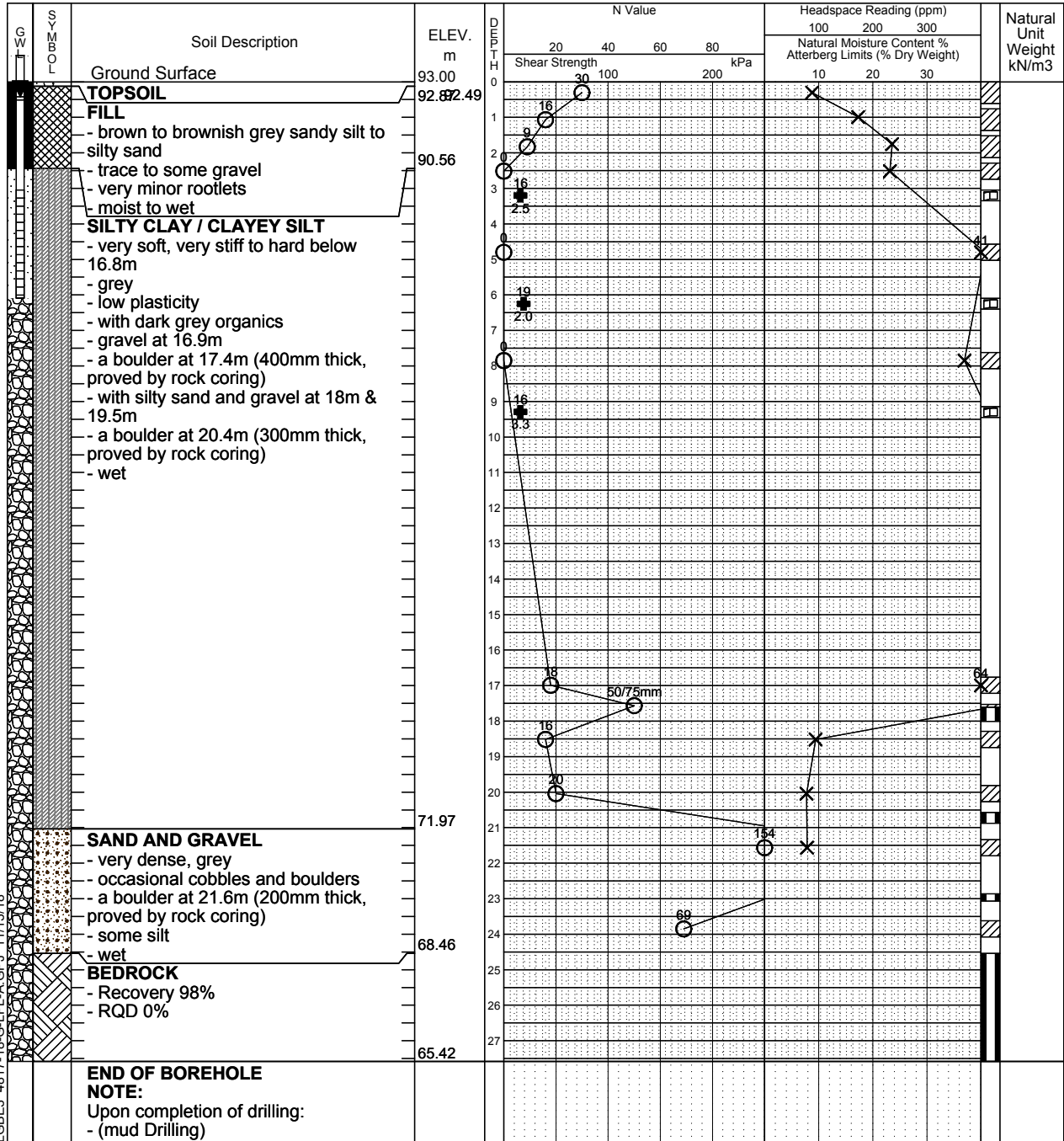
Shelby Tube

Unconfined Compression

Field Vane Test

% Strain at Failure

Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
Nov 5, 2018	0.51m	

Date Drilled: 4/30/18

Auger Sample

Headspace Reading (ppm)

Drill Type: Track Mounted Drill Rig

SPT (N) Value

Natural Moisture

Datum: Geodetic

Dynamic Cone Test

Plastic and Liquid Limit

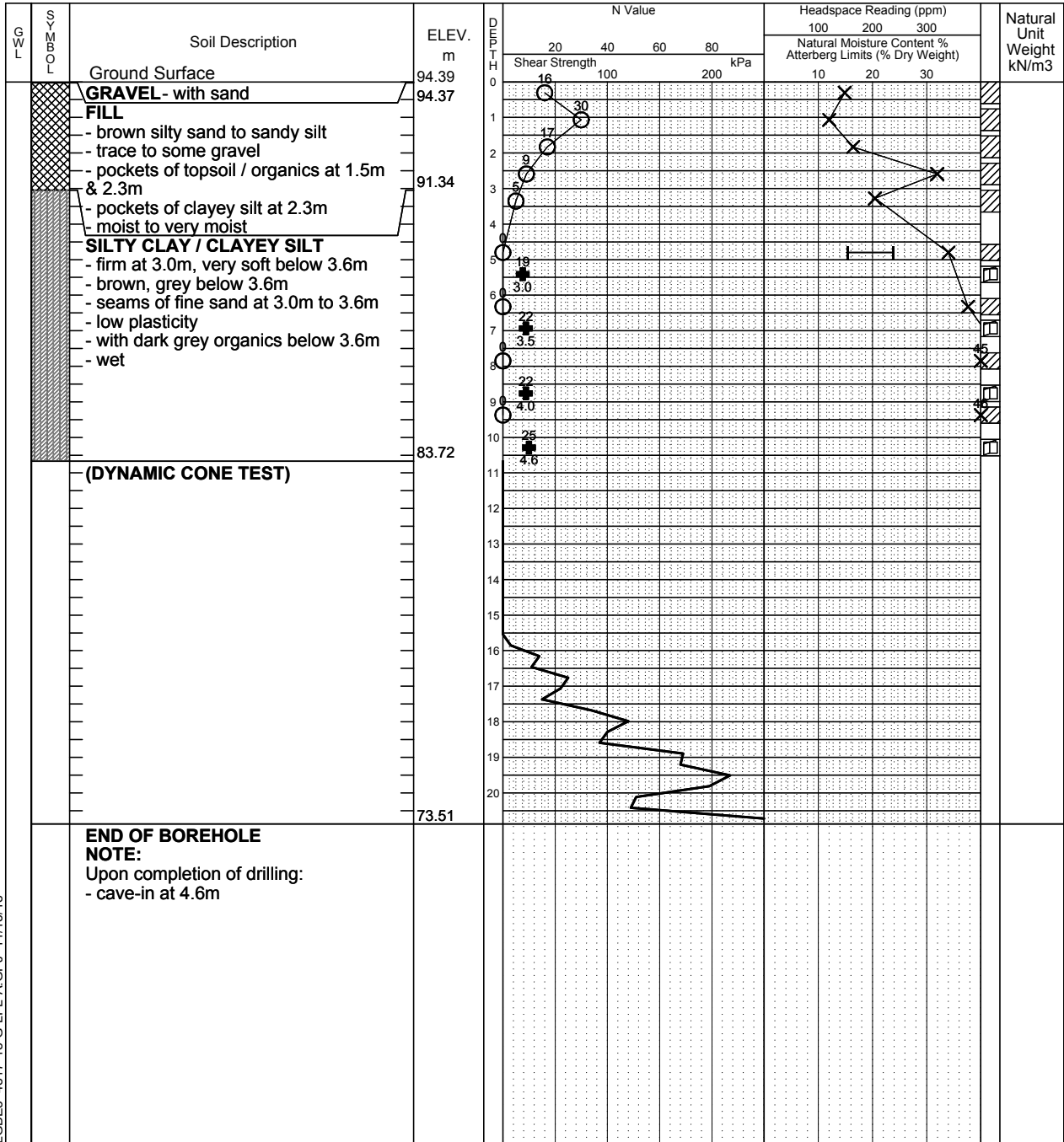
Shelby Tube

Unconfined Compression

Field Vane Test

% Strain at Failure

Penetrometer



LGBE3 4817-18-G-LPL-A.GPJ 11/13/18

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: West of Cambrian Road and Greenbank Road, Barrhaven, Ontario

Date Drilled: 4/30/18

Auger Sample

Headspace Reading (ppm)

Drill Type: Track Mounted Drill Rig

SPT (N) Value

Natural Moisture

Datum: Geodetic

Dynamic Cone Test

Plastic and Liquid Limit

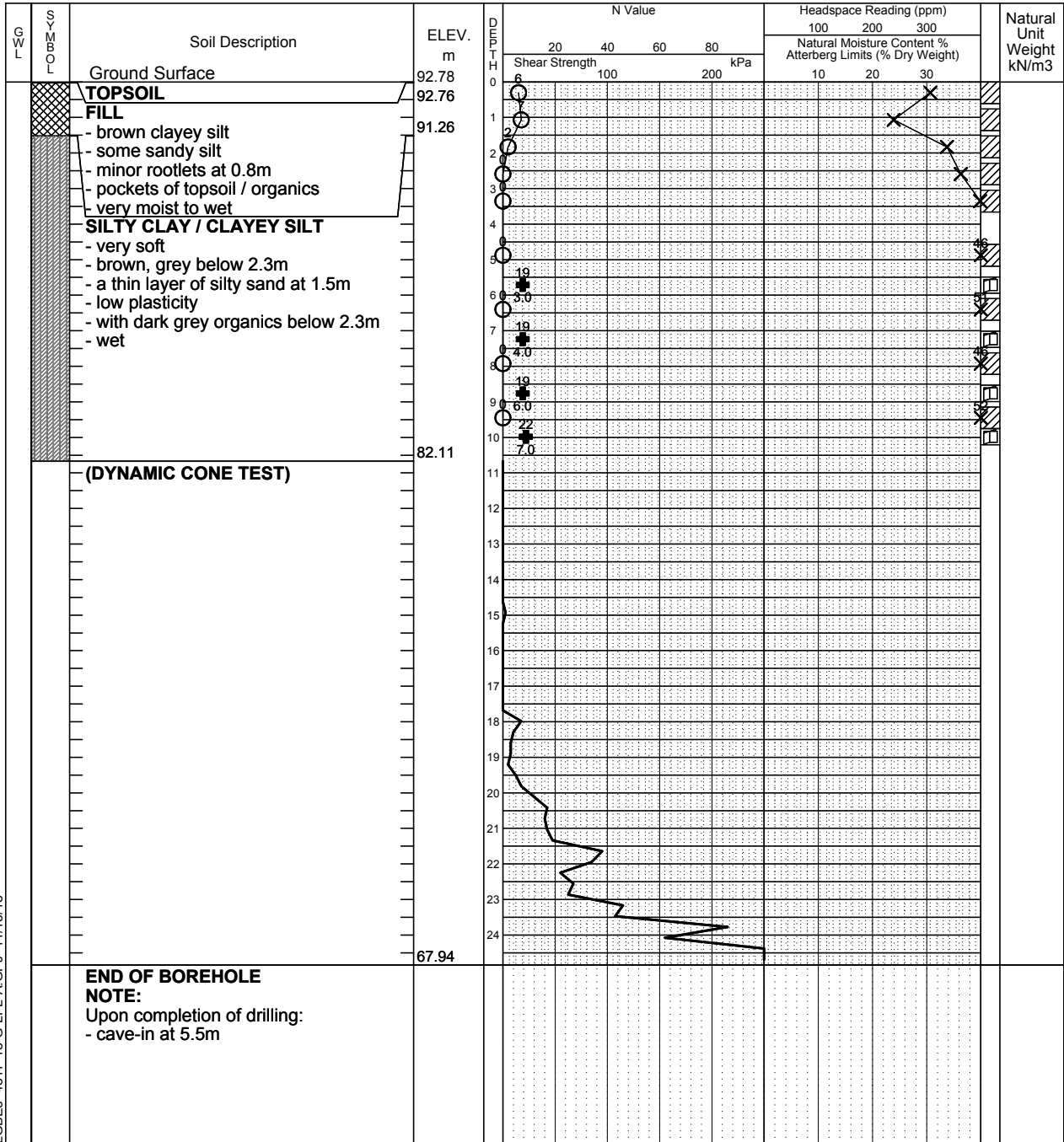
Shelby Tube

Unconfined Compression

Field Vane Test

% Strain at Failure

Penetrometer



LGBE3 4817-18-G-LPL-A-GPJ 11/13/18

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: West of Cambrian Road and Greenbank Road, Barrhaven, Ontario

Date Drilled: 5/1/18

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



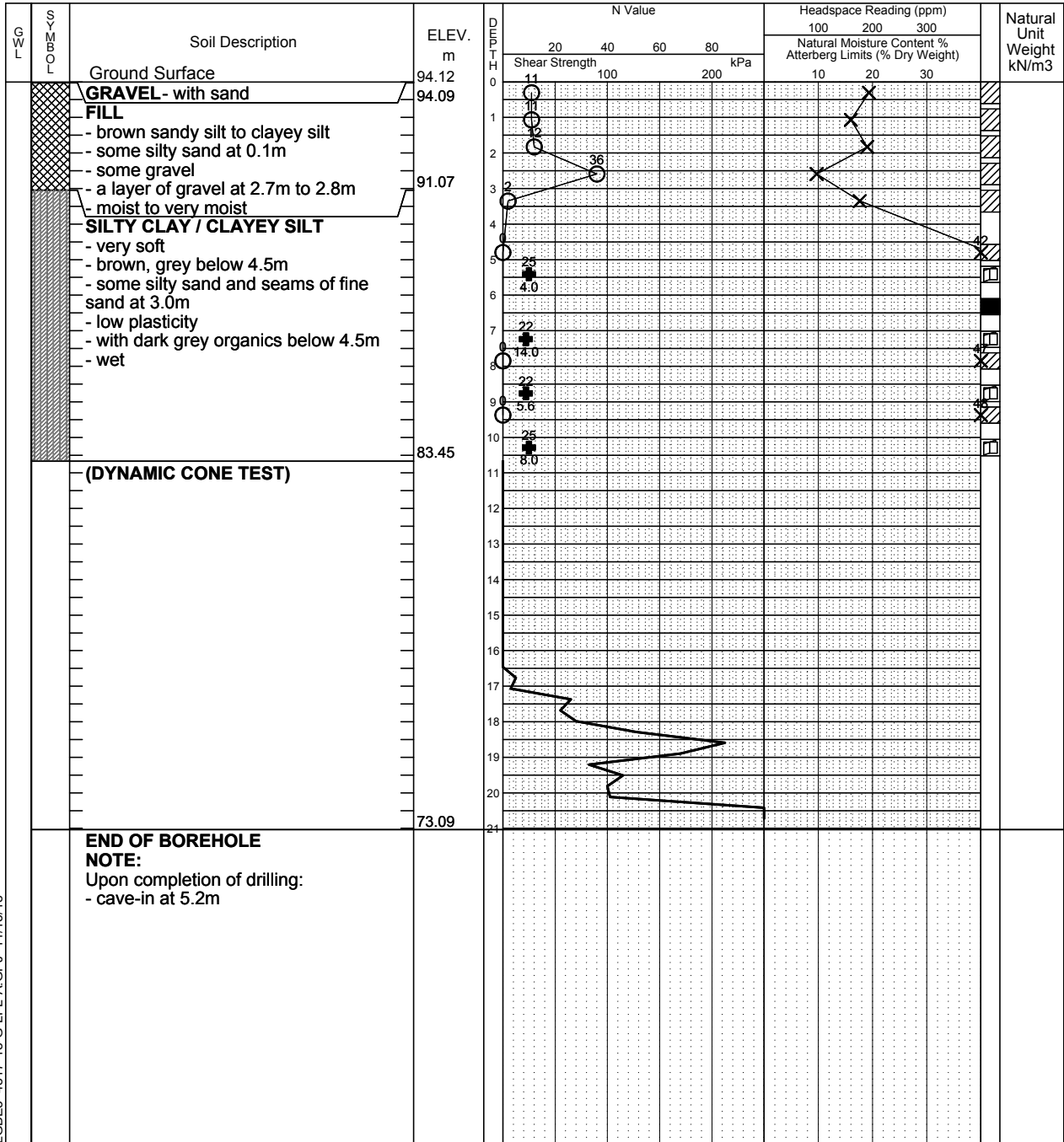
Field Vane Test



% Strain at Failure



Penetrometer



LGBE3 4817-18-G-LPL-A.GPJ 11/13/18

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: West of Cambrian Road and Greenbank Road, Barrhaven, Ontario

Date Drilled: 5/2/18

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



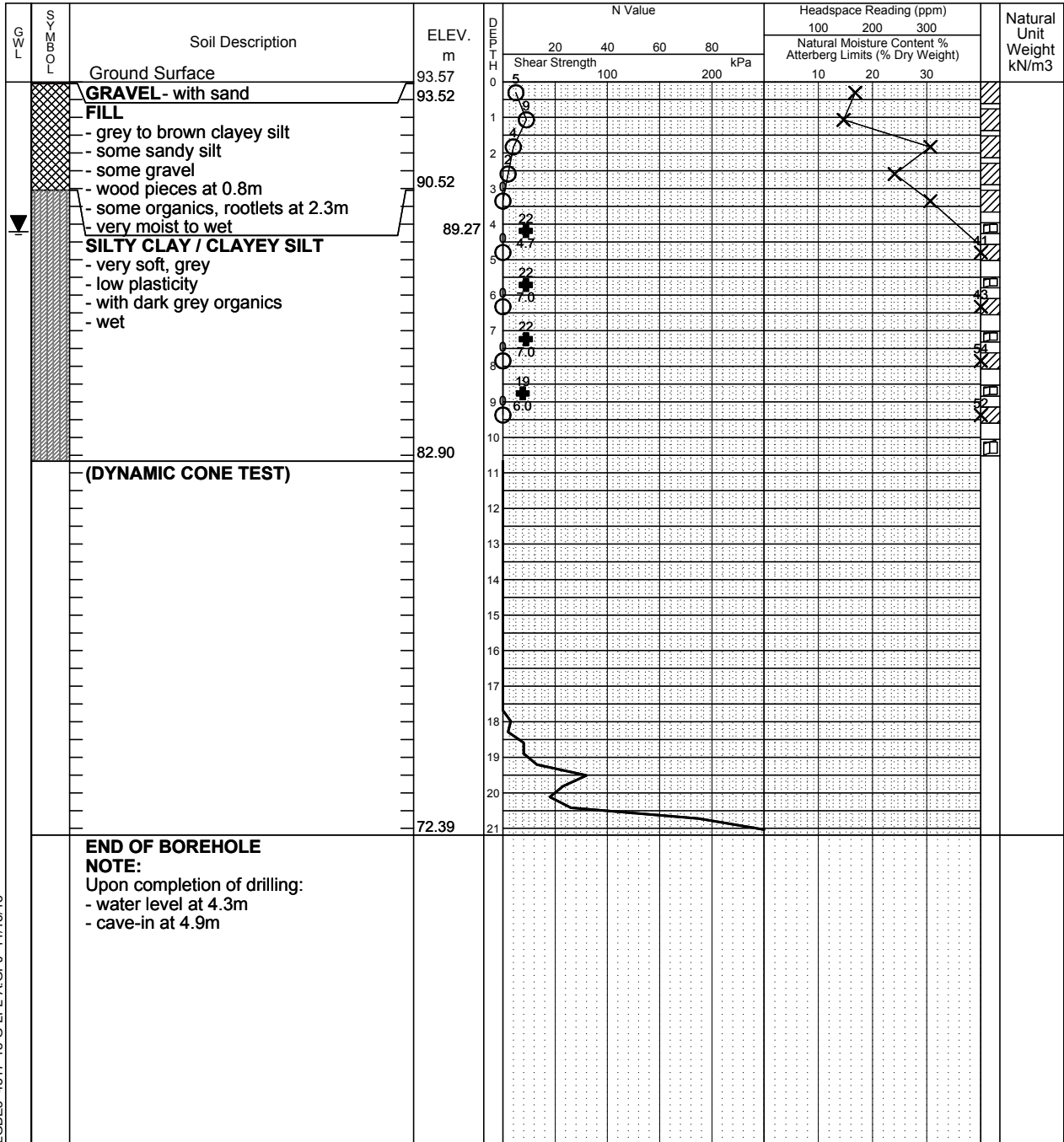
Field Vane Test



% Strain at Failure



Penetrometer



LGBE3 4817-18-G-LPL-A.GPJ 11/13/18

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: West of Cambrian Road and Greenbank Road, Barrhaven, Ontario

Date Drilled: 4/30/18

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



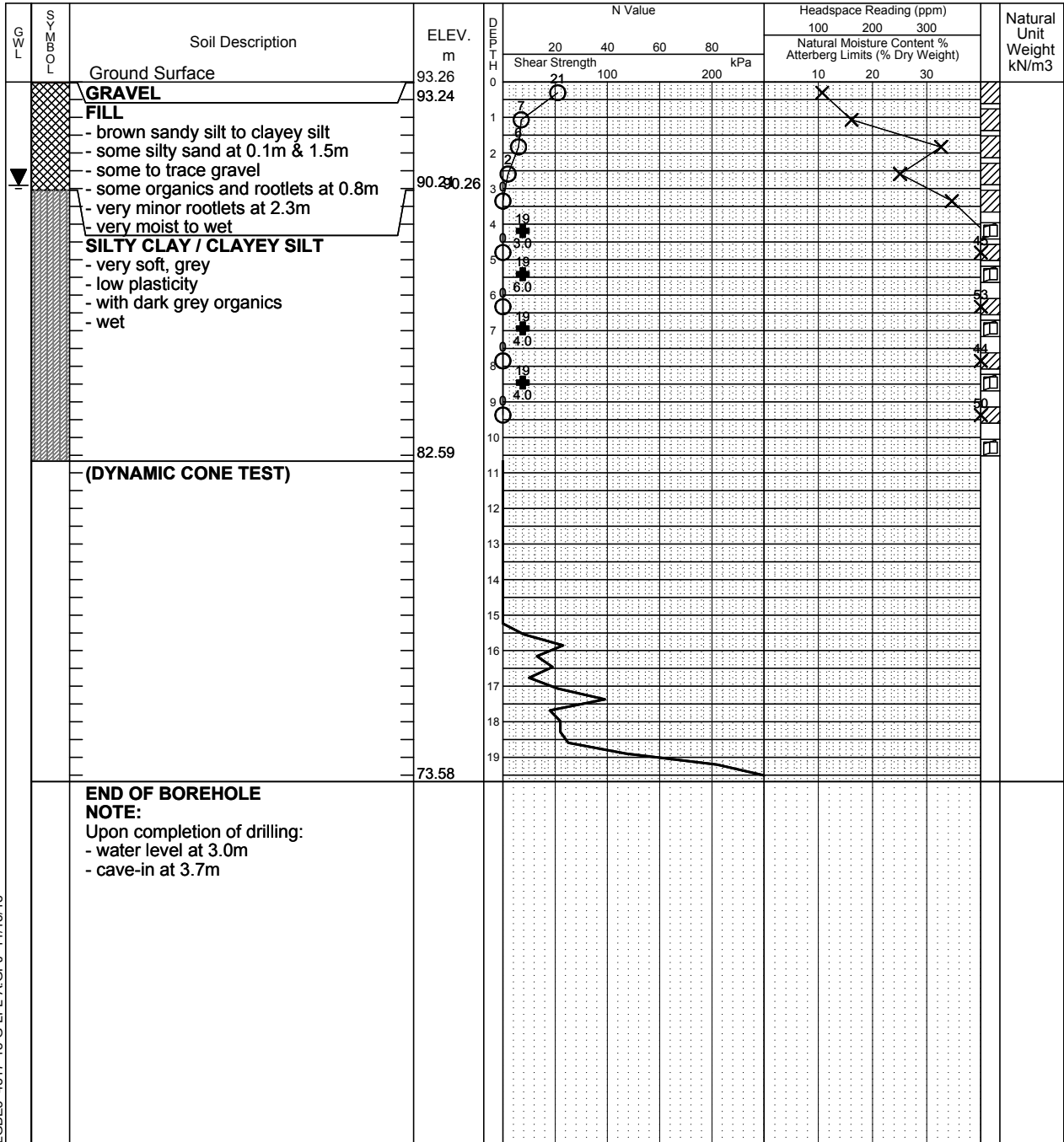
Field Vane Test



% Strain at Failure



Penetrometer



LGBE3 4817-18-G-LPL-A.GPJ 11/13/18

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Date Drilled: 5/1/18

Auger Sample

Headspace Reading (ppm)

Drill Type: Track Mounted Drill Rig

SPT (N) Value

Natural Moisture

Datum: Geodetic

Dynamic Cone Test

Plastic and Liquid Limit

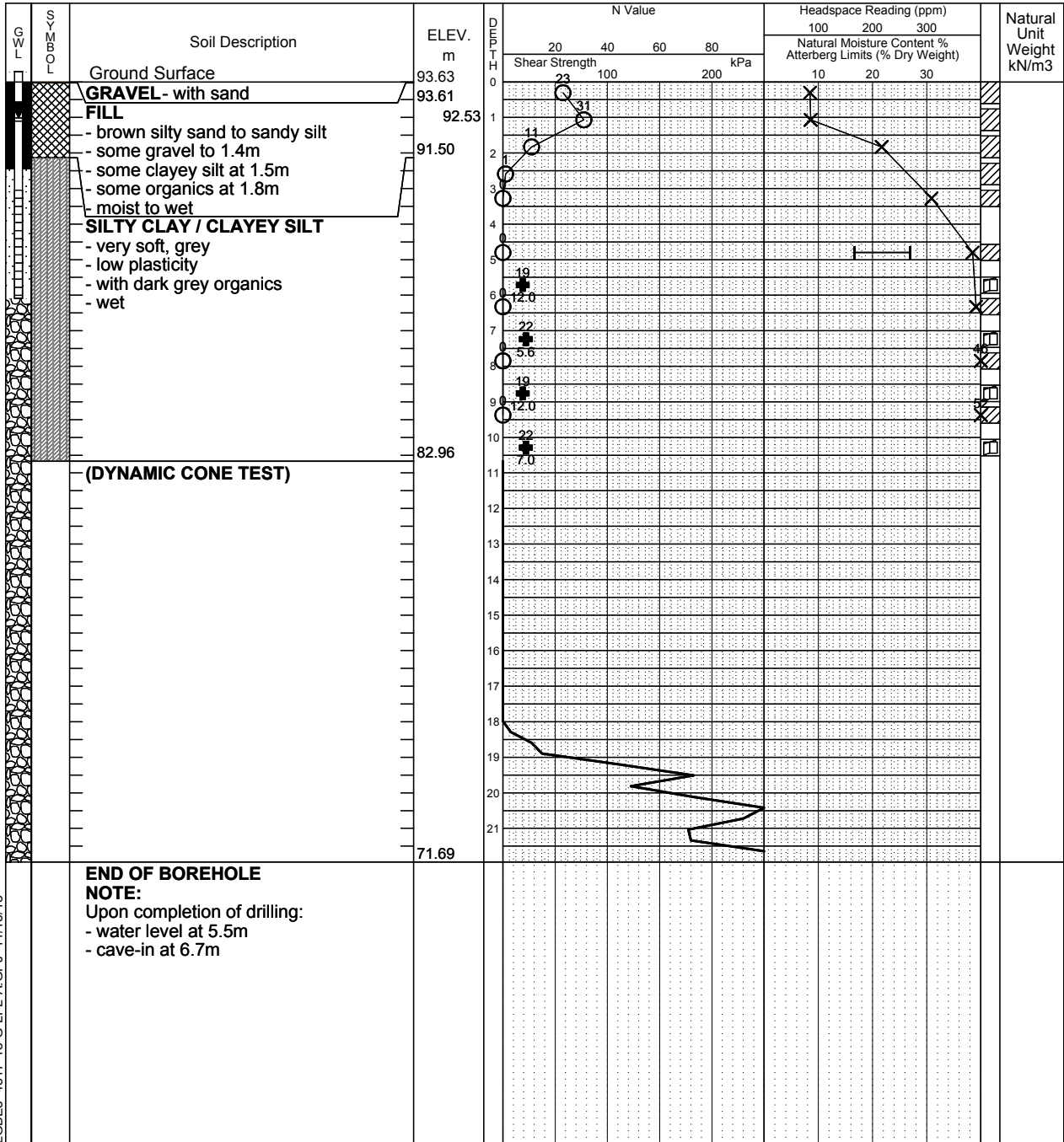
Shelby Tube

Unconfined Compression

Field Vane Test

% Strain at Failure

Penetrometer



LGBE3 4817-18-G-LPL-A.GPJ 11/13/18

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
Nov 5, 2018	1.10m	

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: West of Cambrian Road and Greenbank Road, Barrhaven, Ontario

Date Drilled: 5/2/18

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



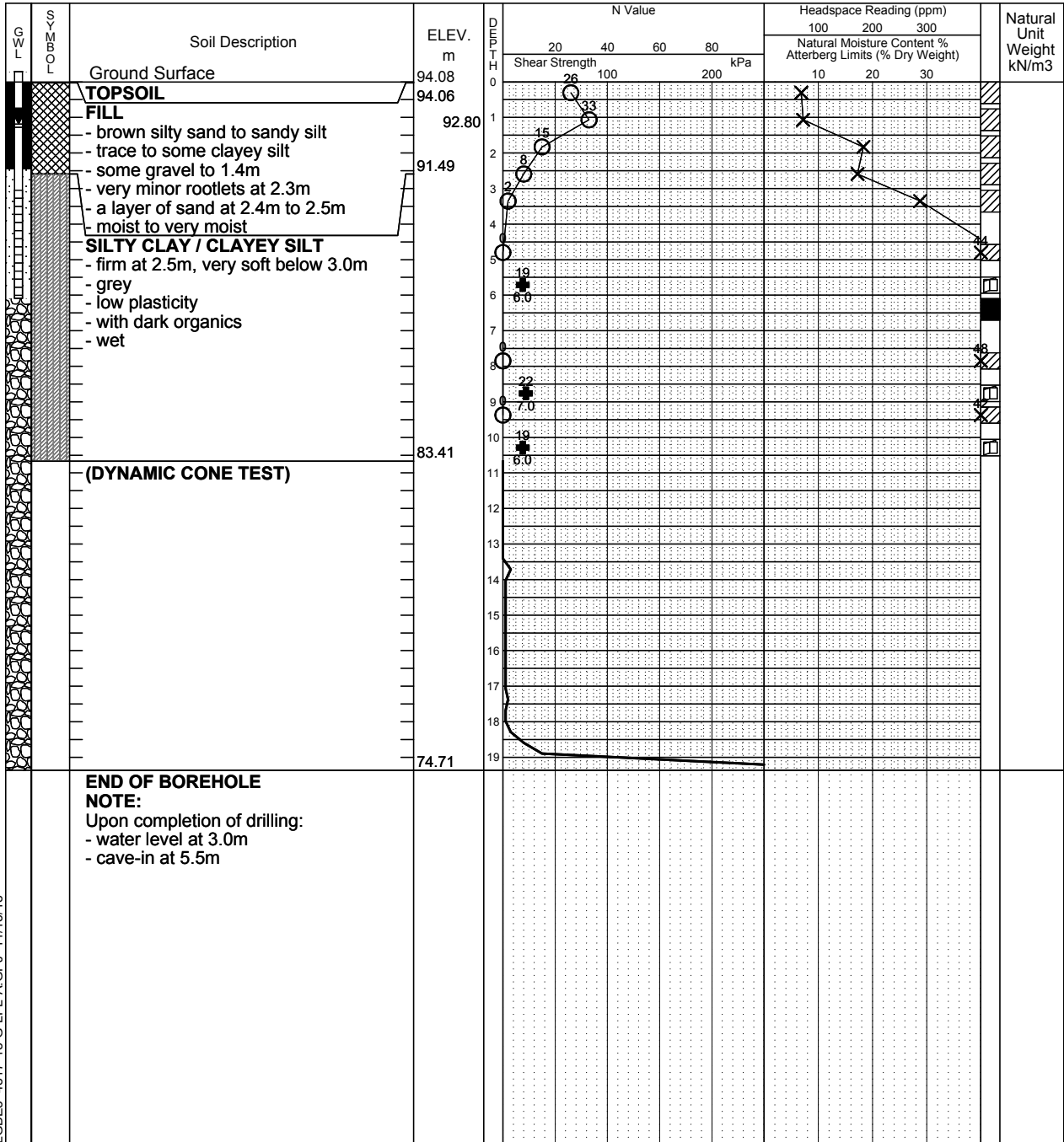
Field Vane Test



% Strain at Failure



Penetrometer



LGBE3 4817-18-G-LPL-A.GPJ 11/13/18

END OF BOREHOLE NOTE:
 Upon completion of drilling:
 - water level at 3.0m
 - cave-in at 5.5m

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
Nov 5, 2018	1.28m	

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: West of Cambrian Road and Greenbank Road, Barrhaven, Ontario

Date Drilled: 8/17/18

Auger Sample

☒

Headspace Reading (ppm)

•

Drill Type: Track Mounted Drill Rig

SPT (N) Value

○

Natural Moisture

×

Datum: Geodetic

Dynamic Cone Test

—

Plastic and Liquid Limit

—|—

Shelby Tube

■

Unconfined Compression

⊗

Field Vane Test

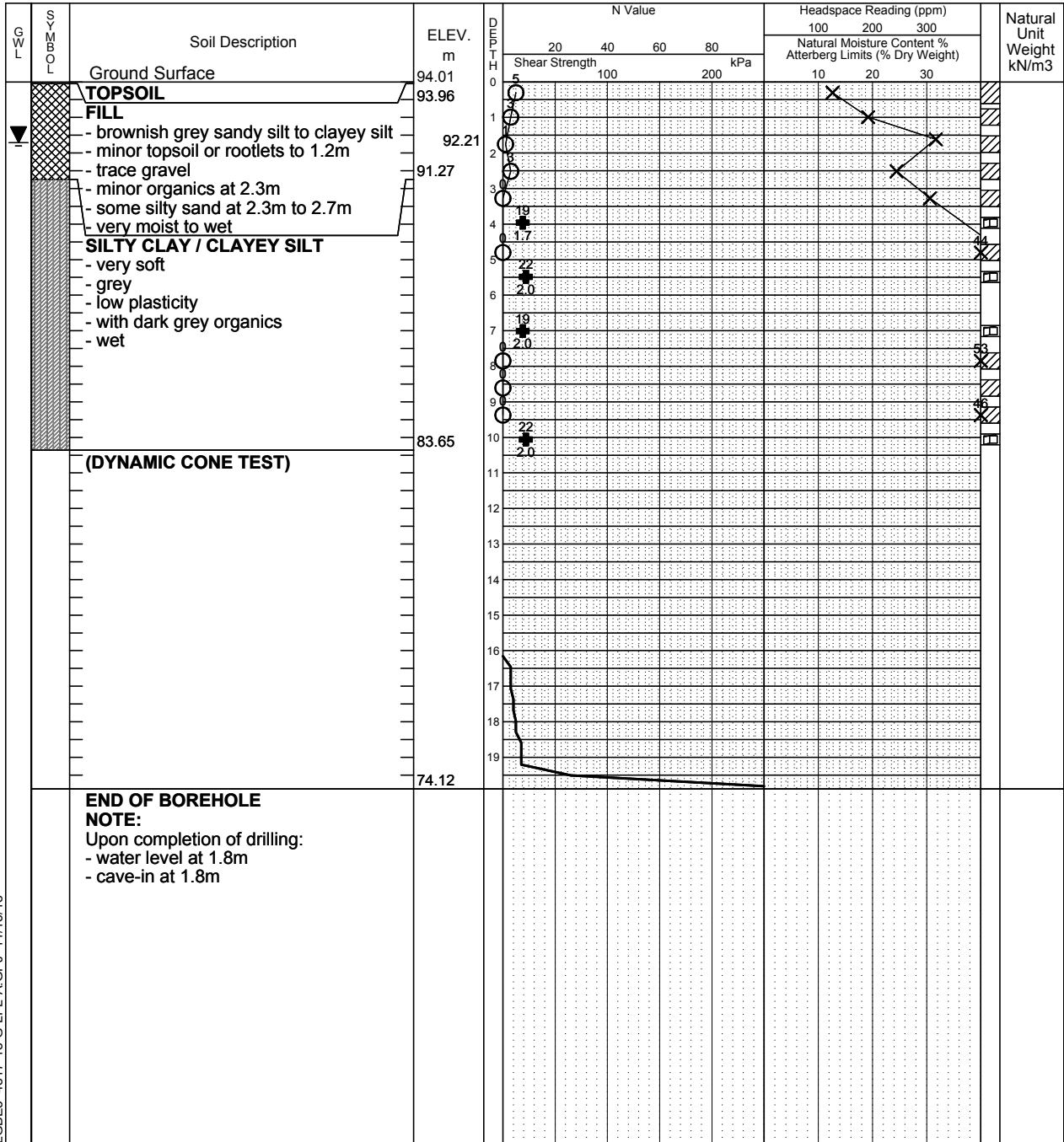
⊕

% Strain at Failure

⊗

Penetrometer

▲



LGBE3 4817-18-G-LPL-A.GPJ 11/13/18

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Date Drilled: 9/25/18

Auger Sample

Headspace Reading (ppm)

Drill Type: Track Mounted Drill Rig

SPT (N) Value

Natural Moisture

Datum: Geodetic

Dynamic Cone Test

Plastic and Liquid Limit

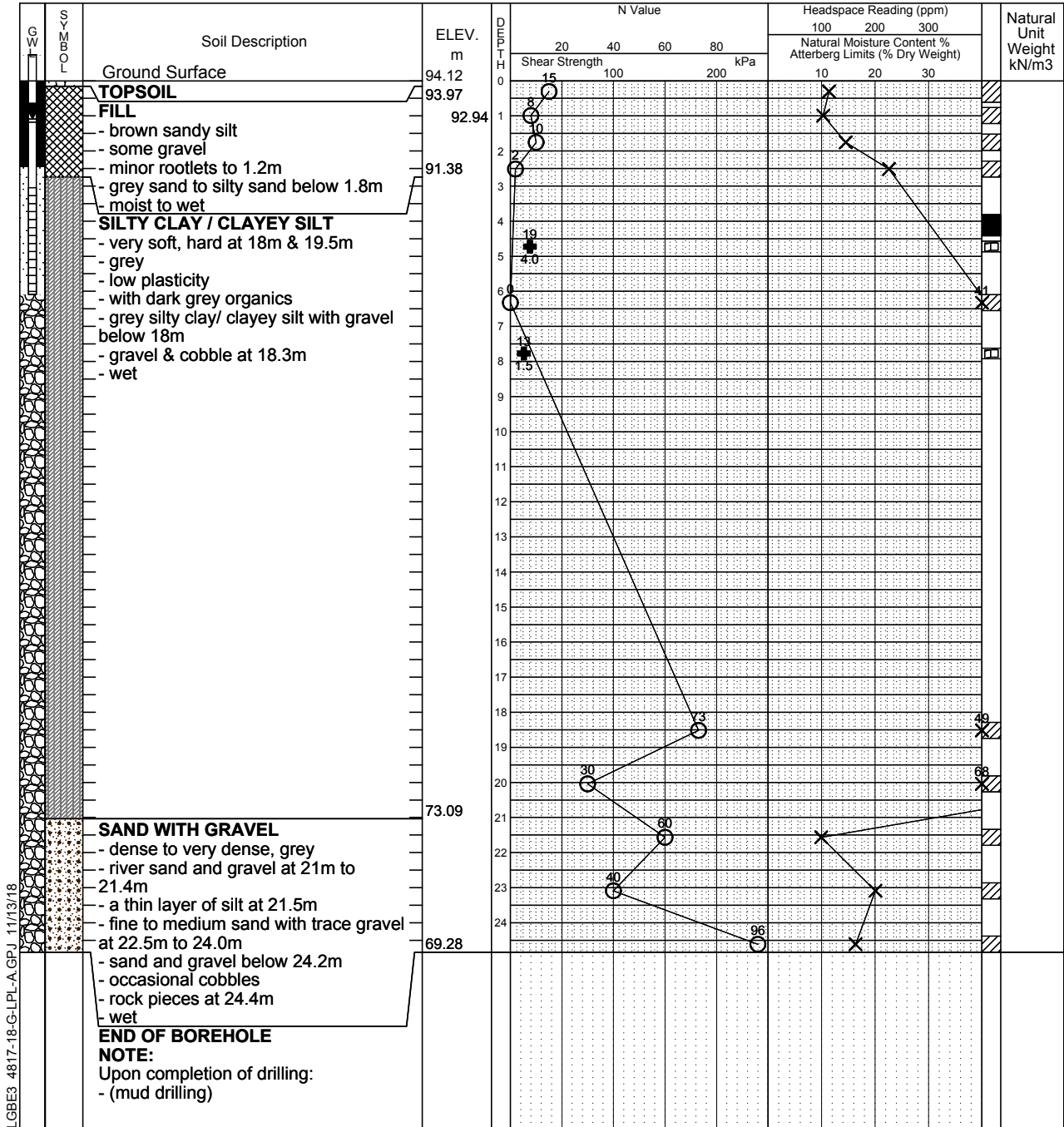
Shelby Tube

Unconfined Compression

Field Vane Test

% Strain at Failure

Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
Nov 5, 2018	1.18m	

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: West of Cambrian Road and Greenbank Road, Barrhaven, Ontario

Date Drilled: 8/16/18

Auger Sample

Headspace Reading (ppm)

Drill Type: Track Mounted Drill Rig

SPT (N) Value

Natural Moisture

Datum: Geodetic

Dynamic Cone Test

Plastic and Liquid Limit

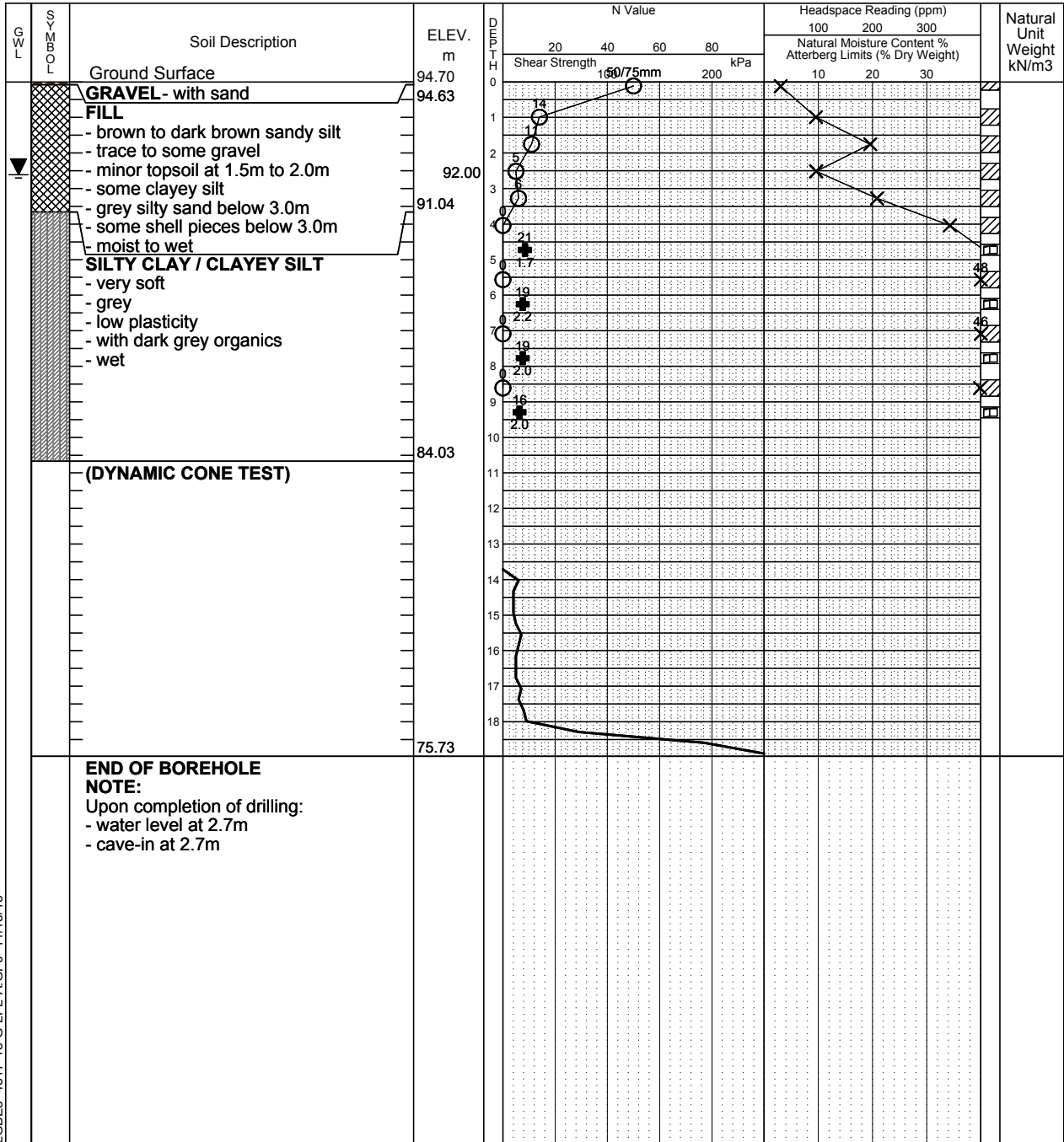
Shelby Tube

Unconfined Compression

Field Vane Test

% Strain at Failure

Penetrometer



LGBE3 4817-18-G-LPL-A.GPJ 11/13/18

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)



Toronto Inspection Ltd.

FIGURES

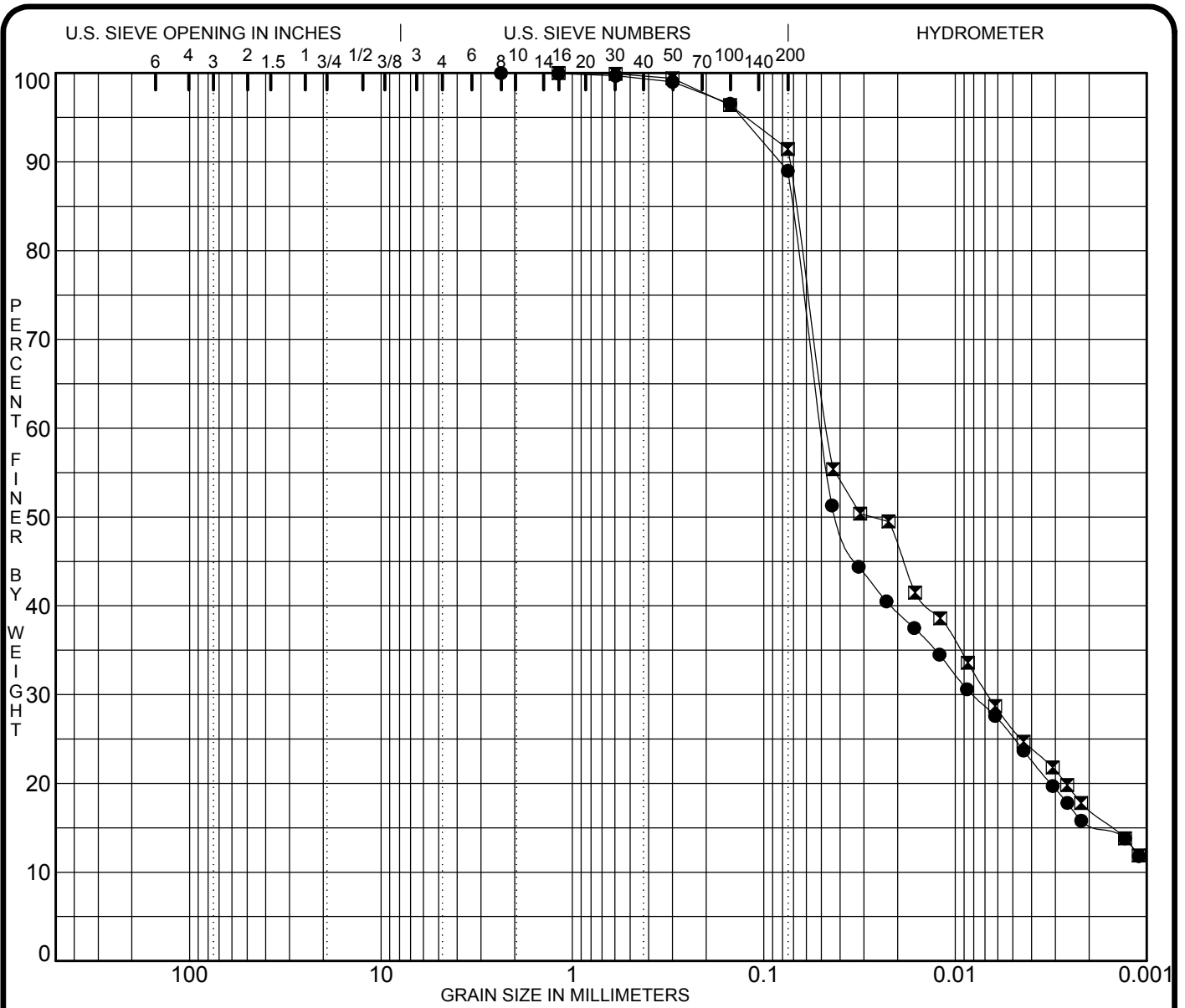
Gradation Curves

Atterberg Limits

Consolidation Tests

Detail For Subdrain At CB's & DBMH's

Concrete Ramp/Sidewalk Sections



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu	
● BH-2 4.6	LEAN CLAY CL		24	15	8			
☒ BH-7 4.6	LEAN CLAY CL		27	17	10			
Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● BH-2 4.6	2.36	0.05	0.008		0.0	11.0	63.8	25.2
☒ BH-7 4.6	1.18	0.05	0.007		0.0	8.5	65.3	26.2

PROJECT **Geotechnical Investigation - NW Quadrant Road of Cambrian Road and Greenbank Road, Barrhaven, Ontario** JOB NO. **4817-18-G-LPL-A** DATE **5/10/18**

GRADATION CURVES
Toronto Inspection Ltd.

FIGURE NO. 1

Figure No. 3 - Void Ratio vs Log Pressure

Project : Proposed Retail Buildings
 Location : NW-Cambrian Road and Greenbank Road, Barrhaven, Ontario
 Sample Description : Silty Clay / Clayey Silt, BH-4, at a depth of 6.6m (20.5')

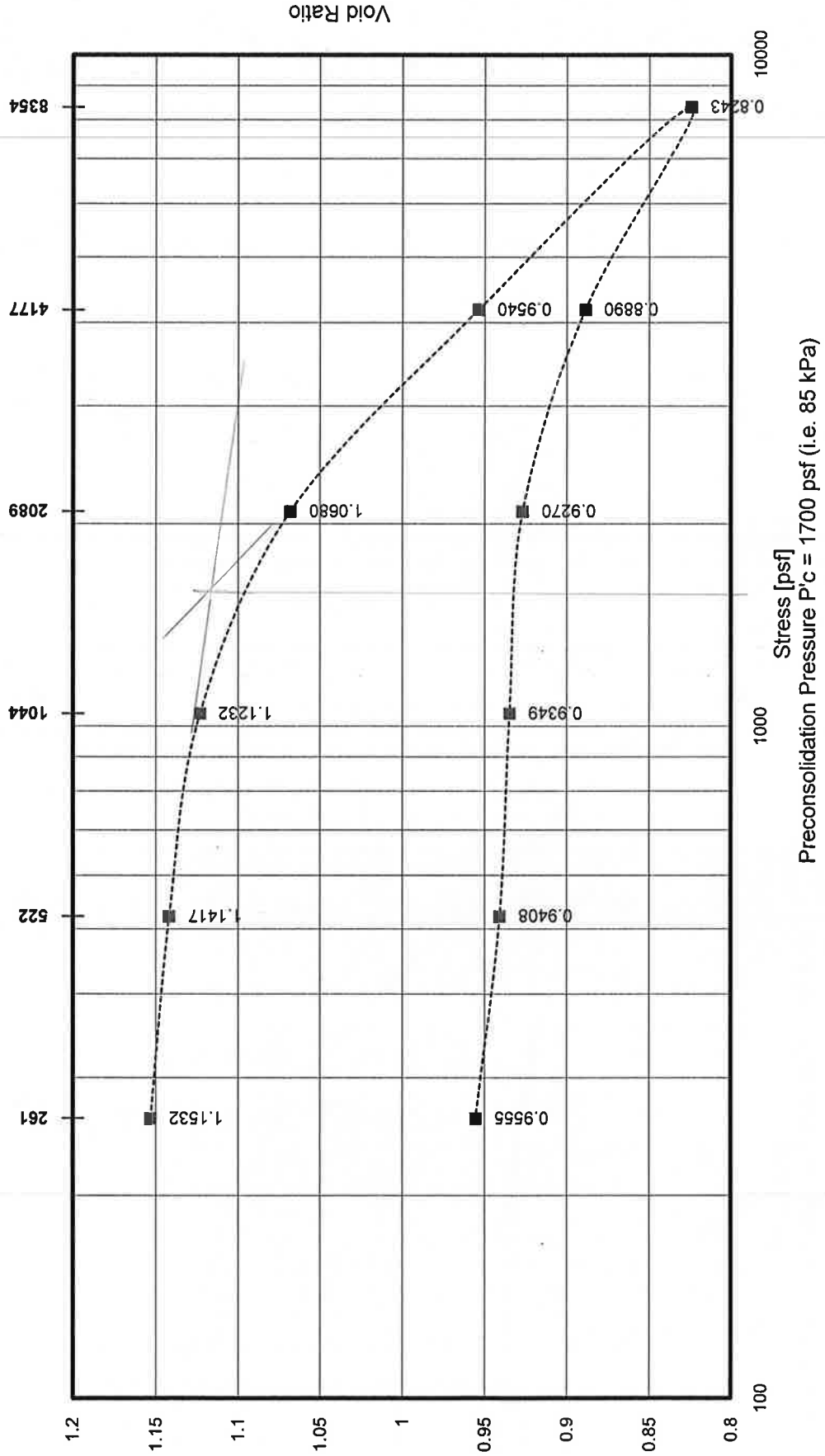
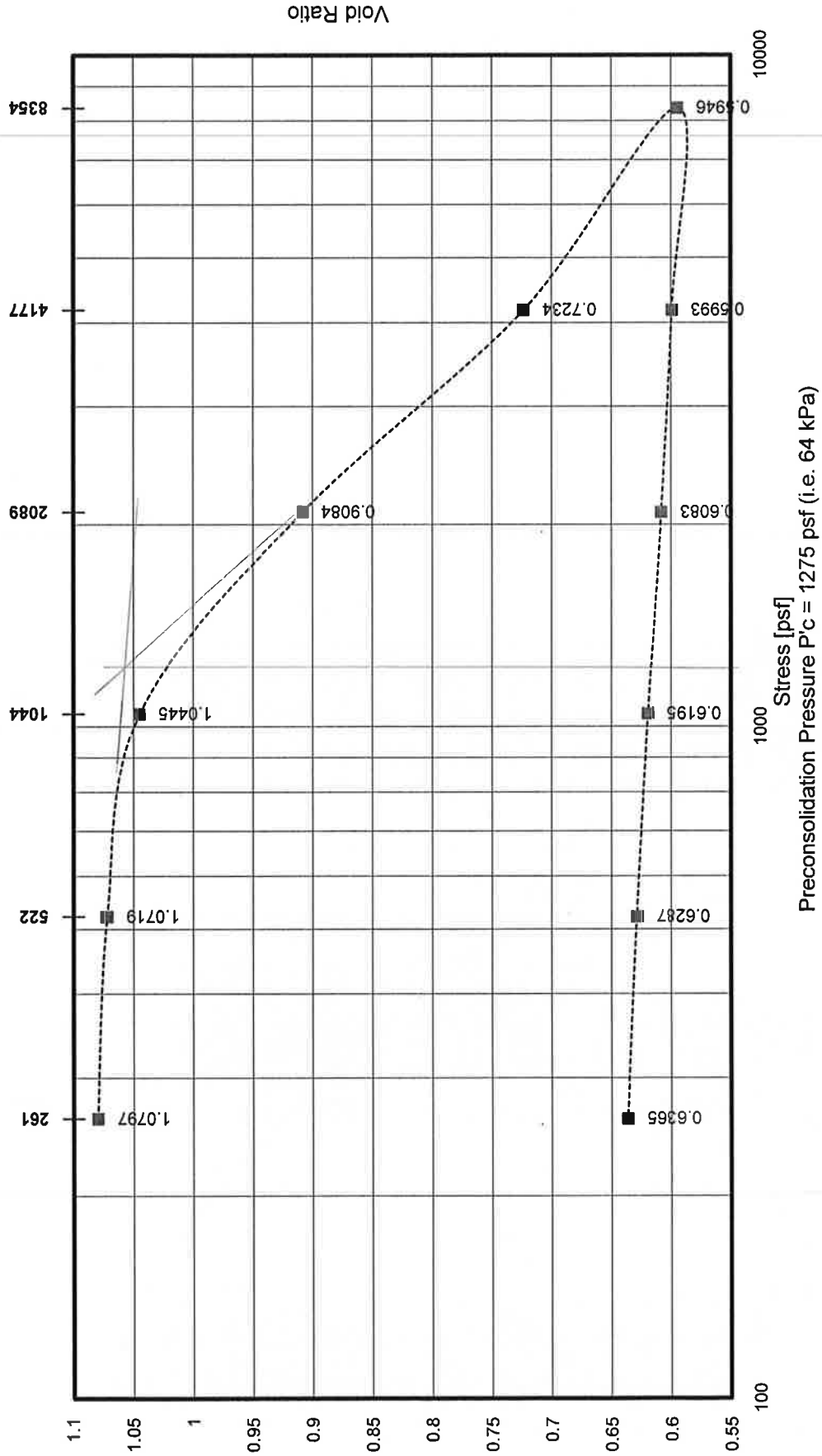
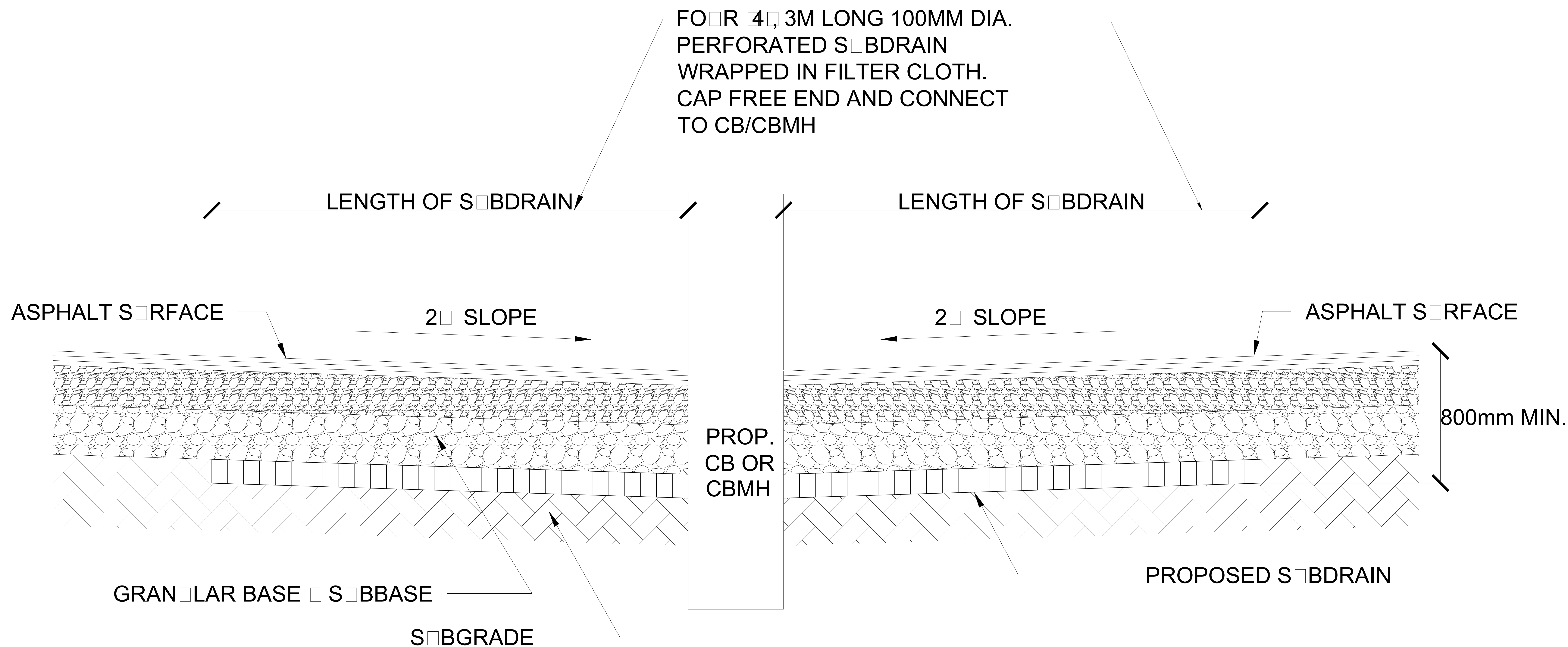


Figure No. 4 - Void Ratio vs Log Pressure

Project : Proposed Retail Buildings
Location : SW-Cambrian Road and Greenbank Road, Barrhaven, Ontario
Sample Description : Silty Clay / Clayey Silt, BH-11, at a depth of 4.3m (14')





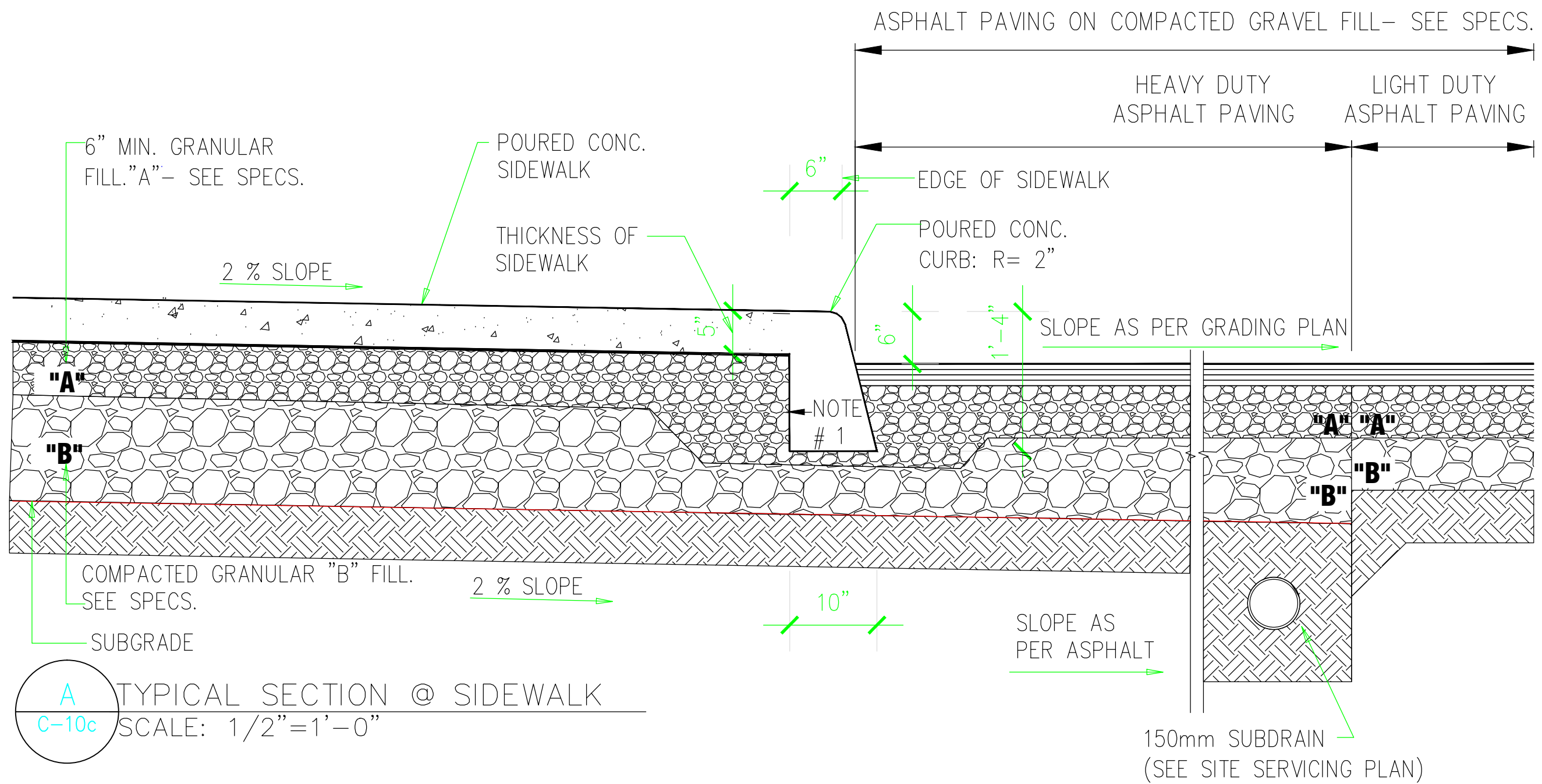
NOT TO SCALE

GENERAL NOTES:

- G1. SIDEWALK SLOPES TO BE 1/8" / FT. (1%) MAX. IN ALL DIRECTIONS ALONG STOREFRONT, 1/4" / FT. (2%) MAX. IN OTHER AREAS.
- G2. EXPANSION JOINTS ARE TO BE AS SPECIFIED & AS LOCATED ON ARCHITECTURAL DOCUMENTS.
- G3. REINFORCING TO BE AS PER STRUCTURAL DOCUMENTS.
- G4. CONCRETE TO BE BROOM FINISHED AS PER ARCHITECTURAL SPECIFICATIONS.

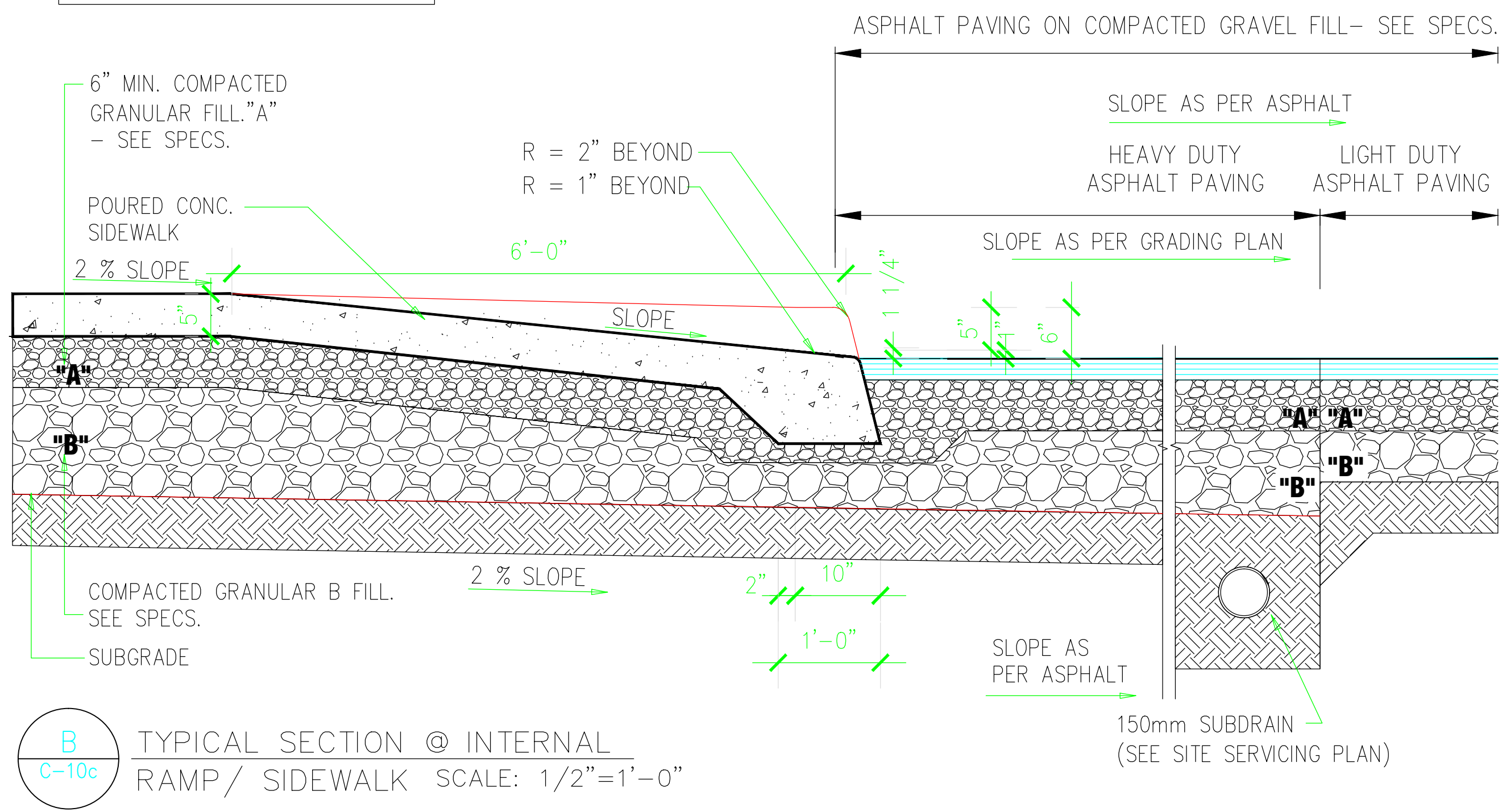
NOTES:

- 1 FOR SLIPFORMING PROCEDURE A 5% BATTER IS ACCEPTABLE
- A TREATMENT AT ENTRANCES SHALL CONFORM WITH OPSD-351.01
- B OUTLET TREATMENT SHALL CONFORM WITH OPSD-610 SERIES
- C THE LENGTH OF TRANSITION FROM ONE CURB TYPE TO ANOTHER SHALL BE 3.0M, EXCEPT IN CONJUNCTION WITH GUIDE RAIL, IT SHALL CONFORM TO OPSD-900 SERIES



A TYPICAL SECTION @ SIDEWALK
 C-10c SCALE: 1/2" = 1'-0"

REFER TO SITE GRADING PLAN



B TYPICAL SECTION @ INTERNAL RAMP / SIDEWALK
 C-10c SCALE: 1/2" = 1'-0"

NOT TO SCALE



Toronto Inspection Ltd.

APPENDIX A
Guidelines of Engineered Fill

GUIDELINES FOR ENGINEERED FILL

The information presented in this guideline is intended for general guidance only. Site specific and prevailing weather conditions may require modification of the material(s) to be used and the compaction standards or procedures changed. The site preparation and the material(s) to be used must be discussed and procedures agreed with *Toronto Inspection Ltd.* prior to the start of the earthworks and must be subjected to on going review during construction.

For fill to be classified as engineered fill, suitable for supporting structural loads, a number of conditions must be satisfied, including but not necessarily limited to the following:

1. Areal Extent

The engineered fill must extend beyond the envelope of the structure to be supported. The minimum extent should be 2.0m beyond the envelope in all directions at the foundation level, including the loading dock pad and the front sidewalk, and sloping downwards to the sub-grade at 45°. Once the envelope is set, the structure cannot be moved out of the envelope without consultation with *Toronto Inspection Ltd.* Similarly, no excavation should encroach on the engineered fill envelope without consultation with *Toronto Inspection Ltd.*

2. Survey Control

Accurate survey control is essential to the success of an engineered fill project. The boundaries of the engineered fill must be laid out by a surveyor. During construction, it is necessary to have qualified surveyors providing control stations on the three-dimensional extent of the engineered fill.

3. Subsurface Preparation

Prior to placement of the engineered fill, the sub-grade must be prepared to the satisfaction of *Toronto Inspection Ltd.* All deleterious material must be removed and in some cases excavation of native mineral soils may also be required. Particular attention must be paid to wet sub-grade and possible additional measures required to achieve sufficient compaction. Where fill is placed against a slope, benching will be necessary and natural drainage paths must not be blocked.

4. Suitable Fill Material

All material to be used as fill must be approved by *Toronto Inspection Ltd.* Such approval will be influenced by weather factors. External sources of fill material must be sampled, tested and approved prior to material being hauled to the job site.

5. Trial Test Section

In advance of the construction of the engineered fill pad, the contractor should conduct a trial test section. The compaction criterion will be assessed for the backfill material to be used, using specified lift thicknesses and number of passes for the compaction equipment proposed by the contractor. To achieve a uniform degree of compaction of each layer, the lift thickness of loose

material, prior to start of compaction, must not exceed 200mm (8 inches). Additional trial test section(s) may be required throughout the course of the project to reflect changes in material sources, the moisture content of the material and the weather conditions.

6. Degree of Compaction

The minimum degree of compaction for the engineered fill should not be less than 100% of the Standard Proctor maximum dry density, or 95% of the Modified Proctor maximum dry density, to the level at or above 0.3m from proposed footing founding level. Each layer must be tested and approved by this office before the next layer is placed.

7. Inspection and Testing

Uniform and thorough compaction is crucial to the performance of the fill and the supported structure. Hence, all subgrade preparation, filling and compacting must be done with full time inspection and to the satisfaction of *Toronto Inspection Ltd.* All founding surfaces must be inspected and approved by *Toronto Inspection Ltd.* prior to placement of concrete.

8. Protection of Fill

Fills are generally more susceptible to the effects of weather than are natural soils. Fill placed and approved to the level at which structural support is required must be protected from excessive wetting, drying, erosion or freezing. Where inadequate protection had been provided, it may be necessary to provide deeper founding level for footings or to strip and re-compact some of the filled layers.

9. Limitations

The engineered fill is subjected to the following limitations:

- i. Proper drainage must be maintained at all times within the engineered fill pad.
- ii. If the engineered fill is left in place during the winter months, adequate protection must be provided against frost penetration to the proposed footing depths.
- iii. If the engineered fill depth exceeds 5m below the foundation depth, the construction of the foundations might have to be delayed for a period of 1 year after placement, depending on the type of fill material used.
- iv. Strip footings and foundation walls founded on engineered fill must be reinforced continuously with a minimum of two 15mm steel bars with at least 1m of overlap.