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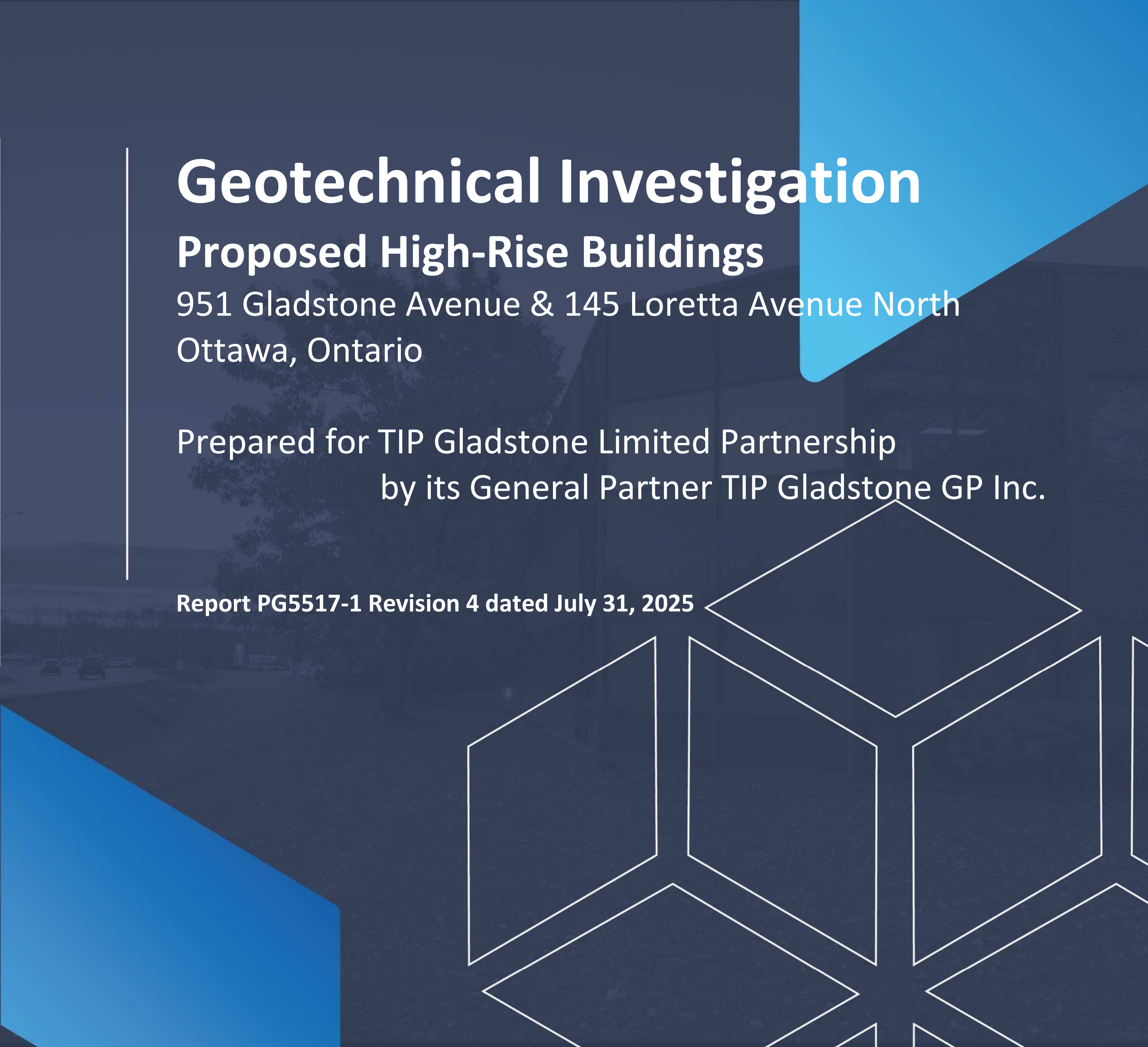

Geotechnical Investigation

Proposed High-Rise Buildings

951 Gladstone Avenue & 145 Loretta Avenue North
Ottawa, Ontario

Prepared for TIP Gladstone Limited Partnership
by its General Partner TIP Gladstone GP Inc.

Report PG5517-1 Revision 4 dated July 31, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by TIP Gladstone Limited Partnership by its General Partner TIP Gladstone GP Inc. to conduct a geotechnical investigation for the proposed development to be located at 951 Gladstone Avenue and 145 Loretta Avenue North in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

☐ Determine the subsurface soil and groundwater conditions by means of boreholes and to;

☐ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Development

Based on the currently available drawings, it is understood that the proposed development will consist of high-rise buildings with 3 levels of underground parking.

Phase 1 will consist of Tower A, a 35 storey building. Future phases are expected to include Tower B and Tower C with 38 and 40 storeys, respectively.

At finished grades, the proposed buildings will be surrounded by paver walkways and asphalt-paved access lanes with landscaped areas. The proposed development is expected to be municipally serviced.

It is further understood that the existing 3-storey Standard Bread Building located at the south-east corner of the subject site will remain as part of the proposed development, however, the other existing buildings on-site will be demolished.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation was carried out on September 14, September 22, and September 23, 2020. At that time, 5 boreholes (BH 1 through BH 5) were advanced to a maximum depth of 12.2 m below the existing ground surface. A previous geotechnical investigation by others during July 2017 included 13 boreholes advanced throughout the subject site to a maximum depth of 16.6 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the test holes are shown on Drawing PG5517-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. The samples were then transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Standard Penetration Tests (SPT) were conducted and recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sample 300 mm into the soil after the initial penetration of 150 mm using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling was completed at all boreholes as part of the current investigation, with the exception of BH2, to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage. The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

The subsurface conditions observed in the boreholes were recorded in detail in the field and are presented on the Soil Profile and Test Data sheets in Appendix 1.

Groundwater

Groundwater monitoring wells were installed in all boreholes completed as part of the current investigation to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The test hole locations and elevations were surveyed in the field by Paterson. The ground surface elevations at the test hole locations were referenced to a geodetic datum. The borehole locations and the ground surface elevation of the borehole locations are presented on Drawing PG5517-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and bedrock samples recovered from the subject site were visually examined in our laboratory to review the field logs. Laboratory testing consisting of Atterberg Limits, grain size distributions, and rock core unconfined compressive strength testing was also conducted by others as part of the previous geotechnical investigation at the site. The results of the laboratory testing by others are provided in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and are discussed in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is located at the northeast corner of the intersection of Gladstone Avenue and Loretta Avenue North. The site is currently occupied by several one and two-storey commercial buildings, and a three-storey commercial building. The buildings are generally surrounded by asphalt-paved access lanes and parking areas.

The site is bordered by the Trillium Rail Corridor to the east, Loretta Avenue North to the west, a commercial property to the north, and Gladstone Avenue to the south. The existing ground surface across the site slopes gradually from south to north from approximate geodetic elevation of 67 to 64 m.

It is also understood that a 1,372 mm diameter watermain is located underlying Loretta Avenue and Gladstone Avenue in the vicinity of the subject site.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of asphalt underlain by fill extending to an approximate depth of 2.2 to 3.9 m below the existing ground surface. The fill was generally observed to consist of a compact brown silty sand or silty clay with crushed stone.

A silty clay deposit was encountered underlying the fill. This deposit was observed to consist of a very stiff to stiff, brown silty clay, becoming a firm to stiff, grey silty clay with depth.

Glacial till was encountered underlying the silty clay deposit below approximate depths of 3.8 to 6.9 m. The glacial till was observed to consist of interbedded layers of compact grey sandy silt, silty sand, sand and/or silty clay with some gravel, and occasional cobbles.

Specific details of the subsoil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Bedrock was encountered underlying the overburden at approximate depths of 4.6 m at the south end of the site, increasing to depths of 8.5 m at the north end of the site.

The bedrock was cored at all boreholes during the current investigation, with the exception of BH 2, to approximate depths ranging from 10.6 to 12.2 m and was observed to consist of limestone with interbedded shale. Based on the RQDs of the recovered rock core, the bedrock generally increases in quality from poor to excellent with depth.

Based on available geological mapping, the bedrock at the subject site consists of limestone and dolomite of Verulam formation.

4.3 Groundwater

Groundwater levels were measured on September 30, 2020 for boreholes completed as part of the current investigation. The results are presented in Table 1. It should be further noted that the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH1	64.97	5.03	59.94	Sept 30, 2020
BH2	66.79	5.05	61.74	
BH3	64.24	4.18	60.06	
BH4	64.46	4.60	59.86	
BH9	64.92	4.82	60.10	

Note: Ground surface elevations at monitoring well locations were surveyed by others using a temporary benchmark of 100 m.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes.

Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 4.5 to 5.5 m below ground surface within the low permeability silty clay and glacial till layer. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed buildings be founded on conventional spread footing foundations placed on clean, surface sounded bedrock, or on lean concrete in-filled trenches which extend to the clean, surface sounded bedrock.

Bedrock removal will be required to complete the proposed underground parking levels and site servicing, particularly on the south end of the site. All contractors should be prepared for bedrock removal within the subject site.

Due to a 1,372 mm diameter watermain and trunk sewers located in the vicinity of the subject site, a vibration monitoring program will be required to include these utilities.

Due to the presence of a silty clay layer, the proposed development will be subjected to grade raise restrictions. Our permissible grade raise recommendations are discussed in Section 5.3.

The above and other considerations are discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. However, the site excavation is expected to occupy the majority of the site to a depth significantly below the existing grade, therefore, all topsoil and fill materials will be removed from within the perimeter of the proposed building.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming where large quantities of bedrock need to be removed.

Prior to considering blasting operations, the blasting effects on the existing services, buildings, and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in the proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries or claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations must be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Watermain Monitoring Program

The following vibration monitoring program is recommended to ensure that excessive movements and vibrations do not occur at the watermain location:

☐ Install 2 inclinometers located adjacent to the 1,372 mm diameter watermain and the shoring face. Daily monitoring events should be completed during the excavation program until the tiebacks are stressed and then weekly during the construction program until the foundation extends above exterior finished grade. An alert level with 10 mm of movement will require an assessment. An action level with movement greater than 15 mm will require immediate attention and possible mitigation measures. A visual inspection of the excavation side slopes will also be completed along with the inclinometer monitoring events.

☐ Periodically monitor the vibration levels within an existing valve chamber along the subject section of watermain. If the vibration monitor cannot be placed within the valve chamber, the monitor will be placed at ground surface in the immediate area of shoring works.

☐ If the vibration limits noted in Table 2 are exceeded, the site superintendent will be notified by Paterson personnel of the exceedance and the shoring/excavation operation will be stopped. The project surveyor will survey the watermain level (within the valve chamber) to ensure pipe movement has not occurred. If pipe movement is not observed based on the survey results, the shoring/excavation operation will resume.

The vibration limits in Table 2 on the next page are recommended for the shoring/excavation operation to be completed adjacent to the 1,372 mm diameter watermain.

Table 2 - Vibration Limits for Work Completed Adjacent to Watermain

Location of Vibration Monitor	Peak Particle Velocity (mm/s)	Frequency (Hz)
Inside the Valve Chamber	15	4 to 12
	25	>40
At Ground Surface (within 3 m of watermain)	10	4 to 12
	25	>40

Note: The values should be interpolated between 12 and 40 Hz.

Weekly reporting of our findings and recommendations will be provided to the owner and the City of Ottawa. Any mitigation measures contemplated for implementation will be discussed with the owner and City of Ottawa personnel. A detailed Vibration Monitoring and Control Plan (VMCP) will be prepared by Paterson prior to construction which will contain additional details about the vibration monitoring program.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids.

Lean Concrete Filled Trenches

Where bedrock overbreak occurs below the design underside of footing elevation (USF), lean concrete (minimum **17 MPa** 28-day compressive strength) can be used to reinstate grades from the clean-surface sounded bedrock up to the USF elevation. Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on the clean, surface sounded bedrock, or on lean concrete which is supported directly on the clean, surface sounded bedrock, **at or below geodetic elevation 53.5 m** can be designed for a factored bearing resistance value at serviceability limit states (SLS) and ultimate limit states (ULS) of **4,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

Footings with a **USF above elevation 53.5 m**, supported on clean, surface-sounded bedrock, or on lean concrete placed directly over such rock, can be designed using a bearing resistance value of **3,000 kPa** under both serviceability limit states (SLS) and ultimate limit states (ULS) conditions.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and should not contain surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Footings bearing directly or indirectly on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **2 m** is recommended for grading at the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

Seismic shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed buildings in accordance with Table 4.1.8.4.-B of the Ontario Building Code (OBC) 2024. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity testing are provided on Figures 3 and 4 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as shown on Drawing PG5517-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 1 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations were 1.5 and 15 m away from the first geophone and 1.5 and 10 m away from the last geophone of the seismic array.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods.

The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the foundation of the buildings.

The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on our testing results, the bedrock shear wave velocity is **2,416 m/s**. Further, it is expected that footings will be founded directly or indirectly (lean concrete trenches) on the bedrock surface for all buildings.

Based on the above, the V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below.

$$V_{s30} = \frac{Depth_{of\ interest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{s_{Layer1}}(m/s)} + \frac{Depth_{Layer2}(m)}{V_{s_{Layer2}}(m/s)}\right)}$$

$$V_{s30=} \frac{30\ m}{\left(\frac{30\ m}{2{,}416\, m/s}\right)}$$

$$V_{s30=}\ 2{,}416\, m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} for the proposed buildings with foundations bearing directly on the bedrock surface is **2,416 m/s**. Therefore, a **Site Class X_{2416}** is applicable for design of the proposed buildings, as per Table 4.1.8.4.-B of the OBC 2024. The soil underlying the subject site is not susceptible to liquefaction.

5.5 Basement Floor Slab

With the removal of all topsoil and deleterious fill from within the footprints of the proposed buildings, the bedrock will be considered an acceptable subgrade on which to commence backfilling for basement slab construction.

It is anticipated that the lowest underground level for the proposed building will be mostly parking, and the recommended pavement structures noted in Section 5.7 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 300 mm of underslab fill is recommended to consist of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the anticipated groundwater conditions, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided underlying the lowest level floor slab. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed building. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained material
γ = unit weight of fill of the applicable retained material (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a \cdot H^2/g$ where:

$a_c = (1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.31g according to OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o \cdot \gamma \cdot H^2$, where K = 0.5 for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes.

The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that centre to centre spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Based on the testing results completed by others, the unconfined compressive strength of the limestone bedrock below the subject site ranges between 95 and 125 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 69** was assigned to the bedrock, and Hoek and Brown parameters **(m and s)** were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required.

For our calculations the following parameters were used.

Table 3 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR)-Good quality Limestone Hoek and Brown parameters	69 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	60 MPa
Unit weight - Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 4, on the next page.

Table 4 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	3.2	1.2	4.4	750
	4.5	2	6.5	1000
	7.5	2.5	10	1750
	10	3	13	2250
125	2.3	0.9	3.2	900
	3	1.3	4.3	1200
	6	2.2	8.2	2250
	8.6	2.8	11.4	3250

Based on discussions with the structural engineer, it is understood that rock anchors are required for foundation uplift resistance. In summary, there are single rock anchors along with groups of 2 and 3. The specific free length and bond length for each rock anchor in the various scenarios are provided below:

- Single Rock Anchors
  Free Length: 1.5 m
  Bonded Length: 7.0 m

- Groups of 2
  Free Length: 2.5 m
  Bonded Length: 7.0 m

- Groups of 3
  Free Length: 3.0 m
  Bonded Length: 7.0 m

Each rock anchor should consist of a 75 mm cold-rolled, steel threadbar (Yield Strength 830 MPa and Ultimate Strength 1035 MPa) with 40 MPa grout and installed in a minimum 150 mm diameter drilled hole.

The technical data sheet from Dywidag Systems International (DSI) is attached for reference, although equivalent steel threadbars from other manufacturers are also considered acceptable.

Each rock anchor installed in this manner will have a capacity of 3,350 kN under serviceability limit states (SLS) and ultimate limit states (ULS) conditions.

Other considerations

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Design

Lowest Underground Parking Level

For design purposes, it is recommended that the rigid pavement structure for the lowest underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 5, on the next page.

Table 5 - Recommended Rigid Pavement Structure – Underground Parking Level

Thickness (mm)	**Material Description**
150	**Exposure Class C2** – **32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m).

The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Pavement Structure Over Podium Deck

The pavement structures presented in Tables 6 and 7 should be used for car only parking areas, at grade access lanes and heavy loading parking areas over the top of the podium structure.

Table 6 - Recommended Pavement Structure - Car Only Parking Areas Over Podium Deck

Thickness (mm)	**Material Description**
50	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
200*	**BASE** - OPSS Granular A Crushed Stone
See below**	**Thermal Break** -** Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**

SUBGRADE – Reinforced concrete podium deck

* Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph

** If specified by others, not required from a geotechnical perspective

Table 7 - Recommended Pavement Structure – Access Lanes, Fire Truck Lane, Ramp, and Heavy Loading Areas Over Podium Deck

Thickness (mm)	**Material Description**
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
300*	**BASE** - OPSS Granular A Crushed Stone
See below**	**Thermal Break** -** Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**

SUBGRADE – Reinforced concrete podium deck

* Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph

** If specified by others, not required from a geotechnical perspective

Pavement Structure over Soil Subgrade

The following pavement structures in Tables 8 and 9 given below may be used for car only parking and heavy traffic areas on a soil subgrade.

Table 8 - Recommended Pavement Structure – Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II
SUBGRADE – Either in-situ soils, existing imported fill or OPSS Granular B Type I or II material placed over in-situ soil or bedrock.	

Table 9 - Recommended Pavement Structure – Access Lanes, Garage Ramp and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II
SUBGRADE – Either in-situ soils, existing imported fill or OPSS Granular B Type I or II material placed over in-situ soil or bedrock.	

Other Considerations

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Water Suppression System and Foundation Drainage

For the proposed underground parking levels, it is understood that the building foundation walls will be placed in close proximity to the site boundaries. Therefore, it is recommended that the foundation walls be blind poured against a drainage system and waterproofing system fastened to the shoring system or vertical bedrock face. To manage and control groundwater water infiltration over the long term, the following water suppression system is recommended to be installed for the exterior foundation walls:

- ☐ A waterproofing membrane will be required to lessen the effect of water infiltration for the lower underground parking levels starting from a geodetic elevation of 62 m. The waterproofing membrane will consist of a bentonite waterproofing such as Tremco Paraseal, or equivalent, which is securely fastened to the temporary shoring system or the vertical bedrock surface. The membrane should extend to the bottom of the excavation at the founding level and extend horizontally over the bedrock surface a minimum of 600 mm prior to the placement of the footings.

- ☐ A composite drainage layer will be placed from finished grade to the bottom of the foundation wall. It is recommended that the composite drainage system (such as DeltaDrain 6000, MiraDrain G100N or equivalent) extend down to the bottom of the foundation. It is recommended that 150 mm diameter sleeves placed at 3 m centres be cast in the concrete footings or in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to the sump pit(s) within the lower basement area. Water infiltration will result from two sources. The first will be water infiltration for the portion of the foundation walls above the waterproofing membrane. The second source will be water breaching the waterproofing membrane.

Reference should be made to Figure 2 - Foundation Drainage and Water Suppression System in Appendix 2 for an overview of the proposed foundation waterproofing and drainage system. A groundwater infiltration system should also be provided for any elevator shafts and sump pump pits (pit bottoms and walls) located within the lowest basement level.

Underslab Drainage

Sub-slab drainage will be required to control water infiltration below the lowest level floor slab. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where space is available for conventional wall construction, backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular A, should be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

It is anticipated that temporary shoring will be required to support the overburden soils.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. The shoring designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation. The design of the temporary shoring system should also take into consideration sub-excavation under proposed footings which may be required to extend to the bedrock surface for the placement of lean concrete.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist to failure, if required, by means of tieback anchors or extending the piles into the bedrock through pre-augured holes, if a soldier pile and lagging system is the preferred method.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The temporary shoring system design should also consider that trenches excavated to the bedrock for lean concrete placement may be excavated in close proximity to the temporary shoring system.

The earth pressures acting on the shoring system may be calculated with the parameters provided in Table 10, on the next page.

Table 10 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight , kN/m_3	20
Submerged Unit Weight , kN/m_3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Bedrock Stabilization

Where required, excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

Horizontal rock anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The requirement for temporary rock anchors, shotcrete, and/or chainlink fencing should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage of the project.

6.4 Pipe Bedding and Backfill

A minimum of 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. The bedding layer should be increased to a minimum of 300 mm of OPSS Granular A when placed on bedrock subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the pipe obvert should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 99% of the SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Due to the existing groundwater level and inferred depths of the proposed footings, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps.

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, and EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Long-term Groundwater Control

Our recommendations for the proposed building’s long-term groundwater control are presented in Subsection 6.1. Any groundwater which breaches the building’s perimeter groundwater infiltration control system will be directed to the sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that long-term groundwater flow will be very low to negligible (ie.- less than 25,000 L/day).

Impacts on Neighbouring Properties

Given the elevation of the groundwater encountered in the monitoring wells, and the anticipated depths of excavation, minimal dewatering is anticipated during the construction period. Further, for the permanent condition, the lower portion of the foundation will have a groundwater infiltration control system in place.

Due to the presence of a groundwater infiltration control system, long-term groundwater lowering is anticipated to be negligible for the area. Therefore, no adverse effects to neighbouring properties or nearby utilities are expected.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a very aggressive corrosive environment.

6.8 Slope Stability Assessment

A slope stability assessment has been conducted to determine the geotechnical slope stability for the proposed conditions at the subject site, given that there is more than a 2 m grade difference from north to south across the site.

One slope cross-section (Section A) was studied for the proposed conditions at the site under static and seismic conditions. It should be noted that assumptions were made for finished grades based on surrounding road grades and borehole elevation data collected from the geotechnical investigation. Actual finished grades planned for the proposed development were not available at the time of preparation of this report. The cross-section location is presented on Drawing PG5517-1 - Test Hole Location Plan, which is included in Appendix 2. The analysis is discussed further below.

Slope Stability Analysis

The slope stability analysis for the proposed conditions was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods, including the Bishop's simplified method which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favouring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is marginally stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation. The effective strength soil parameters used for static analysis are presented in Table 11 below.

Table 11 - Effective Strength Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m³,	Friction Angle (degrees)	Cohesion (kPa)
Fill	19	33	2
Brown Silty Clay Crust	17	33	5
Glacial Till	19	35	0
Bedrock	23	0	1000

The total strength parameters for seismic analysis were chosen based on the subsurface conditions encountered within the completed at the time of our geotechnical investigation, and based on our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 12 below:

Table 12 - Total Strength Soil and Material Parameters (Seismic Analysis)

Soil Layer	Unit Weight (kN/m^3,	Friction Angle (degrees)	Cohesion (kPa)
Fill	19	33	2
Brown Silty Clay Crust	17	0	80
Glacial Till	19	35	0
Bedrock	23	0	1000

Static Loading (Effective Strength) Analysis

A minimum factor of safety of 1.5 is generally recommended for static conditions where the failure of the slope would endanger permanent structures.

The slope stability analysis for static conditions was completed at the slope cross-section under a conservative scenario by assigning cohesive soils which are fully saturated.

The results of the static analysis at Section A are shown on the attached Figure 5 in Appendix 2. The results indicate that the factor of safety exceeds 1.5, and is considered acceptable from a geotechnical perspective.

Seismic Loading (Total Stress) Analysis

An analysis considering seismic loading for the proposed site conditions was also completed at Section A. A horizontal seismic coefficient of 0.16 g was considered for the slope. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the seismic analysis for Section A are shown on Figure 6 in Appendix 2. The results indicate that the factor of safety exceeds 1.1 and is considered acceptable, from a geotechnical perspective.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

☐ Review the grading plan, from a geotechnical perspective.

☐ Review the water suppression system design and implementation.

☐ Review proposed foundation drainage design and requirements.

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Sampling and testing of the concrete and fill materials used.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than TIP Gladstone Limited Partnership by its General Partner TIP Gladstone GP Inc., or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Deepak k Rajendran, E.I.T.

LICENSED PROFESSIONAL ENGINEER
July 31, 2025
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Scott Dennis
PROVINCE OF ONTARIO

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Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA SHEETS

LABORATORY TESTING BY OTHERS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
951 Gladstone Avenue and 145 Loretta Avenue North
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 14, 2020

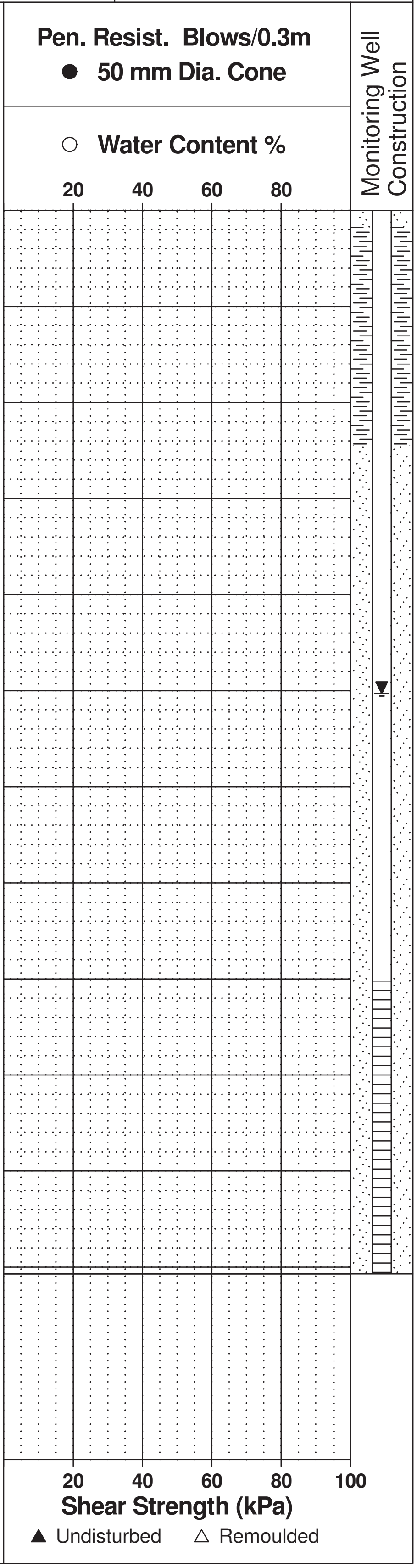
FILE NO. PG5517

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand with shale, some gravel, cobbles, trace wood	3.45
Very stiff to stiff, brown **SILTY CLAY**	5.33
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	6.86
GLACIAL TILL: Grey sandy silt with clay, gravel, cobbles and boulders	7.03
BEDROCK: Fair to good quality, grey limestone with interbedded shale	11.07
End of Borehole (GWL @ 5.03m - Sept. 30, 2020)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	17	4
SS	3	21	10
SS	4	46	32
SS	5	50	27
SS	6	88	9
SS	7	100	9
SS	8	38	9
SS	9	100	3
SS	10	50	50+
RC	1	100	57
RC	2	100	73
RC	3	100	90
RC	4	100	82

DEPTH (m)	ELEV. (m)
0	64.97
1	63.97
2	62.97
3	61.97
4	60.97
5	59.97
6	58.97
7	57.97
8	56.97
9	55.97
10	54.97
11	53.97



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
951 Gladstone Avenue and 145 Loretta Avenue North
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 14, 2020

FILE NO. PG5517

HOLE NO. BH 2

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Monitoring Well Construction

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone	0.66
FILL: Brown silty sand with gravel, some crushed stone	2.29
Very stiff, brown **SILTY CLAY**	3.81
GLACIAL TILL: Compact, brown silty clay, some gravel, cobbles and boulders	4.57
GLACIAL TILL: Compact to dense, grey silty sand, some clay, gravel, cobbles and boulders	6.17

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	6
SS	3	33	8
SS	4	92	15
SS	5	100	13
SS	6	46	18
SS	7	62	11
SS	8	38	8
SS	9	0	50+

DEPTH (m)	ELEV. (m)
0	66.79
1	65.79
2	64.79
3	63.79
4	62.79
5	61.79
6	60.79

End of Borehole

Practical refusal to augering at 6.17m depth

(GWL @ 5.05m - Sept. 30, 2020)

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
951 Gladstone Avenue and 145 Loretta Avenue North
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 22, 2020

FILE NO. PG5517

HOLE NO. BH 3

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08	AU	1		
FILL: Brown silty sand with crushed stone		SS	2	58	11
	2.29	SS	3	29	14
FILL: Brown silty clay, trace sand and gravel	3.05	SS	4	12	11
FILL: Brown silty sand with clay, trace gravel	3.50	SS	5	71	10
Very stiff to stiff, brown **SILTY CLAY**		SS	6	79	13
		SS	7	100	6
- grey by 5.3m depth	6.10	SS	8	100	3
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders		SS	9	25	3
		SS	10	33	13
	7.72	SS	11	100	50+
BEDROCK: Good quality, grey limestone with interbedded shale		RC	1	82	75
		RC	2	100	55
	12.24	RC	3	100	85
End of Borehole					
(GWL @ 4.18m - Sept. 30, 2020)					

DEPTH (m)	ELEV. (m)
0	64.24
1	63.24
2	62.24
3	61.24
4	60.24
5	59.24
6	58.24
7	57.24
8	56.24
9	55.24
10	54.24
11	53.24
12	52.24

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
951 Gladstone Avenue and 145 Loretta Avenue North
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5517

REMARKS

HOLE NO. BH 4

BORINGS BY CME-55 Low Clearance Drill

DATE September 22, 2020

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with crushed stone	1.37
FILL: Brown silty clay with sand and gravel	3.81
Stiff, brown **SILTY CLAY** - grey with some sand by 6.1m depth	6.86
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders	8.33
BEDROCK: Good quality, grey limestone with interbedded shale	10.67
End of Borehole (GWL @ 4.60m - Sept. 30, 2020)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	27
SS	3	29	12
SS	4	4	23
SS	5	54	14
SS	6	100	8
SS	7	88	7
SS	6	100	4
SS	9	100	3
SS	10	33	26
SS	11	50	3
RC	1	97	75
RC	2	100	85

DEPTH (m)	ELEV. (m)
0	64.46
1	63.46
2	62.46
3	61.46
4	60.46
5	59.46
6	58.46
7	57.46
8	56.46
9	55.46
10	54.46

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

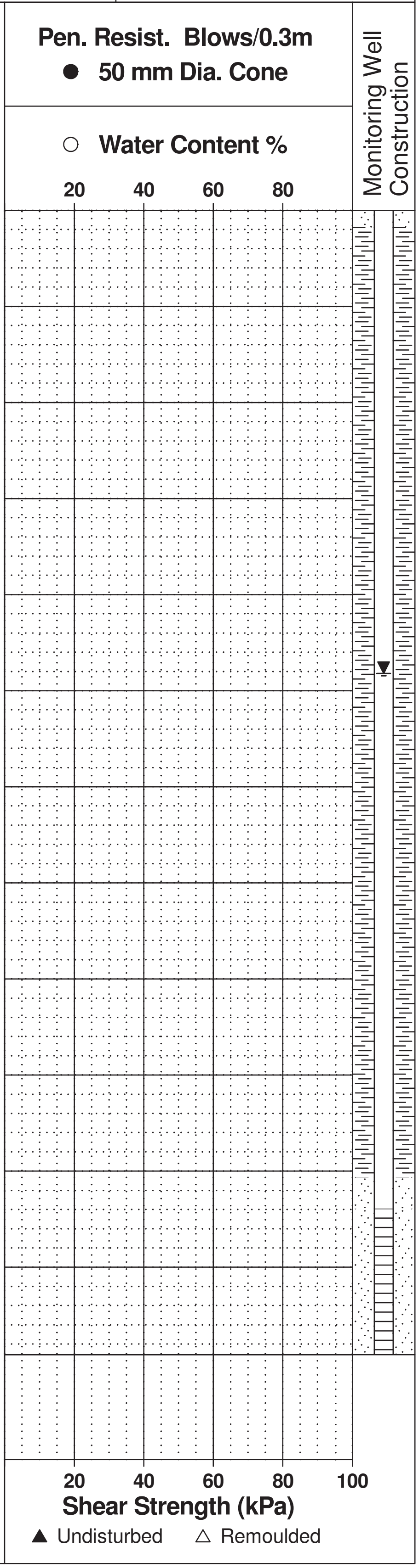
Geotechnical Investigation
951 Gladstone Avenue and 145 Loretta Avenue North
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE September 23, 2020
FILE NO. PG5517
HOLE NO. BH 5

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10	AU	1		
FILL: Brown silty sand with crushed stone	0.56				
FILL: Brown silty sand with crushed stone and gravel, trace clay	1.37	SS	2	21	9
FILL: Brown silty sand and gravel, trace cobbles	3.81	SS	3	29	27
		SS	4	46	27
		SS	5	58	47
Stiff, brown **SILTY CLAY**	4.57	SS	6	100	4
GLACIAL TILL: Grey silty clay, some sand, gravel, cobbles and boulders	6.10	SS	7	92	5
		SS	8	71	3
GLACIAL TILL: Dense, grey sandy silt with gravel, cobbles and boulders	6.93	SS	9	62	38
BEDROCK: Poor to excellent quality, grey limestone with interbedded shale	11.91	SS	10	100	50+
		RC	1	100	33
		RC	2	93	40
		RC	3	100	92
		RC	4	100	100
End of Borehole					

(GWL @ 4.82m - Sept. 30, 2020)

DEPTH (m)	ELEV. (m)
0	64.92
1	63.92
2	62.92
3	61.92
4	60.92
5	59.92
6	58.92
7	57.92
8	56.92
9	55.92
10	54.92
11	53.92



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = $D60 / D10$

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

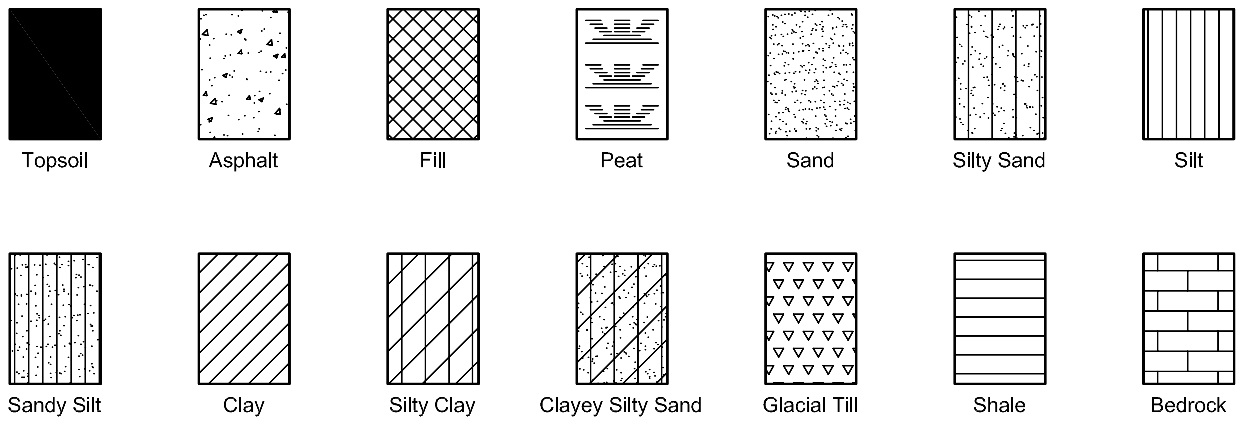
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

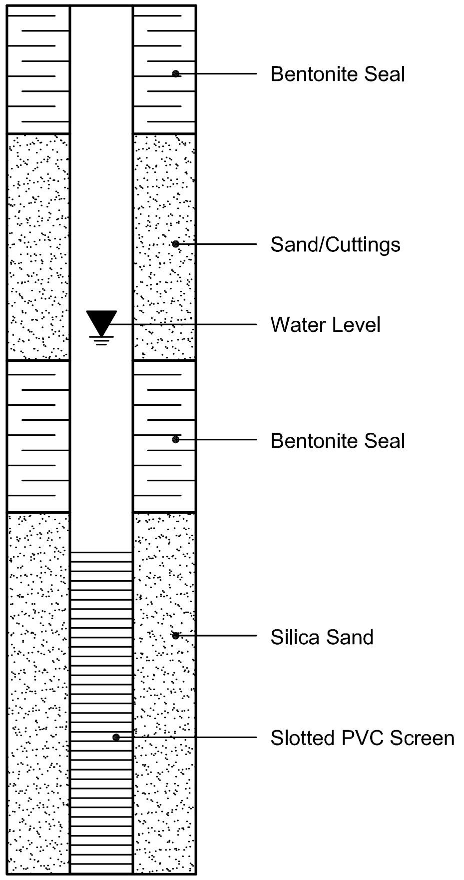
SYMBOLS AND TERMS (continued)

STRATA PLOT

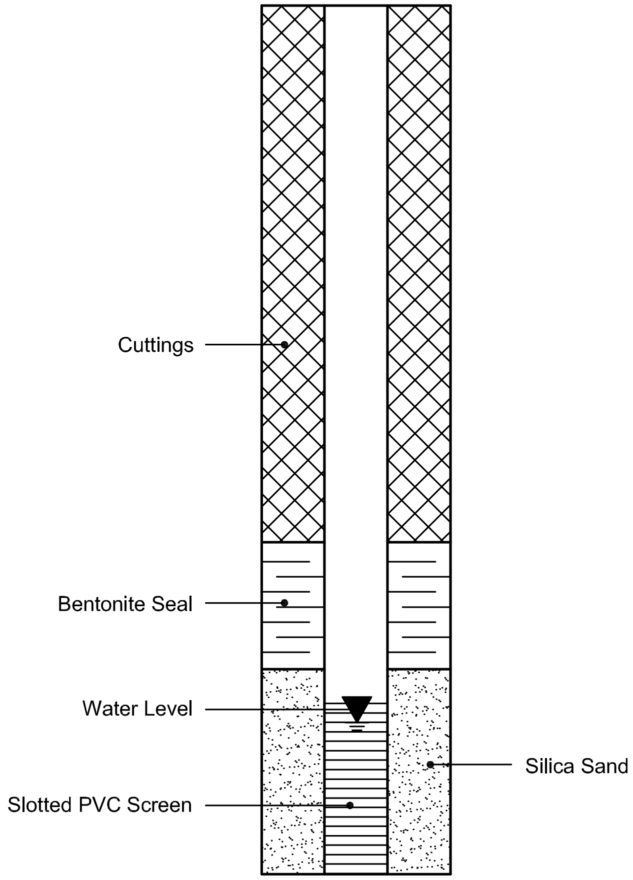


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



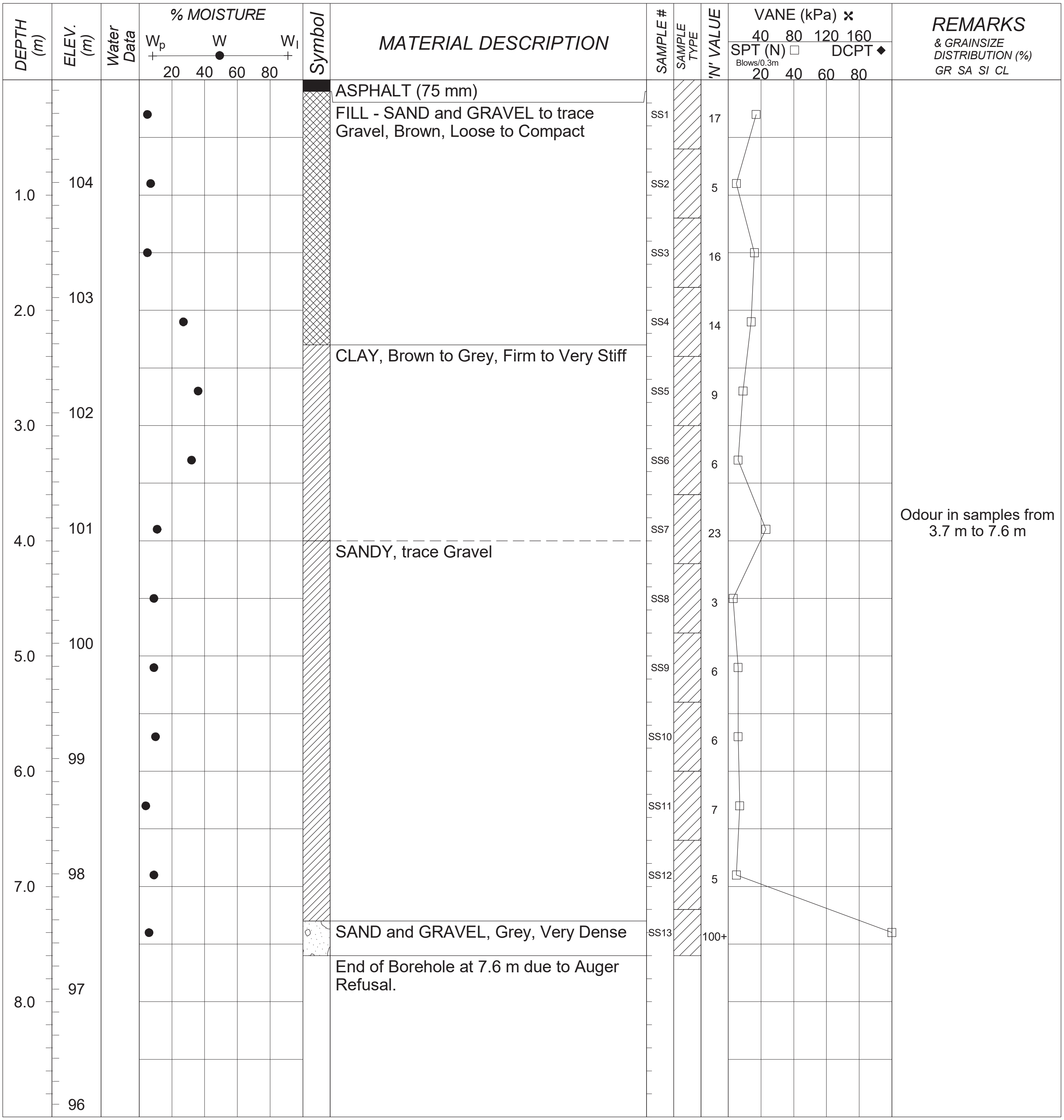
PIEZOMETER CONSTRUCTION



LOG OF BOREHOLE BH2017-01

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **104.9 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/5/2017**
COMPLETION DATE: **7/5/2017**
COORDINATES: **5028029** m N, **443991** m E



ASPHALT (75 mm)

FILL - SAND and GRAVEL to trace Gravel, Brown, Loose to Compact

CLAY, Brown to Grey, Firm to Very Stiff

SANDY, trace Gravel

SAND and GRAVEL, Grey, Very Dense

End of Borehole at 7.6 m due to Auger Refusal.

SAMPLE #	'N' VALUE
SS1	17
SS2	5
SS3	16
SS4	14
SS5	9
SS6	6
SS7	23
SS8	3
SS9	6
SS10	6
SS11	7
SS12	5
SS13	100+

REMARKS: Odour in samples from 3.7 m to 7.6 m

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SAMPLE TYPE LEGEND

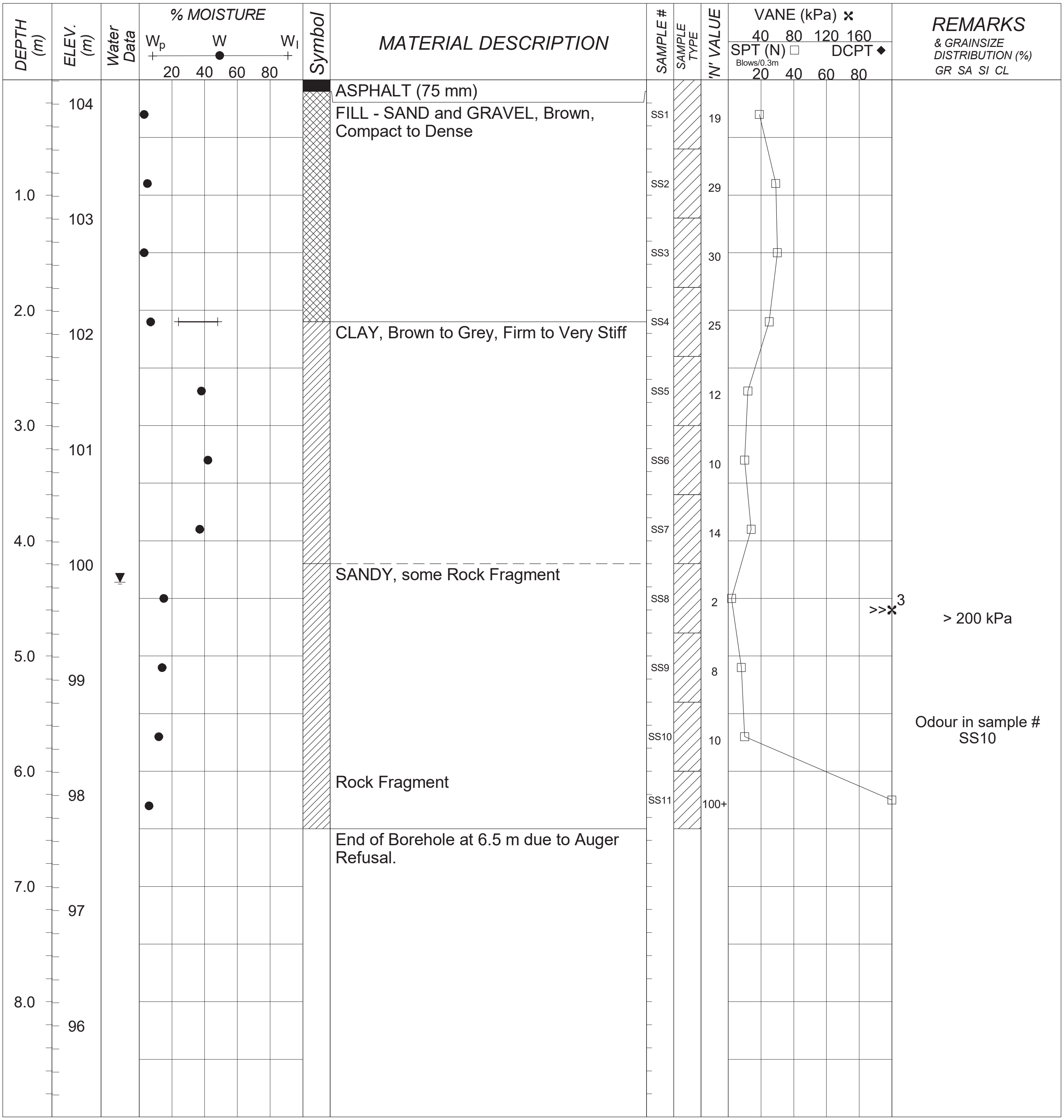
- ☰ Auger Sample
- ▨ Split Spoon Sample
- ▥ Bulk Sample
- ▩ Rock Core
- ▧ Hiller Peat Sampler
- ▯ Shelby Tube
- ■ Bentonite
- ░ Sand
- ×3 Numbers refers to Sensitivity

ENCLOSURE 1

LOG OF BOREHOLE BH2017-02

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **104.2 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/6/2017**
COMPLETION DATE: **7/6/2017**
COORDINATES: **5028045** m N, **444017** m E



MATERIAL DESCRIPTION	SAMPLE #	SAMPLE TYPE	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
ASPHALT (75 mm)				
FILL - SAND and GRAVEL, Brown, Compact to Dense	SS1	▨	19	
	SS2	▨	29	
	SS3	▨	30	
CLAY, Brown to Grey, Firm to Very Stiff	SS4	▨	25	
	SS5	▨	12	
	SS6	▨	10	
	SS7	▨	14	
SANDY, some Rock Fragment	SS8	▨	2	> 200 kPa
	SS9	▨	8	
	SS10	▨	10	Odour in sample # SS10
Rock Fragment	SS11	▨	100+	
End of Borehole at 6.5 m due to Auger Refusal.				

BOREHOLE (OTTAWA) TS-SO-29563.GPJ DATA TEMPLATE.GDT 28/7/17

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SAMPLE TYPE LEGEND

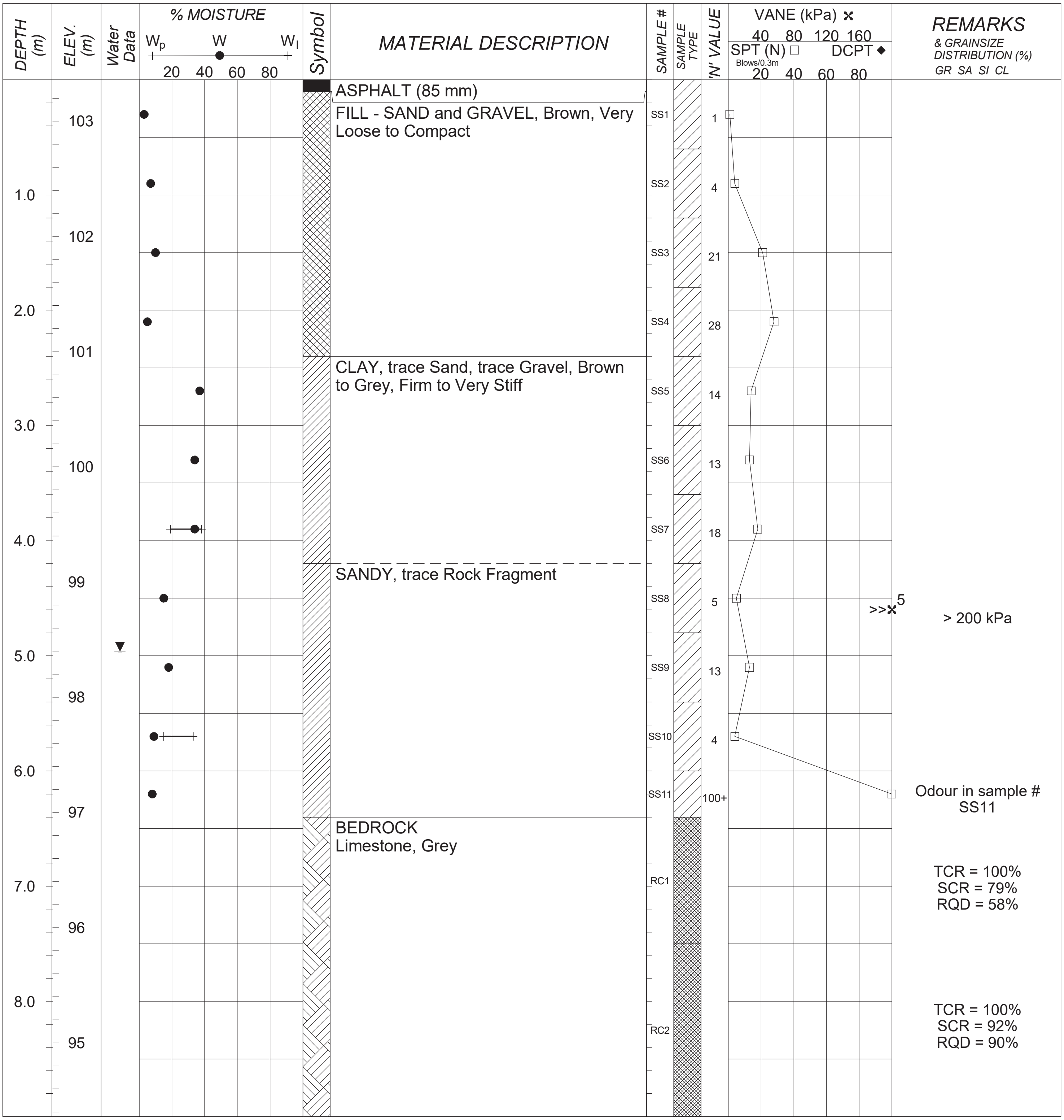
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	▯	Shelby Tube	×3	Numbers refers to Sensitivity

ENCLOSURE 2

LOG OF BOREHOLE BH2017-03

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **103.4 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/5/2017**
COMPLETION DATE: **7/5/2017**
COORDINATES: **5028054** m N, **444057** m E



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SAMPLE TYPE LEGEND

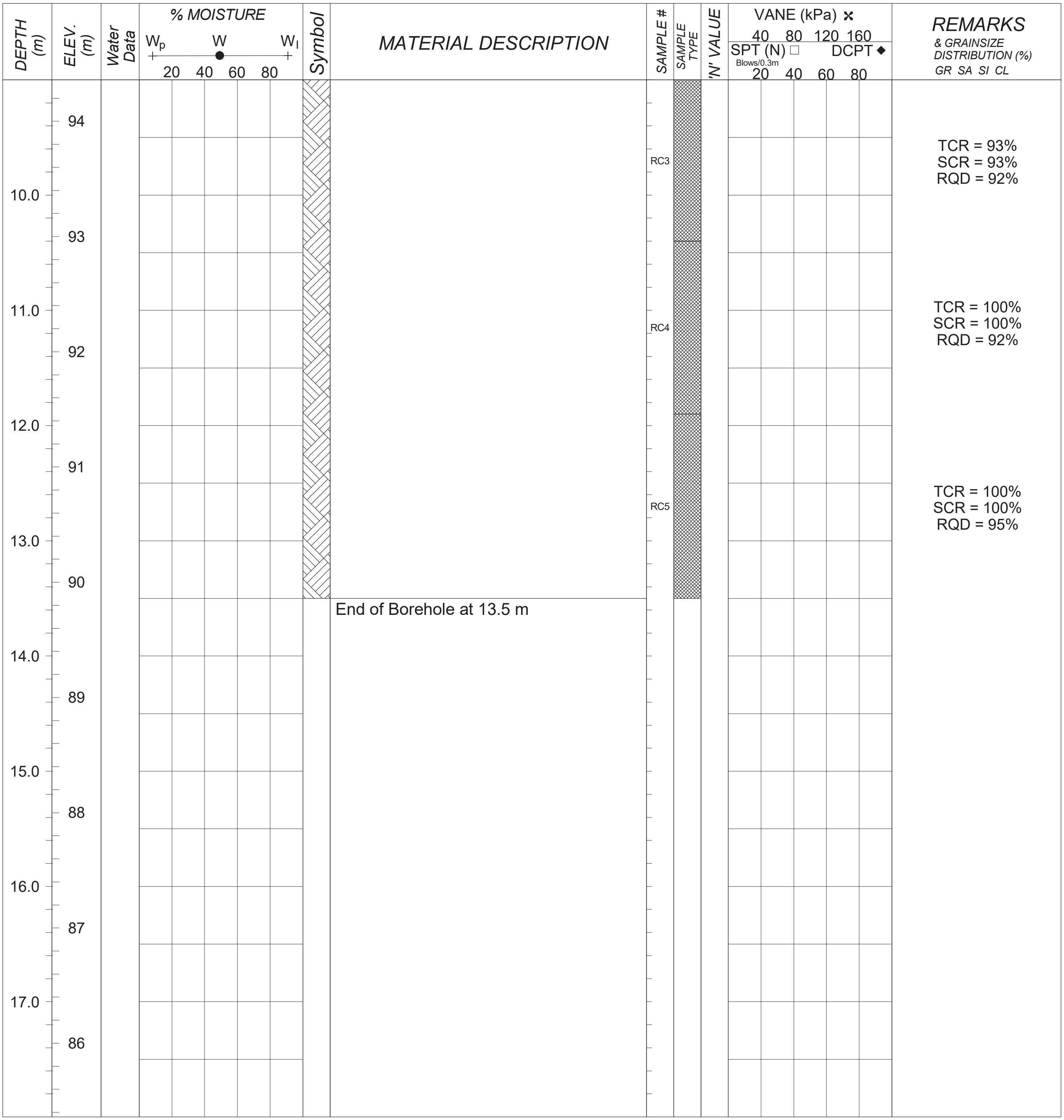
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	▯	Shelby Tube	×3	Numbers refers to Sensitivity

ENCLOSURE 3

LOG OF BOREHOLE BH2017-03

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **103.4 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/5/2017**
COMPLETION DATE: **7/5/2017**
COORDINATES: **5028054** m N, **444057** m E



End of Borehole at 13.5 m

Sample #	Remarks
RC3	TCR = 93% SCR = 93% RQD = 92%
RC4	TCR = 100% SCR = 100% RQD = 92%
RC5	TCR = 100% SCR = 100% RQD = 95%

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SAMPLE TYPE LEGEND

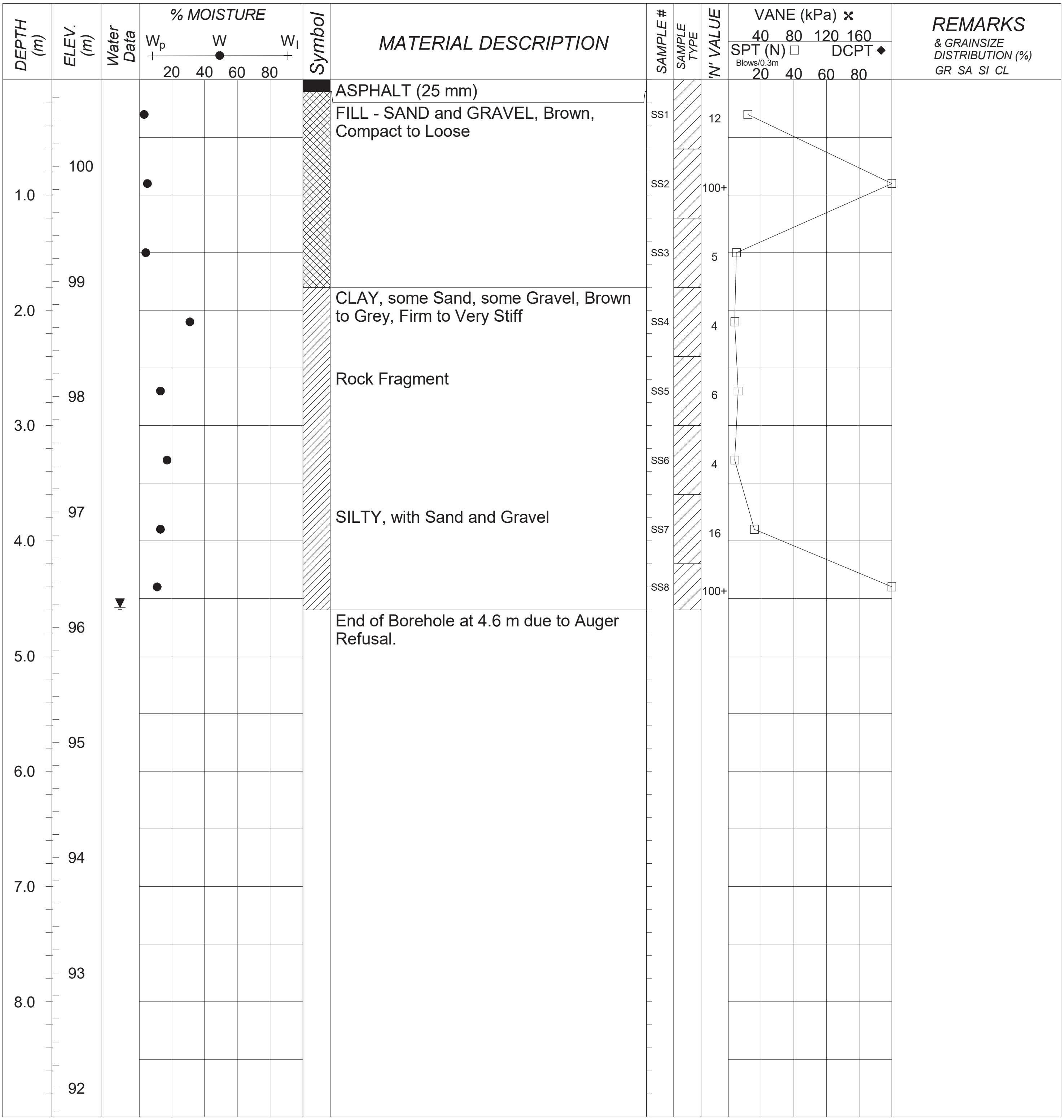
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	▯	Shelby Tube	✕³	Numbers refers to Sensitivity

ENCLOSURE 4

LOG OF BOREHOLE BH2017-04

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **100.8 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/6/2017**
COMPLETION DATE: **7/6/2017**
COORDINATES: **5028076** m N, **444058** m E



ASPHALT (25 mm)

FILL - SAND and GRAVEL, Brown, Compact to Loose

CLAY, some Sand, some Gravel, Brown to Grey, Firm to Very Stiff

Rock Fragment

SILTY, with Sand and Gravel

End of Borehole at 4.6 m due to Auger Refusal.

SS1	12
SS2	100+
SS3	5
SS4	4
SS5	6
SS6	4
SS7	16
SS8	100+

BOREHOLE (OTTAWA) TS-SO-29563.GPJ DATA TEMPLATE.GDT 28/7/17

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Web: www.dstgroup.com

SAMPLE TYPE LEGEND

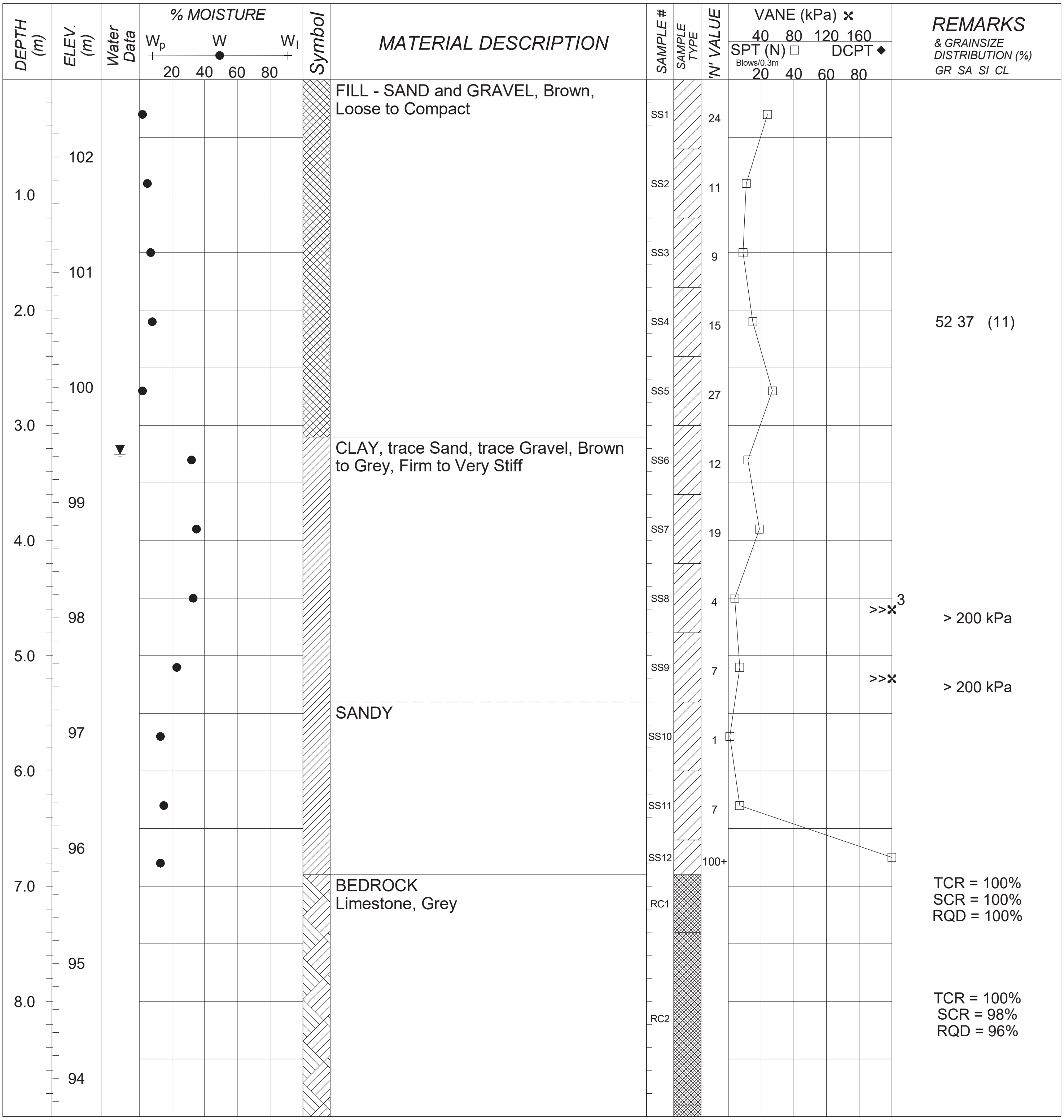
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	◫	Shelby Tube	✕3	Numbers refers to Sensitivity

ENCLOSURE 5

LOG OF BOREHOLE BH2017-05

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **102.7 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/7/2017**
COMPLETION DATE: **7/7/2017**
COORDINATES: **5028096** m N, **444017** m E



FILL - SAND and GRAVEL, Brown, Loose to Compact

CLAY, trace Sand, trace Gravel, Brown to Grey, Firm to Very Stiff

SANDY

BEDROCK
Limestone, Grey

52 37 (11)

> 200 kPa

> 200 kPa

TCR = 100%
SCR = 100%
RQD = 100%

TCR = 100%
SCR = 98%
RQD = 96%

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SAMPLE TYPE LEGEND

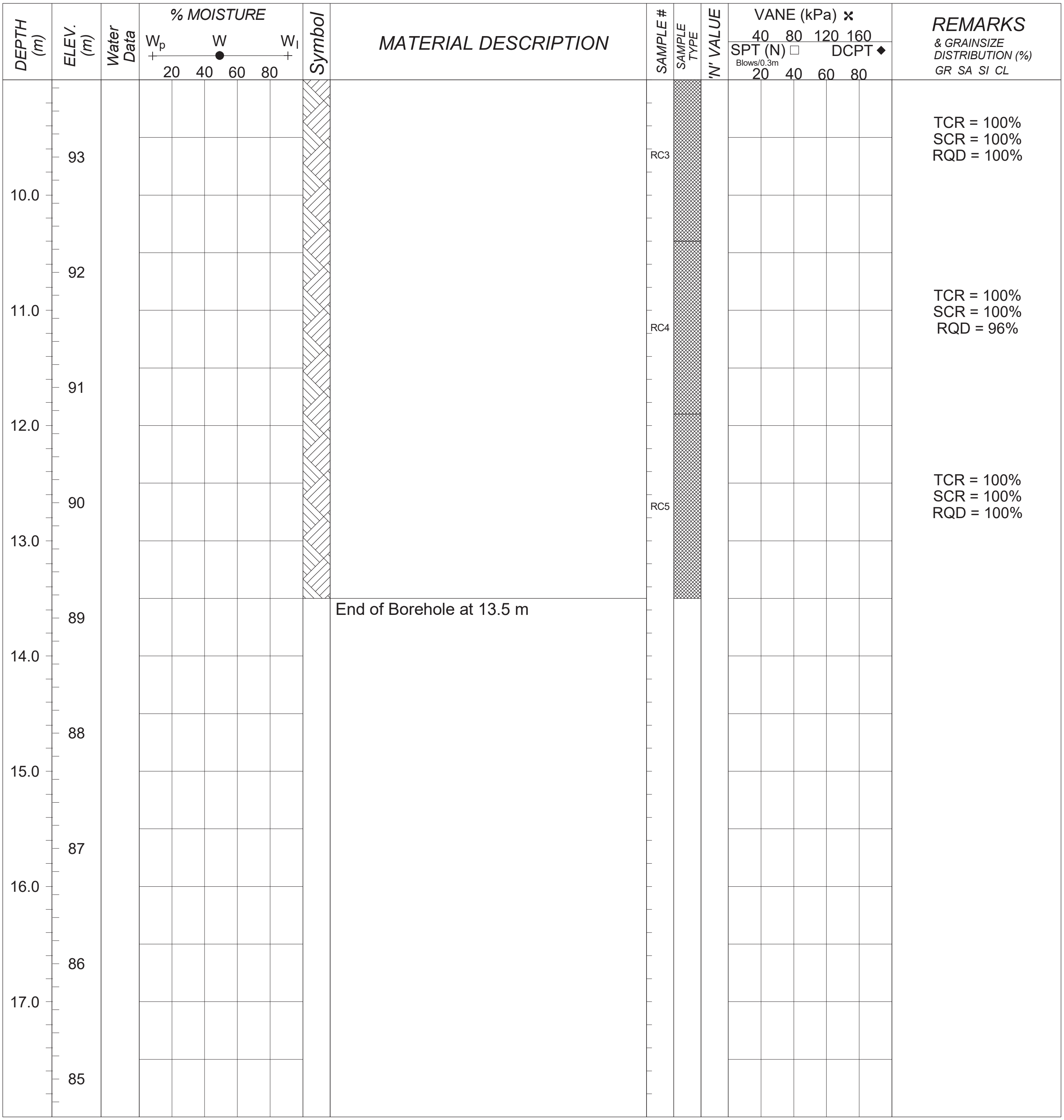
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	⛆	Sand
▥	Bulk Sample	◫	Shelby Tube	×3	Numbers refers to Sensitivity

ENCLOSURE 6

LOG OF BOREHOLE BH2017-05

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **102.7 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/7/2017**
COMPLETION DATE: **7/7/2017**
COORDINATES: **5028096** m N, **444017** m E



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SAMPLE TYPE LEGEND

☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	▧ Hiller Peat Sampler	░ Sand
▥ Bulk Sample	▯ Shelby Tube	×3 Numbers refers to Sensitivity

ENCLOSURE 7

LOG OF BOREHOLE BH2017-05A

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **102.7 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/7/2017**
COMPLETION DATE: **7/7/2017**
COORDINATES: **5028096** m N, **444019** m E

DEPTH (m)	ELEV. (m)	Water Data	% MOISTURE W	Symbol	MATERIAL DESCRIPTION	SAMPLE #	SAMPLE TYPE	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
	102.7			▩	FILL - SAND and GRAVEL, Brown, Loose to Compact	SS1	▨	11	
1.0	102			▩		SS2	▨	12	
	101			▩		SS3	▨	4	
1.8					End of Borehole at 1.8 m due to Auger Refusal.				Hit possibly old engine tank; smells like engine oil

% MOISTURE: W_p W W_l — 20 40 60 80

VANE (kPa) ✕ 40 80 120 160; SPT (N) ☐ Blows/0.3m; DCPT ◆ 20 40 60 80

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SAMPLE TYPE LEGEND

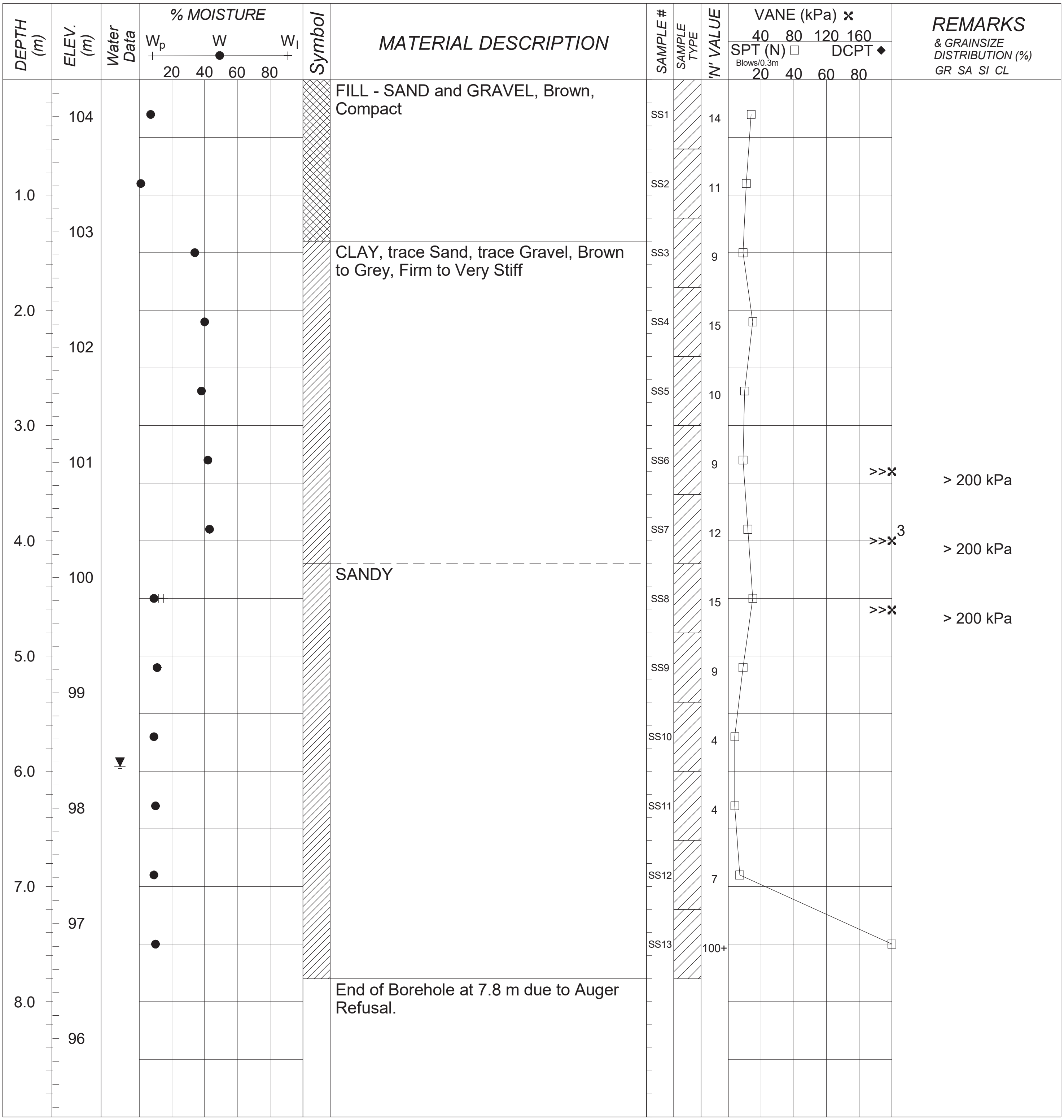
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	▧ Hiller Peat Sampler	░ Sand
▥ Bulk Sample	▯ Shelby Tube	✕3 Numbers refers to Sensitivity

ENCLOSURE 8

LOG OF BOREHOLE BH2017-06

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **104.3 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/7/2017**
COMPLETION DATE: **7/7/2017**
COORDINATES: **5028066** m N, **443975** m E



MATERIAL DESCRIPTION

FILL - SAND and GRAVEL, Brown, Compact

CLAY, trace Sand, trace Gravel, Brown to Grey, Firm to Very Stiff

SANDY

End of Borehole at 7.8 m due to Auger Refusal.

SAMPLE #	'N' VALUE	VANE (kPa)	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
SS1	14		
SS2	11		
SS3	9		
SS4	15		
SS5	10		
SS6	9	>>✕	> 200 kPa
SS7	12	>>✕ 3	> 200 kPa
SS8	15	>>✕	> 200 kPa
SS9	9		
SS10	4		
SS11	4		
SS12	7		
SS13	100+		

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Web: www.dstgroup.com

SAMPLE TYPE LEGEND

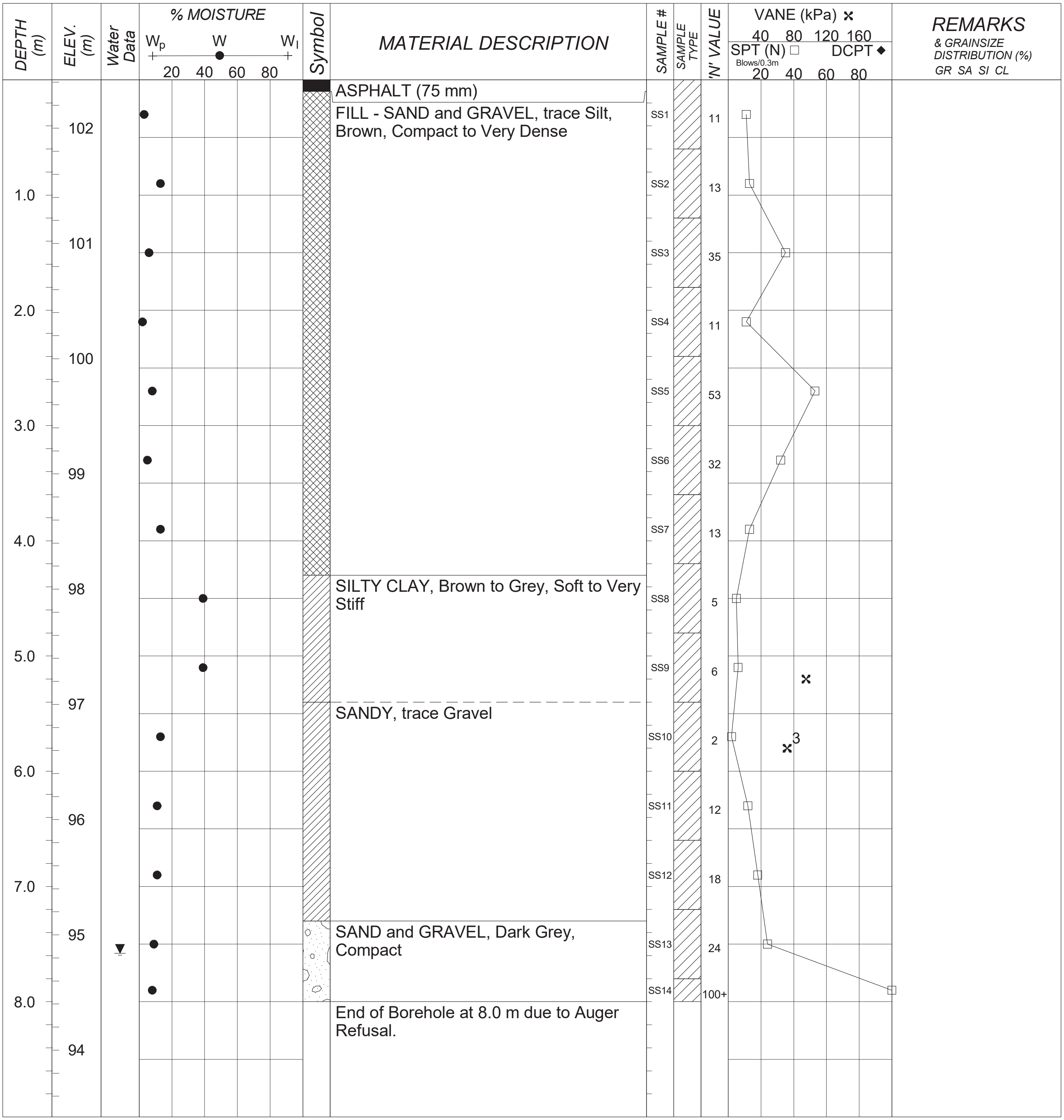
☰ Auger Sample
▨ Split Spoon Sample
▥ Bulk Sample
▩ Rock Core
▧ Hiller Peat Sampler
◫ Shelby Tube
■ Bentonite
⬚ Sand
✕3 Numbers refers to Sensitivity

ENCLOSURE 9

LOG OF BOREHOLE BH2017-07

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **102.4 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **6/27/2017**
COMPLETION DATE: **6/27/2017**
COORDINATES: **5028127** m N, **443952** m E



MATERIAL DESCRIPTION
ASPHALT (75 mm)
FILL - SAND and GRAVEL, trace Silt, Brown, Compact to Very Dense
SILTY CLAY, Brown to Grey, Soft to Very Stiff
SANDY, trace Gravel
SAND and GRAVEL, Dark Grey, Compact
End of Borehole at 8.0 m due to Auger Refusal.

SAMPLE #	'N' VALUE
SS1	11
SS2	13
SS3	35
SS4	11
SS5	53
SS6	32
SS7	13
SS8	5
SS9	6
SS10	2
SS11	12
SS12	18
SS13	24
SS14	100+

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SAMPLE TYPE LEGEND

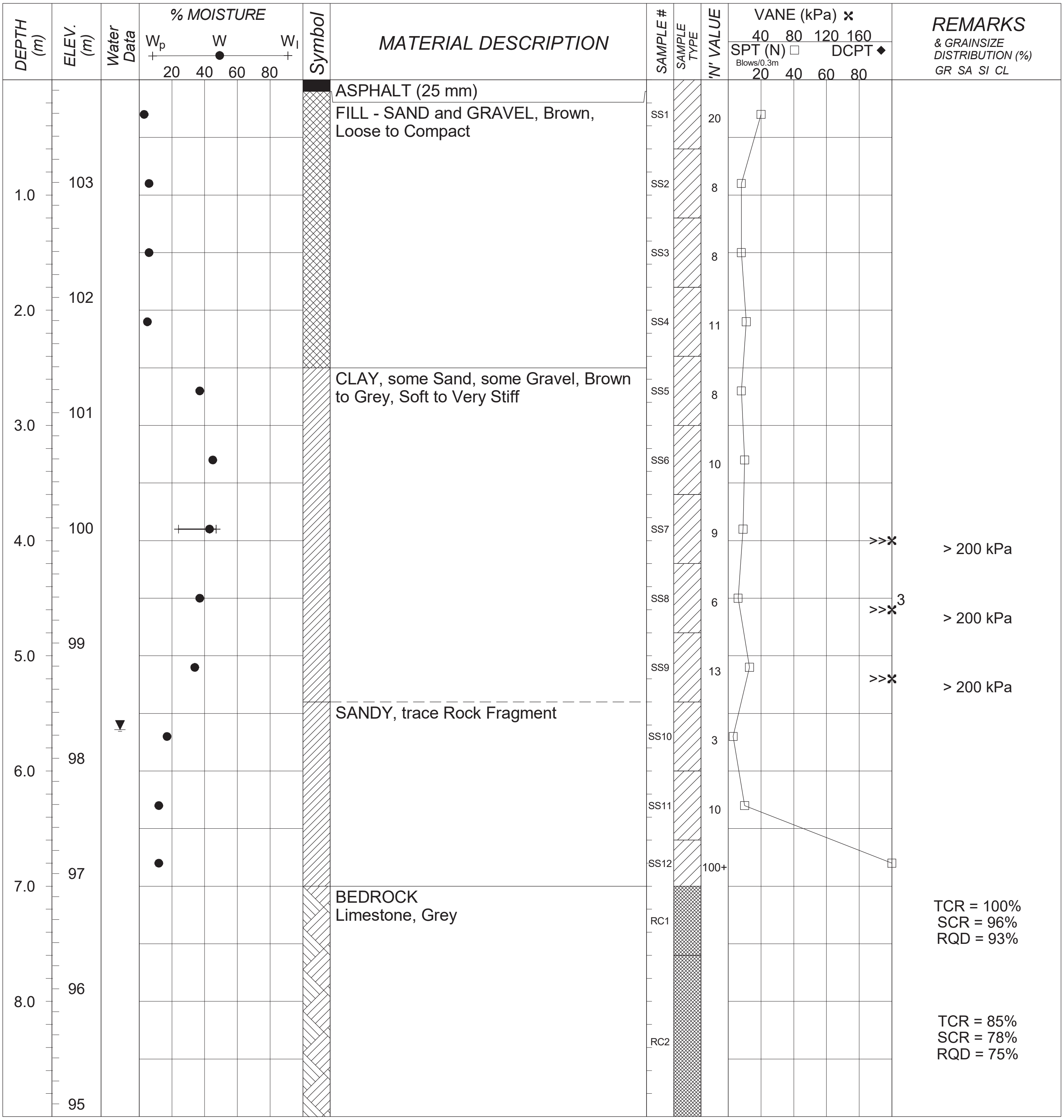
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	▯	Shelby Tube	×3	Numbers refers to Sensitivity

ENCLOSURE 10

LOG OF BOREHOLE BH2017-08

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **103.9 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/10/2017**
COMPLETION DATE: **7/10/2017**
COORDINATES: **5028091** m N, **443980** m E



MATERIAL DESCRIPTION	**SAMPLE #**	**'N' VALUE**	**REMARKS**
ASPHALT (25 mm)			
FILL - SAND and GRAVEL, Brown, Loose to Compact	SS1	20	
	SS2	8	
	SS3	8	
	SS4	11	
CLAY, some Sand, some Gravel, Brown to Grey, Soft to Very Stiff	SS5	8	
	SS6	10	
	SS7	9	> 200 kPa
	SS8	6	> 200 kPa
	SS9	13	> 200 kPa
SANDY, trace Rock Fragment	SS10	3	
	SS11	10	
	SS12	100+	
BEDROCK Limestone, Grey	RC1		TCR = 100% SCR = 96% RQD = 93%
	RC2		TCR = 85% SCR = 78% RQD = 75%

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Email: ottawa@dstgroup.com
Web: www.dstgroup.com

SAMPLE TYPE LEGEND

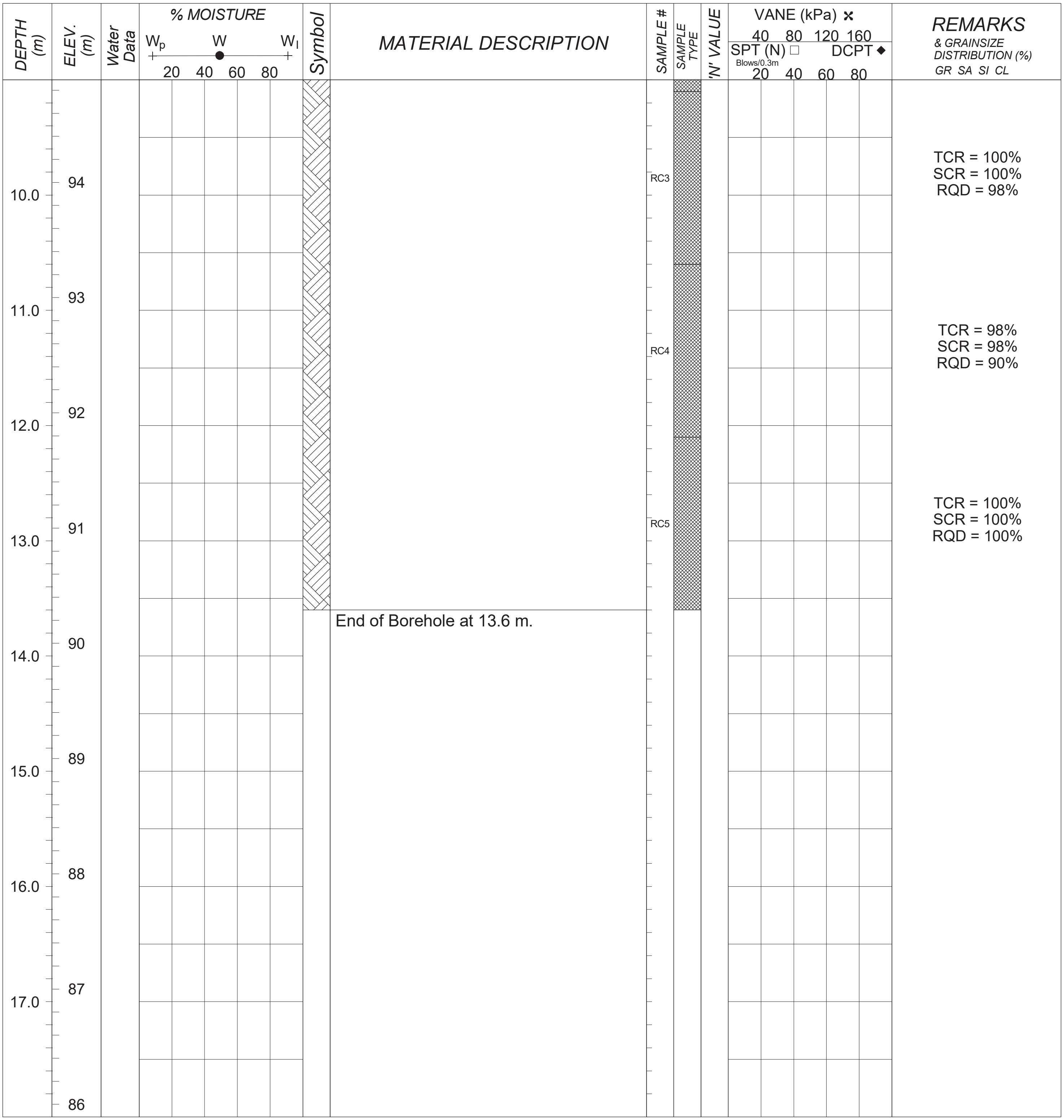
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	▧ Hiller Peat Sampler	░ Sand
▥ Bulk Sample	▯ Shelby Tube	$\times^3$ Numbers refers to Sensitivity

ENCLOSURE 11

LOG OF BOREHOLE BH2017-08

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **103.9 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/10/2017**
COMPLETION DATE: **7/10/2017**
COORDINATES: **5028091** m N, **443980** m E



End of Borehole at 13.6 m.

Sample #	Remarks
RC3	TCR = 100% SCR = 100% RQD = 98%
RC4	TCR = 98% SCR = 98% RQD = 90%
RC5	TCR = 100% SCR = 100% RQD = 100%

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SAMPLE TYPE LEGEND

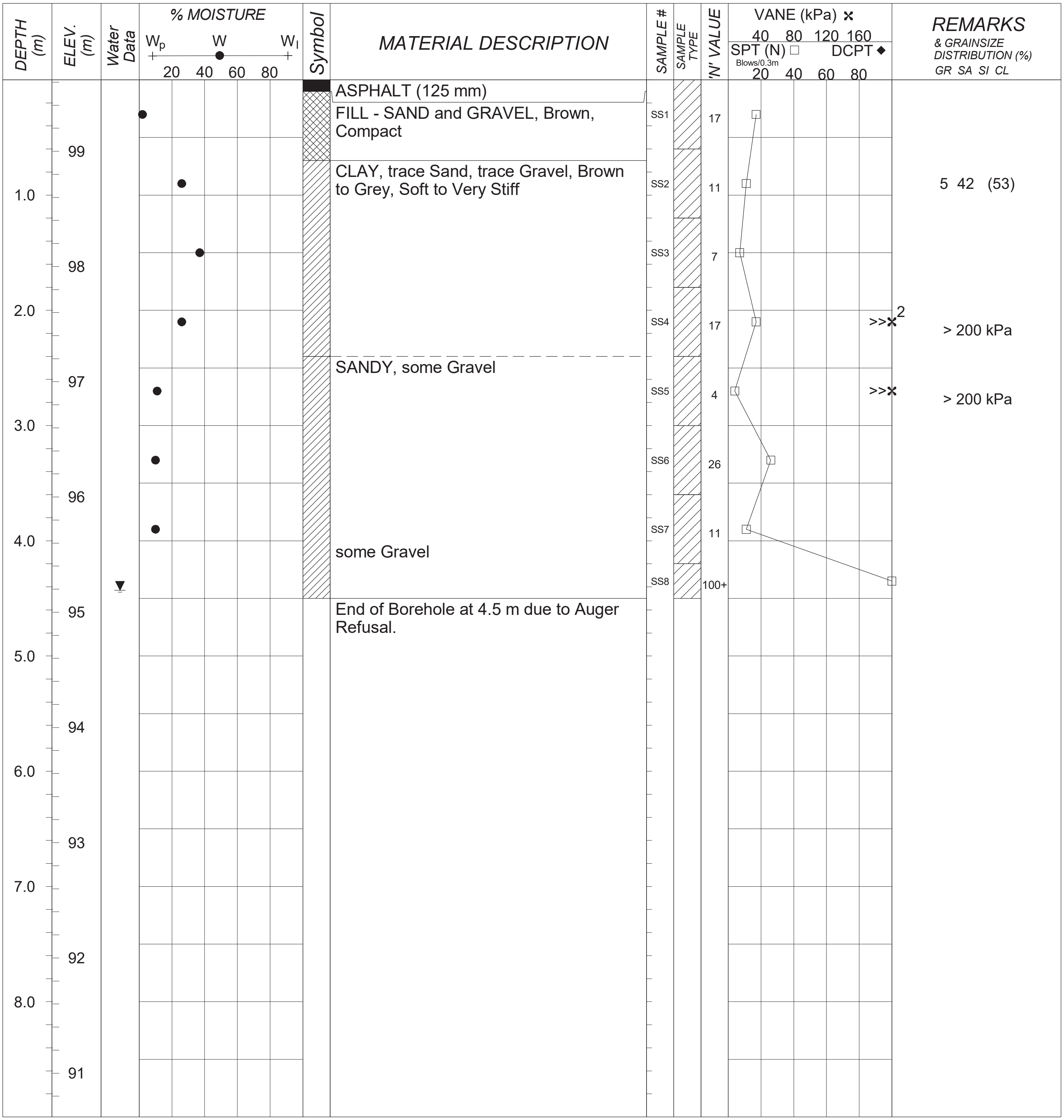
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	⬚	Sand
▥	Bulk Sample	◫	Shelby Tube	×3	Numbers refers to Sensitivity

ENCLOSURE 12

LOG OF BOREHOLE BH2017-09

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **99.6 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/6/2017**
COMPLETION DATE: **7/6/2017**
COORDINATES: **5028115** m N, **444005** m E



MATERIAL DESCRIPTION

ASPHALT (125 mm)

FILL - SAND and GRAVEL, Brown, Compact

CLAY, trace Sand, trace Gravel, Brown to Grey, Soft to Very Stiff

SANDY, some Gravel

some Gravel

End of Borehole at 4.5 m due to Auger Refusal.

SAMPLE #	SAMPLE TYPE	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
SS1	▨	17	
SS2	▨	11	5 42 (53)
SS3	▨	7	
SS4	▨	17	> 200 kPa
SS5	▨	4	> 200 kPa
SS6	▨	26	
SS7	▨	11	
SS8	▨	100+	

BOREHOLE (OTTAWA) TS-SO-29563.GPJ DATA TEMPLATE.GDT 28/7/17

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SAMPLE TYPE LEGEND

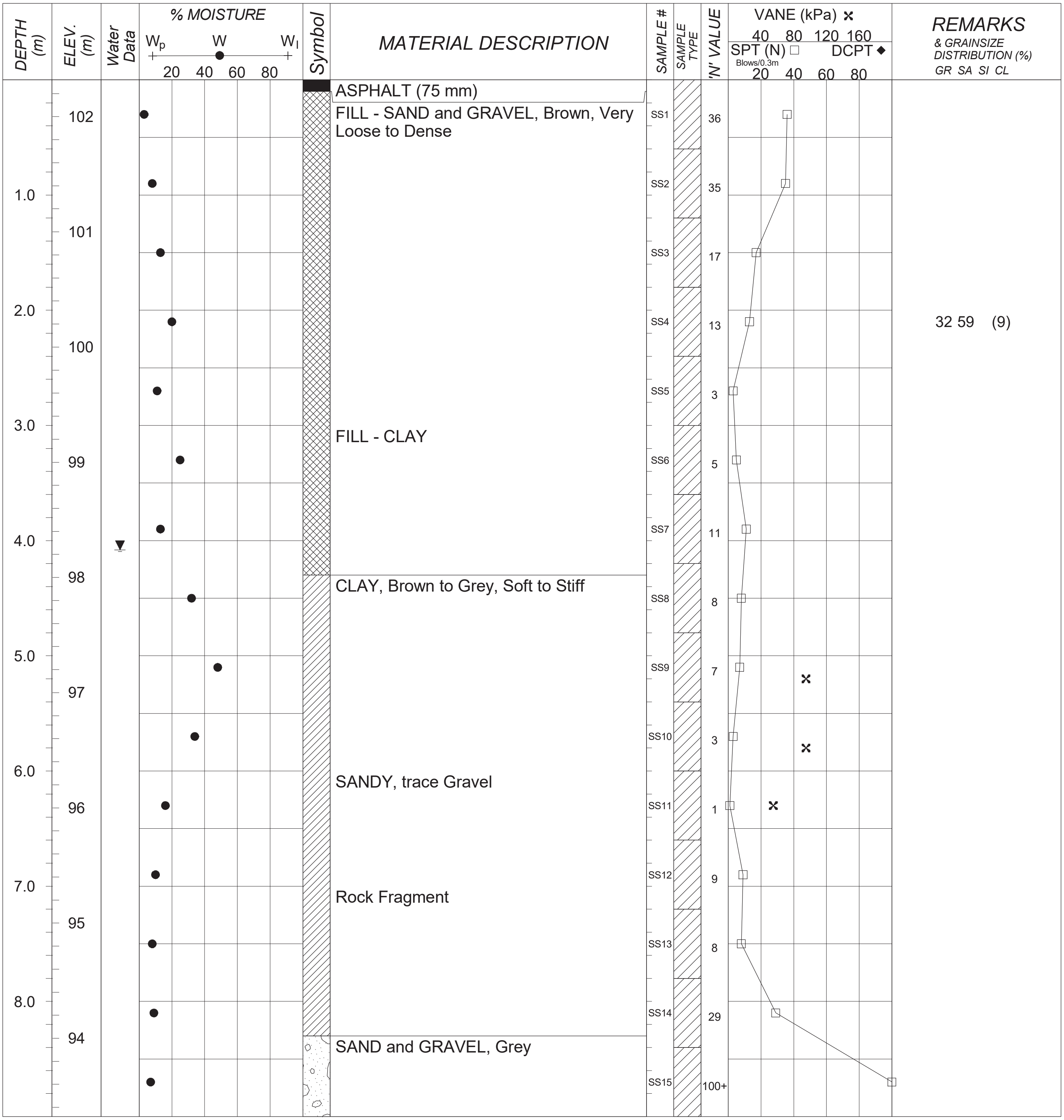
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	◫	Shelby Tube	×3	Numbers refers to Sensitivity

ENCLOSURE 13

LOG OF BOREHOLE BH2017-10

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **102.3 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **6/27/2017**
COMPLETION DATE: **6/27/2017**
COORDINATES: **5028139** m N, **443966** m E



BOREHOLE (OTTAWA) TS-SO-29563.GPJ DATA TEMPLATE.GDT 28/7/17

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SAMPLE TYPE LEGEND

☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	⬚	Sand
▥	Bulk Sample	◫	Shelby Tube	✕3	Numbers refers to Sensitivity

ENCLOSURE 14

LOG OF BOREHOLE BH2017-10

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **102.3 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **6/27/2017**
COMPLETION DATE: **6/27/2017**
COORDINATES: **5028139** m N, **443966** m E

DEPTH (m)	ELEV. (m)	Water Data	% MOISTURE Wp W Wl 20 40 60 80	Symbol	MATERIAL DESCRIPTION	SAMPLE #	SAMPLE TYPE	'N' VALUE	VANE (kPa) × 40 80 120 160 / SPT (N) □ Blows/0.3m DCPT ◆ 20 40 60 80	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
	93				BEDROCK Limestone, Grey		▨			
						RC1	▨			TCR = 98% SCR = 92% RQD = 85%
10.0							▨			
	92						▨			
11.0							▨			
	91					RC2	▨			TCR = 100% SCR = 100% RQD = 100%
12.0							▨			
	90						▨			
						RC3	▨			TCR = 98% SCR = 98% RQD = 98%
13.0							▨			
	89						▨			
14.0							▨			
	88					RC4	▨			TCR = 100% SCR = 100% RQD = 100%
15.0							▨			
	87						▨			
						RC5	▨			TCR = 100% SCR = 100% RQD = 100%
16.0							▨			
	86						▨			
					End of Borehole at 16.6 m.					
17.0										
	85									

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SAMPLE TYPE LEGEND

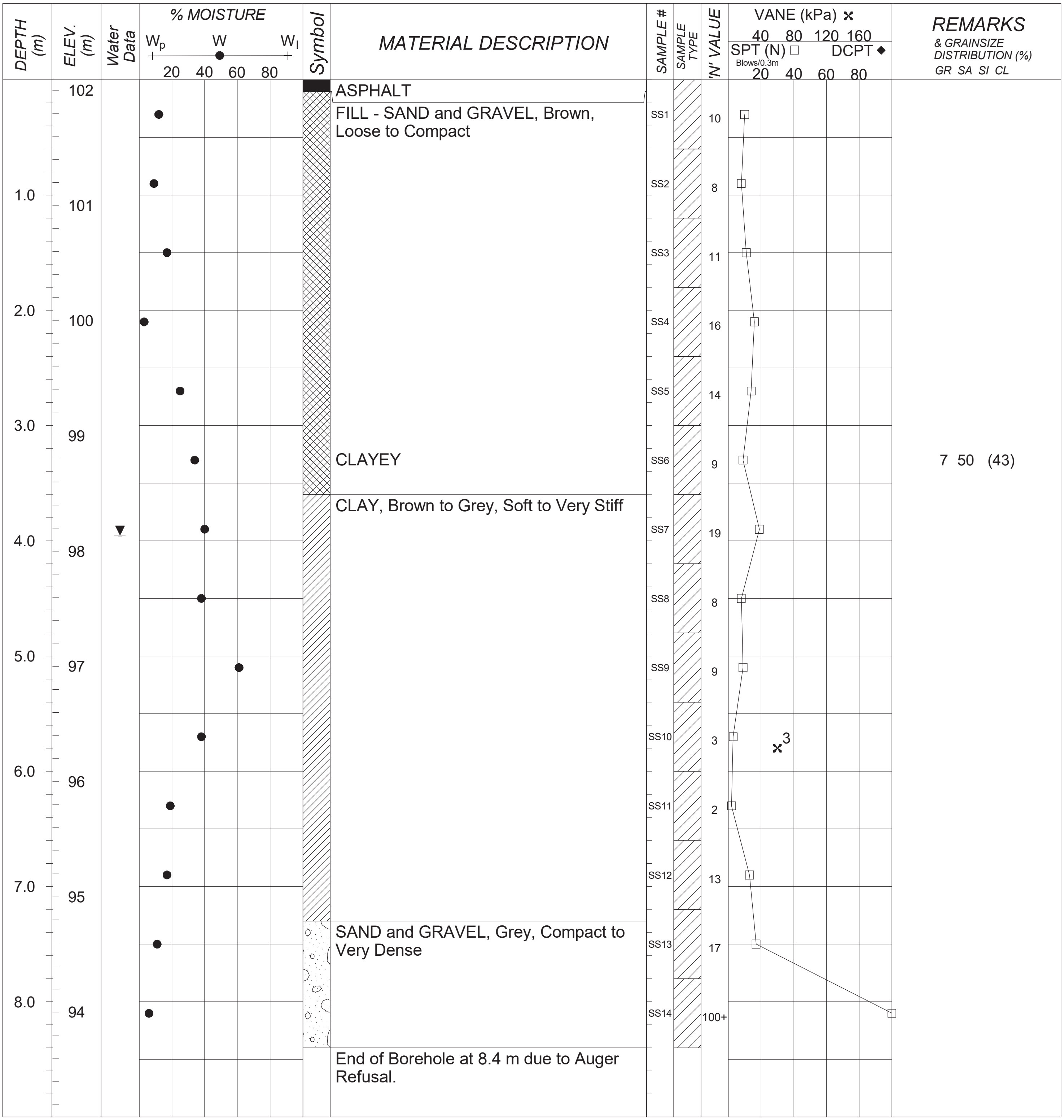
☰	Auger Sample	▨	Rock Core	■	Bentonite
▨	Split Spoon Sample	▨	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	▯	Shelby Tube	×3	Numbers refers to Sensitivity

ENCLOSURE 15

LOG OF BOREHOLE BH2017-11

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **102.1 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/4/2017**
COMPLETION DATE: **7/4/2017**
COORDINATES: **5028155** m N, **443948** m E



MATERIAL DESCRIPTION
ASPHALT
FILL - SAND and GRAVEL, Brown, Loose to Compact
CLAYEY
CLAY, Brown to Grey, Soft to Very Stiff
SAND and GRAVEL, Grey, Compact to Very Dense
End of Borehole at 8.4 m due to Auger Refusal.

SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%)
SS1	10	
SS2	8	
SS3	11	
SS4	16	
SS5	14	
SS6	9	7 50 (43)
SS7	19	
SS8	8	
SS9	9	
SS10	3	
SS11	2	
SS12	13	
SS13	17	
SS14	100+	

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SAMPLE TYPE LEGEND

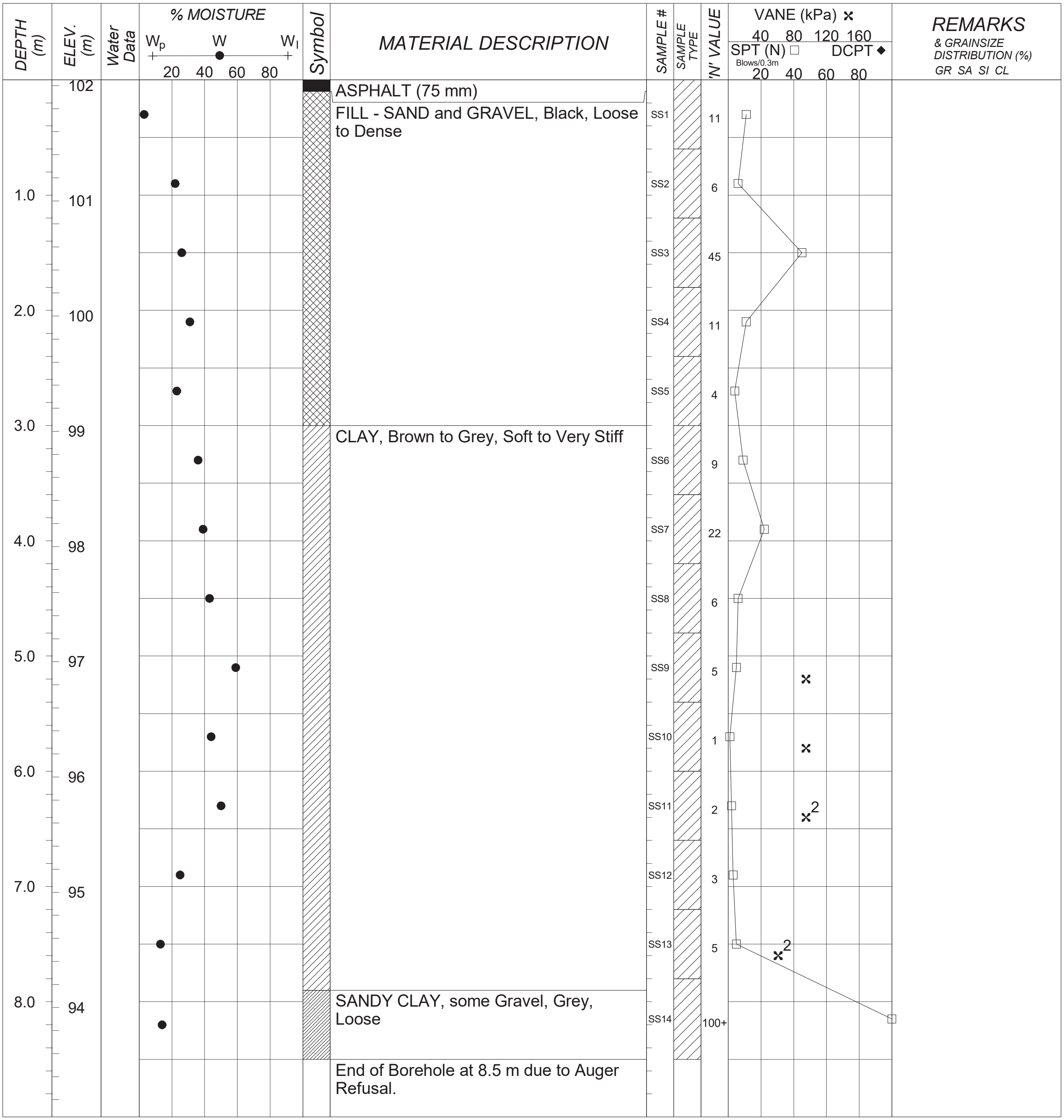
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	◫	Shelby Tube	×3	Numbers refers to Sensitivity

ENCLOSURE 16

LOG OF BOREHOLE BH2017-12

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **102.1 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **7/4/2017**
COMPLETION DATE: **7/4/2017**
COORDINATES: **5028159** m N, **443963** m E



MATERIAL DESCRIPTION
ASPHALT (75 mm)
FILL - SAND and GRAVEL, Black, Loose to Dense
CLAY, Brown to Grey, Soft to Very Stiff
SANDY CLAY, some Gravel, Grey, Loose
End of Borehole at 8.5 m due to Auger Refusal.

SAMPLE #	'N' VALUE
SS1	11
SS2	6
SS3	45
SS4	11
SS5	4
SS6	9
SS7	22
SS8	6
SS9	5
SS10	1
SS11	2
SS12	3
SS13	5
SS14	100+

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SAMPLE TYPE LEGEND

☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	▯	Shelby Tube	×3	Numbers refers to Sensitivity

ENCLOSURE 17

LOG OF BOREHOLE BH2017-13

DST REF. No.: **TS-SO-29563**
CLIENT: **Trinity Development Group Inc.**
PROJECT: **Geotechnical Drilling for the Proposed Development**
LOCATION: **951 Gladstone Avenue, Ottawa, ON**
SURFACE ELEV.: **102.2 metres**

Drilling Data
METHOD: **Hollow Stem Auger**
START DATE: **6/28/2017**
COMPLETION DATE: **6/28/2017**
COORDINATES: **5028143** m N, **443978** m E

DEPTH (m)	ELEV. (m)	Water Data	% MOISTURE Wp W Wl 20 40 60 80	Symbol	MATERIAL DESCRIPTION	SAMPLE #	SAMPLE TYPE	'N' VALUE	VANE (kPa) ✕ 40 80 120 160 / SPT (N) ☐ Blows/0.3m DCPT ◆ 20 40 60 80	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
				■	ASPHALT (75 mm)					
	102			▩	FILL - SAND and GRAVEL, Brown	SS1	▨	15	☐	
1.0	101			▩		SS2	▨	14	☐	
				▨	CLAY, trace Gravel, Brown, Stiff	SS3	▨	9	☐	
2.0	100				End of Borehole at 1.8 m due to Auger Refusal.					
3.0	99									
4.0	98									
5.0	97									
6.0	96									
7.0	95									
8.0	94									

BOREHOLE (OTTAWA) TS-SO-29563.GPJ DATA TEMPLATE.GDT 28/7/17

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SAMPLE TYPE LEGEND

☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	◫	Shelby Tube	✕3	Numbers refers to Sensitivity

ENCLOSURE 18

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 32474

Report Date: 21-Jul-2021

Order Date: 16-Jul-2021

Project Description: PG5517

	Client ID:	TP 2-21 G3	-	-	-
	Sample Date:	14-Jul-21 09:00	-	-	-
	Sample ID:	2129723-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	74.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.67	-	-	-
Resistivity	0.10 Ohm.m	8.32	-	-	-
Anions					
Chloride	5 ug/g dry	488	-	-	-
Sulphate	5 ug/g dry	355	-	-	-

PARACEL
LABORATORIES LTD.

Paracel ID: 2129723

Paracel Order Number
(Lab Use Only)
2129723

Chain Of Custody
(Lab Use Only)
Nº 132967

Client Name: PATERSON GROUP
Contact Name: SCOTT DENNIS
Address: 154 COLONNADE RD S
OTTAWA ON
Telephone: 613-726-7381

Project Ref: PG5517
Quote #:
PO #: 32474
E-mail: sdennis@patersongroup.ca

Page 1 of 1

Turnaround Time
☐ 1 day ☐ 3 day
☐ 2 day ☒ Regular
Date Required:

☐ REG 153/04 ☐ REG 406/19
☐ Table 1 ☐ Res/Park ☐ Med/Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/Other
☐ Table
For RSC: ☐ Yes ☐ No

Other Regulation
☐ REG 558 ☐ PWQO
☐ CCME ☐ MISA
☐ SU - Sani ☐ SU - Storm
Mun:
☐ Other:

Matrix Type: **S** (Soil/Sed.) **GW** (Ground Water)
SW (Surface Water) **SS** (Storm/Sanitary Sewer)
P (Paint) **A** (Air) **O** (Other)

					Sample Taken		Required Analysis											
	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Date	Time	PHCs F1-F4+BTEX	VOCs	PAHs	Metals by ICP	Hg	CrVI	B (HWS)	CHLORIDE	SULPHATE	RESISTIVITY	PH	
1	TP 2-21 G3	S		1	JULY 14/21									X	X	X	X	
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

Comments:

Method of Delivery: Drop Box

Relinquished By (Sign):
Relinquished By (Print): RICHARD GRANGER
Date/Time: JULY 16 2021

Received By Driver/Depot:
Date/Time:
Temperature: °C

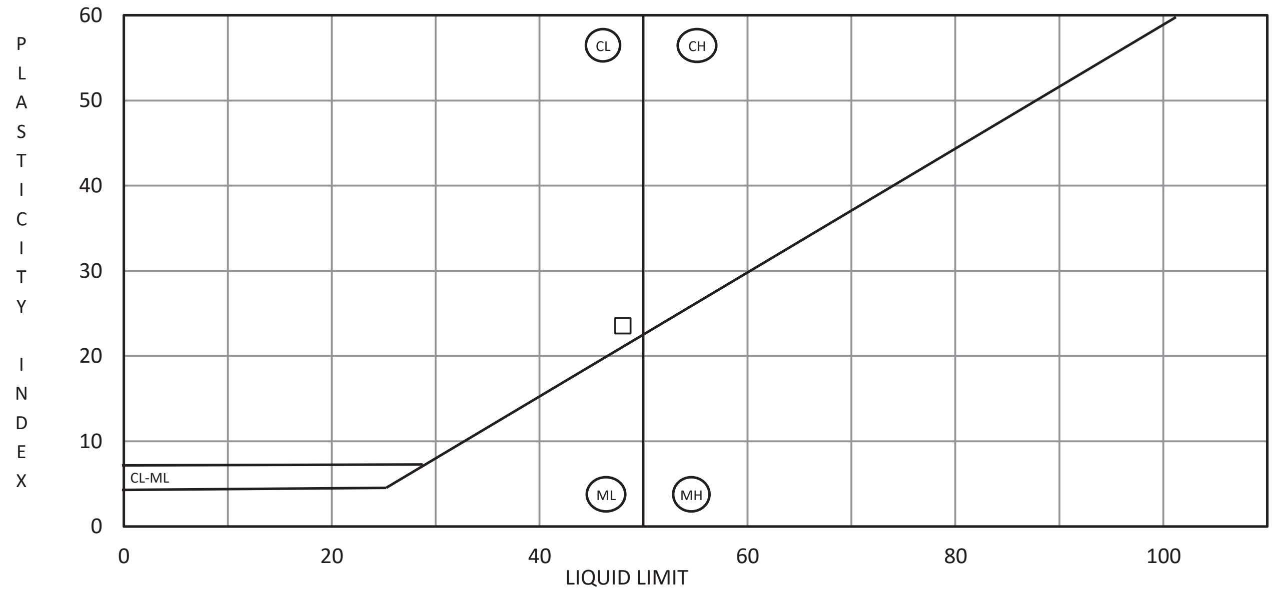
Received at Lab:
Date/Time: JUL 16, 2021 17:00
Temperature: 19.8 °C

Verified By:
Date/Time: July 16, 2021 17:32
pH Verified: ☐ By:

consulting engineers

DST CONSULTING ENGINEERS INC.
550 Parkside Drive, Unit C1-B
Waterloo ON, N2L 5V4
Tel: 519-772-4521
Fax: 519-725-3789
waterloo@dstgroup.com
www.dstgroup.com

DST Ref. No.:	TS-SO-29563	**Date Sampled:**	
Project:	Trinity Development Group Geotech Investiga	**Sampled By:**	
Client:	Trinity Development Group	**Source:**	BH2017-2, SS-4
Project Location:		**Location:**	
Sample #:	**KWG-016-4**		
Description:			



ATTERBERG LIMIT AND MOISTURE RESULTS:

SUMMARY OF ATTERBERG AND MOISTURE CONTENT:	
Liquid Limit, LL	48
Plastic Limit, PL	24
Plasticity Index, PI	24
In Place Moisture Content (ASTM D2216) %	0.0

TESTING EQUIPMENT USED		
Plastic Limit	Hand Rolled	X
	Mechanical Rolling Device	
Liquid Limit Apparatus	Manual	X
	Mechanical	
Casagrande ASTM Tool	Metal	X
	Plastic	

SPECIMEN PREPARATION			
Wet		Washed on #40 Sieve	
Dry (Air)		Dry Sieved on #40 Sieve	X
Dry (Oven)	X		

25-Jul-17

DISTRIBUTION:

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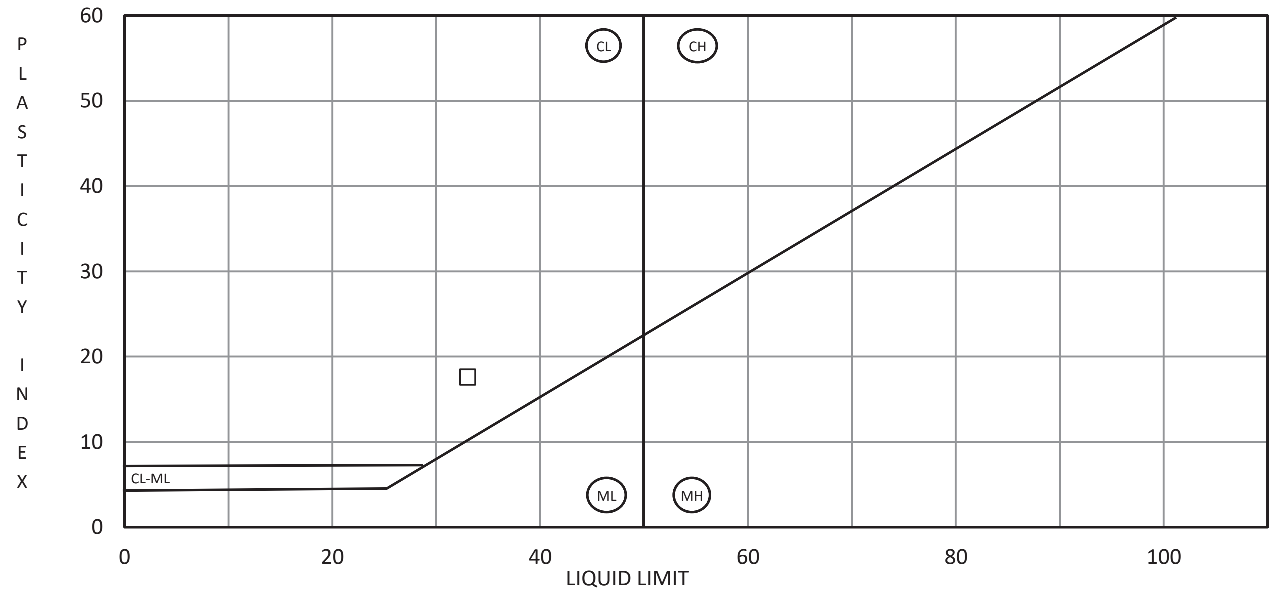
DST Ref. No.:	TS-SO-29563	**Date Sampled:**	
Project:	Trinity Development Group Geotech Investiga	**Sampled By:**	
Client:	Trinity Development Group	**Source:**	BH2017-3, SS-10
Project Location:		**Location:**	
Sample #:	**KWG-016-13**		
Description:			



ATTERBERG LIMIT AND MOISTURE RESULTS:

SUMMARY OF ATTERBERG AND MOISTURE CONTENT:	
Liquid Limit, LL	33
Plastic Limit, PL	15
Plasticity Index, PI	18
In Place Moisture Content (ASTM D2216) %	0.0

TESTING EQUIPMENT USED		
Plastic Limit	Hand Rolled	X
	Mechanical Rolling Device	
Liquid Limit Apparatus	Manual	X
	Mechanical	
Casagrande ASTM Tool	Metal	X
	Plastic	

SPECIMEN PREPARATION			
Wet		Washed on #40 Sieve	
Dry (Air)		Dry Sieved on #40 Sieve	X
Dry (Oven)	X		

25-Jul-17

Hugh Arthur - Laboratory Supervisor

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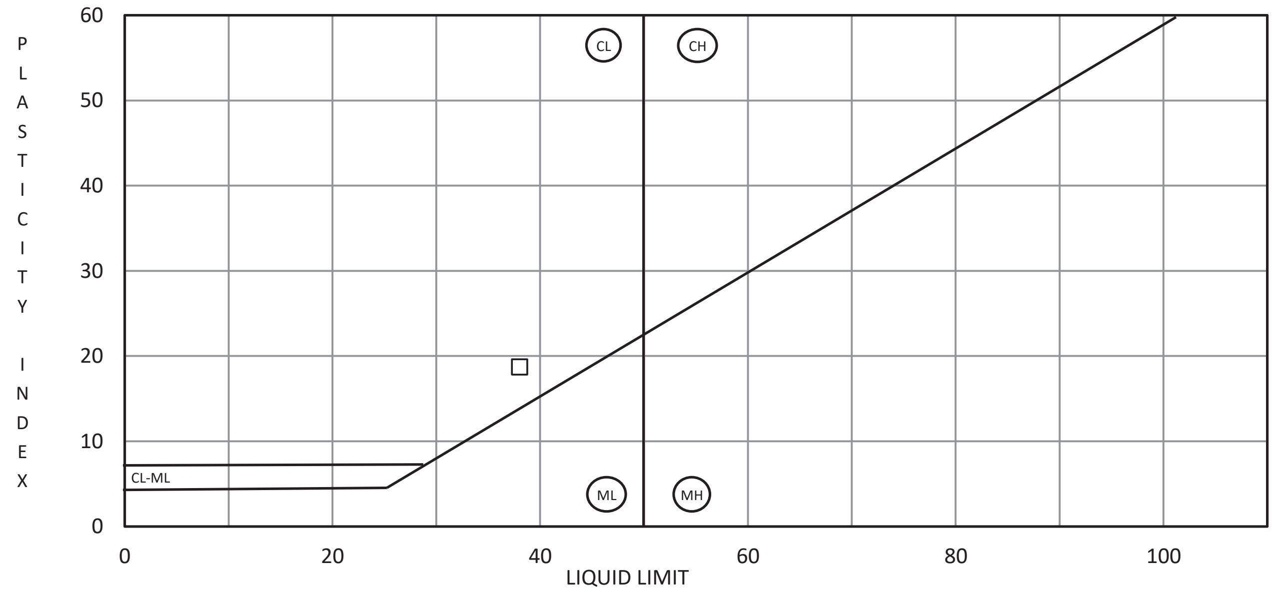
DST Ref. No.: TS-SO-29563
Project: Trinity Development Group Geotech Investiga
Client: Trinity Development Group
Project Location:
Sample #: **KWG-016-5**
Description:

Date Sampled:
Sampled By:
Source: BH2017-3, SS-7
Location:



ATTERBERG LIMIT AND MOISTURE RESULTS:

SUMMARY OF ATTERBERG AND MOISTURE CONTENT:	
Liquid Limit, LL	38
Plastic Limit, PL	19
Plasticity Index, PI	19
In Place Moisture Content (ASTM D2216) %	0.0

TESTING EQUIPMENT USED		
Plastic Limit	Hand Rolled	X
	Mechanical Rolling Device	
Liquid Limit Apparatus	Manual	X
	Mechanical	
Casagrande ASTM Tool	Metal	X
	Plastic	

SPECIMEN PREPARATION			
Wet		Washed on #40 Sieve	
Dry (Air)		Dry Sieved on #40 Sieve	X
Dry (Oven)	X		

25-Jul-17

Hugh Arthur - Laboratory Supervisor

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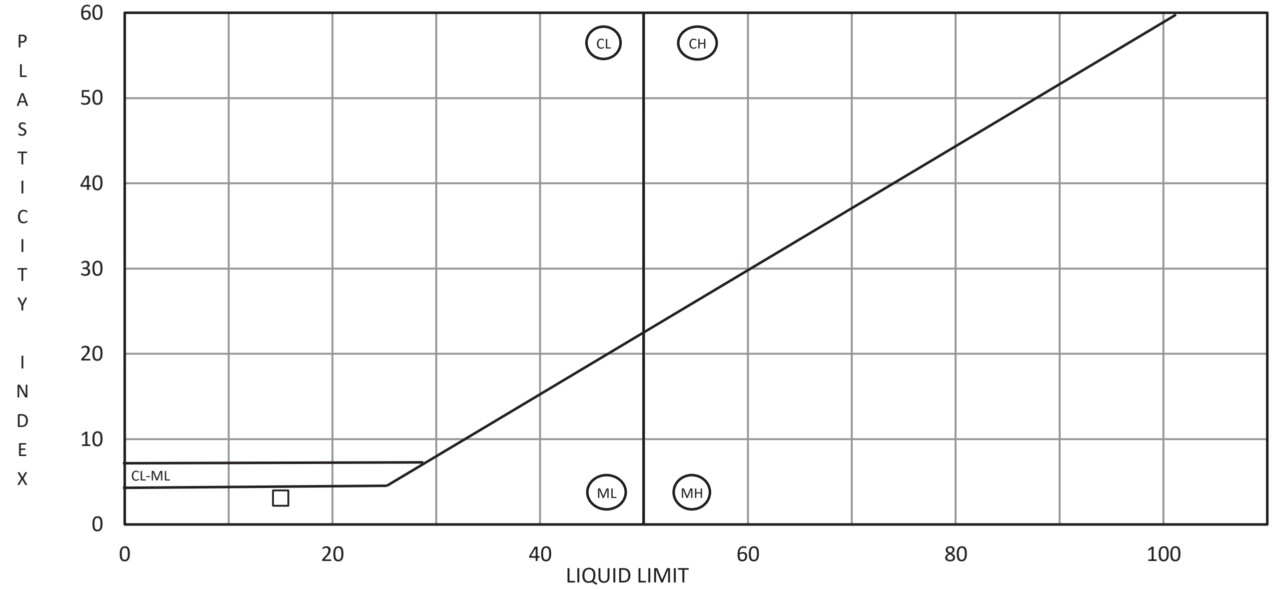
www.dstgroup.com

DST Ref. No.:	TS-SO-29563	**Date Sampled:**	
Project:	Trinity Development Group Geotech Investiga	**Sampled By:**	
Client:	Trinity Development Group	**Source:**	BH2017-6, SS-8
Project Location:		**Location:**	
Sample #:	**KWG-016-7**		
Description:			



ATTERBERG LIMIT AND MOISTURE RESULTS:

SUMMARY OF ATTERBERG AND MOISTURE CONTENT:	
Liquid Limit, LL	15
Plastic Limit, PL	12
Plasticity Index, PI	3
In Place Moisture Content (ASTM D2216) %	0.0

TESTING EQUIPMENT USED		
Plastic Limit	Hand Rolled	X
	Mechanical Rolling Device	
Liquid Limit Apparatus	Manual	X
	Mechanical	
Casagrande ASTM Tool	Metal	X
	Plastic	

SPECIMEN PREPARATION			
Wet		Washed on #40 Sieve	
Dry (Air)		Dry Sieved on #40 Sieve	X
Dry (Oven)	X		

25-Jul-17

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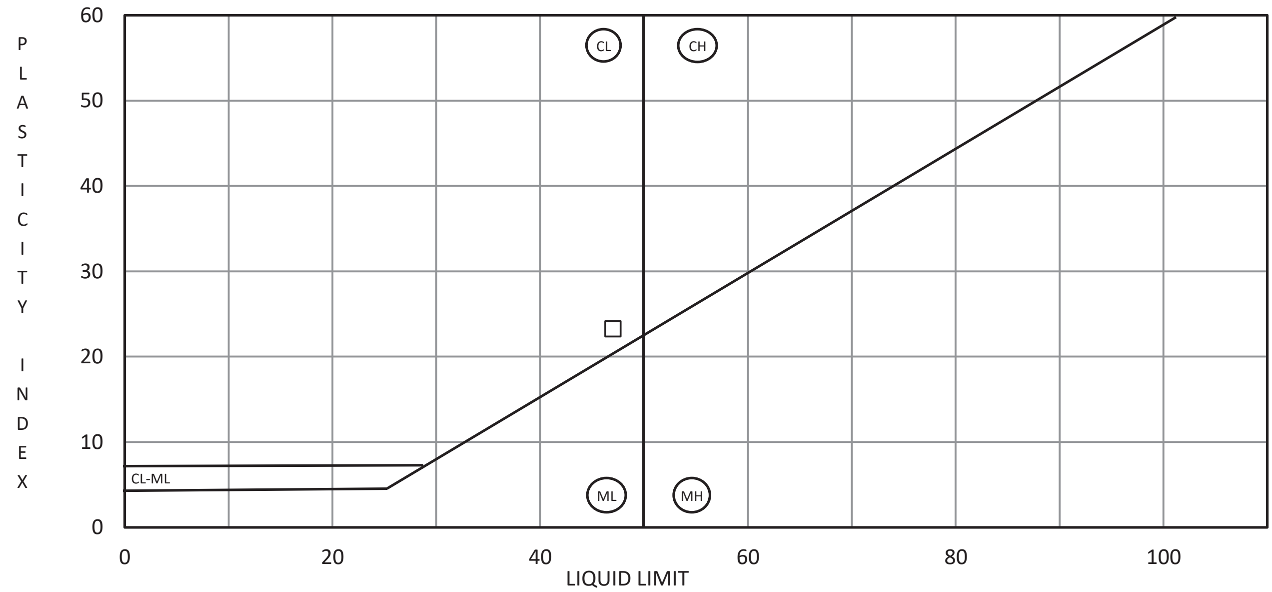
DST Ref. No.:	TS-SO-29563	**Date Sampled:**	
Project:	Trinity Development Group Geotech Investiga	**Sampled By:**	
Client:	Trinity Development Group	**Source:**	BH2017-8, SS-7
Project Location:		**Location:**	
Sample #:	**KWG-016-8**		
Description:			



ATTERBERG LIMIT AND MOISTURE RESULTS:

SUMMARY OF ATTERBERG AND MOISTURE CONTENT:	
Liquid Limit, LL	47
Plastic Limit, PL	24
Plasticity Index, PI	23
In Place Moisture Content (ASTM D2216) %	0.0

TESTING EQUIPMENT USED		
Plastic Limit	Hand Rolled	X
	Mechanical Rolling Device	
Liquid Limit Apparatus	Manual	X
	Mechanical	
Casagrande ASTM Tool	Metal	X
	Plastic	

SPECIMEN PREPARATION			
Wet		Washed on #40 Sieve	
Dry (Air)		Dry Sieved on #40 Sieve	X
Dry (Oven)	X		

25-Jul-17

DISTRIBUTION:

Hugh Arthur - Laboratory Supervisor

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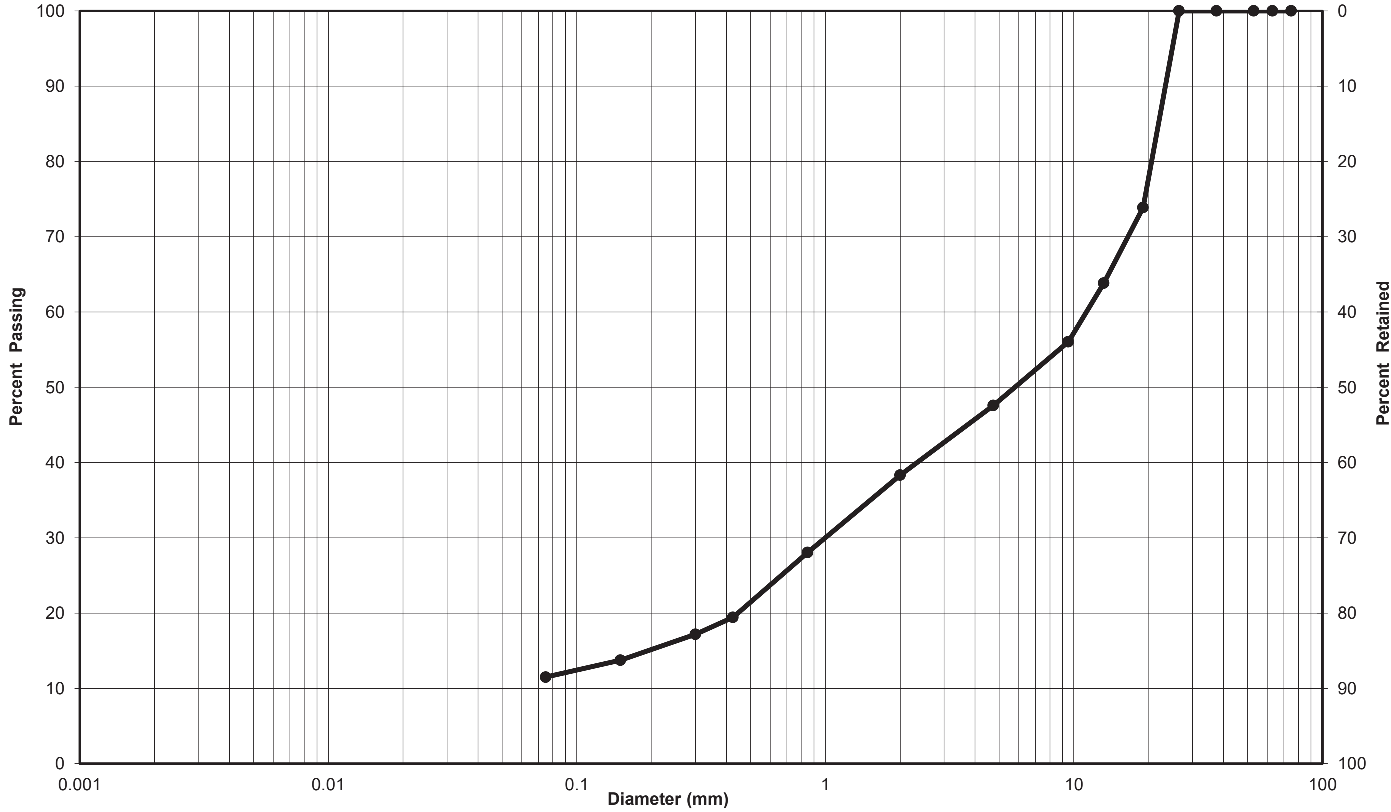
PARTICLE SIZE ANALYSIS OF SOILS

DST Ref. No.:	TS-SO-29563	**Date Sampled:**	January 0, 1900
Project:	Trinity Development Group Geotech Investig	**Sampled By:**	1900-01-00
Client:	Trinity Development Group	**Source:**	BH2017-5, SS-4
Project Location:	Ottawa, ON	**Location:**	0
Sample #:	KWG-016-6	**Description:**	Gravel and Sand, trace Clay/Silt



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Gravel and Sand, trace Clay/Silt	52	37	11

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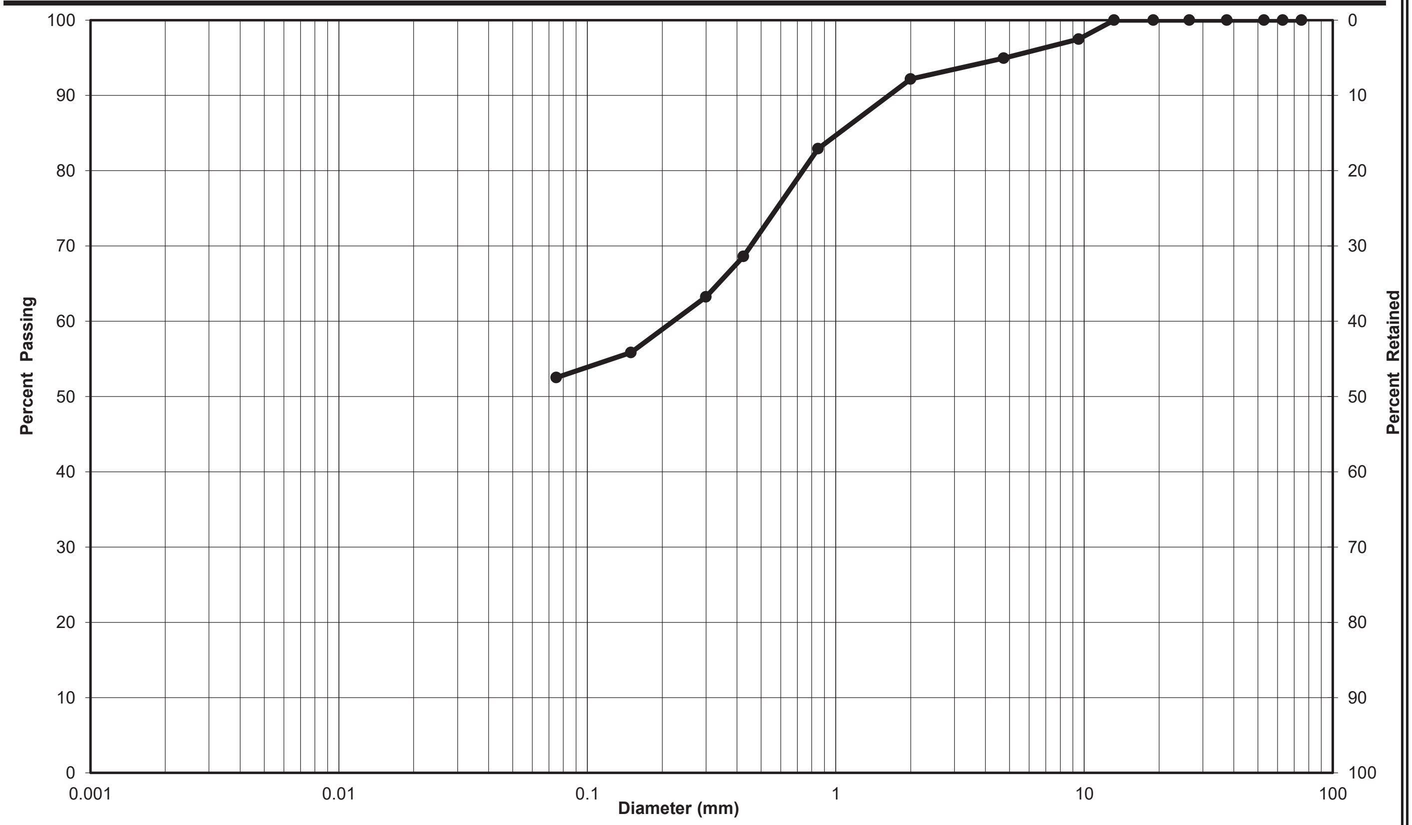
PARTICLE SIZE ANALYSIS OF SOILS

DST Ref. No.:	TS-SO-29563	**Date Sampled:**	January 0, 1900
Project:	Trinity Development Group Geotech Investig	**Sampled By:**	1900-01-00
Client:	Trinity Development Group	**Source:**	BH2017-9, SS-2
Project Location:	Ottawa, ON	**Location:**	0
Sample #:	KWG-016-9	**Description:**	Sand and Clay/Silt, trace Gravel



<table>
<tr><td rowspan="2">Clay & Silt</td><td colspan="3">Sand</td><td colspan="2">Gravel</td></tr>
<tr><td>Fine</td><td>Medium</td><td>Coarse</td><td>Fine</td><td>Coarse</td></tr>
<tr><td colspan="6">Particle-Size Limits as per USCS (ASTM D-2487)</td></tr>
</table>

Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Sand and Clay/Silt, trace Gravel	5	42	53

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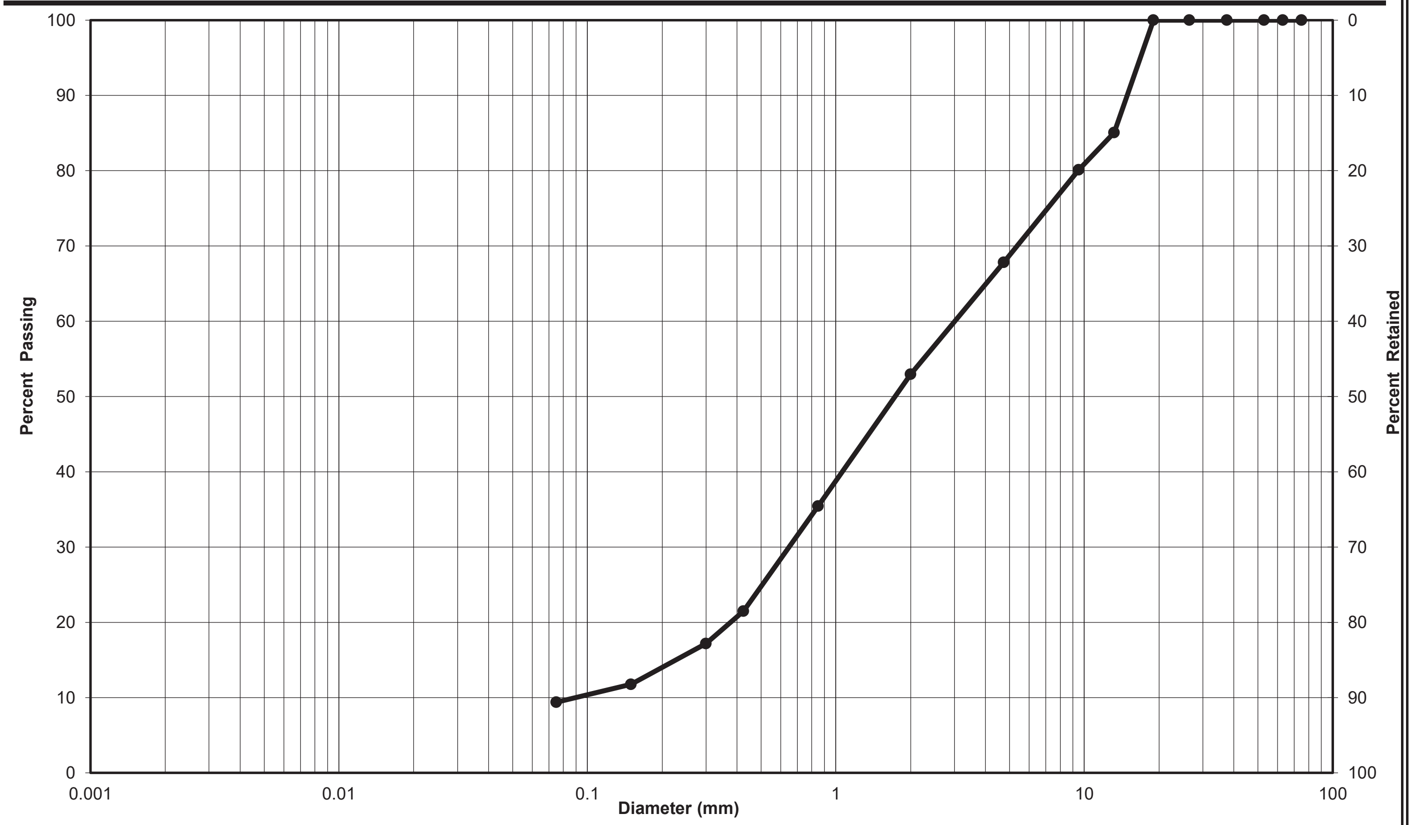
PARTICLE SIZE ANALYSIS OF SOILS

DST Ref. No.:	TS-SO-29563	**Date Sampled:**	January 0, 1900
Project:	Trinity Development Group Geotech Investig	**Sampled By:**	1900-01-00
Client:	Trinity Development Group	**Source:**	BH2017-10, SS-4
Project Location:	Ottawa, ON	**Location:**	0
Sample #:	KWG-016-10	**Description:**	Gravelley Sand, trace Clay & Silt



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Gravelley Sand, trace Clay & Silt	32	59	9

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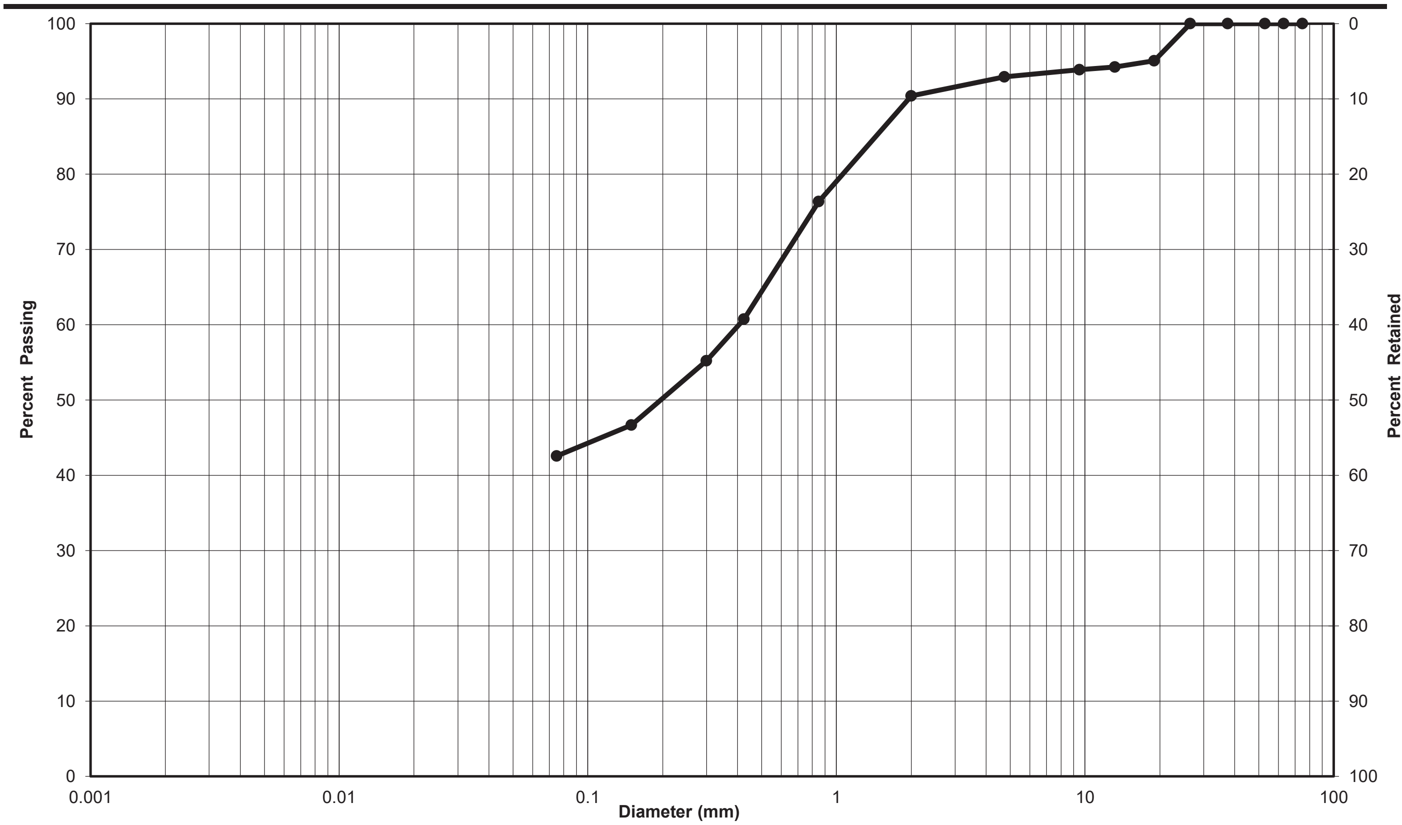
PARTICLE SIZE ANALYSIS OF SOILS

DST Ref. No.:	TS-SO-29563	**Date Sampled:**	January 0, 1900
Project:	Trinity Development Group Geotech Investig	**Sampled By:**	1900-01-00
Client:	Trinity Development Group	**Source:**	BH2017-11, SS-6
Project Location:	Ottawa, ON	**Location:**	0
Sample #:	KWG-016-12	**Description:**	Clay/Silt and Sand, trace Gravel



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

Soil Description	Gravel (%)	Sand (%)	Clay & Silt (%)
Clay/Silt and Sand, trace Gravel	7	50	43

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Table No. 1
Trinity Development – Geotechnical Investigation
DST Project No.: TS-SO-29563
Rock Core Unconfined Compressive Strength Test Result Summary

Davroc Sample No.	Borehole/Core No.	Depth	Unconfined Compressive Strength (MPa)
C974-1	BH2017-3/CR1	96.081 – 95.916	*127.6
C974-2	BH2017-3/CR5	90.124 – 89.845	125.7
C974-3	BH2017-5/CR2	94.468 – 94.138	121.5
C974-4	BH2017-8/CR3	93.526 – 93.348	*113.1
C974-5	BH2017-10/CR1	93.198 - 92.919	97.2
C974-6	BH2017-10/CR4	88.117 – 87.787	129.6

*- L/D ratio for these samples were <2.0.

We trust the above is satisfactory. Should you require any further information, please do not hesitate to contact the undersigned.

Yours very truly,
Davroc Testing Laboratories Inc.

Kateryna Fiyalko, C.E.T.
Concrete Laboratory Supervisor

Sal Fasullo, C.E.T.
Vice President

SF/kf
17-0460-1RC

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - FOUNDATION DRAINAGE AND WATER SUPPRESSION SYSTEM

FIGURES 3 & 4 - SEISMIC SHEAR WAVE VELOCITY PROFILES

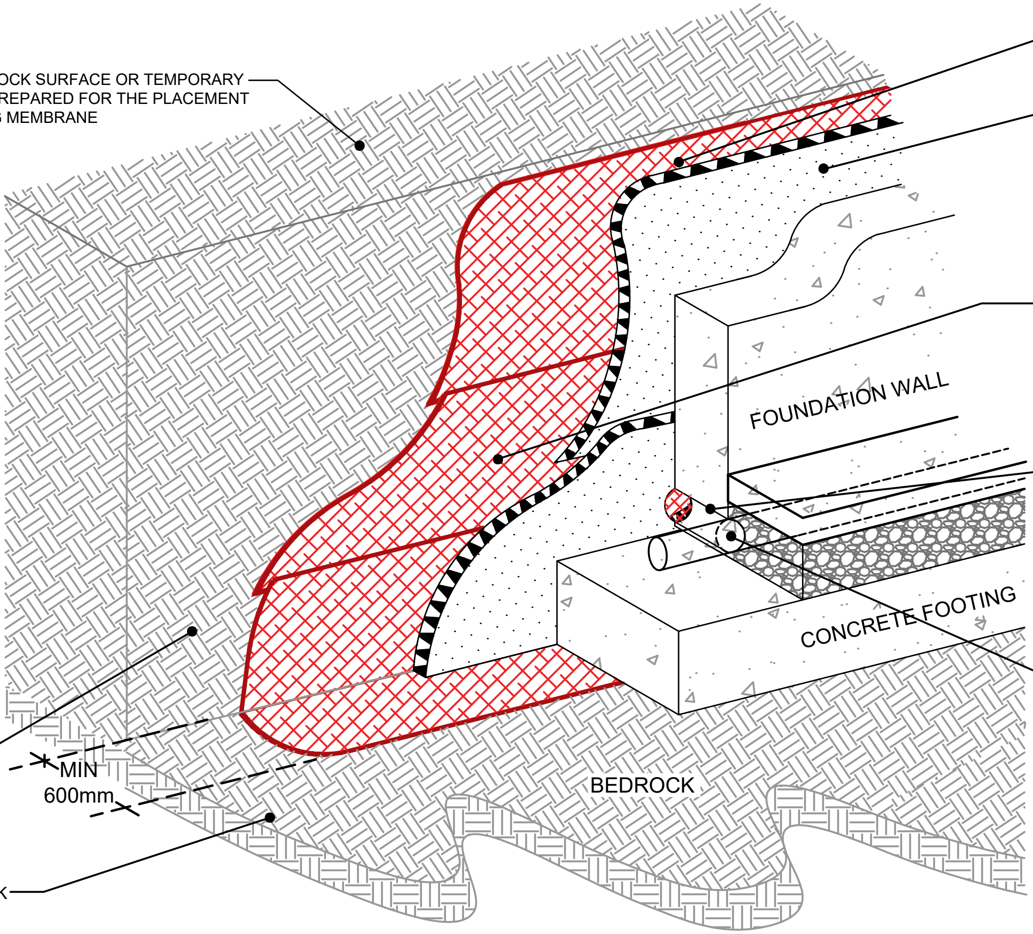
FIGURES 5 & 6 - SLOPE STABILITY ANALYSIS

DRAWING PG5517-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON
GROUP



SMOOTH AND FLAT BEDROCK SURFACE OR TEMPORARY SHORING ADEQUATELY PREPARED FOR THE PLACEMENT OF THE WATERPROOFING MEMBRANE

GEOTEXTILE LINED SIDE OF THE DRAINAGE BOARD FACING THE BEDROCK

COMPOSITE FOUNDATION DRAINAGE MEMBRANE 6000 SERIES MEMBRANE BY DELTA DRAIN, G100N BY MIRADRAIN OR EQUIVALENT. INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150mm HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR. MECHANICALLY SECURE VERTICAL AND HORIZONTAL JOINTS WITH BLUESKIN WP200 AND BLUESKIN PRIMER OR EQUIVALENT.

15miL THICK EXPANDABLE, GRANULAR BENTONITE AND HDPE MEMBRANE. INSTALL IN HORIZONTAL LIFTS TO MANUFACTURERS SPECIFICATIONS IN SHINGLE FASHION WITH THE HDPE SIDE FACING APPLICATOR TO AN ADEQUATELY PREPARED SUBSTRATE SURFACE. WATER PROOFING MEMBRANE TO EXTEND FROM AN ELEVATION OF 60m

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE BASE OF THE EXTERIOR FOUNDATION WALL AT APPROXIMATELY 3.0m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE

THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER INTERIOR SUBFLOOR PERIMETER DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT

SMOOTH AND FLAT BEDROCK SURFACE OR TEMPORARY SHORING ADEQUATELY PREPARED FOR THE PLACEMENT OF THE WATERPROOFING MEMBRANE

CLEAN SURFACE SOUNDED BEDROCK APPROVED BY THE GEOTECHNICAL CONSULTANT

IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE FOUNDATION DRAINAGE SYSTEM .

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PROPOSED MIXED-USE DEVELOPMENT
951 GLADSTONE AVENUE & 145 LORETTA AVENUE NORTH
OTTAWA, ONTARIO

Title:
FOUNDATION DRAINAGE AND WATER SUPPRESSION SYSTEM

Scale: N.T.S.
Date: 03/2021
Drawn by: NFRV
Report No.: PG5517-1
Checked by: NP
Drawing No.: FIGURE 2
Approved by: DG
Revision No.:

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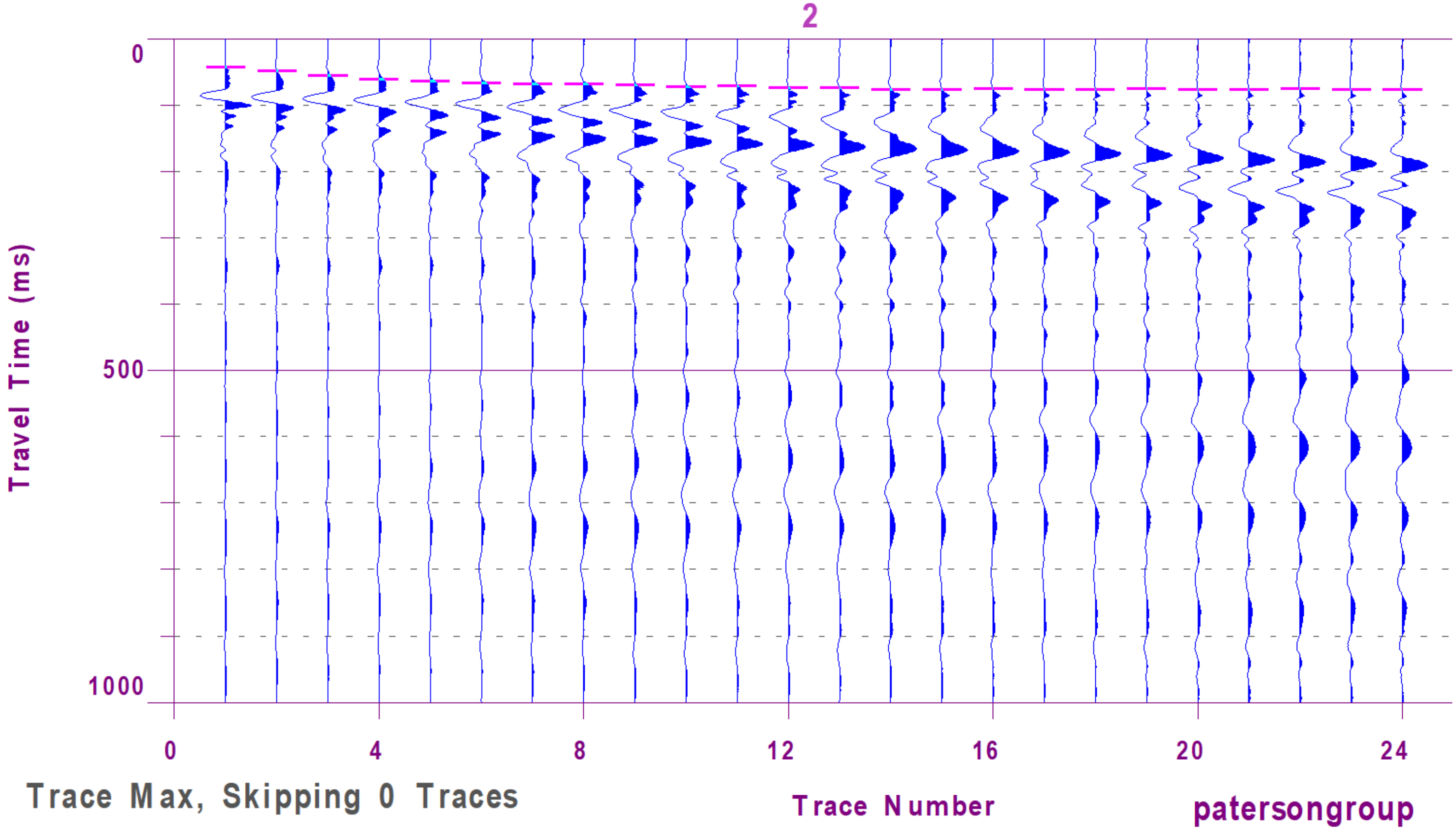

Figure 3 – Shear Wave Velocity Profile at Shot Location -15 m

PATERSON GROUP

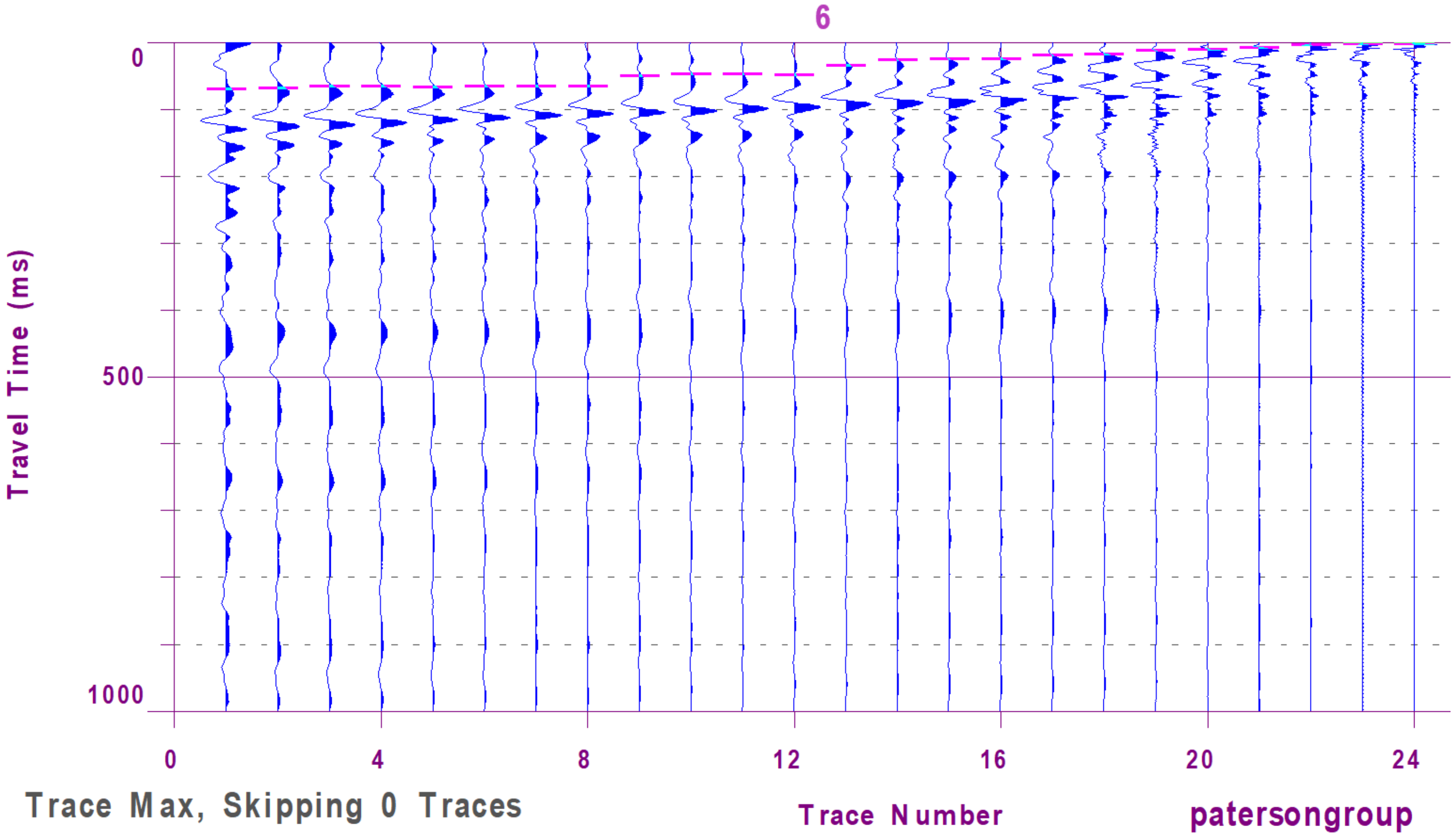

Figure 4 – Shear Wave Velocity Profile at Shot Location 24.5 m

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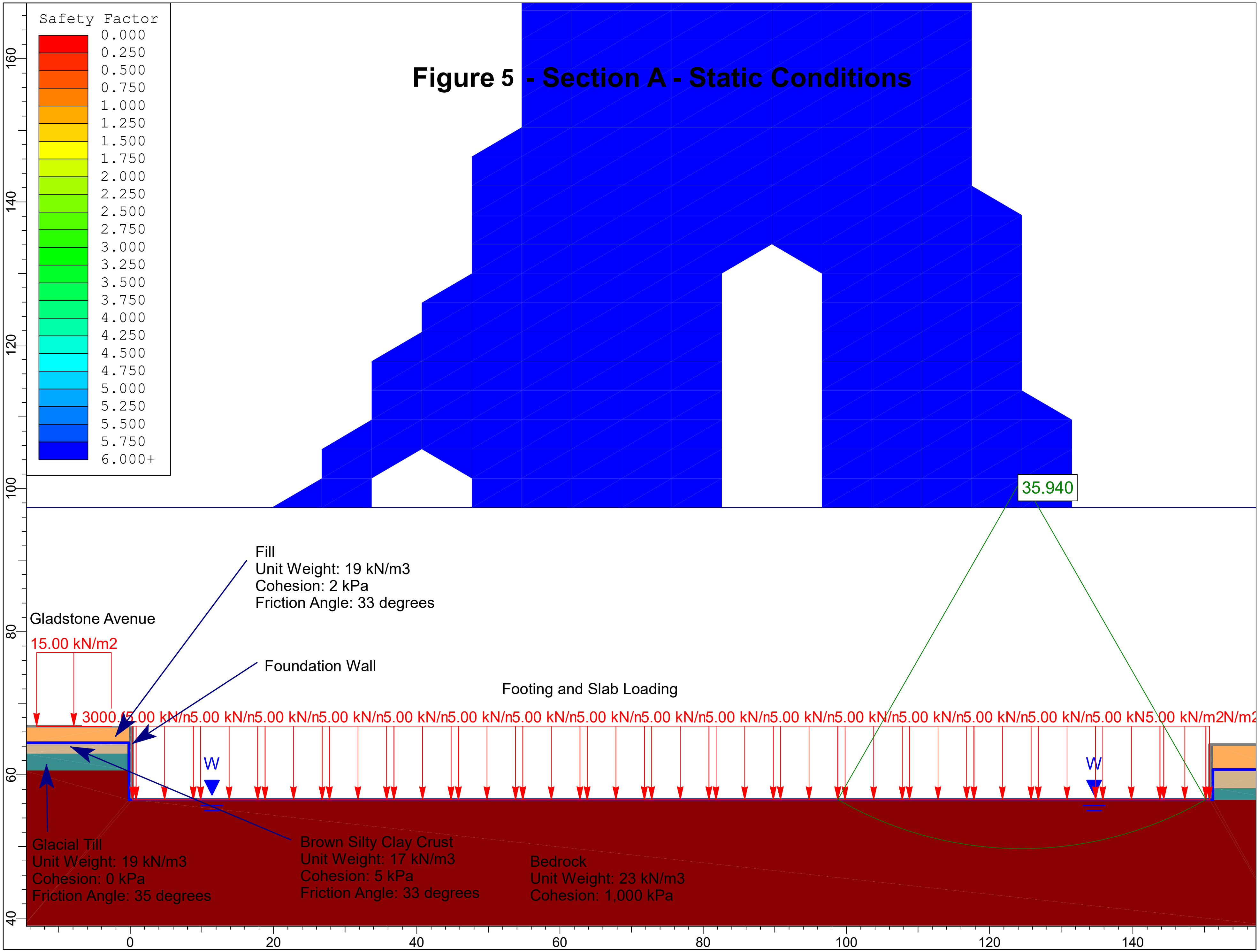


Figure 5 - Section A - Static Conditions

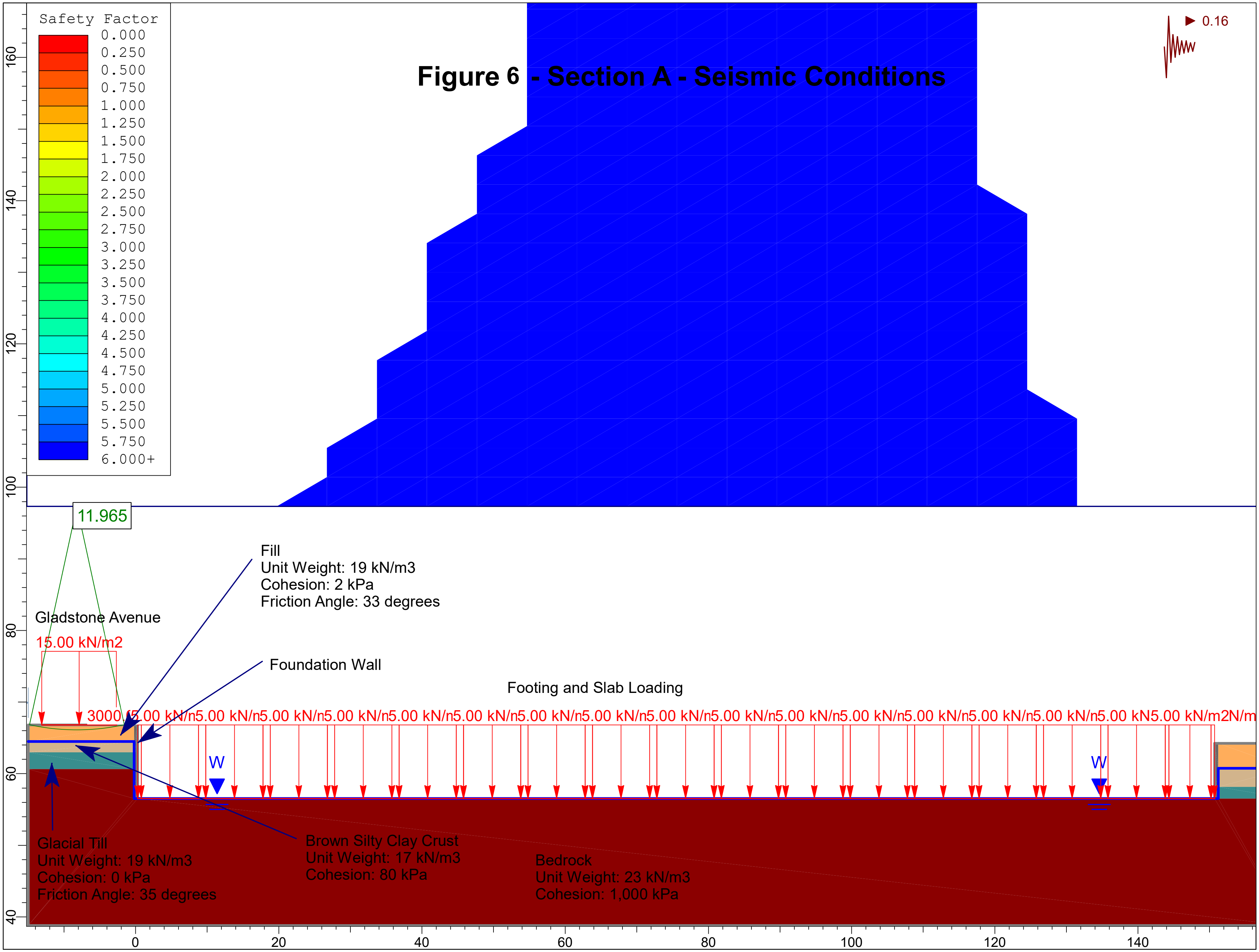
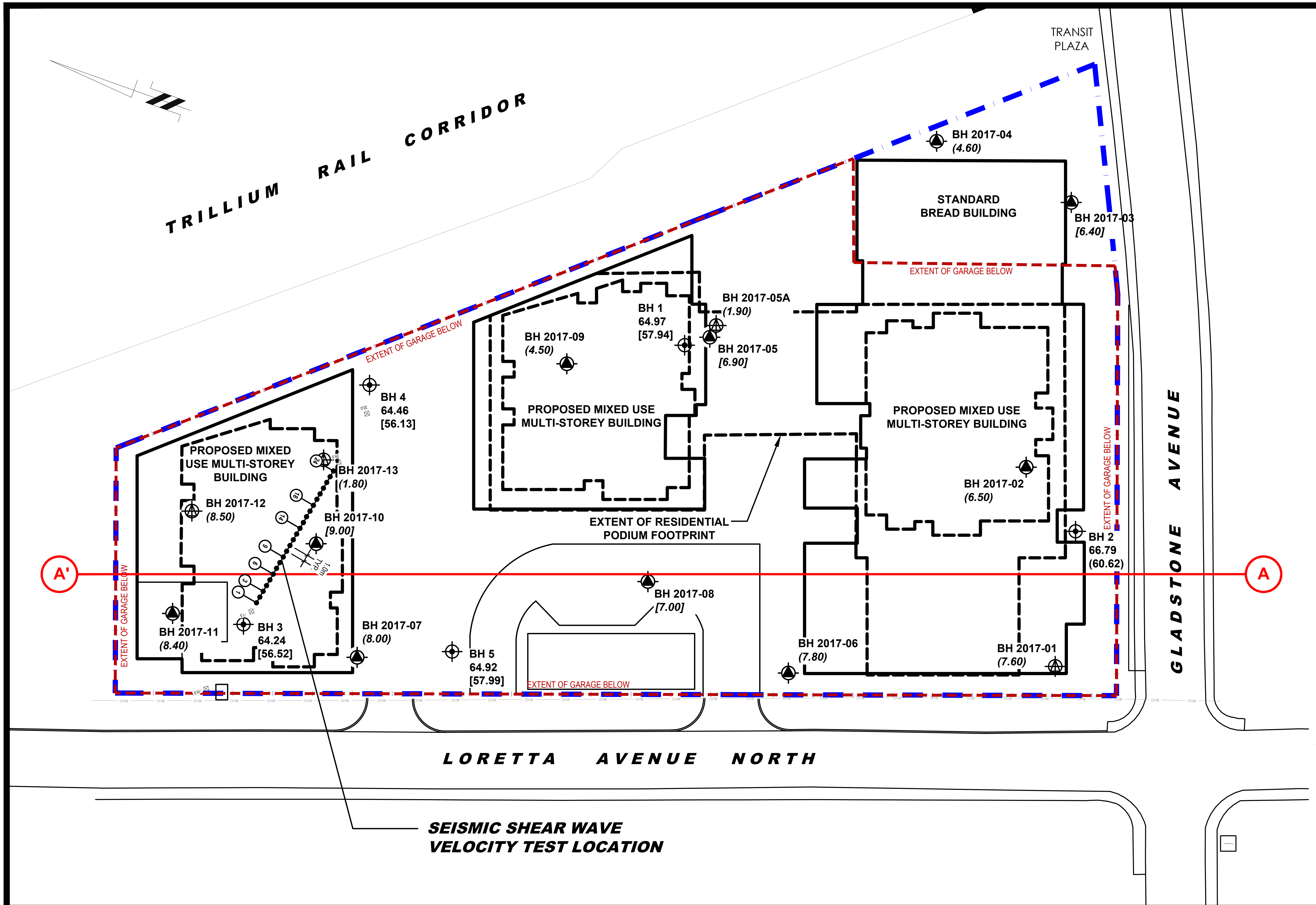


Figure 6 - Section A - Seismic Conditions



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION, CURRENT INVESTIGATION
⌖	BOREHOLE LOCATION (DST CONSULTING ENGINEERS, TS-SO-29563, JULY 2017)
▲	MONITORING WELL LOCATION BY DST CONSULTING ENGINEERS, TS-SO-29563, JULY 2017)
A'—A	CROSS SECTION LOCATION
64.97	GROUND SURFACE ELEVATION (m)
[57.94]	BEDROCK SURFACE ELEVATION (m)
[6.90]	*DEPTH TO BEDROCK SURFACE (m)*
(60.62)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
(7.60)	*DEPTH TO PRACTICAL REFUSAL (m)*
●—●—●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
⌖ +10.0	SHOT LOCATION

BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:600

0 5 10 15 20 25 30m

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PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
3	UPDATED CONCEPTUAL SITE PLAN AND ADDED SEISMIC SHEAR WAVE VELOCITY TEST LOCATION	17/05/2024	FC
2	CROSS SECTION ADDED	01/11/2021	NP
1	UPDATED CONCEPTUAL SITE PLAN	26/03/2021	NP

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GEOTECHNICAL INVESTIGATION
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OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600
Date: 09/2020
Drawn by: NFRV
Report No.: PG5517-1
Checked by: NP
Dwg. No.: PG5517-1
Approved by: SD
Revision No.: 3

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