PRELIMINARY GEOTECHNICAL INVESTIGATION FOR
PROPOSED NEW BUILDINGS, TRUCK SCALES, AND PAVEMENT
2555 SHEFFIELD ROAD
OTTAWA, ON

DST Reference No: TS-SO-37029

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Prepared for:
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Table of Contents

List of Tables

List of Appendices

1. INTRODUCTION

DST Consulting Engineers Inc. (DST), a division of Englobe, is pleased to present the findings of this preliminary geotechnical investigation for the proposed new buildings, truck scales, and pavement (Project) at the existing recycling facility at 2555 Sheffield Road (Site) in Ottawa, Ontario.

DST was retained by AIM Recycling Ottawa East (Client), a division of American Iron & Metal Company Inc., to carry out a geotechnical investigation to evaluate the subsurface conditions at the Site for proposed new buildings, new truck scales. and new pavement. The investigation included the drilling, sampling, and testing of fifteen (15) boreholes across the Site.

Written authorization to proceed with this geotechnical investigation was provided by Mr. Dan Dupelle of the Client February 27, 2019, by means of Purchase Order No: 12-123044DD.

This report is prepared for the sole use of the Client and Designers. The use of the report, or any reliance on it by any third party, is the responsibility of such third party. This report is subject to the limitations shown in Appendix A. It is understood that the Project will be performed in accordance with all applicable codes and standards present within its jurisdiction.

It is important to emphasize that at the time of this geotechnical investigation, DST has not been provided with any structural plans of the proposed new structures or civil plans of the proposed new facilities. Therefore, this is a pre-design geotechnical investigation and should be considered as preliminary in nature. DST requests to be retained again once the Project has progressed to the detailed design stage to ensure the proposed designs meet with the intent of the geotechnical comments and recommendations provided in this report.

2. SITE AND PROJECT DESCRIPTION

The Site is located at a municipal address of 2555 Sheffield Road in Ottawa, Ontario. The location of the Site is shown on the Site Location Map attached as Figure 1 in Appendix B.

The Site is an approximately rectangular property in an industrial part of the City. It is currently occupied by a metals recycling facility including two existing buildings on the Site. One building is located on the south-west and the second building on the north-west corner of the Site. The existing building at the south-west corner of the Site includes a two-storey portion and one-storey structure with slab on grade and it is mainly used as indoor recycling warehouse and offices. The existing building at the north-west corner of the Site is a one-storey slab on grade structure mainly used as a garage and mechanical warehouse. The east side of the Site has an existing on-grade concrete pavement structure and is used as an outdoor recycling yard. The balance of the Site has gravel as surficial cover and is mainly used for storage and traffic circulation purposes. DST has not been provided with any structural drawings or details of the foundations of the existing structures.

DST's understanding of the Site and the proposed Project are based on the preliminary architectural drawings "Recycling Ontario Sheffield Road Layout Development" (Ref No: AIM-ONT-Sheffield-Lay, dated December 5, 2018) prepared by Novatech Architects (Architect). This drawing was provided by the Client at the time of proposal and is presented in Appendix E at the end of this report for reference.

The proposed Project includes demolition of two existing buildings and the construction of two new buildings, as shown on Figure 3. One of the buildings will be a two-storey slab on grade structure with an approximate footprint of 6,040 m^2. It will house a warehouse, storage, office and mechanical spaces. The second building will be a one-storey slab on grade garage building with a mezzanine. It will have an approximate footprint of 930 m^2. In addition to the new buildings, the Project includes the construction of two new truck scales, a new parking lot and a new pavement structure for the outdoor recycling yard and heavy traffic circulation areas. The Client is intending to use a Roller Compacted Concrete (RCC) for the pavement structure. The approximate location of the proposed buildings and truck scales are shown on the Borehole Location Plan attached as Figure 2 in Appendix B.

The following assumptions about the proposed Project were made by DST during the preparation of this preliminary report. Designers should review these assumptions and inform DST if there are any discrepancies, as they may affect the recommendations shown herein:

- The buildings and truck scales will have shallow foundations consisting of conventional pad and strip footings founded on native soils, and below the design frost depth;
- The buildings will be shallow slab on grade structures with no basements or below-ground levels;
- The structures are governed under Part 4 of the Ontario Building Code (OBC-2012) and therefore will require a sit classification for seismic site response;

- There are no new slopes or retaining walls being proposed, and therefore slope stability analyses are not required;
- No significant global grade raises (i.e. greater than 0.5 m) are being planned for this Site, and therefore consolidation settlement analyses are not required; and
- Floor slabs will be lightly loaded with no special point loading from racking, stationary equipment or process machinery. A typical distributed loading of 24 kPa (maximum applied pressure) is anticipated.

The purpose of this geotechnical investigation was to evaluate the subsurface and groundwater conditions at fifteen discrete borehole locations and to provide geotechnical parameters and recommendations to assist in the design of the foundations for the proposed Project.

3. SCOPE OF WORK

The scope of the work was outlined in DST Proposal Ref No: TS-SO-37029 dated February 8, 2019 and was agreed to by the Client on February 27, 2019 by means of a purchase order. DST's mandate consisted of the follow activities:

- To retain a subcontractor to provide both public and private underground utility clearances;
- To retain a geotechnical drilling subcontractor to drill fifteen boreholes within the footprint of the proposed new buildings, truck scales and pavement. Boreholes were to range in depth from approximately 1.5 m below the existing ground surface (mbgs) to approximately 30.0 mbgs or auger refusal.
    - To drill and sample two (2) boreholes to auger refusal or to a maximum depth of 30.0 mbgs. If auger refusal was encountered in these boreholes, 3.0 m of rock coring was to be performed in each,
    - To advance four (4) boreholes to auger refusal or to a maximum depth of 8.0 mbgs, with one of the boreholes instrumented with a groundwater monitoring well,
    - To advance five (5) boreholes to auger refusal or to a maximum depth of 6.0 mbgs, with one of the boreholes instrumented with a groundwater monitoring well; and
    - To advance four (4) boreholes to auger refusal or to a maximum depth of 1.5 mbgs.
- To supervise the fieldwork and log the soil conditions in the boreholes, based on the samples recovered;
- Submit soil samples to the geotechnical laboratory for the following testing:

  - Standard corrosion package testing on two (2) soil samples,
  - Atterberg limits testing on six (6) soil samples,
  - Grain size analyses on eight (8) soil samples; and
- Prepare this geotechnical investigation report.

Borehole locations MW19-05, BH19-05 and BH19-07 were found to have an approximately 0.3 m concrete slab present in on the surface, in the existing yard area. DST was required to retain a coring company to core the concrete at these locations prior to undertaking the drilling.

4. FIELD INVESTIGATION AND LABORATORY TESTING

4.1 Geotechnical Drilling Fieldwork

The fieldwork component of this geotechnical investigation consisted of the drilling of fifteen (15) boreholes at the locations shown on Figure 2 to varying depths across the Site, which were labelled as BH19-01 through BH19-15. The drilling was performed between March 18 and 21, 2019. A geotechnical drilling subcontractor, George Downing Estate Drilling Ltd., was retained to perform the work.

Boreholes were drilled using a truck-mounted CME-55 drill rig, adapted for geotechnical sampling. Boreholes were advanced through the overburden using hollow-stem continuous-flight augers, and into the bedrock using wireline diamond coring methods. Soil samples from overburden were collected using standard 50 mm split-spoon sampler driven by an automatic Standard Penetration Test (SPT) hammer. The compaction of cohesionless soil was assessed using recorded SPT N-values and the consistency of the cohesive soil was assessed using a combination of Field Vane Testing (FVTs) and/or Pocket Penetrometer (PP) resistance values.

Borehole locations MW19-05, BH19-05 and BH19-07 were found to have an approximately 0.3 m concrete slab present on the surface, in the existing yard area. Therefore, DST retained Portelance Concrete Cutting and Coring, on March 20, 2019 to core through the concrete pavement before drilling.

Boreholes MW19-04, MW19-06, and MW19-09 had monitoring well installed, with screens sealed into the native overburdened at approximate depths of 8.0 m 6.4 m and 6.1 mbgs, respectively. All boreholes were backfilled with a combination of bentonite hole-plug, well materials, or/and auger cuttings as necessary.

The soil conditions encountered in the boreholes were described by DST field staff based on the samples that were recovered. The recovered soil samples were labelled and submitted to DST's geotechnical laboratory for further visual examination and geotechnical laboratory testing on selected soil samples.

The location of the boreholes is shown on the Borehole Location Plan attached as Figure 2 at the end of this report. The location and elevation of the ground surface at the borehole locations of the boreholes was surveyed by Annis O'Sullivan Vollebekk Ltd. who was retained directly by the by Client.

4.2 Geotechnical Laboratory Testing

The laboratory testing component of this geotechnical investigation consisted of determination of moisture contents of all recovered soil samples, except for those soil samples which were submitted for environmental testing under the coinciding Phase II Environmental Site Assessment (ESA). Grain size analysis were conducted on four representative fill soil samples between 0.1 and 1.4 m depth and four representative native soil samples between 6.1 and 9.4 m depth. Atterberg limit testing was performed on four representative soil samples between 1.5 and 3.8 m depth. Hydrometer analysis were conducted on four representative soil samples between 4.6 and 6.7 depth. Standard corrosion package testing was conducted on two representative soil samples from approximately 1.5 to 2.1 m depth. The results of the laboratory testing are presented on the Borehole Logs in Appendix C and as Laboratory Test Results in Appendix D.

Environmental or analytical testing of the soil samples was not part of DST's scope of work for this geotechnical investigation. However, this investigation was performed in conjunction with a Phase II ESA, the results of which are provided under separate cover. Designers and Contractors are referred to the Phase II ESA report for comments on the management and disposal of excess soils on this Site.

5. GENERALIZED DESCRIPTION OF SUBSURFACE CONDITIONS

The subsoil conditions encountered at the borehole locations are briefly discussed in the following subsections with a graphical representation of each location presented on the Borehole Logs attached in Appendix C. Soils were classified in according to the Unified Soil Classification System.

A summary of boreholes drilled at this site with the soil layer encountered in each borehole is presented in Table 5-0 below.

Table 5-0: Summary of Borehole Stratigraphy

Borehole	Asphalt/ Concrete Thickness (mm)	Depth of Fill (m)	Silty Clay (Crust) Depth (m)	Grey Silty Clay Depth (m)	Clayey Silt to Silt Depth (m)	Silty Sandy (Till) Depth (m)	Refusal Depth (m)
BH19-01	40	0.9	0.9 - 2.4	2.4 - 4.0		4.0 - 8.2*	
BH19-02		1.2	1.2 - 3.0	3.0 - 4.6	4.6 - 6.1	6.1 - 8.2*	
BH19-03	50	3.6			3.6 - 5.3	5.3 - 9.3	9.3 (DCPT)
MW19-04	270	2.4	2.4 - 4.4		4.4 - 6.2	6.2 - 8.2*	
BH19-05	300	1.5	1.5 - 3.8	3.8 - 5.3	5.3 - 6.1	6.1 - 8.8	8.8 (confirmed rock by coring)
MW19-06		1.8	1.8 - 3.8	3.8 - 5.0	5.0 - 6.4*		
BH19-07	250	2.6	2.6 - 4.6	4.6 - 5.5	5.5 - 10.2		10.2 (DCPT)
BH19-08		3.0	3.0 - 5.0		5.0 - 6.4*		
MW19-09		3.0	3.0 - 5.2		5.2 - 6.4*		
BH19-10A	Auger refusal at approximately 1.2. mbgs, moved borehole 6 m west						
BH19-10B	Auger refusal at approximately 1.3. mbgs						
BH19-11A	Auger refusal at approximately 1.7 mbgs, moved borehole 15 m west						
BH19-11B		2.3	2.3 - 4.6		4.6 - 6.4	6.4 - 10.0	10.0 (confirmed rock by coring)
BH19-12	Auger refusal at approximately 1.3. mbgs						
BH19-13		2.4	2.4 - 3.5*				
BH19-14		1.2					1.2
BH19-15	40	0.9	0.9 - 2.1*				

* End of borehole, refusal not encountered within drilled depth.

It is important to note that the soil descriptions presented below and on the Borehole Logs represent the soils encountered at the test locations only. They may vary between and beyond borehole locations. This is especially true in previously excavated and/or filled areas such as near existing and former utility trenches and building foundations.

5.1 Asphalt and Concrete Pavement

Three boreholes, BH19-01, BH 19-03, and BH19-15 were located in areas which had asphaltic pavement present at the surface. Within these boreholes, the pavement ranged in thickness from approximately 40 to 50 mm.

Three boreholes, BH19-04, BH1-05, and BH19-07were located in an area with concrete pavement present at the surface. In these locations the concrete pavement was found to range in thickness form approximately 250 to 300 mm

In the remaining borehole locations, there were fill soils present directly on the surface and there were no surficial coverings It is important to note that the thickness and descriptions of surficial coverings and pavement structures noted above are for planning purposes only. They should not be used for quality assessments or quantity take-offs.

5.2 FILL

At different borehole locations, there were two distinct FILL types present. These were described as either granular fills, or reworked local soils.

The granular FILL consisted of a silty sand and gravel. It extended to depths ranging from approximately 0.9 mbgs in borehole BH10-01 and BH19-15 to approximately 2.4 mbgs in borehole BH19-04.

The second FILL type was mainly reworked material consisted of silty sand and silty clay soil extended to considerable depths ranging from approximately 0.9 to 3.0 m in borehole BH19-3. It is important to note that borehole BH19-3 was approximately 1.5 m away from the wall of the existing office/administration building. In terms of elevation, the fill was found to extend from approximately 66.1 to 63.4 masl. Borehole BH19-10, BH19-12 and BH19-14 were terminated within this layer due to auger refusal.

The moisture contents of the recovered samples were determined in DST Laboratory and presented on the detailed borehole logs. It should be noted that the first 0.8 m of the fill layer was augered because the frost extended up to 1.2 mbgs in some areas of the Site. This was witnessed in borehole BH19-07 where SPT refusal was encountered at 0.9 mbgs due to frost layer. Generally, where frost and cobbles/boulder size soil were not encountered, the SPT N-values of the fill varied between 12 and 24 indicating compact conditions.

Four soil samples from the FILL were tested for grain size distribution and summary of the results are presented in Table 5-1 below and attached in Appendix D. A soil sample from this fill layer was also tested for corrosivity and the result is presented in Appendix D.

Table 5-1: Summary of Grain Size Analyses in FILL

Sample ID	Sample Depth (mbgs)	% Gravel	% Sand	% (Silt & Clay)
BH19-02, GS1	0.1 – 0.8	27	46	27
BH19-03, GS1	0.1 – 0.8	39	36	25
BH19-06, GS1	0.1 – 0.7	18	51	31
BH19-11B, SS2	0.8 – 1.4	36	38	26

5.3 Buried Structures

Based on our conversations with the Client’s contact on Site, DST understands that there are possible existing foundation elements and/or buried concrete present in the north east corner of the building. A review of the aerial publicly available aerial photographs from 2007 suggest that this may have been a concrete lined storage area. Boreholes BH19-10A, BH19-10B, BH19-11A, and BH19-12 were located in this area, and had encountered shallow auger refusals at depths ranging from approximately 1.2 to 1.7 mbgs.

5.4 Silty Clay

In all borehole locations, the fill soils were underlain by a native deposit of silty clay with trace sand seams, extending to depths ranging from approximately 0.9 to 5.5 mbgs (elevations near 66.4 to 62.1 masl). This deposit was brown to grey in color and the moisture contents generally indicates moist to wet. The moisture contents of the recovered samples were determined in DST Laboratory and presented on the detailed borehole logs in Appendix C. Borehole BH19-13 and BH19-15 were terminated within this layer at the target depth.

Field vane and penetrometer testing were performed on this cohesive layer and the undrained shear values varied significantly from approximately 196 to 50 kPa which generally indicates a very stiff to stiff/firm consistency from upper to lower clay layer, respectively. Based on the Atterberg limit tests, the silty clay layer has low to high plasticity limit. This deposit was classified high plasticity clay (CH) near the higher elevations and the low plasticity clay (CL) near the lower elevations of the deposit.

Four soil samples were tested for Atterberg limits and the results are summarized in Table 5-4 below and included in Appendix D.

Table 5-2: Summary of Atterberg Limits in Silty Clay

Sample ID	Sample Depth (mbgs)	LL (%)	PL (%)	PI (%)
BH19-5, SS2	1.5 – 2.1	53	19	34
BH19-5, SS5	3.8 – 4.4	28	20	8
BH19-11B, SS4	2.3 – 2.9	67	21	46
BH19-11B, SS6	3.8 – 4.4	36	18	18

5.5 Silt

The silty clay deposit was found to be underlain by a relatively thin layer of silt deposit extending to depths ranging from approximately 3.6 to 6.4 mbgs (near elevation 62.2 to 60.5 masl). The silt was described as grey and wet to saturated. The moisture contents of the recovered samples were determined in DST Laboratory and presented on the detailed borehole logs in Appendix C. Borehole BH19-6, BH19-8 and BH19-9 were terminated within this layer at the target depth.

The recorded SPT N-values for this layer ranged from 4 to 11, which generally indicates a loose to compact soil relative density. Four soil samples were tested for Hydrometer analysis and the results are summarized in Table 5-5 below and included in Appendix D. Based on the Hydrometer analysis results, the silt layer deposit contains some clay with trace to some gravel and sand. The silt layer was found to be a transition zone between the silty clay layer and silty sand till deposits.

Table 5-3 Summary of Hydrometer Tests in Silt

Sample ID	Sample Depth (mbgs)	% Gravel	% Sand	% Silt
BH19-05, SS6	5.5 - 5.9	10	16	67
BH19-11B, SS7	4.6 - 5.2	1	1	85
BH19-11B, SS8	5.3 - 5.9	3	3	91
BH19-11B, S9B	6.4 - 6.7*	6	42	37

5.6 Clayey Silt to Silty Sand (Till)

The silt deposit was found to be underlain mainly by a till deposit in all boreholes from approximately 5.3 to 10.0 mbgs depths (approximate elevations near 61.7 to 57.4 masl). The till deposit consisted of main silty sand with some gravel to gravelly, trace to some clay and possible cobbles and/or boulders at the lower elevations of this deposit confirmed through coring in borehole BH19-5 and BH11B. It was described as fine to medium grained, brown to grey in color and moist to wet. Borehole BH19-1 to BH19-4 were terminated within this layer at the target depth.

The recorded SPT N-values for this till deposit ranged from 4 to refusal, N-values greater than 50 for 150 mm penetration or less, indicating a significant variation in soil density from loose near the top elevation to very dense near the bottom elevation of the till deposit.

The moisture contents of the recovered samples were determined in DST Laboratory and presented on the detailed borehole logs in Appendix C. Four soil samples from this sand layer were tested for grained size and the summary of the result is presented in Table 5-6 and attached in Appendix D.

Table 5-4: Summary of Grain Size Analyses in Till

Sample ID	Sample Depth (mbgs)	% Gravel	% Sand	% (Silt & Clay)
BH19-05, SS7	6.1 - 6.7	33	33	34
BH19-5, SS8	7.8 – 8.2	15	45	39
BH19-11, SS10	7.6 – 8.2	21	44	35
BH19-11, SS11	9.1 – 9.4	33	36	31

5.7 Bedrock

The clayey silty sand till was found to be underlain by bedrock, which was confirmed by coring in boreholes BH19-05 and BH19-11. Approximate depth of r inferred bedrock was also estimated in borehole BH19-7 and BH19-3 through overburden by means of dynamic cone penetration test. The bedrock was found to extend from approximately 8.8 mbgs (approximate elevation 58.3 masl) in BH19-05 and approximately 10.0 mbgs (approximate elevation 57.4 masl) in BH19-11. Dynamic cone penetration refusal encountered at approximately 9.3 (approximate elevation 57.8 masl) in BH19-03 and approximately 10.0 mbgs (approximate elevation 56.9 masl) in BH19-07. Therefore, the top of bedrock extends from approximate depth of 8.8 to 10.2 mbgs or approximate elevation of

58.3 to 56.9 masl. The bedrock was generally consisted of limestone interbedded with shale. It was visually described as slightly weathered to fresh and was generally fair to good quality rock based on Rock Quality Designation (RQD) index.

5.8 Groundwater

Throughout the drilling and sampling operations, the recovered soil samples were described in the field as being damp to moist. Wet soil samples were recovered from approximately 3.0 to 8.8 mbgs. The variations of measured moisture contents with depth are shown on detailed borehole logs in Appendix C.

Monitoring well was installed in BH18-04, BH19-06 and BH19-09. DST returned to the Site on March 29, 2019 and measured the water level at approximate depth from 2.3 to 3.6 mbgs or approximate elevation 65.0 to 63.5 masl. The recorded water elevations are summarized in Table 5-5 below.

Table 5-5: Monitoring Well Observations

Location	Screened Interval (mbgs)	Soil Description at Screened Interval	Water Level Observations		
			Depth (mbgs)	Elevation (masl)	Date
MW19-4	4.3-7.6	Native Clays and Silts	3.6	63.7	March 29, 2019
MW19-6	2.3-6.4	Native Clays and Silts	2.3	64.7	
MW19-9	4.0-6.1	Native Clays and Silts	2.3	65.0	

It should be noted that groundwater levels are subject to seasonal fluctuations and in response to precipitation, flooding, and snowmelt events. Typically, they are at their highest during the spring thaw.

6. DISCUSSION AND RECOMMENDATIONS

Based on the results of the field investigation and laboratory testing performed at the fifteen discrete borehole locations, the following discussion is provided to assist the Client and Designers with the design of the foundations for the proposed Project. The recommendations provided within this report are based on our current understanding of the proposed Project which is summarised above in Section 2. If any of these understandings or assumptions change, then DST should be contacted to assess the implications of those changes on the recommendations provided herein.

Based on the soil conditions encountered in fifteen discrete boreholes, and assuming them to be representative of the soil conditions across the Site, the most important geotechnical considerations for the design of the foundations for proposed Project are expected to be the following:

- **Removal of Existing Foundation Elements:** There will be existing foundation elements present on Site once the building is demolished. Furthermore, there was shallow refusal on assumed buried concrete in the proposed garage location. DST recommends that several test pits be performed in these areas before construction, so that Designers and Contractors can assess the depth of the existing foundations and fill soils;

- **Variable Depths of FILL soils:** The existing FILL soils on Site were found to range in depth from approximately 1.0 to 3.7 mbgs. FILL soils are not suitable to support any new footings. Excavations for footings should extend below all FILL soils down to the native undisturbed stiff silty clay

- **Grade Raise Limitations:** The native clayey soils on this Site are subject to consolidation settlement. The bearing pressures provided within this report are based on the assumption that there are no significant (i.e. greater than 0.5 m) grade raises planned for this Site. If grade raises are envisioned, then additional consolidation testing and settlement analyses will be required;

- **Management and Disposal of Excess Soils:** Management of excess soils and evaluation of the environmental quality of subsoils is not within the scope of this Geotechnical Investigation. Designers and Contractors are referred to the Phase II ESA report prepared under separate cover by DST for further guidance on soil quality.

- **Assumed Slab on Grade Loadings:** A typical floor slab loading for a lightly loaded slab on grade would involve a maximum pressure of 24 kPa. DST has not been provided with any specific floor slab requirements such as racking, process equipment or other concentrated loadings. If higher loadings are envisioned, then DST should be retained to perform additional consulting in regard to design of the floor slab.

Again, it is important to reiterate that at the time of this geotechnical investigation, DST has not been provided with any structural plans of the proposed structures or civil plans of the proposed Project. It is understood that this is a pre-design geotechnical investigation and should therefore be considered as preliminary in nature. DST requests to be retained once designs have progressed to ensure the designs meet with the intent of the comments are in conformance with this report.

6.1 Site Preparation

The Site should be graded in the earlier stages of construction to provide for positive control of surface water and directing it away from excavations and subgrades. Appropriate provisions should be made for collection and disposal of surface and ground water and runoff including an adequate pumping system, if necessary.

6.1.1 Removal of Existing Foundations and Fill Soils

DST understands that the existing structure on the south west corner of the Site is being demolished in order to construct the new two-storey structure. Therefore, once the building is demolished, there will be foundations, floor slabs, utilities, and associated disturbed material and FILL that will need to be removed and disposed of. DST recommends that several test pits be performed along the existing foundation before construction, so that Designers and Contractors can assess the depth of the existing foundations and fill soils in the building area. In boreholes BH19-01 through BH19-05 the fill soils were found to extend to depths ranging from approximately 1.0 to 3.7 mbgs, or to an approximate elevation near 65.5 to 63.3 masl.

On the north-east corner of the Site, in the location of the proposed garage, boreholes BH19-11 and BH19-12 had encountered shallow auger refusals at depths ranging from approximately 1.2 to 1.7 mbgs. A review of the publicly available aerial photograph from 2007 suggest that this may have been a concrete lined storage area. DST recommends that several test pits be performed in this area before construction, so that Designers and Contractors can assess the depth and nature of the existing concrete

All existing foundation elements and fill soils will need to be removed from within the footprint of the proposed new structures, to expose a native undisturbed subgrade. The use of Engineered Fill may be required to raise the resultant excavation back up to the design foundation level.

These items, as well as any corresponding excess soils should be disposed of in a manner consistent with all applicable environmental legislation. Management of excess soils and evaluation of the environmental quality of subsoils is not within the scope of this Geotechnical Investigation. Designers and Contractors are referred to the Phase II ESA report prepared under separate cover by DST for further guidance on soil quality and the necessary environmental management.

6.1.2 Interference with Existing Underground Services

DST understands that there are existing underground services present within the footprint of the proposed new structures, particularly the new two-story structure. Designers and Contractors must be aware of these and ensure the design and construction properly identify and address potential conflict or interference. Typically, existing underground services will need to be removed and re-routed around the future structures. The existing trench excavations will need to have existing fills and services removed from the trenches and then have the proposed subgrades evaluated and approved by the Geotechnical Engineer before placing new appropriate fill soils. These excavations need to be backfilled with Engineered Fill to ensure proper support where footings and/or floor slabs are proposed.

6.1.3 Subgrade Preparation and Footing Base Evaluation

The existing FILL soils are not suitable to support any new footings. Therefore, excavations for footings should extend below all FILL soils down to the native undisturbed stiff silty clay. Based on the boreholes, the native silty clay is expected at approximate depths of 0.9 to 3.0 mbgs, or approximately elevation of 66.4 to 63.4 masl.

All footing subgrades must be evaluated and approved by a Geotechnical Engineer to ensure that the native subgrade is free of any organics, roots, fill, loose or disturbed soils and can support the design bearing pressure. Any identified local anomalies or soft spots should be subsequently sub-excavated, replaced with Engineered Fill in accordance to the comments in Section 6.10.

The existing silty clay on this Site is sensitive to strength loss upon disturbance. If it is disturbed by over-excavation, remoulding, equipment and foot traffic, or subjected to excess water, it will lose its initial strength and will need to be sub-excavated. Contractors should use excavation methods that minimize disturbance to the clay subgrades. Final excavations should be performed with a smooth-edged ditching bucket. It is recommended that designs incorporate the use of a lean mix concrete mud mat on the approved subgrade surfaces to protect the clay and to provide for a clean dry working surface.

6.1.4 Subgrade Preparation for Floor Slabs

The existing fill soils are typically not suitable to remain under future floor slabs. Therefore, excavations should extend below all fill to expose native undisturbed silty clay. The resultant subgrade must be reviewed and approved by the Geotechnical Engineer.

Subgrades must be free of fills, disturbed or loosened soils, roots, and any organics. The exposed subgrades should be proof-rolled using a loaded dump truck or large vibratory roller to identify soft spots, deflection, rutting, or local anomalies. Any identified local anomalies or soft spots should be subsequently sub-excavated, replaced with suitable imported fill, and compacted. Any imported fill underlying floor slabs should be considered as Engineered Fill and treated in accordance to the comments in Section 6.10.

If the Client or Designers wish to consider the possibility of allowing a portion of the existing fill soils to remain under the slab on grade, then the existing fill must be further assessed at the outset of construction. The assessment would require further evaluation of the quality and compactness of existing fill material via test pits at appropriate locations through the fill, as well as a thorough proof rolling of the surface.

6.2 Excavations

Based on DST’s current understanding of the Project, we anticipate that the deepest excavations will be for the building excavations and will be a maximum of approximately 3.0 m deep. Therefore, it is expected that excavations will be performed using sloped open excavations.

All excavations must be undertaken in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA) Regulations for Construction. The comments within this subsection are intended to be an addition to, and not a replacement of the current OHSA requirements.

- The existing fill soils would be considered as a “Type 3 Soil” according to the regulations. However, if it becomes wet or muddy it would become a “Type 4 Soil”;
- The existing native silty clay would also be considered as a “Type 3 Soil” according to the regulations. However, if it becomes wet or muddy it would become a “Type 4 Soil”;
- According to the OHSA, excavations which penetrate through multiple soil types should be considered as having the highest soil type.

The stability of the excavation side slopes is highly dependent on the Contractor’s methodology and layout. No surface surcharges should be placed closer to the edge of the excavation than a distance equal to twice the depth of the excavation, unless an excavation support system has been designed to accommodate such a surcharge.

No excavations should penetrate below an imaginary line drawn downwards and outwards at 7V:10H slope from the toe of any existing footing or load bearing elements, in order to avoid undermining them. If the limitation of not undermining adjacent structures or infrastructure is unavoidable then and Engineered Shoring system would need to be employed, along with a specific backfilling plan.

The existing silty clay on this Site is sensitive to strength loss upon disturbance. If it is disturbed by over-excavation, remoulding, equipment movement, foot traffic, or subjected to excess water, it will lose its initial strength and will need to be sub-excavated. Contractors should use excavation methods that minimize disturbance to the clay subgrades.

6.3 Temporary Construction Dewatering

Contractors should be prepared to handle any surface or groundwater infiltration by ditching, pumping and/or other methods in order to maintain dry working conditions. There may be perched water at the fill/native interface. Furthermore, if excavations intercept existing or former service trenches, then the backfill in these trenches could act as a drain supplying unexpected offsite water into excavations.

The deepest excavations are understood to be for the building excavations and are anticipated to be a maximum of approximately 3.0 m deep. This corresponds to an approximate elevation near 63.3 masl. The water levels in the monitoring wells were measured at approximately 2.3 to 3.6 mbgs or near an approximate elevation near 65.0 to 63.5 masl. Therefore, the deepest excavations may be below the water. Designers and Contractors should be aware of this condition and plan accordingly.

Hydrogeological consulting in support of a Permit to Take Water (PTTW) or an Environmental Activity Sector Registry (EASR) application were not within DST's scope of work. Assessments of the quantity of water to be expected in excavations, was not part of DST's scope of work.

6.4 Foundations

Based on our understanding of the proposed Project and the soil conditions encountered within the boreholes it is recommended that the foundations for the proposed structures consist of conventional shallow pad and strip footings founded on native undisturbed very stiff/stiff silty clay below the design frost depth.

The existing FILL soils are not suitable for bearing of any foundation elements. Therefore, excavations for footings should extend below all FILL soils and down to the native undisturbed silty clay or alternatively, be placed on new Engineered Fill directly in contact with native undisturbed silty clay. Based on the boreholes this would be encountered at approximate depths of 1.0 to 3.0 mbgs, or approximate elevations near 66.1 masl to 63.4 masl.

Footings at varying levels and/or constructed adjacent to utility trenches, sump pits or similar should be constructed such that the higher footings be set at a level below an imaginary line constructed 7V:10H from the base of the lower excavation. Step footings should be designed with benching no steeper than 2H:1V along their length, and steps no higher than 0.3 m.

6.4.1 Recommended Bearing Pressures on Native Silty Clay Crust

For conventional pad footings up to 2.0 m by 2.0 m and strip footings up to 1.0 m wide founded on the native undisturbed very stiff Silty Clay Crust, a factored bearing capacity of 150 kPa under Ultimate Limit States (ULS) conditions is recommended. This includes for a geotechnical resistance factor of $\Phi = 0.5$. A Serviceability Limit States (SLS) design bearing pressure of 100 kPa is generally recommended, subject to settlement checks during detail design stage. This assumes a maximum tolerable differential settlement in the order of 19 mm and a maximum tolerable total settlement in the order of 25 mm. These assumptions need to be confirmed by detailed settlement calculations by the Geotechnical Consultant during detailed design and prior to any foundation drawing issued for construction.

It is important to emphasize that this assessment was performed considering that there are no significant (i.e. greater than 0.5 m) grade raises planned for this Site. If grade raises, larger dimensioned footings, or higher bearing pressures are used, then this layer has the potential to undergo consolidation and would result in intolerable foundation settlements. If grade raises, larger dimensioned footings, or higher bearing pressures are necessary then DST should be retained to perform additional fieldwork, testing, and Engineering analysis to support a consolidation settlement analysis. Alternatively, piled foundations may also be considered.

Subgrade preparation for silty clay subgrades will involve the removal of all fills, organics, disturbed or previously excavated soil to expose a native undisturbed silty clay. The exposed surface should be examined by the Geotechnical Engineer to assess the competency. Any identified local anomalies or soft spots should be subsequently excavated, replaced with new Engineered Fill.

Final excavations should be performed with a smooth-edged ditching bucket. The use of a lean mix concrete mud mat on the approved subgrade surfaces is recommended to protect the clay and to provide for a clean dry working surface.

6.4.2 Recommended Bearing Pressures on Engineered Fill

Engineered Fill may be used, if necessary, to raise the grade between the approved native silty clay subgrade and the design footing elevation, or to correct irregularities in the design subgrades.

For this Site, if Engineered Fill is required below footings, then the recommended bearing pressures are recommended to be the same as for the native undisturbed very stiff Silty Clay Crust (i.e., 150 kPa factored ULS and 100 kPa SLS). Again, this assumes a maximum tolerable differential settlement in the order of 19 mm and a maximum tolerable total settlement in the order of 25 mm.

Designers and Contractors must ensure that any Engineered Fill used to raise the grade below the footings and above the silty clay, has sufficient lateral extent. At a minimum, the Engineered Fill beneath foundations should extend laterally a distance of 0.3 m beyond the edge of the footing and then be sloped downward and outward at 1H:1V slope. Designers and Contractors are cautioned that the resultant excavation can be quite large if a significant thickness of Engineered Fill is required. Alternatives would be to lengthen foundation walls and step footing down if necessary.

Subgrade preparation below Engineered Fill will be similar to that for footings as noted above. The exposed surface should be examined by the Geotechnical Engineer or a qualified technologist working under the supervision of a Geotechnical Engineer to assess the competency. Engineered Fill must be treated in accordance to the requirements in Section 6.10.

6.5 Frost Protection

All footings for heated structures must be provided with a minimum of 1.5 m of earth cover for heated and 1.8 m of earth cover for unheated or isolated structures in the Ottawa area. Otherwise an equivalent insulation detail would be required in order to provide adequate protection against frost action. Where soil cover cannot be provided, an insulation detail should be designed or approved by a Geotechnical Engineer. Contractors must be aware that this detail may be such that the insulation may need to be placed below the footing and then the footing poured on top, and therefore pre-approval is recommended to ensure excavations and backfill are properly planned.

In building areas with garage doors that remain open for significant periods of the day, it is recommended to use the 1.8 m design frost depth notes above. It is anticipated that the Structural Designer will be designing an under-slab insulation detail specific for the garage door areas.

Should construction take place during the winter, surfaces that support foundations or Engineered Fill must be protected by Contractors against freezing for the entire duration of construction or until adequate soil cover is in place. Backfill soils should not be placed in a frozen condition or placed on frozen subgrades.

6.6 Seismic Site Classification

In accordance with the OBC-2012, structures designed under Part Four of the Code must be designed to resist a minimum earthquake force. Based upon the results of the drilling program, we recommend that this structure be designed to “Site Class D”, with respect to Table 4.1.8.4.A of the OBC-2012, and subject to the limitations of the code.

DST has performed a preliminary screening of the liquefaction potential of the soils on this Site and has concluded that the existing soils are not considered to be liquefiable under the current design earthquake for a structure of “normal importance”.

6.7 Corrosion Potential of Soils

Analytical testing was carried out on two soil samples collected from borehole BH19-02, sample number SS3 and BH19-9 sample number SS3 and SS4, to determine corrosion potential of the subsurface soils. The selected soil samples were tested for pH, resistivity, chlorides, sulphides, sulphates, and redox potential. The test results are summarized in the following table.

Table 6-1: Corrosion Parameter Results

Parameter	Tested Value (BH19-02, SS3)	Tested Value (BH19-03, SS3 and SS4)
pH	7.39	7.98
Chloride (ug/g)	94	480
Sulphate (ug/g)	148	174
Resistivity (Ohm-cm)	2400	800
Redox Potential (mV)	+437	-328
Sulphide (%)	0.02	0.02

The American Water Works Association (AWWA) publication 'Polyethylene Encasement for Ductile-Iron Pipe Systems' ANSI/AWWA C105/A21.5-10 dated October 1, 2010 assigns points based on the results of the above tests. A soil that has a total point score of 10 or more is considered to be potentially corrosive to ductile iron pipe. Based on the results obtained for the sample submitted, the Site soils from borehole BH19-9, garage building, are considered to be potentially corrosive to ductile iron pipe based on Resistivity value and Redox Potential value. The soils from borehole BH19-02, two-story building, are considered to be slightly corrosive to ductile iron pipe based on low Resistivity values.

The analytical results of the soil samples were compared with applicable Canadian Standards association (CSA Standards A23.1-04) standards and are given in Table 6-2 below.

Table 6-2: Additional Requirement for Concrete Subjected to Sulphate Attack

Class of Exposure	Degree of Exposure	Water soluble Sulphate in soil sample (%)	Cementing Material to be used
S-1	Very Severe	> 2.0	HS or HSb
S-2	Severe	0.20 – 2.0	HS or HSb
S-3	Moderate	0.10 – 0.20	MS, MSb, LH, HS, or HSb

The chemical sulphate content analyses for selected soil samples tested indicate a sulphate concentration of maximum of a 480 ug/g (less than 0.10 %) in soil, as shown in Table 6-1. The results were compared with Canadian Standards Association (CSA) Standards A23.1 for sulphate attack potential on concrete structures and possesses a “negligible” risk for sulphate attack on concrete material. Therefore, sulphate resistance concrete is not required for concrete substructures.

6.8 Floor Slabs

DST was not provided with any design criteria for floor slab loadings and therefore we have assumed that floor slabs are lightly loaded with no heavy racking or process machinery that require specific support. A typical floor slab loading for a lightly loaded slab on grade would involve a maximum pressure of 24 kPa. If this is not the case, then DST should be retained to perform additional consulting in regard to design of the floor slab. For design purposes and based upon a properly prepared native subgrade surface covered with 200 mm of Ontario Provincial Standard Specification (OPSS) 1010 ‘Granular A’, a typical preliminary modulus of subgrade reaction appropriate for the slab design would be approximately 25,000 kN/m^3 on Engineered Fill compacted to 100%of its Standard Proctor Maximum Dry Density (SPMDD). Alternative values would require additional analysis and testing.

A capillary moisture barrier consisting of a layer of either 19 mm clear stone or an OPSS 1010 ‘Granular A’ at least 200 mm thick should underlie the slab. This layer should be compacted to 100%of its SPMDD and placed on approved subgrade surfaces.

If floor coverings are to be used, vapour barriers are also recommended to be incorporated beneath the slab. Floor toppings may be impacted by curing and moisture conditions of the concrete. Floor finish manufacturer’s specifications and requirements should be consulted, and procedures outlined in the specifications should be followed. The slabs should be free floating and should not be tied into the foundation walls. The placement of construction and control joints in the concrete should be in accordance with generally accepted practice.

6.9 Permanent Drainage

Under floor drainage is typically not necessary for structures with no basement level and which have floor slabs at a minimum of 0.3 m above the exterior grade. Perimeter drainage around the proposed building is recommended. Perimeter drainage may consist of either a conventional weeping tile with clear stone and geotextile, or alternatively a composite drainage blanket may be used. Regardless of the drainage system used, it is recommended that the foundation walls be backfilled with a free-draining non-frost-susceptible soil such as an OPSS 1010 “Granular B, Type I”. The perimeter drain should be connected to a frost-free outlet for year-round drainage.

If the floor slab is set at or below the exterior grade, then both perimeter and under-floor drainage is recommended. Perimeter and underfloor drainage systems should not be connected.

6.10 Engineered Fill

All new fill soils that underlie floor slabs, footings, are in building interiors, or other structural applications is considered as Engineered Fill. For this project, Engineered Fill may be required below footings to raise the grade between approved subgrade and the design footing level, for interior foundation wall backfill, to correct deficiencies in the footing subgrade, and below floor slabs. In order to be considered Engineered Fill. The placement must meet the strict requirements as shown below:

- The proposed material must be tested for grain size and Proctor and reviewed and approved by the Geotechnical Engineer before being considered as Engineered Fill. Typically, a crushed well-graded material such as an OPSS 1010 “Granular A”-type material is suitable. However, other suitable granular materials may be proposed and considered depending on the Site-specific conditions;
- Prior to placing any Engineered Fill, all unsuitable fill materials must be removed, and the subgrade approved by the Geotechnical Engineer. Any deficient areas should be repaired prior to placement;
- Engineered Fill should be placed in maximum loose lifts of 200 mm and adequately compacted to achieve 100% of its SPMDD. Engineered fill must have full-time compaction testing of every lift performed by geotechnical personnel; and
- At a minimum, the Engineered Fill beneath foundations should extend laterally a distance of 0.3 m beyond the edge of the footings and then be sloped downward and outward at 1H:1V slope. Designers and contractors are cautioned that the resultant excavation can be quite large if a significant thickness of Engineered Fill is required.

6.11 Exterior Foundation Backfill

The backfill placed against exterior foundations in landscaped or non-structural areas should be a free draining granular material meeting the grading requirements of an OPSS 1010 “Granular B, Type I” or “Granular B, Type II”. Exterior foundation backfill should be placed and compacted as outlined below:

- Backfill should not be placed in a frozen condition, or place on a frozen subgrade;
- Backfill should be placed and compacted in maximum loose lift thickness compatible with the selected construction equipment, but not thicker than 0.3 m;
- In landscaped areas the upper 0.3 m of backfill below landscape details should be a low permeable soil to reduce surface water infiltration;
- Backfill should be placed uniformly on both sides of the foundation walls to avoid build-up of unbalanced lateral pressures;
- For backfill that would underlie paved areas, sidewalks or exterior slabs-on-grade, each lift should be uniformly compacted to achieve 98 % percent of its SPMDD.
- For backfill on the building exterior that would underlie landscaped areas, each lift should be uniformly compacted to at least 95 % of its SPMDD;
- Exterior grades should be sloped away from the foundation wall, and roof drainage downspouts should be placed so that water flows away from the foundation wall;
- Entrance slabs should be placed founded on frost walls or alternatively have insulation details developed to prevent frost heaving at the building entrances;
- In areas where the building backfill underlies a pavement, sidewalk, or other hard landscaping, the excavation should have a frost taper incorporated to prevent differential heaving around the building.

6.12 Underground Utilities

The recommendations within this section are intended to be a supplement to, and not a replacement of the most recent local municipal requirements.

6.12.1 Bedding and Cover

The following are recommendations for service trench bedding and cover materials:

- Bedding for buried utilities should consist of an OPSS 1010 "Granular A" material and placed in accordance with municipal requirements, assuming the subgrade soils are not allowed to become disturbed;
- The use of clear stone is not recommended for use as pipe bedding. The voids in the stone may result in a low gradient water flow and infiltration of fines from the surrounding soils and cover materials, causing settlement and loss of support to pipes and structures;
- The cover material should be a service sand material or an OPSS 1010 "Granular A". The dimensions should comply with pertinent specification section;
- The bedding, springline, and cover should be compacted to at least 95% of its SPMDD;
- Compaction equipment should be used in such a way that the utility pipes are not damaged during construction.

6.12.2 Trench Backfill

Backfill above the cover for buried utilities should be in accordance with the following recommendations:

- For service trenches underlying pavement areas, the backfill should be placed and compacted in uniform lift thickness compatible with the selected compaction equipment and not thicker than 300 mm. Each lift should be compacted to a minimum of 98% of its SPMDD.
- The backfill placed in the upper 0.3 m below the pavement subgrade elevation should be compacted to a minimum of 100% of its SPMDD.
- Excavation backfill should attempt to match texture of the existing adjacent soils. If imported materials are used, side slopes with frost tapers are recommended. Frost tapers should be a back-slope of 10H:1V through the frost zone, (i.e.,1.8 m from finished grade)
- During backfilling, care should be taken to ensure the backfill proceeds in equal stages simultaneously on both sides of the pipe.
- No frozen material should be used as backfill; neither should the trench base be allowed to freeze.

The quality and workmanship in the construction is as important as the compaction standards themselves. It is imperative that the guidelines for the compaction be followed for the full depth of the trench to achieve satisfactory performance.

6.12.3 Clay Seals

Clay seals should be incorporated into the design of the any utility trenches. If clay seals are not used, then there is the potential for the trench to act as a drain and dewater the local clays causing settlement. The location of the clay seals should be at a frequency prescribed by the Civil Engineer.

Ontario Provincial Standard Drawing (OPSD) 1205 and OPSD 802.095 are referred to both the Designers and Contractor for guidance on clay seals. Acceptable imported clay material may be used for the construction of the clay seals.

6.13 Recommended Asphalt Pavement

All existing asphalt pavement and granular courses should be excavated down to the proposed new subgrade level. The final subgrade should be proof-rolled to look for deflection, soft spots, or local anomalies. Typically, a heavy-duty steel drum roller or a loaded dump truck is sufficient for proof rolling. Proof rolling of proposed subgrades should be witnessed by geotechnical staff. Any non-performing areas should be sub-excavated and replaced with an appropriate new fill soil. An appropriate fill soil would be a free-draining non-frost susceptible soil similar to a Granular ‘B’ Type I or Granular ‘B’ Type II material.

Newly backfilled soils should attempt to match the texture of the existing adjacent soils. Localized sub-excavations should have frost tapers to avoid concentrated frost heaves across the roadway at the transition zones between sub-excavated and un-excavated subgrades.

In order to accommodate the recommended thicknesses, designers will need to review existing and proposed grades and determine where stripping or filling is necessary. Drainage of the pavement layers is important. Surface runoff should be directed to storm sewers or surface ditches where possible. The subgrade surface and each layer of the pavement section should also be provided with a suitable cross fall (approximately 3%) to prevent water from ponding on each layer. The installation of subdrains may be recommended as designs progress based on the surrounding topography and drainage conditions to assist in the long-term performance of the pavement structures. Non-woven geotextile as a separation medium may be prudent based on the observations during proof rolling.

For the proposed pavement base and subbase courses the material should consist of a Granular ‘A’ and Granular ‘B’ Type II material, respectively. The material should be placed in maximum loose lifts of 300 mm and compacted to 100 % of its SPMDD.

Sufficient field-testing should be carried out during construction to assess compaction of each lift of the pavement structure layers. This should be accompanied by laboratory testing of the proposed granular materials and asphalt materials.

In the case of winter work, which is not recommended, no frozen material should be used as backfill, and backfill should not be placed on frozen subgrades.

Based on the results of the field and laboratory testing, DST is recommending the following preliminary minimum pavement sections. It is important to note that at the time of this investigation, DST has not been provided with any traffic counts, or level of service requirements or equipment loadings for pavement structures. The pavement sections being provided are what we would consider to be suitable for a private development within this part of Ottawa. Table 6-3 below summarizes proposed asphalt designs for the parking lot and fire route respectively.

Table 6-3: Recommended Minimum Pavement Sections

Material	Layer Thickness
Parking Lots – Light Duty (Parking Stalls)	
Asphalt Wearing Course	50 mm
Well Graded Granular Base Course (Granular ‘A’)	150 mm
Well Graded Granular Sub-Base Course (Granular ‘B’ Type II)	300 mm
Parking Lots – Heavy Duty (Aisles and Fire Routes)	
Asphalt Wearing Course	40 mm
Asphalt Binder Course	50 mm
Well Graded Granular Base Course (Granular ‘A’)	150 mm
Well Graded Granular Sub-Base Course (Granular ‘B’ Type II)	450 mm

Annual or regular maintenance will be required to achieve maximum life expectancy. Generally, the asphalt pavement maintenance will involve periodic crack sealing and repair of local distress.

It is important to emphasize that the pavement sections described above are for the proposed end use condition, including light vehicular traffic and occasional service trucks. It may be necessary to over-design these sections if they are intended to support heavy construction equipment throughout construction.

6.14 Recommendations for RCC Pavement

DST was not provided with any design criteria for pavement loadings. We have assumed that the concrete pavement is loaded with heavy machineries and traffic circulation. Roller-Compacted Concrete (RCC) pavement can be considered for heavy duty traffic. The RCC pavement has the same basic ingredients as conventional concrete and it is cost-effective and easy to construct with no reinforcement and joints requirement. According to “Guide for Roller-Compacted Concrete Pavement” dated August 2010 prepared for National Concrete Pavement Technology Centre, typically the compressive strength of RCC is comparable to that of conventional concrete, ranging from 28 to 41 MPa. RCC mixtures have a low water content and require compaction using a vibratory and/or static roller to achieve a target density. The subgrade for RCC need to be inspected and approved by a qualified Geotechnical Engineer. The base of the RCC pavement is recommended to consisted of at least 300 mm granular consisting of an OPSS1010 “Granular A” and should be compacted to at least 98% of its SPMDD.

Due to moisture sensitivity and other factors, choosing the right aggregate gradation is critical to a achieve successful design mix. These mixtures have well-distributed aggregate size throughout which helps in the process of placement and compaction. Mixture design procedures involve the following steps:

- Selection of the well-graded aggregates;
- Selection of the cement content as a percentage of total weight;
- Development of maximum dry density and optimum moisture content; and
- Strength specimens at maximum density and optimum moisture content

Immediately after placement of RCC, it needs to be compacted to at least 100% of its SPMDD. During the curing process of the RCC pavement, only occasional light traffic should be allowed. The pavement should be saw cut during early stages of curing if any pre-determined expansion joints are required, or alternately it can be allowed to crack naturally.

7. MONITORING DURING CONSTRUCTION

DST requests to be retained once the plans and specifications are finalized to review the documents and ensure the recommendations in this report are adequately addressed.

The recommendations presented in this report are based on the assumption that an adequate level of construction monitoring by qualified geotechnical personnel during construction will be provided. Based on our understanding of the scope of the Project, an adequate level of construction monitoring is considered to be as follows:

- Review and approval of all footing subgrades by the Geotechnical Engineer;
- Proof rolling, review, and approval of subgrades below the floor slab;
- Laboratory testing and pre-approval of fill soils that are proposed to be used;
- Full time compaction testing of Engineered Fill and part time compaction testing of exterior foundation wall backfill; and
- Periodic testing of concrete.

An important purpose of providing an adequate level of monitoring is to check that recommendations, based on data obtained at the discrete borehole locations, are relevant to other areas of the site.

8. CLOSURE

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix A and forms an integral part of this report.

We trust this report meets your present requirements. Should you have any questions, please do not hesitate to contact our office.

Sincerely,

DST CONSULTING ENGINEERS INC.

LICENSED PROFESSIONAL ENGINEER
May 21, 2019
S. W. DUNSTAN
100152399
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
F. SAADAT
May 23, 2019
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Shane Dunstan, P.Eng.
Geotechnical Project Manager

Farbod Saadat, Ph.D., P.Eng.
Chief Geotechnical Engineer

APPENDIX A

LIMITATIONS OF REPORT

LIMITATIONS OF REPORT

GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that DST Consulting Engineers Inc. be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the boreholes.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid. Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and DST Consulting Engineers Inc. cannot warranty their accuracy. Similarly, DST cannot warranty the accuracy of information supplied by the Client.

APPENDIX B
SITE LOCATION MAP
BOREHOLE LOCATION PLAN

Source:
Google Earth 2019

Revision	Date	Issue	Approval
0	04/16/19	Original	

Tuesday, April 16, 2019 @ 10:24 by Joven Mendoza

Folder: L:\TS\CAD\Projects\TS\TS-SO-037029 2555 Sheffield\2019 Geotech Investigation\DWGs

Drawing: 1 Site Location Plan.dwg

Client
AIM Recycling Ottawa East

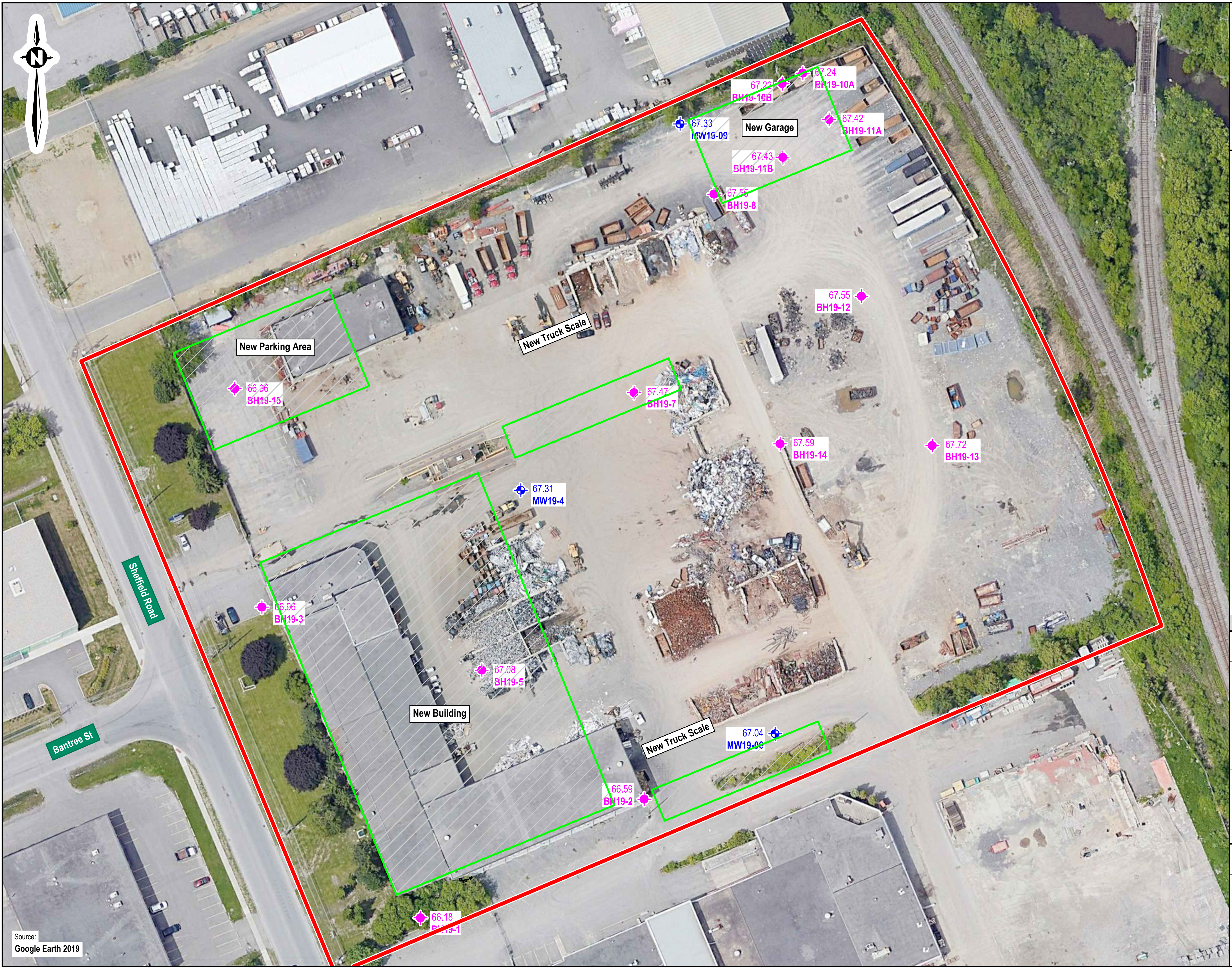
Site
2555 Sheffield Road, Ottawa, ON

DST consulting engineers
a division of Englobe

Report Title
Geotechnical Investigation for New Buildings,
Truck Scales and Pavement

Drawing Title
Site Location Plan

Designed By S.N.	Date April 2019
Drawn By J.M.	Project No. TS-SO-37029
Approved By S.D.	Figure No. 1
Scale As shown	



Source:
Google Earth 2019

DST
consulting engineers
a division of Englobe

Note

1. This drawing shall be read in conjunction with the associated technical report.

Legend

🟥	Site Boundary
⌖	Monitoring Well Location
●	Borehole Location
▨	Approximate Building Locations

1:1 000
0 10 20 30 40 50 m
0 *1:1* 5 cm
Scale accurate when printed at 100% using paper size ANSI full bleed B (11.0 x 17.0 Inches)

Revision	Date	Issue	Approval
0	04/16/19	Original	

Client
AIM Recycling Ottawa East

Site
2555 Sheffield Road, Ottawa, ON

Report Title
Geotechnical Investigation for New Buildings, Truck Scales and Pavement

Drawing Title
Borehole Location Plan

Designed By S.N.	Scale As shown
Drawn By J.M.	Date April 2019
Approved By S.D.	Project No. TS-SO-37029
Figure No.	2

Drawing: **2 Borehole Location Plan**.dwg *Folder:* **L:\TS\CAD\Projects\TS\TS-SO-037029 2555 Sheffield\2019 Geotech Investigation\DWGs** Tuesday, April 16, 2019 @ 10:23 by Joven Mendoza

APPENDIX C
LIST OF SYMBOLS AND DEFINITIONS FOR GEOTECHNICAL SAMPLING
BOREHOLE LOGS

LIST OF SYMBOLS AND DEFINITIONS FOR GEOTECHNICAL SAMPLING AND COMMON LITHOLOGIES

The following is a reference sheet for commonly used symbols and definitions within this report and in any figures or appendices, including borehole logs and test results. Symbols and definitions conform to the standard proposed by the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) wherever possible. Discrepancies may exist when comparing to third-party results using the Unified Soil Classification System (USCS).

PART A – SOILS

Standard Penetration Test (SPT) 'N'

The number of blows required to drive a 50-mm (2 in) split barrel sampler 300 mm (12 in). The standard hammer has a mass of 63.5 kg (140 lbs) and is dropped vertically from a height of 760 mm (30 in). Additional information can be found in ASTM D1586-11 and in §4.5.2 of the CFEM 4th Ed.

For penetration less than 300 mm, 'N' is recorded with the penetration that was achieved.

Non-Cohesive Soils

The relative density of non-cohesive soils relates empirically to SPT 'N' as follows:

Relative Density	'N'
Very Loose	0 – 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	> 50

Cohesive Soils

The consistency and undrained shear strength of cohesive soils relates empirically to SPT 'N' as follows:

Consistency	Undrained Shear Strength (kPa)	'N'
Very Soft	< 12	0 – 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

PART B – ROCK

The following parameters are used to describe core recovery and to infer the quality of a rockmass.

Total Core Recovery, TCR (%)

The total length of solid drill core recovered, regardless of the quality or length of the pieces, taken as a percentage of the length of the core run.

Solid Core Recovery, SCR (%)

The total length of solid, full-diameter drill core recovered, taken as a percentage of the length of the core run.

Rock Quality Designation, RQD (%)

The sum of the lengths of solid drill core greater than 100 mm long, taken as a percentage of the length of the core run. RQD is commonly used to infer the quality of the rockmass, as follows:

Rockmass Quality	RQD (%)
Very Poor	< 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	> 90

Weathering

The terminology used to describe the degree of weathering for recovered rock core is defined as follows, as suggested by the *Geological Society of London*:

Completely weathered: All rock material is decomposed and/or disintegrated to soil. The original mass structure is largely intact.

Highly weathered: More than half the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as a discontinuous framework or as core stone.

Moderately weathered: Less than half the rock material is decomposed and/or disintegrates to soil. Fresh or discolored rock is present ether as a continuous framework or as core stone.

Slightly weathered: Discoloration indicates weathering of rock material and discontinuity of surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than its fresh condition.

Fresh: No visible signs of weathering.

PART C – SAMPLING SYMBOLS

Symbol	Description
SS	Split spoon sample
TW	Thin-walled (Shelby Tube) sample
PH	Sampler advanced by hydraulic pressure
WH	Sampler advanced by static weight
SC	Soil core

PART D – IN-SITU AND LAB TESTING

SOIL NAMING CONVENTIONS

Particle sizes are described as follows:

Particle Size Descriptor		Size (mm)
Boulder		> 300
Cobble		75 – 300
Gravel	Coarse	19 – 75
	Fine	4.75 – 19
Sand	Coarse	2.0 – 4.75
	Medium	0.425 – 2.0
	Fine	0.075 – 0425
Silt		0.002 – 0.075
Clay		< 0.002

The principle constituent of a soil is written in uppercase. The minor constituents of a soil are written according to the following convention:

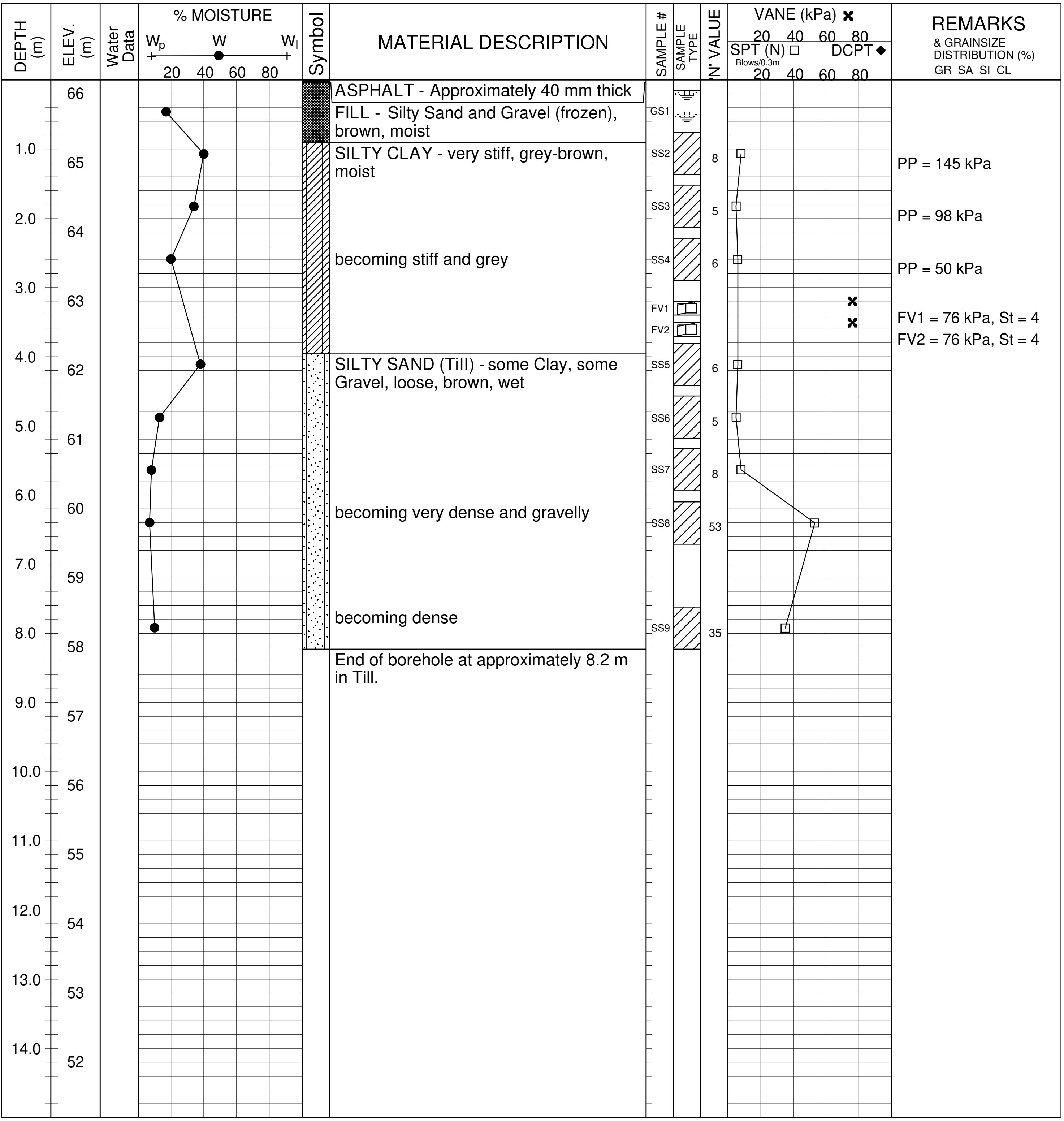
Descriptive Term	Proportion of Soil (%)
Trace	1 – 10
Some	10 – 20
(ey) or (y)	20 – 35
And	35 – 50

Eg.: A soil comprising 65% Silt, 21% Sand and 14% Clay would be described as a: Sandy SILT, Some Clay

LOG OF BOREHOLE BH19-01

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 66.20 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 19, 2019
COORDINATES: 5029984m N, 374710m E



MATERIAL DESCRIPTION	SAMPLE #	'N' VALUE	REMARKS
ASPHALT - Approximately 40 mm thick			
FILL - Silty Sand and Gravel (frozen), brown, moist	GS1		
SILTY CLAY - very stiff, grey-brown, moist	SS2	8	PP = 145 kPa
	SS3	5	PP = 98 kPa
becoming stiff and grey	SS4	6	PP = 50 kPa
	FV1		FV1 = 76 kPa, St = 4
	FV2		FV2 = 76 kPa, St = 4
SILTY SAND (Till) - some Clay, some Gravel, loose, brown, wet	SS5	6	
	SS6	5	
	SS7	8	
becoming very dense and gravelly	SS8	53	
becoming dense	SS9	35	
End of borehole at approximately 8.2 m in Till.			

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SAMPLE TYPE LEGEND

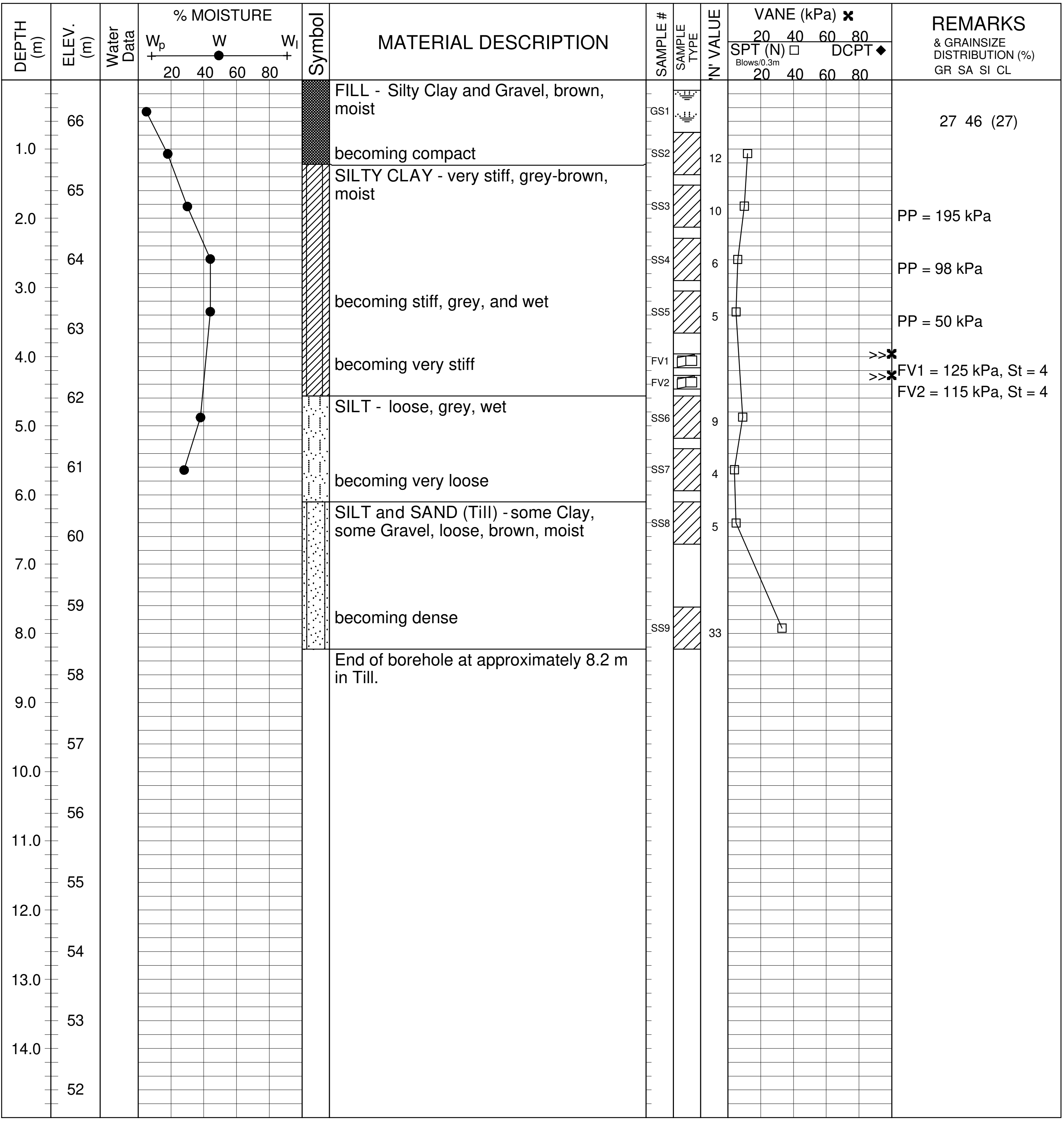
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	◫ Field Vane	▒ Sand
▥ Bulk Sample	▯ 70mm Thin Wall Tube	

ENCLOSURE 1

LOG OF BOREHOLE BH19-02

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 66.60 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 19, 2019
COORDINATES: 5030015m N, 374771m E



MATERIAL DESCRIPTION

FILL - Silty Clay and Gravel, brown, moist
becoming compact

SILTY CLAY - very stiff, grey-brown, moist
becoming stiff, grey, and wet
becoming very stiff

SILT - loose, grey, wet
becoming very loose

SILT and SAND (Till) - some Clay, some Gravel, loose, brown, moist
becoming dense

End of borehole at approximately 8.2 m in Till.

SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
GS1		27 46 (27)
SS2	12	
SS3	10	PP = 195 kPa
SS4	6	PP = 98 kPa
SS5	5	PP = 50 kPa
FV1		FV1 = 125 kPa, St = 4
FV2		FV2 = 115 kPa, St = 4
SS6	9	
SS7	4	
SS8	5	
SS9	33	

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SAMPLE TYPE LEGEND

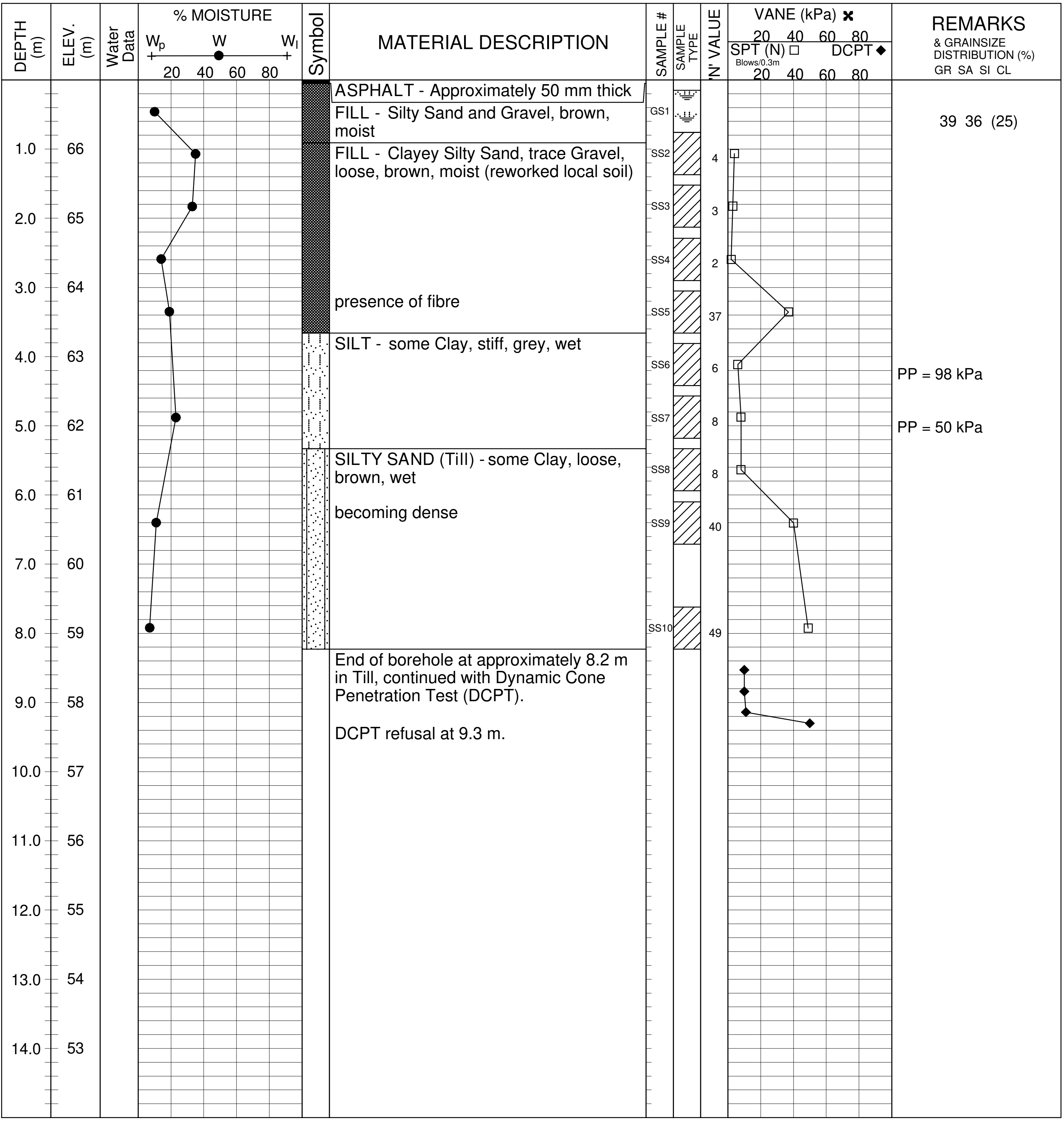
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	Field Vane	░ Sand
Bulk Sample	70mm Thin Wall Tube	

ENCLOSURE 2

LOG OF BOREHOLE BH19-03

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.00 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 19, 2019
COORDINATES: 5030070m N, 374669m E



ASPHALT - Approximately 50 mm thick

FILL - Silty Sand and Gravel, brown, moist

FILL - Clayey Silty Sand, trace Gravel, loose, brown, moist (reworked local soil)

presence of fibre

SILT - some Clay, stiff, grey, wet

SILTY SAND (Till) - some Clay, loose, brown, wet

becoming dense

End of borehole at approximately 8.2 m in Till, continued with Dynamic Cone Penetration Test (DCPT).

DCPT refusal at 9.3 m.

SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
GS1		39 36 (25)
SS2	4	
SS3	3	
SS4	2	
SS5	37	
SS6	6	PP = 98 kPa
SS7	8	PP = 50 kPa
SS8	8	
SS9	40	
SS10	49	

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SAMPLE TYPE LEGEND

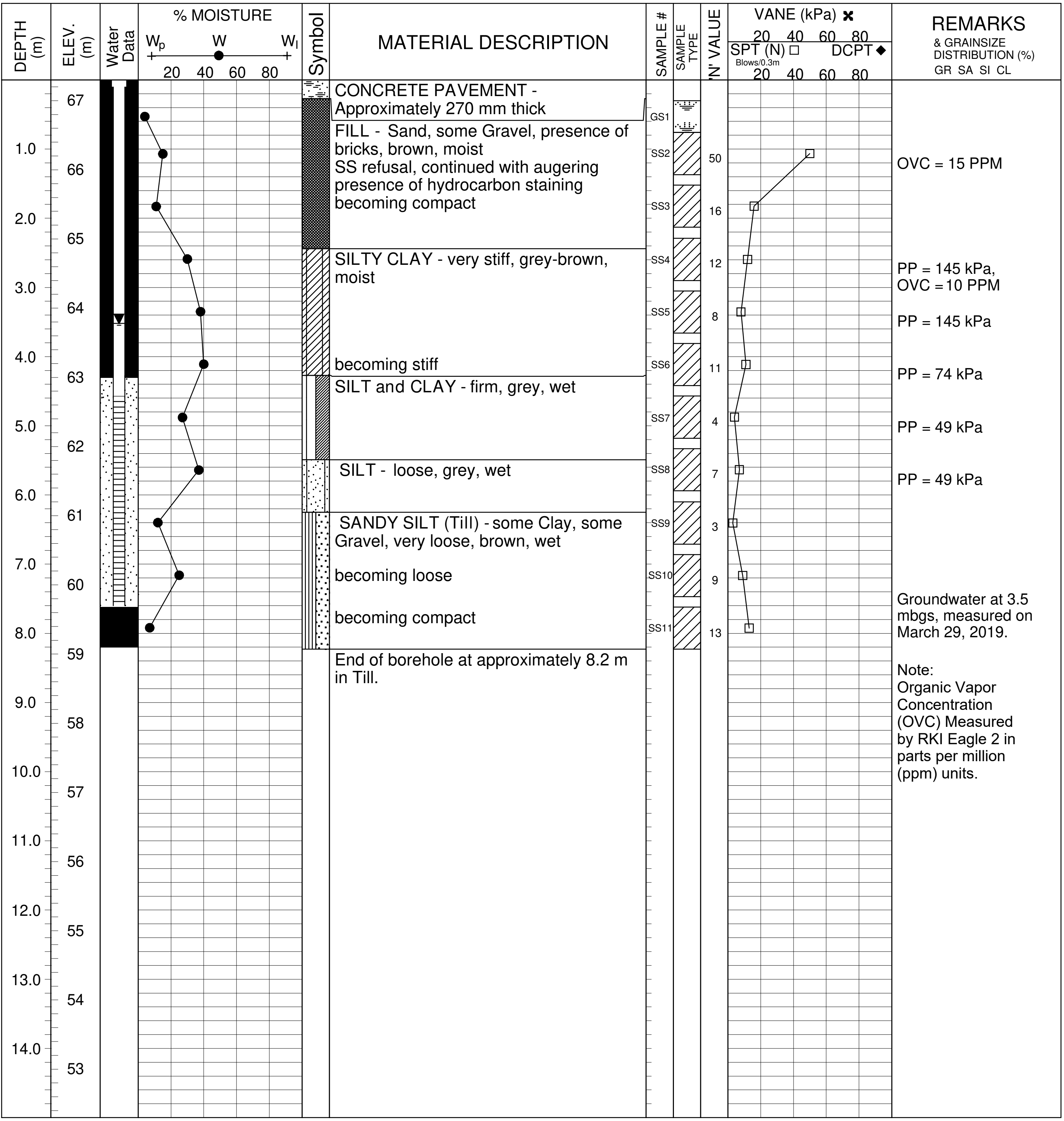
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	◫ Field Vane	⁘ Sand
▥ Bulk Sample	◫ 70mm Thin Wall Tube	

ENCLOSURE 3

LOG OF BOREHOLE MW19-04

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.30 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 21, 2019
COORDINATES: 5030100m N, 374737m E



MATERIAL DESCRIPTION
CONCRETE PAVEMENT - Approximately 270 mm thick
FILL - Sand, some Gravel, presence of bricks, brown, moist SS refusal, continued with augering presence of hydrocarbon staining becoming compact
SILTY CLAY - very stiff, grey-brown, moist becoming stiff
SILT and CLAY - firm, grey, wet
SILT - loose, grey, wet
SANDY SILT (Till) - some Clay, some Gravel, very loose, brown, wet becoming loose becoming compact
End of borehole at approximately 8.2 m in Till.

SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
GS1		
SS2	50	OVC = 15 PPM
SS3	16	
SS4	12	PP = 145 kPa, OVC = 10 PPM
SS5	8	PP = 145 kPa
SS6	11	PP = 74 kPa
SS7	4	PP = 49 kPa
SS8	7	PP = 49 kPa
SS9	3	
SS10	9	
SS11	13	

Groundwater at 3.5 mbgs, measured on March 29, 2019.

Note:
Organic Vapor Concentration (OVC) Measured by RKI Eagle 2 in parts per million (ppm) units.

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SAMPLE TYPE LEGEND

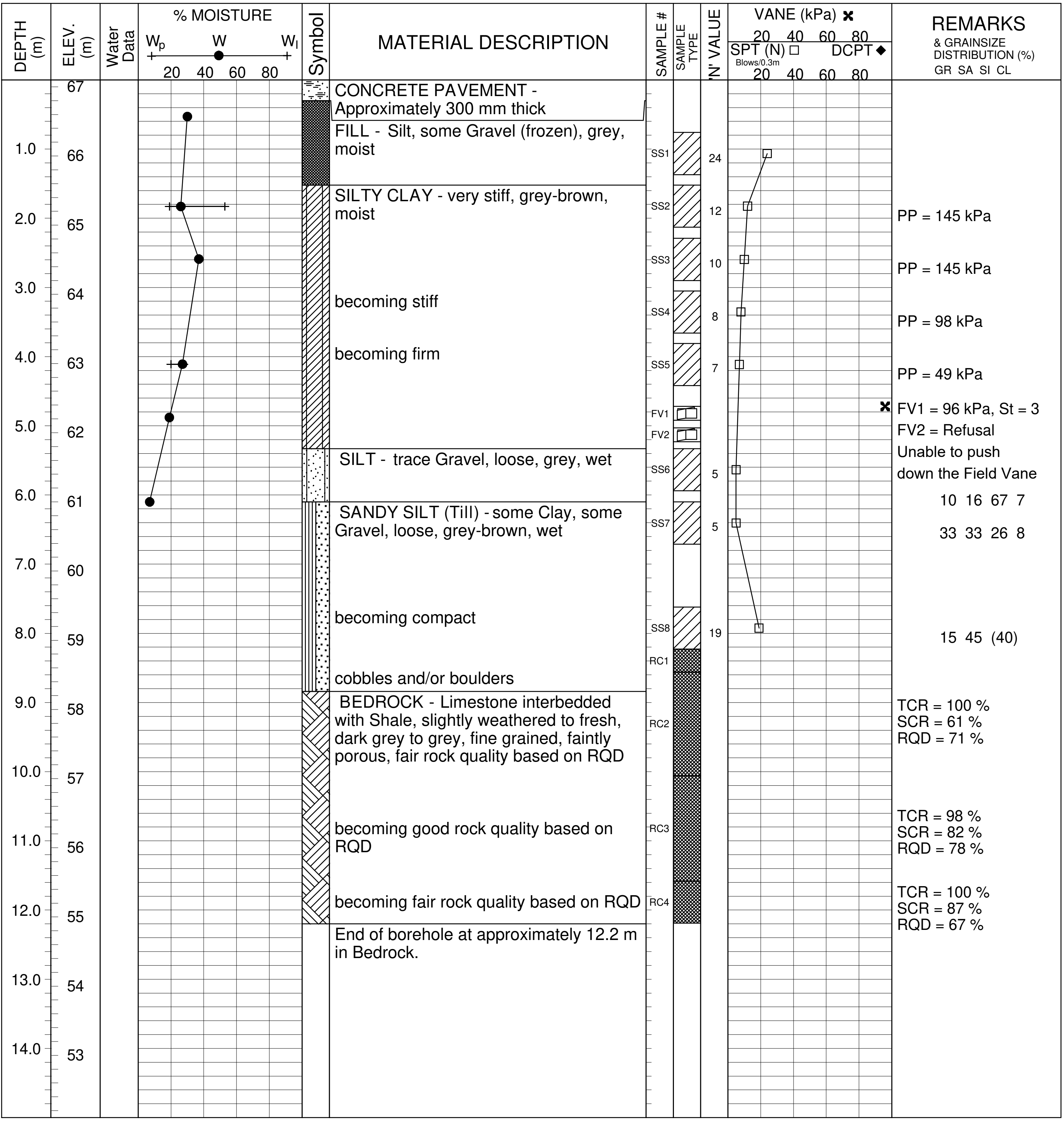
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	◫	Field Vane	░	Sand
▥	Bulk Sample	◫	70mm Thin Wall Tube		

ENCLOSURE 13

LOG OF BOREHOLE BH19-05

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.10 metres

Drilling Data
METHOD: Hollow Stem Auger & Rock Coring
DIAMETER: 200 mm
DATE: March 20, 2019
COORDINATES: 5030051m N, 374727m E



MATERIAL DESCRIPTION
CONCRETE PAVEMENT - Approximately 300 mm thick
FILL - Silt, some Gravel (frozen), grey, moist
SILTY CLAY - very stiff, grey-brown, moist becoming stiff becoming firm
SILT - trace Gravel, loose, grey, wet
SANDY SILT (Till) - some Clay, some Gravel, loose, grey-brown, wet becoming compact cobbles and/or boulders
BEDROCK - Limestone interbedded with Shale, slightly weathered to fresh, dark grey to grey, fine grained, faintly porous, fair rock quality based on RQD becoming good rock quality based on RQD becoming fair rock quality based on RQD
End of borehole at approximately 12.2 m in Bedrock.

SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
SS1	24	
SS2	12	PP = 145 kPa
SS3	10	PP = 145 kPa
SS4	8	PP = 98 kPa
SS5	7	PP = 49 kPa
FV1		FV1 = 96 kPa, St = 3
FV2		FV2 = Refusal Unable to push
SS6	5	down the Field Vane 10 16 67 7
SS7	5	33 33 26 8
SS8	19	15 45 (40)
RC1		
RC2		TCR = 100 % SCR = 61 % RQD = 71 %
RC3		TCR = 98 % SCR = 82 % RQD = 78 %
RC4		TCR = 100 % SCR = 87 % RQD = 67 %

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SAMPLE TYPE LEGEND

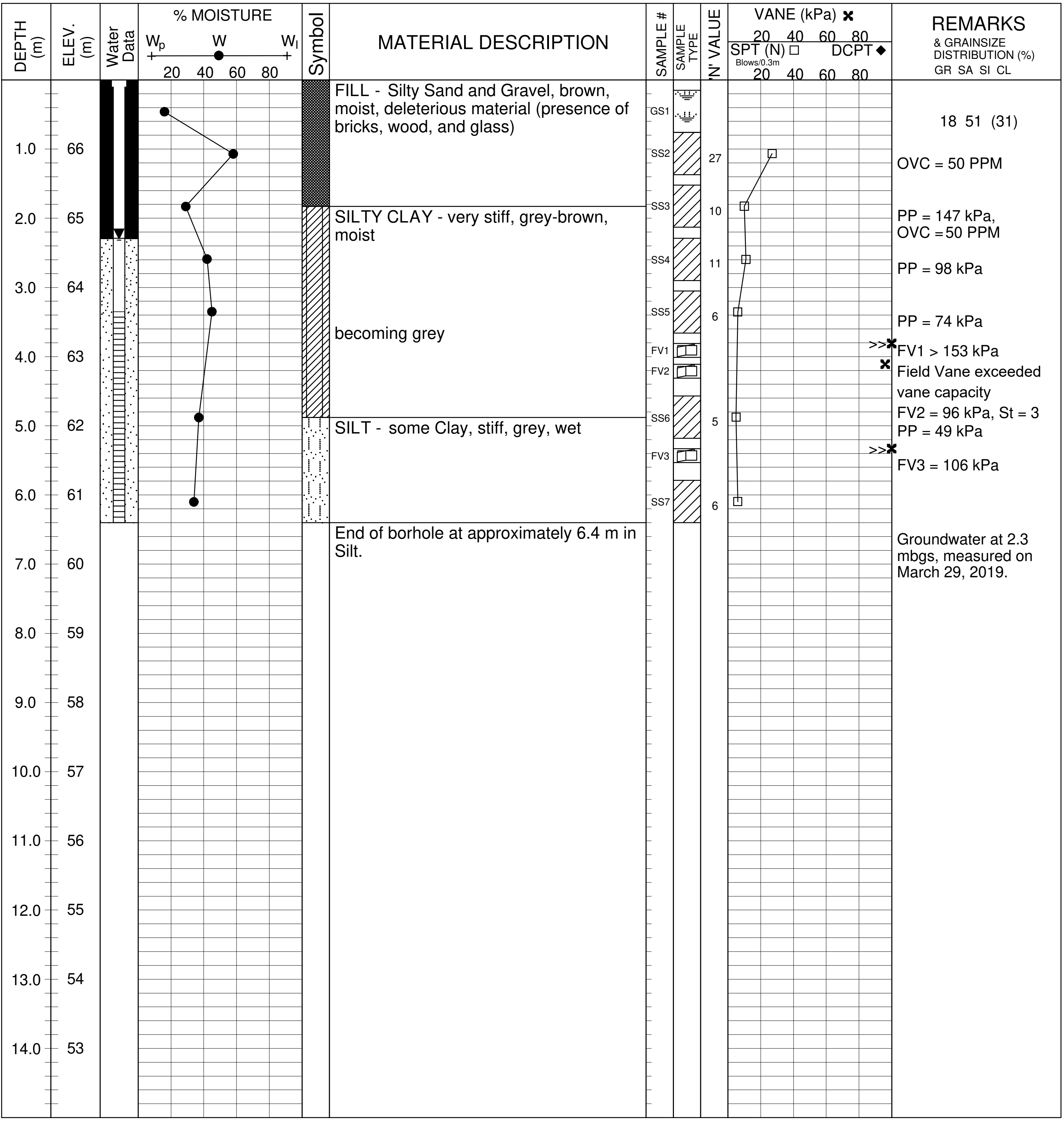
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	◫ Field Vane	░ Sand
▥ Bulk Sample	▯ 70mm Thin Wall Tube	

ENCLOSURE 4

LOG OF BOREHOLE MW19-06

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.00 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 19, 2019
COORDINATES: 5030034m N, 374806m E



MATERIAL DESCRIPTION	SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
FILL - Silty Sand and Gravel, brown, moist, deleterious material (presence of bricks, wood, and glass)	GS1		18 51 (31)
	SS2	27	OVC = 50 PPM
SILTY CLAY - very stiff, grey-brown, moist	SS3	10	PP = 147 kPa, OVC =50 PPM
	SS4	11	PP = 98 kPa
	SS5	6	PP = 74 kPa
becoming grey	FV1		FV1 > 153 kPa Field Vane exceeded vane capacity
	FV2		FV2 = 96 kPa, St = 3
SILT - some Clay, stiff, grey, wet	SS6	5	PP = 49 kPa
	FV3		FV3 = 106 kPa
	SS7	6	
End of borhole at approximately 6.4 m in Silt.			Groundwater at 2.3 mbgs, measured on March 29, 2019.

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SAMPLE TYPE LEGEND

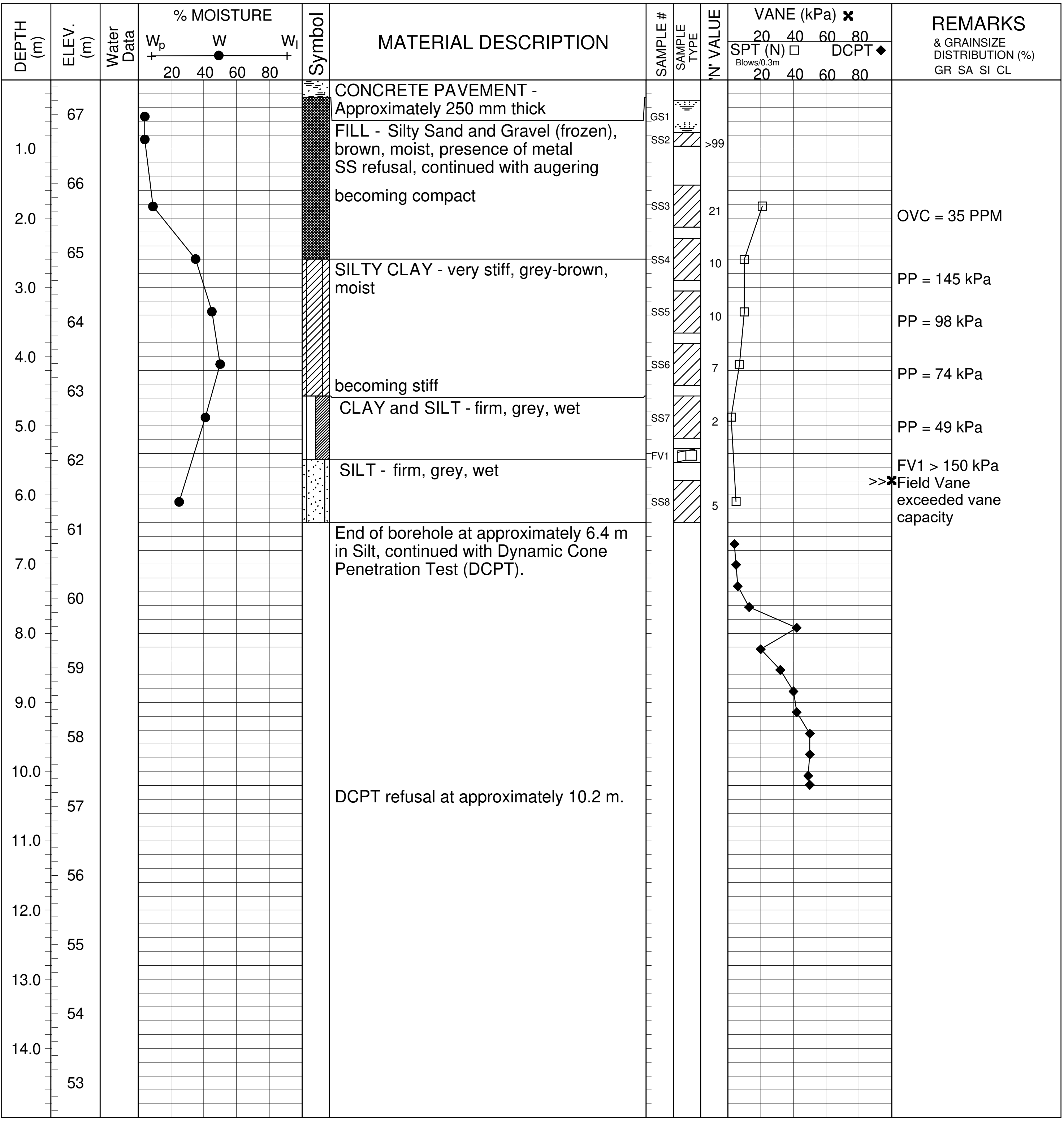
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	◫ Field Vane	░ Sand
▥ Bulk Sample	▯ 70mm Thin Wall Tube	

ENCLOSURE 14

LOG OF BOREHOLE BH19-07

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.50 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 21, 2019
COORDINATES: 5030127m N, 374768m E



CONCRETE PAVEMENT - Approximately 250 mm thick

FILL - Silty Sand and Gravel (frozen), brown, moist, presence of metal
SS refusal, continued with augering

becoming compact

SILTY CLAY - very stiff, grey-brown, moist

becoming stiff

CLAY and SILT - firm, grey, wet

SILT - firm, grey, wet

End of borehole at approximately 6.4 m in Silt, continued with Dynamic Cone Penetration Test (DCPT).

DCPT refusal at approximately 10.2 m.

Sample #	'N' Value	Remarks
GS1		
SS2	>99	
SS3	21	OVC = 35 PPM
SS4	10	PP = 145 kPa
SS5	10	PP = 98 kPa
SS6	7	PP = 74 kPa
SS7	2	PP = 49 kPa
FV1		FV1 > 150 kPa Field Vane exceeded vane capacity
SS8	5	

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SAMPLE TYPE LEGEND

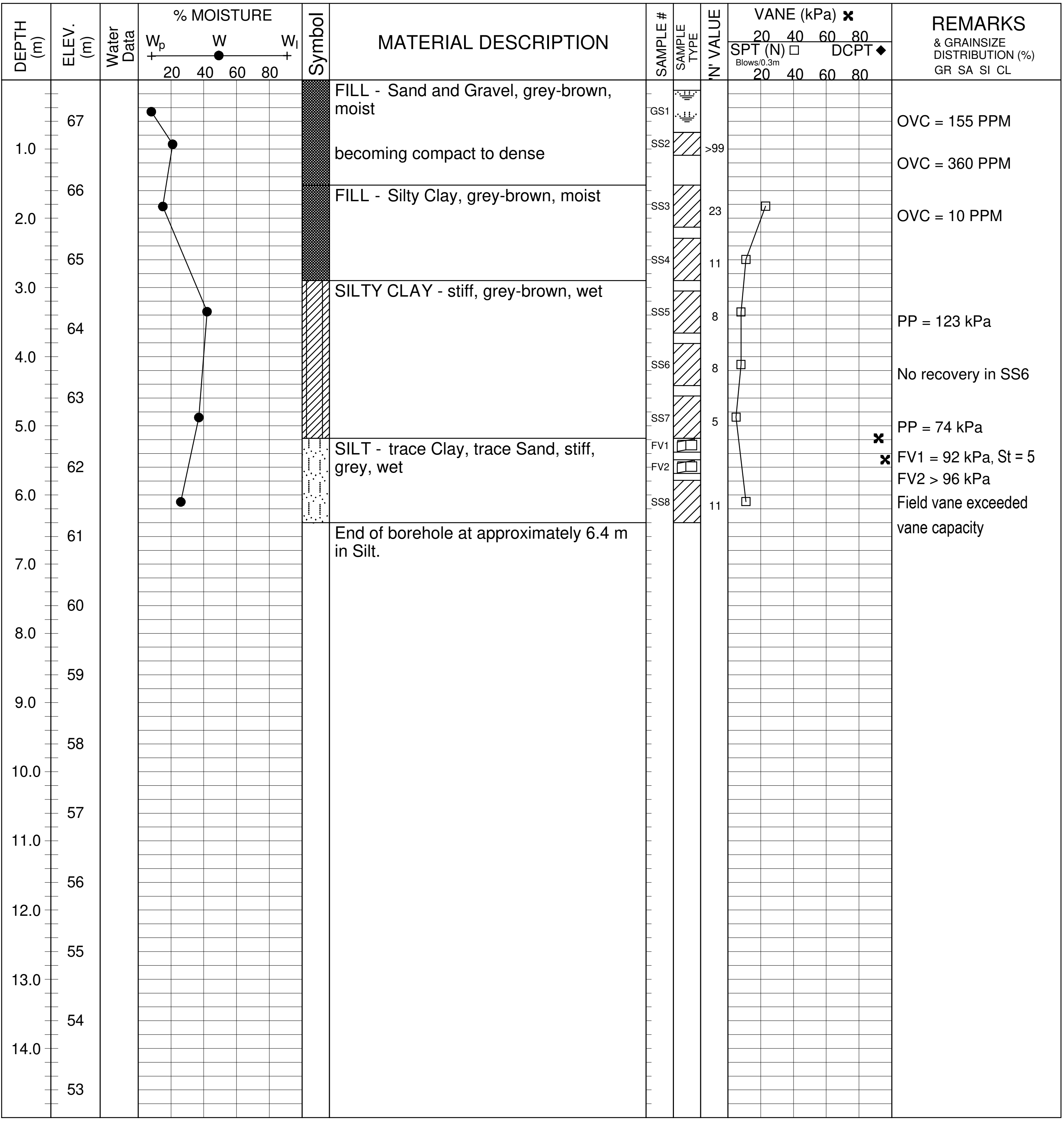
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	◫ Field Vane	▦ Sand
▥ Bulk Sample	▯ 70mm Thin Wall Tube	

ENCLOSURE 5

LOG OF BOREHOLE BH19-08

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.60 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 18, 2019
COORDINATES: 5030181m N, 374789m E



MATERIAL DESCRIPTION
FILL - Sand and Gravel, grey-brown, moist becoming compact to dense
FILL - Silty Clay, grey-brown, moist
SILTY CLAY - stiff, grey-brown, wet
SILT - trace Clay, trace Sand, stiff, grey, wet
End of borehole at approximately 6.4 m in Silt.

SAMPLE #	'N' VALUE	REMARKS
GS1		OVC = 155 PPM
SS2	>99	OVC = 360 PPM
SS3	23	OVC = 10 PPM
SS4	11	
SS5	8	PP = 123 kPa
SS6	8	No recovery in SS6
SS7	5	PP = 74 kPa
FV1		FV1 = 92 kPa, St = 5
FV2		FV2 > 96 kPa
SS8	11	Field vane exceeded vane capacity

BOREHOLE (STANDARD) - OTTAWA TS-SO-37029 - BOREHOLE LOGS - APR. 10, 2019.GPJ DATA TEMPLATE.GDT 4/12/19

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SAMPLE TYPE LEGEND

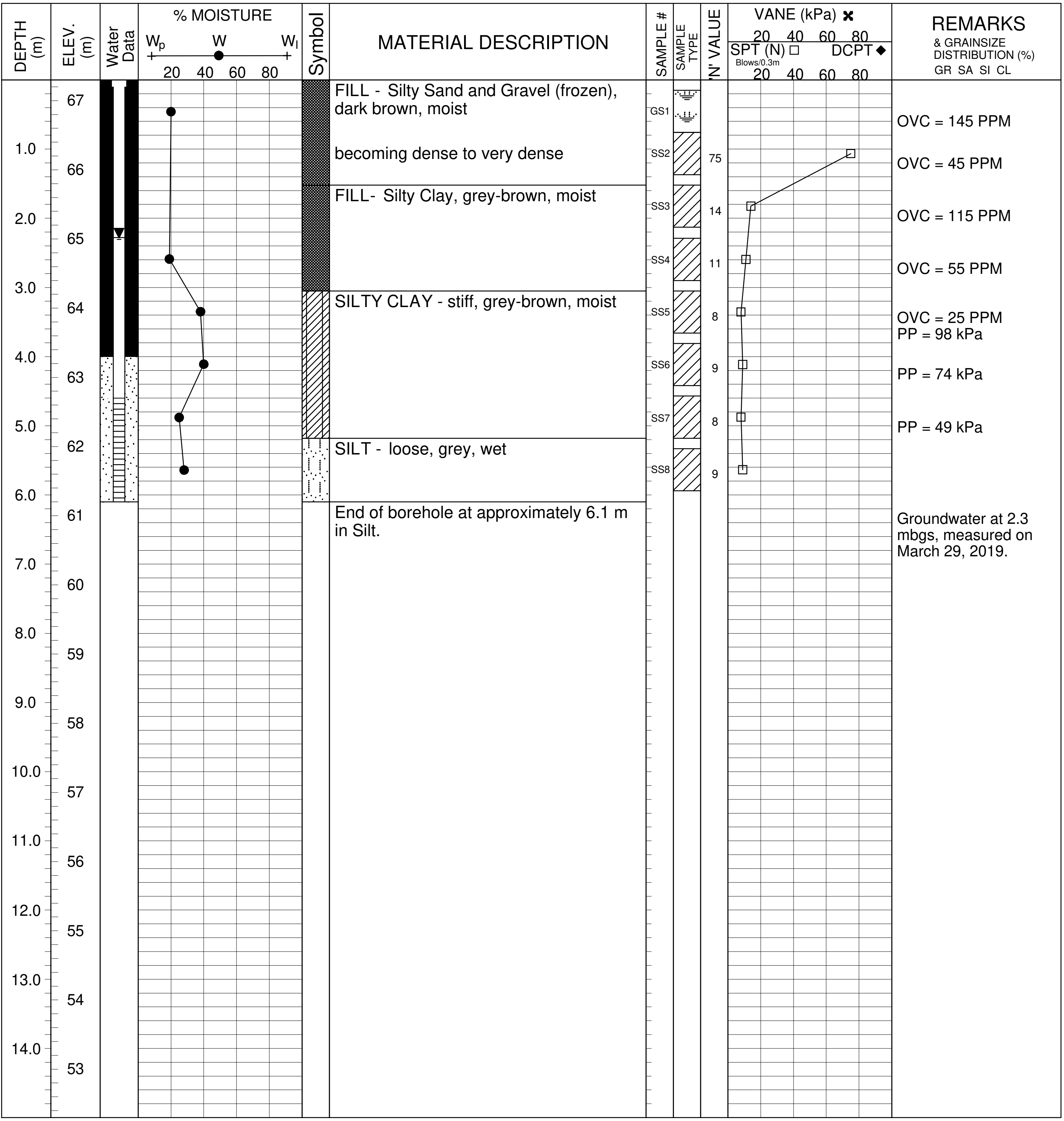
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	◫	Field Vane	▦	Sand
▥	Bulk Sample	▯	70mm Thin Wall Tube		

ENCLOSURE 6

LOG OF BOREHOLE MW19-09

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.30 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 18, 2019
COORDINATES: 5030200m N, 374780m E



FILL - Silty Sand and Gravel (frozen), dark brown, moist

becoming dense to very dense

FILL- Silty Clay, grey-brown, moist

SILTY CLAY - stiff, grey-brown, moist

SILT - loose, grey, wet

End of borehole at approximately 6.1 m in Silt.

SAMPLE #	'N' VALUE	REMARKS
GS1		OVC = 145 PPM
SS2	75	OVC = 45 PPM
SS3	14	OVC = 115 PPM
SS4	11	OVC = 55 PPM
SS5	8	OVC = 25 PPM PP = 98 kPa
SS6	9	PP = 74 kPa
SS7	8	PP = 49 kPa
SS8	9	

Groundwater at 2.3 mbgs, measured on March 29, 2019.

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SAMPLE TYPE LEGEND

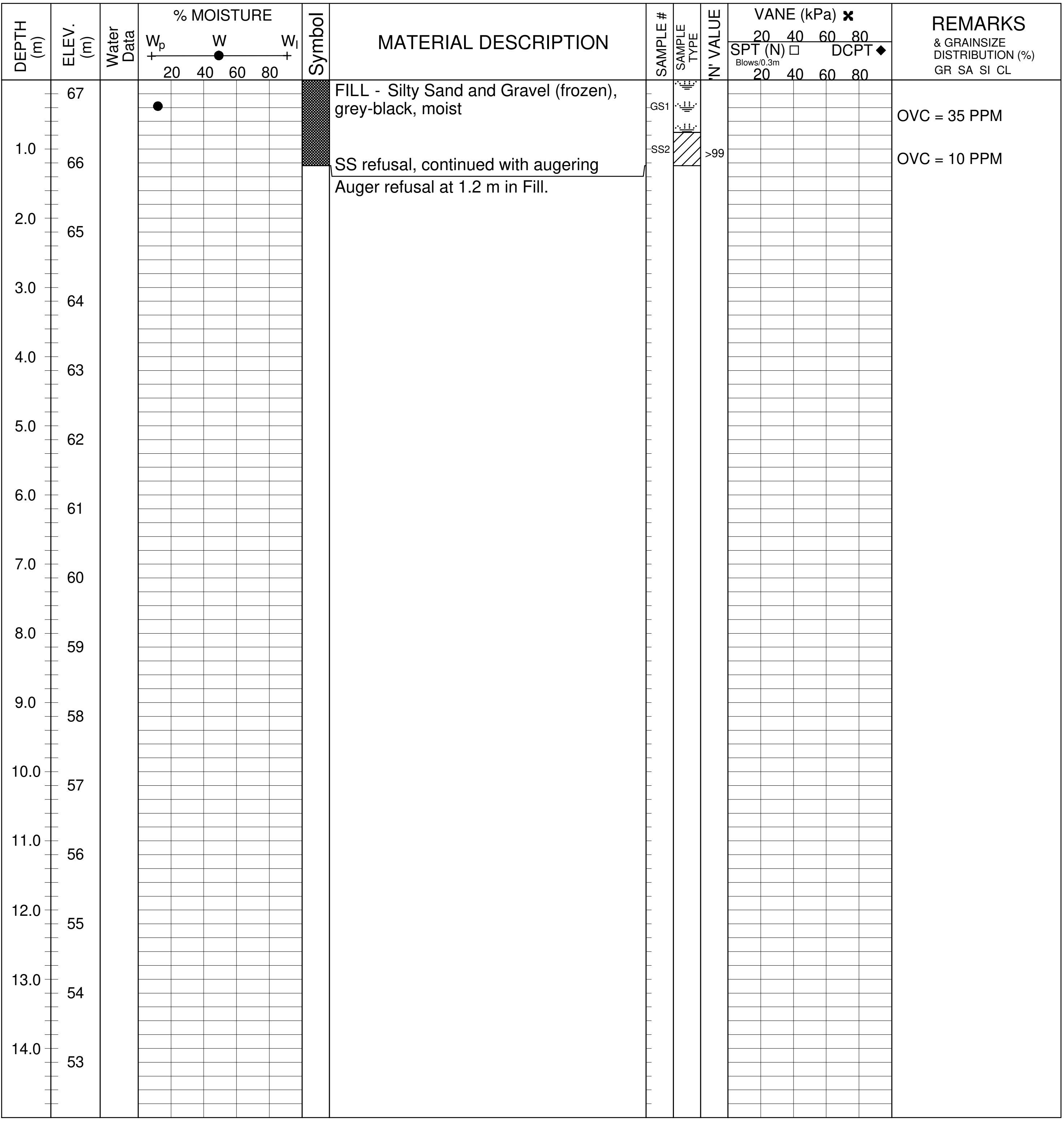
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	☐	Field Vane	░	Sand
▥	Bulk Sample	◫	70mm Thin Wall Tube		

ENCLOSURE 15

LOG OF BOREHOLE BH19-10B

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.20 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 18, 2019
COORDINATES: 5030211m N, 374808m E



MATERIAL DESCRIPTION	SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
FILL - Silty Sand and Gravel (frozen), grey-black, moist	GS1		OVC = 35 PPM
SS refusal, continued with augering	SS2	>99	OVC = 10 PPM
Auger refusal at 1.2 m in Fill.			

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SAMPLE TYPE LEGEND

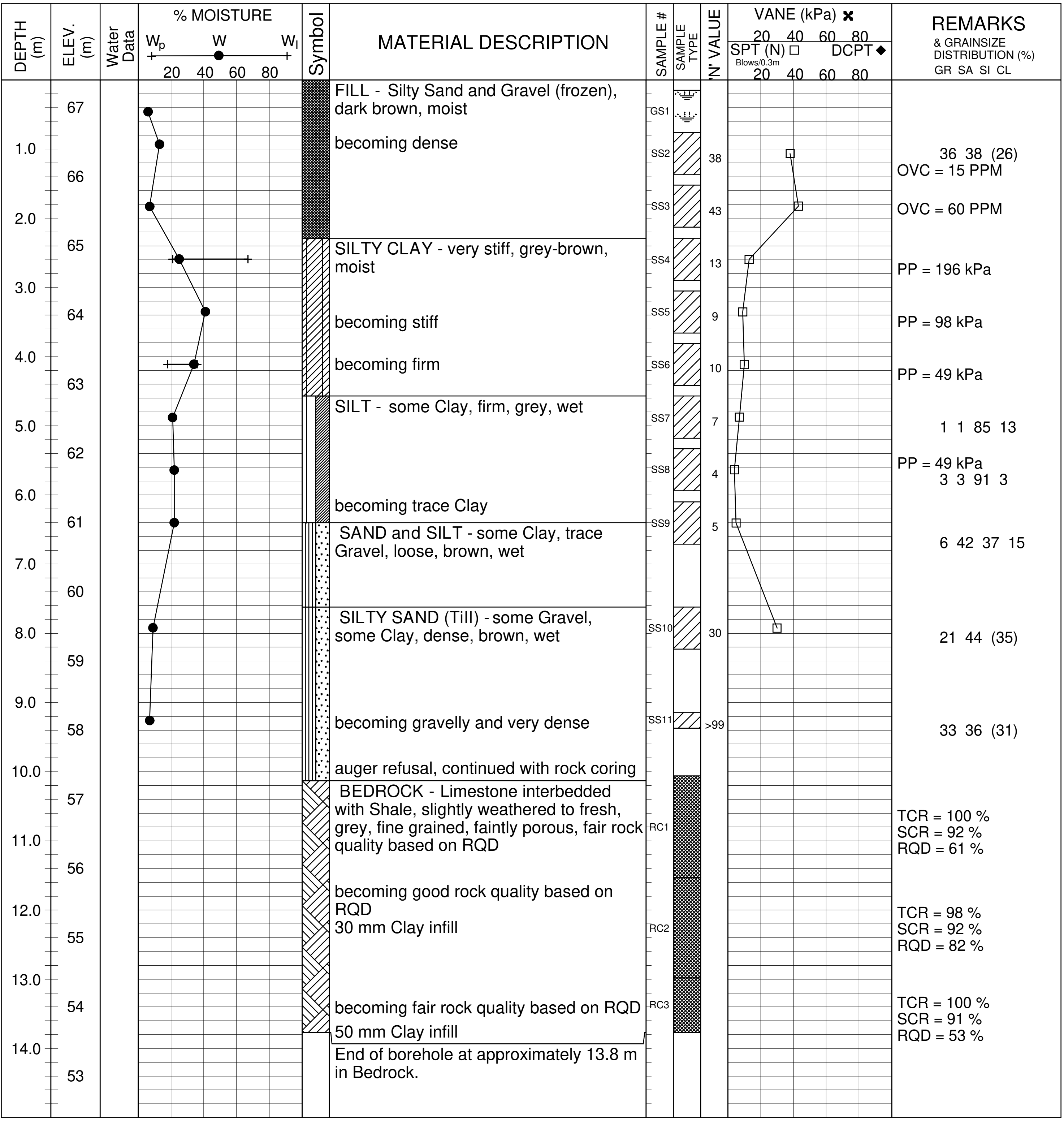
▤	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	◫	Field Vane	░	Sand
▥	Bulk Sample	◫	70mm Thin Wall Tube		

ENCLOSURE 7

LOG OF BOREHOLE BH19-11B

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.40 metres

Drilling Data
METHOD: Hollow Stem Auger & Rock Coring
DIAMETER: 200 mm
DATE: March 20, 2019
COORDINATES: 5030191m N, 374808m E



MATERIAL DESCRIPTION	SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
FILL - Silty Sand and Gravel (frozen), dark brown, moist	GS1		
becoming dense	SS2	38	36 38 (26) OVC = 15 PPM
	SS3	43	OVC = 60 PPM
SILTY CLAY - very stiff, grey-brown, moist	SS4	13	PP = 196 kPa
becoming stiff	SS5	9	PP = 98 kPa
becoming firm	SS6	10	PP = 49 kPa
SILT - some Clay, firm, grey, wet	SS7	7	1 1 85 13
	SS8	4	PP = 49 kPa 3 3 91 3
becoming trace Clay			
SAND and SILT - some Clay, trace Gravel, loose, brown, wet	SS9	5	6 42 37 15
SILTY SAND (Till) - some Gravel, some Clay, dense, brown, wet	SS10	30	21 44 (35)
becoming gravelly and very dense	SS11	>99	33 36 (31)
auger refusal, continued with rock coring			
BEDROCK - Limestone interbedded with Shale, slightly weathered to fresh, grey, fine grained, faintly porous, fair rock quality based on RQD	RC1		TCR = 100 % SCR = 92 % RQD = 61 %
becoming good rock quality based on RQD 30 mm Clay infill	RC2		TCR = 98 % SCR = 92 % RQD = 82 %
becoming fair rock quality based on RQD 50 mm Clay infill	RC3		TCR = 100 % SCR = 91 % RQD = 53 %
End of borehole at approximately 13.8 m in Bedrock.			

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SAMPLE TYPE LEGEND

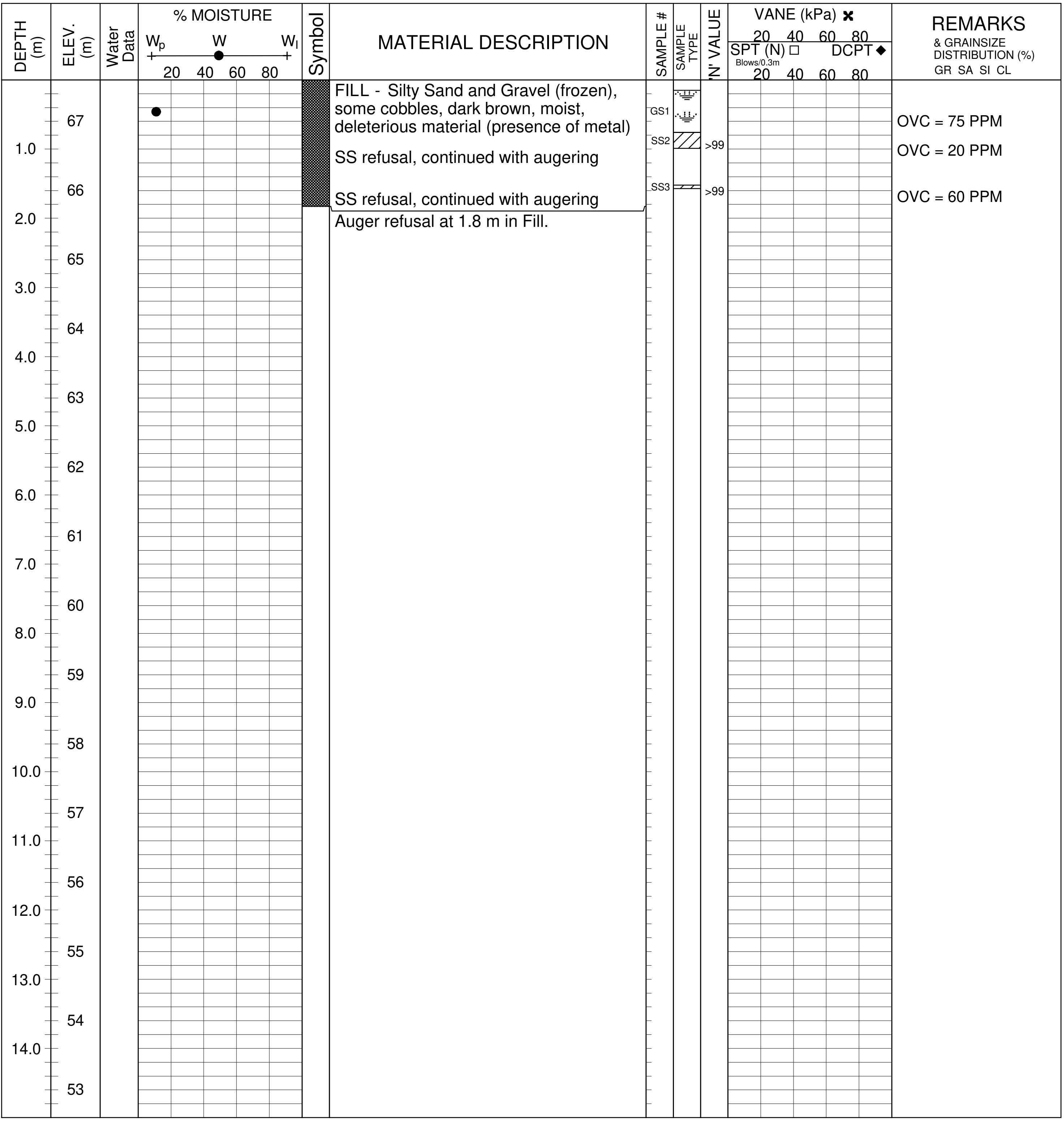
- ☰ Auger Sample
- ▨ Split Spoon Sample
- ▥ Bulk Sample
- ▩ Rock Core
- ☐ Field Vane
- ▯ 70mm Thin Wall Tube
- ■ Bentonite
- ░ Sand

ENCLOSURE 8

LOG OF BOREHOLE BH19-12

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.60 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 21, 2019
COORDINATES: 5030153m N, 374829m E



MATERIAL DESCRIPTION	SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
FILL - Silty Sand and Gravel (frozen), some cobbles, dark brown, moist, deleterious material (presence of metal)	GS1		OVC = 75 PPM
SS refusal, continued with augering	SS2	>99	OVC = 20 PPM
SS refusal, continued with augering	SS3	>99	OVC = 60 PPM
Auger refusal at 1.8 m in Fill.			

BOREHOLE (STANDARD) - OTTAWA TS-SO-37029 - BOREHOLE LOGS - APR. 10, 2019.GPJ DATA TEMPLATE.GDT 4/12/19

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FX: (613)748-1356
Email: ottawa@dstgroup.com
Web: www.dstgroup.com

SAMPLE TYPE LEGEND

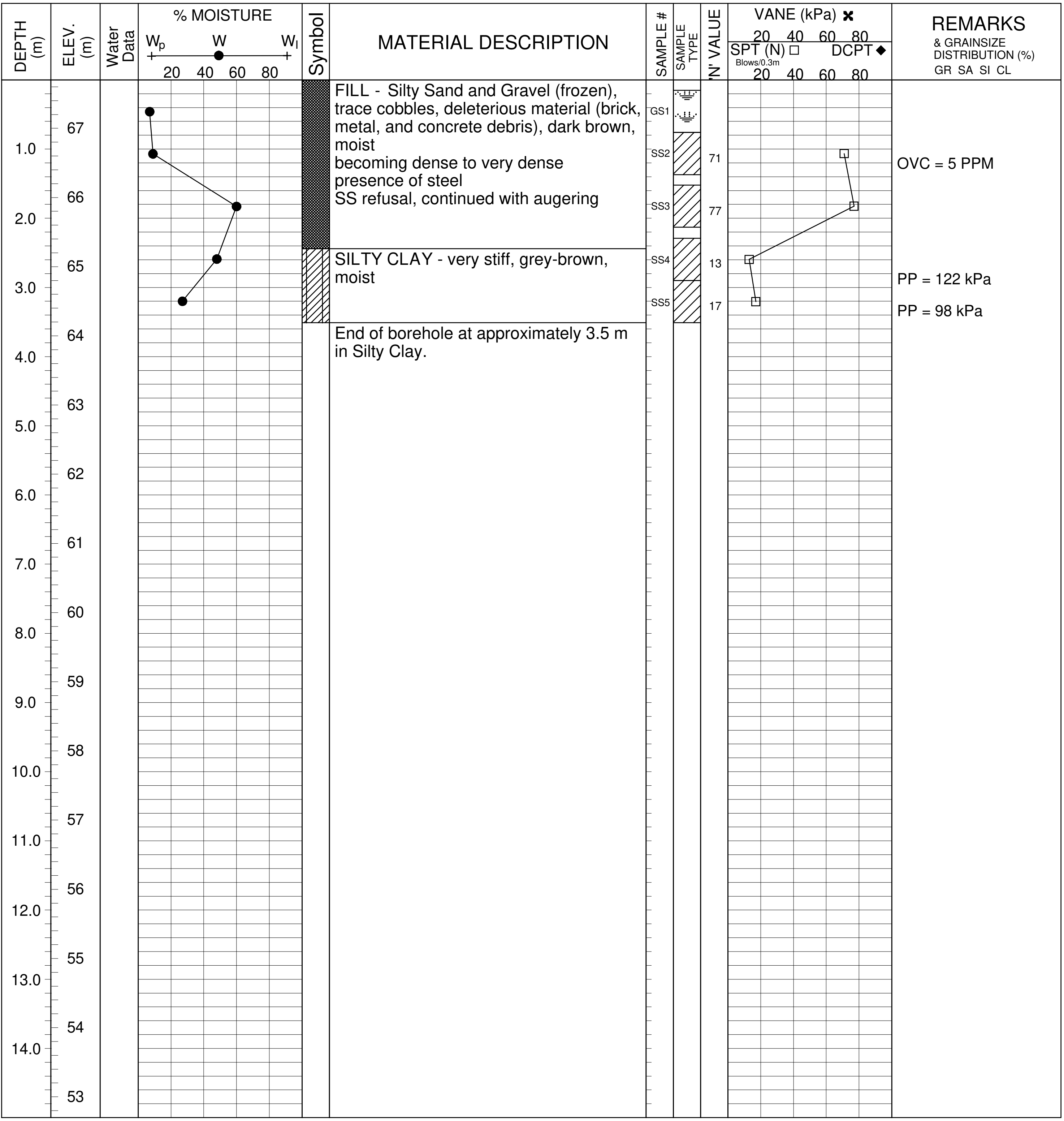
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	◫ Field Vane	▒ Sand
▥ Bulk Sample	◫ 70mm Thin Wall Tube	

ENCLOSURE 9

LOG OF BOREHOLE BH19-13

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.70 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 21, 2019
COORDINATES: 5030112m N, 374849m E



FILL - Silty Sand and Gravel (frozen), trace cobbles, deleterious material (brick, metal, and concrete debris), dark brown, moist
becoming dense to very dense
presence of steel
SS refusal, continued with augering

SILTY CLAY - very stiff, grey-brown, moist

End of borehole at approximately 3.5 m in Silty Clay.

SAMPLE #	'N' VALUE	REMARKS
GS1		
SS2	71	OVC = 5 PPM
SS3	77	
SS4	13	PP = 122 kPa
SS5	17	PP = 98 kPa

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SAMPLE TYPE LEGEND

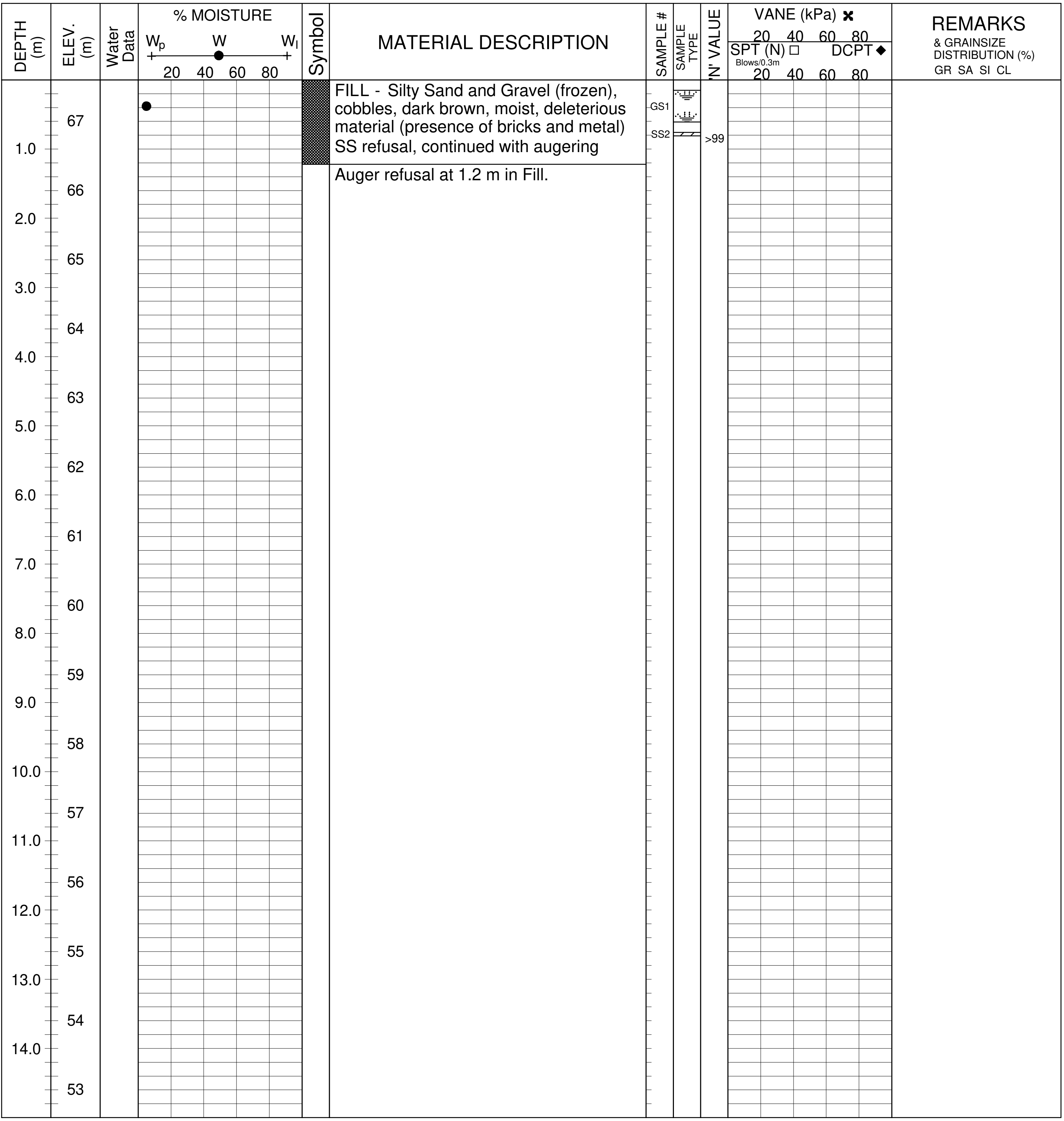
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	◫ Field Vane	░ Sand
▥ Bulk Sample	▯ 70mm Thin Wall Tube	

ENCLOSURE 10

LOG OF BOREHOLE BH19-14

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.60 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 21, 2019
COORDINATES: 5030113m N, 374810m E



FILL - Silty Sand and Gravel (frozen), cobbles, dark brown, moist, deleterious material (presence of bricks and metal) SS refusal, continued with augering

Auger refusal at 1.2 m in Fill.

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

SAMPLE TYPE LEGEND

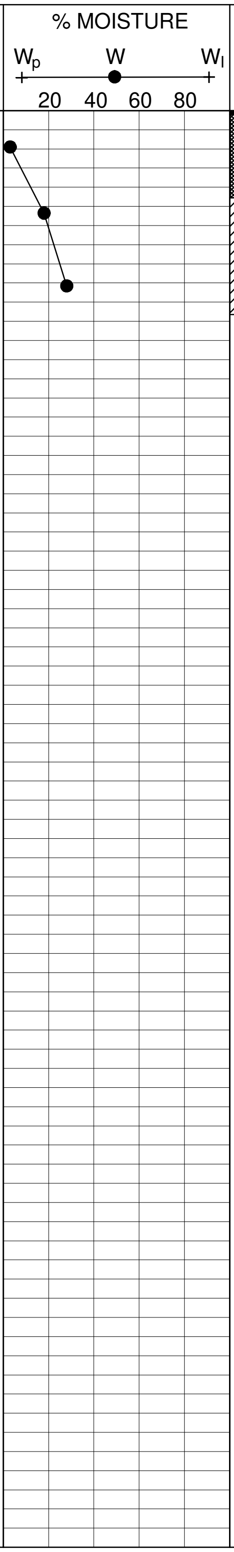
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	◫ Field Vane	▦ Sand
▥ Bulk Sample	◫ 70mm Thin Wall Tube	

ENCLOSURE 11

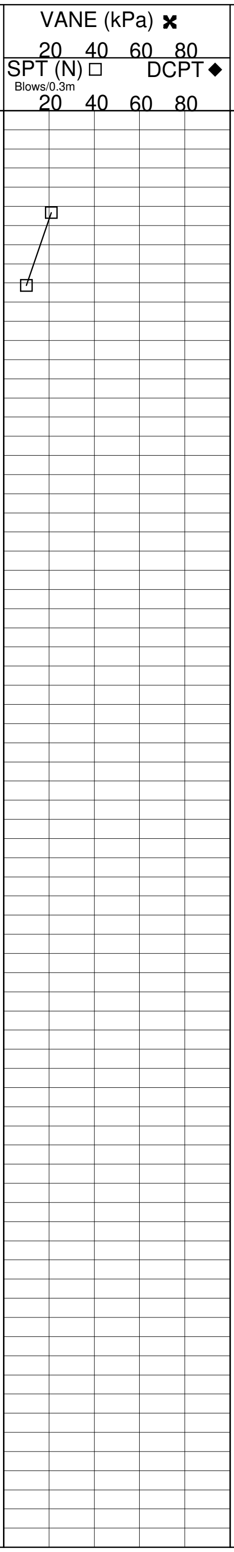
LOG OF BOREHOLE BH19-15

DST REF. No.: TS-SO-037029
CLIENT: AIM Recycling Ottawa East
PROJECT: Geotechnical Investigation
LOCATION: 2555 Sheffield Road, Ottawa, ON
SURFACE ELEV.: 67.00 metres

Drilling Data
METHOD: Hollow Stem Auger
DIAMETER: 200 mm
DATE: March 21, 2019
COORDINATES: 5030128m N, 374659m E



DEPTH (m)	ELEV. (m)	Water Data	MATERIAL DESCRIPTION	SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
			ASPHALT - Approximately 40 mm thick			
			FILL - Silty Sand and Gravel (frozen), brown, damp	GS1		
1.0	66		SILTY CLAY - very stiff, grey-brown, moist	SS2	21	
				SS3	10	PP = 145 kPa
2.0	65		End of borhole at approximately 2.1 m in Silty Clay.			
3.0	64					
4.0	63					
5.0	62					
6.0	61					
7.0	60					
8.0	59					
9.0	58					
10.0	57					
11.0	56					
12.0	55					
13.0	54					
14.0	53					



BOREHOLE (STANDARD) - OTTAWA TS-SO-37029 - BOREHOLE LOGS - APR. 10, 2019.GPJ DATA TEMPLATE.GDT 4/12/19

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SAMPLE TYPE LEGEND

- ☰ Auger Sample
- ▨ Split Spoon Sample
- Bulk Sample
- ▦ Rock Core
- Field Vane
- ☐ 70mm Thin Wall Tube
- ■ Bentonite
- ░ Sand

ENCLOSURE 12

APPENDIX D
GEOTECHNICAL LABORATORY TEST RESULTS

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Ottawa, Ontario, K1G 5T9
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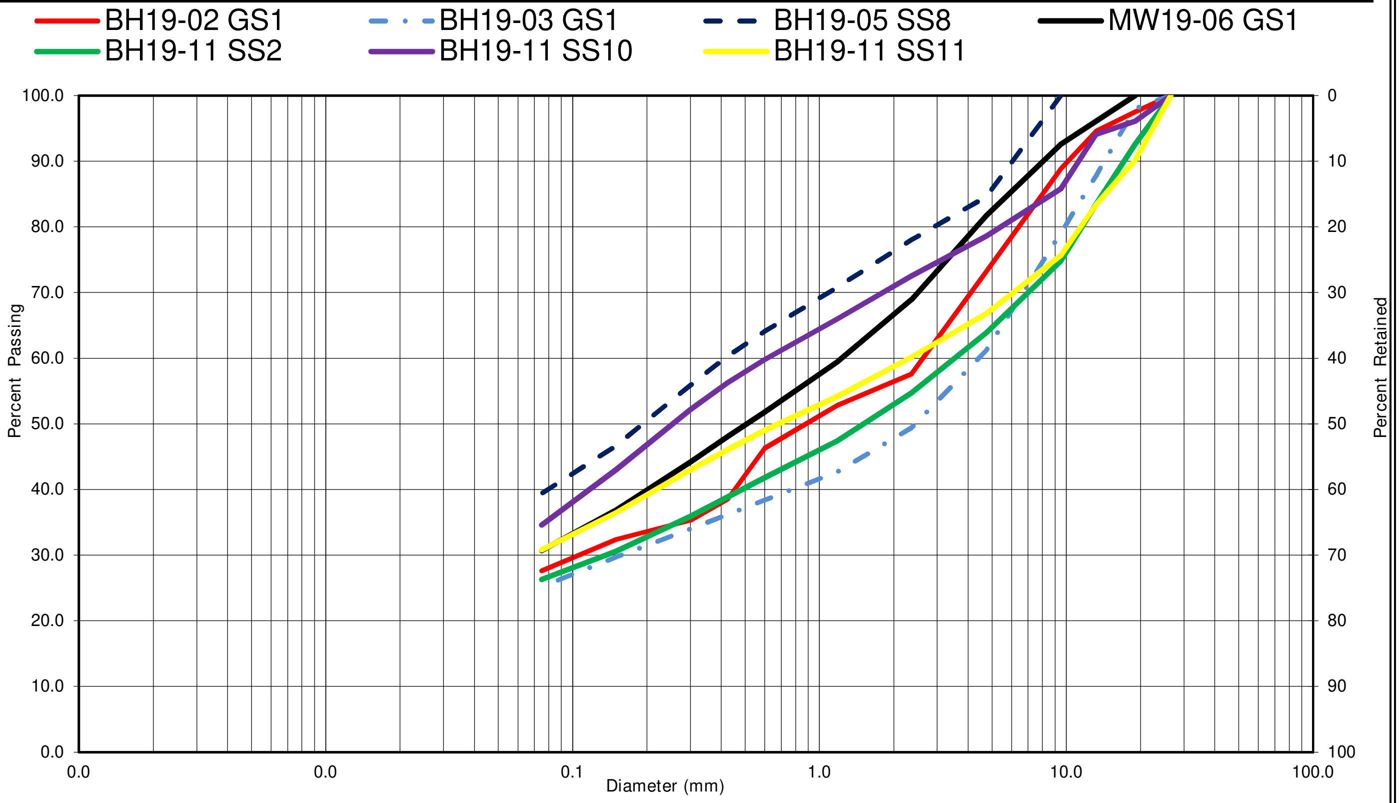
ottawa@dstgroup.com
www.dstgroup.com

PARTICLE SIZE ANALYSIS OF SOILS

DST Ref. No.:	TS-SO-37029	Date Sampled:	March 19 - 21, 2019
Project:	Geotechnical Investigation for AIM Recycling Facility	Sampled By:	Shahab Nowrozi (DST Consulting Engineers Inc.)
Project Location:	2555 Sheffield Road, Ottawa, ON	Field Sample IDs:	Refer to Legend
Client:	American Iron & Metal (AIM)	Lab Sample IDs:	BH19-02 GS1, BH19-03 GS1, BH19-05 SS8, BH19-06 GS1, BH19-11 SS2, BH19-11 SS10, BH19-11 SS11



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

Sample ID	Depth (mbgs)	Description	Particle Size Distribution (%)		
			Gravel	Sand	Silt & Clay
BH19-02 GS1	0.2 - 0.8	Gravelly SAND, some Silt, some Clay	27	46	28
BH19-03 GS1	0.2 - 0.8	GRAVEL and SAND, some Silt, some Clay	39	36	25
BH19-05 SS8	7.6 - 8.2	Silty SAND, some Gravel, trace Clay	15	45	39
MW19-06 GS1	0.2 - 0.8	SAND, some Gravel, some Silt, some Clay	18	51	31
BH19-11 SS2	0.8 - 1.4	SAND and GRAVEL, some Silt, some Clay	36	38	26
BH19-11 SS10	7.6 - 8.2	Silty Gravelly SAND, trace Clay	21	44	35
BH19-11 SS11	9.2 - 9.4	Gravelly Silty SAND, trace Clay	33	36	31

Reviewed By: Amer Mohammad, Laboratory Manager

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waterloo@dstgroup.com
www.dstgroup.com

PARTICLE SIZE ANALYSIS OF SOILS

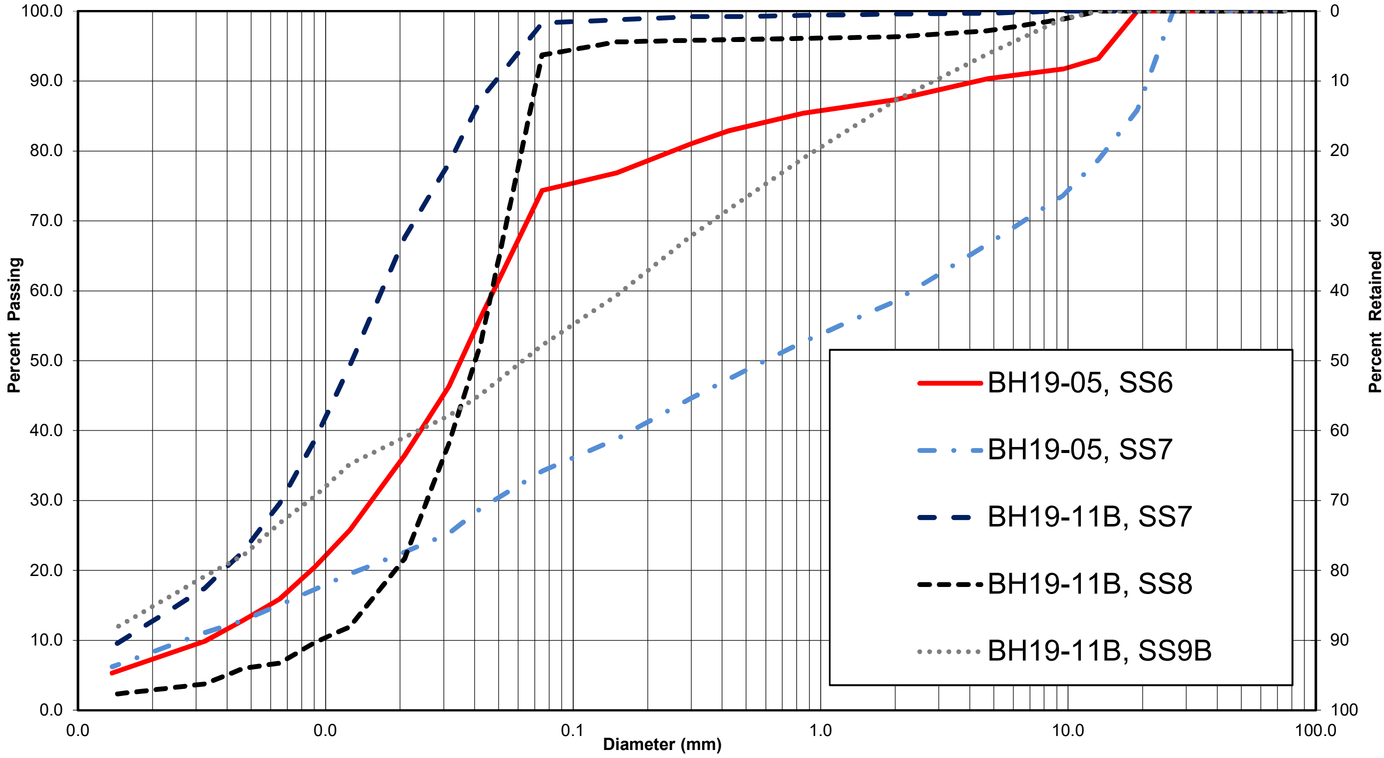
DST Ref. No.: TS-SO-37029 **Date Sampled:** March 20, 2019

Project: Geotechnical Investigation and Phase II ESA **Sampled By:** S. Nowrozi

Project Location: 2555 Sheffield Road, Ottawa, ON **Borehole IDs:** BH19-5 / BH19-11B

Client: American Iron & Metal **Sample IDs:** SS6, SS7 / SS7, SS8, SS9B

Lab Sample IDs: KWG-123-3, -4, -7, -8, -9



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

Sample ID	Depth (mbgs)	Description	Particle Size Distribution (%)		
			Gravel	Sand	Silt & Clay
BH19-05, SS6	5.5 - 5.9	Silt, some Sand, trace Gravel, trace Clay	10	16	67 / 7
BH19-05, SS7	6.1 - 6.7	Silty Sand and Gravel, trace Clay	33	33	26 / 8
BH19-11B, SS7	4.6 - 5.2	Silt, some Clay, trace Sand, trace Gravel	1	1	85 / 13
BH19-11B, SS8	5.3 - 5.9	Silt, trace Sand, trace Clay, trace Gravel	3	3	91 / 3
BH19-11B, SS9B	6.4 - 6.7	Sand and Silt, some Clay, trace Gravel	6	42	37 / 15

HA

Reviewed By: Hugh Arthur, Laboratory Supervisor

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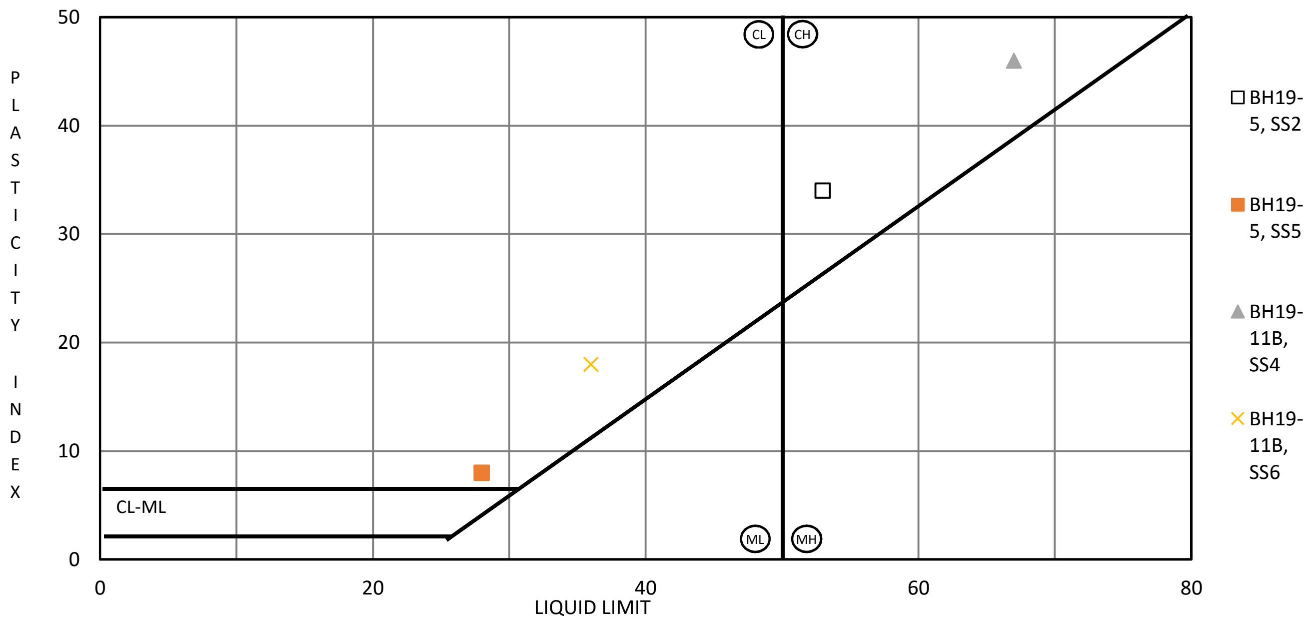
www.dstgroup.com

DST Ref. No.:	TS-SO-37029	**Date Sampled:**	March 20, 2019
Project:	Geotechnical Investigation and Phase II ESA	**Sampled By:**	S. Nowrozi
Client:	American Iron & Metal	**Borehole ID:**	BH19-5 / BH19-11B
Project Location:	2555 Sheffield Road, Ottawa, ON	**Sample IDs:**	SS2, SS5 / SS4, SS6
Lab Sample No:	KWG-123	**Description:**	CL to CH



TESTING EQUIPMENT USED

Plastic Limit	Manual	X
	Mechanical	
Liquid Limit Apparatus	Manual	X
	Mechanical	
Casagrande ASTM Tool	Metal	X
	Plastic	

ATTERBERG LIMITS TEST DATA

Sample ID	LL	PL	PI	MC
BH19-5, SS2	53	19	34	25.7
BH19-5, SS5	28	20	8	26.9
BH19-11B, SS4	67	21	46	25.4
BH19-11B, SS6	36	18	18	33.7

SPECIMEN PREPARATION DETAILS

Specimen was dried by oven and sieved over #40 sieve

ADDITIONAL NOTES

Distribution:

HA

Reviewed by:
Hugh Arthur - Laboratory Supervisor

Date:
2019-04-04

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Certificate of Analysis

DST Consulting Engineers Inc. (Ottawa)

203-2150 Thurston Dr.
Ottawa, ON K1G 5T9
Attn: Shahab Nowrozi

Client PO:
Project: TS SO 37029
Custody:

Report Date: 2-Apr-2019
Order Date: 25-Mar-2019

Order #: 1913082

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1913082-01	BH19-2, SS3
1913082-02	BH19-9, SS3,SS4

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	28-Mar-19	29-Mar-19
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	29-Mar-19	29-Mar-19
Resistivity	EPA 120.1 - probe, water extraction	28-Mar-19	29-Mar-19
Solids, %	Gravimetric, calculation	29-Mar-19	29-Mar-19

Certificate of Analysis

Client: DST Consulting Engineers Inc. (Ottawa)

Client PO:

Report Date: 02-Apr-2019

Order Date: 25-Mar-2019

Project Description: TS SO 37029

	Client ID:	BH19-2, SS3	BH19-9, SS3,SS4	-	-
	Sample Date:	03/20/2019 12:00	03/18/2019 12:00	-	-
	Sample ID:	1913082-01	1913082-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	74.9	72.3	-	-
General Inorganics					
pH	0.05 pH Units	7.39	7.98	-	-
Resistivity	0.10 Ohm.m	24.1	8.10	-	-
Anions					
Chloride	5 ug/g dry	94	480	-	-
Sulphate	5 ug/g dry	148	174	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	11.6	5	ug/g dry	12.0			3.6	20	
Sulphate	51.7	5	ug/g dry	61.0			16.5	20	
General Inorganics									
pH	7.57	0.05	pH Units	7.50			0.9	10	
Resistivity	19.2	0.10	Ohm.m	18.5			4.0	20	
Physical Characteristics									
% Solids	93.2	0.1	% by Wt.	93.3			0.1	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	101	5	ug/g	12.0	89.3	78-113			
Sulphate	151	5	ug/g	61.0	90.5	78-111			

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

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Chain of Custody
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№ 46550

Page __ of __

Client Name: DST Consulting Engineering Inc.
Contact Name: Shahab Nowozi
Address: 203-2150 Thurston Drive Ottawa, ON
Telephone: 613-295-5622

Project Reference: TSSU-37029
Quote #
PO #
Email Address: snowrozi@dstgroup.com

Turnaround Time:
☐ 1 Day ☐ 3 Day
☐ 2 Day ☒ Regular
Date Required: ______

Criteria: ☐ O. Reg. 153/04 (As Amended) Table __ ☐ RSC Filing ☐ O. Reg. 558/00 ☐ PWQO ☐ CCME ☐ SUB (Storm) ☐ SUB (Sanitary) Municipality: ______ ☐ Other: ______

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

Paracel Order Number:
1913082

Required Analyses

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken: Date	Sample Taken: Time	Corrosion	Sulphide + Redox X									
1	BH19-2, SS3	S			March 20/2019	PM	X	X X									
2	BH19-9, SS3, SS4	S			March 18, 2019	PM	X	X X									
3																	
4																	
5																	
6																	
7																	
8																	
9																	
10																	

Comments:

Method of Delivery: Walk in

Relinquished By (Sign): Snowrozi
Relinquished By (Print): Shahab Nowozi
Date/Time: March 25, 2019

Received by Driver/Depot:
Date/Time:
Temperature: ______ °C

Received at Lab: Kim Stewart
Date/Time: 25 March 13.09
Temperature: ______ °C

Verified By:
Date/Time: 3/25/19 13:23
pH Verified [ ] By: ______

Chain of Custody (Blank) - Rev 0.4 Feb 2016

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Subcontracted Analysis

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203-2150 Thurston Dr.
Ottawa, ON K1G 5T9
Attn: Shahab Nowrozi

Tel: (613) 295-5622
Fax: (613) 748-1356

Paracel Report No1913082
Client Project(s): TS SO 37029
Client PO:
Reference: Standing Offer
CoC Number:

Order Date: 25-Mar-19
Report Date: 02-Apr-19

Sample(s) from this project were subcontracted for the listed parameters. A copy of the subcontractor’s report is attached

Paracel ID	Client ID	Analysis
1913082-01	BH19-2, SS3	Redox potential, soil Sulphide, solid
1913082-02	BH19-9, SS3,SS4	Redox potential, soil Sulphide, solid

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

28-March-2019

Paracel Laboratories
Attn : Dale Robertson

300-2319 St.Laurent Blvd.
Ottawa, ON
K1G 4K6, Canada

Phone: 613-731-9577
Fax:613-731-9064

Date Rec. : 26 March 2019
LR Report: **CA15417-MAR19**
Reference: Project#: 1913082

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide %
1: Analysis Start Date		28-Mar-19
2: Analysis Start Time		11:36
3: Analysis Completed Date		28-Mar-19
4: Analysis Completed Time		12:06
5: QC - Blank		< 0.02
6: QC - STD % Recovery		110%
7: QC - DUP % RPD		3%
8: RL		0.02
9: BH19-2, SS3	20-Mar-19	< 0.02
10: BH19-9, SS3, SS4	18-Mar-19	< 0.02

RL - SGS Reporting Limit

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

OnLine LIMS

0698691000

CERTIFICATE OF ANALYSIS

Client:	Dale Robertson	Work Order Number:	368194
Company:	Paracel Laboratories Ltd.- Ottawa	PO #:	
Address:	300-2319 St. Laurent Blvd. Ottawa, ON, K1G 4J8	Regulation:	None
		Project #:	1913082
Phone/Fax:	(613) 731-9577 / (613) 731-9064	DWS #:	
Email:	drobertson@paracellabs.com	Sampled By:	
Date Order Received:	3/26/2019	Analysis Started:	4/1/2019
Arrival Temperature:	11 °C	Analysis Completed:	4/2/2019

WORK ORDER SUMMARY

ANALYSES WERE PERFORMED ON THE FOLLOWING SAMPLES. THE RESULTS RELATE ONLY TO THE ITEMS TESTED.

Sample Description	Lab ID	Matrix	Type	Comments	Date Collected	Time Collected
BH19-2,SS3	1425088	Soil	None		3/20/2019	
BH19-9,SS3,SS4	1425089	Soil	None		3/18/2019	

METHODS AND INSTRUMENTATION

THE FOLLOWING METHODS WERE USED FOR YOUR SAMPLE(S):

Method	Lab	Description	Reference
RedOx - Soil (T06)	Mississauga	Determination of RedOx Potential of Soil	Modified from APHA-2580B

This report has been approved by:

Khaled Omari

Khaled Omari, Ph.D.
Laboratory Director

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd.- Ottawa

Work Order Number: 368194

WORK ORDER RESULTS

Sample Description	BH19 - 2,SS3		BH19 - 9,SS3,SS4			
Lab ID	1425088		1425089			
General Chemistry	Result	MDL	Result	MDL	Units	Criteria: [No Reg - Always Include Reg Report]
RedOx (vs. S.H.E.)	437 [434]	N/A	-328	N/A	mV	~

LEGEND

Dates: Dates are formatted as mm/dd/year throughout this report.

[rr]: After a parameter name indicates a re-run of that parameter. Sample may not have been handled according to the recommended temperature, hold time and head space requirements of the method after the initial analysis.

MDL: Method detection limit or minimum reporting limit.

[ ]: Results for laboratory replicates are shown in square brackets immediately below the associated sample result for ease of comparison.

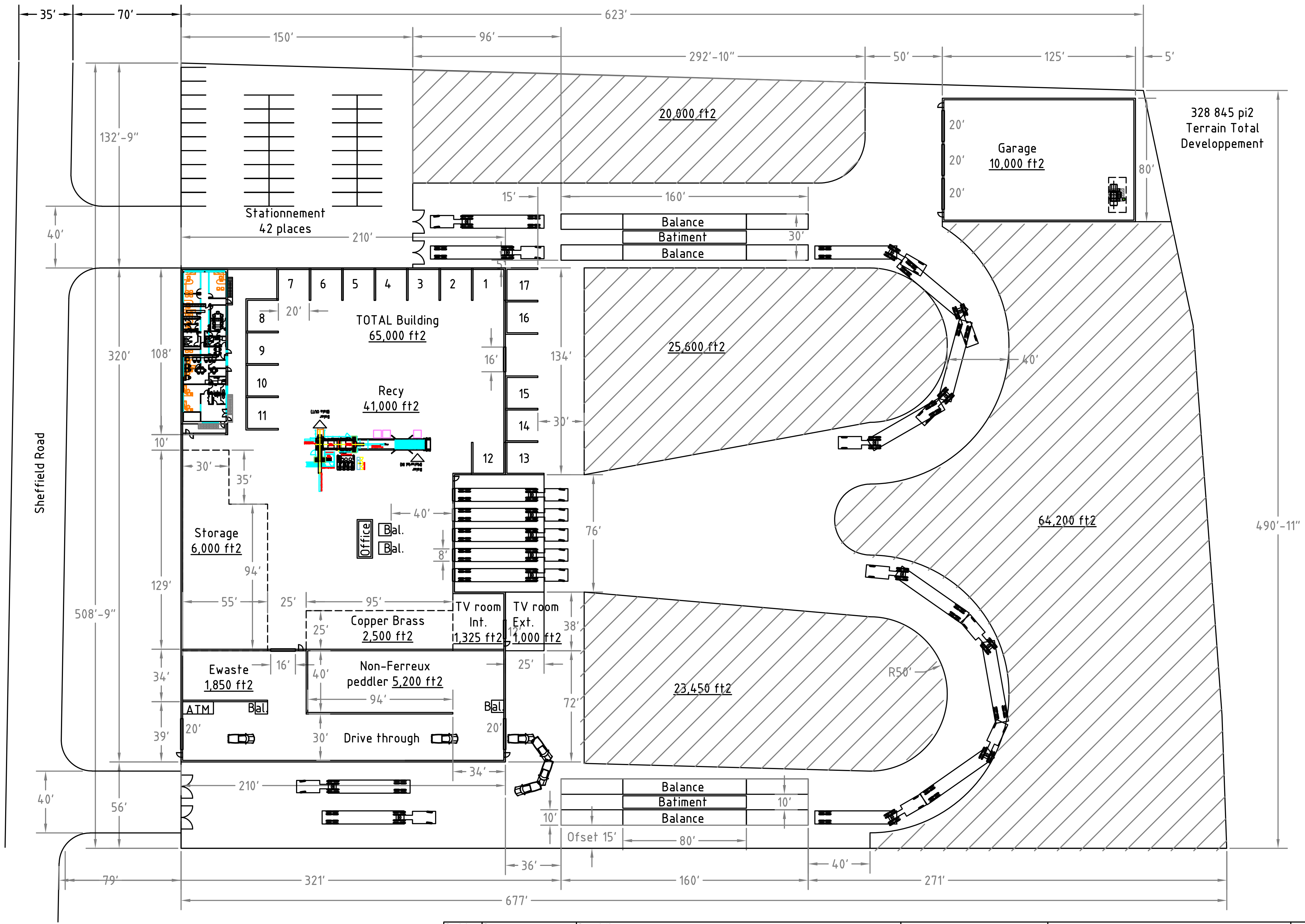
~: In a criteria column indicates the criteria is not applicable for the parameter row.

Quality Control: All associated Quality Control data is available on request.

Exceedences: HIGHLIGHTED CELLS INDICATE THAT THE RESULT EXCEEDS A REGULATORY LIMIT. CALCULATED UNCERTAINTY ESTIMATIONS ARE NOT APPLIED FOR DETERMINING SAMPLE EXCEEDANCES.

Benzo(b)fluoranthene: Results for benzo(b)fluoranthene may include contributions from benzo(j)fluoranthene.

APPENDIX E
CLIENT-PROVIDED DRAWINGS



Revision	Date	Description
8	05/12/2018	Reajustements
7	3/12/2018	Ajout departements, supperficies & traffic
6	28/11/2018	Ajustement & ajout supperficies
5	14/11/2018	Building 60,000ft2 & Stationnement
4	19/10/2018	Building 320' & Ajout d'un garage 10,000 ft2

AIM

Titre / Title:
Recycling
Ontario
Sheffield Road
Layout
Developement

Plan no: AIM-ONT-SHEFFIELD-LAY

Approuvé par / Approved by:

Vérifié par / verified by: J.Corbeil

Dessiné par / Drawing by: E.Derkacz

Vérifié par / verified by: C.Brisebois

Echelle / Scale: 1:800

Département: Projets

Date: 16/10/2018

Version: 0

Format: 11x17

1 of 1

APPENDIX F
SEISMIC SITE CLASSIFICATION

Seismic Site Classification for Seismic Site Response (Based on BH19-11B)

Site Class for Cohesionless Layers (N_{60})

Elevation		Soil	Layer Thickness t	Corrected N-Value N_{60}	t/N_{60}	
From (masl)	To (masl)		(m)	( )		
65.2	64.5	Silty Clay				(1)
64.5	63.8					
63.8	62.8					
62.8	62.2	Silt	0.6	6		
62.2	61.4		0.8	3	0.2667	
61.4	61.0		0.4	4	0.1067	
61.0	59.8	Till	1.2	4	0.3200	
59.8	58.2		1.6	23	0.0711	
58.2	57.2		1.0	74	0.0135	
57.2	35.2	Bedrock	22.0	100	0.2200	(2)
		TOTAL =	**27.6**	Sum t/N_{60} =	**0.9979**	

NOTES:

(1) The founding elevation is assumed as 65.1 masl based on the fill depth in this borehole.

(2) The N-Value of bedrock is conservatively taken as 100.

The average standard penetration resistance is calculated using the following formula:

(as per OBC 2006 Table 4.1.8.4.A.):

$$\text{Avg}(N_{60}) = \frac{\text{Total Thickness of all Layers}}{\sum \frac{\text{Layer Thickness } (t)}{\text{Layer Corrected N-Value } (N_{60})}}$$

$$\textbf{Avg(N60)} = \frac{\textbf{27.6}}{\textbf{0.9979}}$$

Avg(N60) = 27.7

Average standard penetration resistance for the borehole is between 15 and 50.

Site Class for Cohesive Layers (Su)

Elevation		Soil	Layer Thickness t	Undrained Shear Resist. S_u	t/Su	
From (masl)	To (masl)		(m)	( )		
65.2	64.5	Silty Clay	0.7	196.0	0.0036	(1)
64.5	63.8		0.7	98.0	0.0071	
63.8	62.8		1.0	49.0	0.0204	
62.8	62.2	Silt				
62.2	61.4					
61.4	61.0					
61.0	59.8	Till				
59.8	58.2					
58.2	57.2					
57.2	35.2	Bedrock	22.0	250.0	0.0880	(2)
		TOTAL =	**24.4**	Sum t/Su =	**0.1191**	

NOTES:

(1) The founding elevation is assumed as 65.1 masl based on the fill depth in this borehole.

(2) The shear strength of bedrock is conservatively taken as 250 kPa.

The average undrained shear strength is calculated using the following formula:

(as per OBC 2006 Table 4.1.8.4.A.):

$$\text{Avg}(Su) = \frac{\text{Total Thickness of all Layers}}{\sum \frac{\text{Layer Thickness } (t)}{\text{Layer Shear Strength } (Su)}}$$

$$\textbf{Avg(Su)} = \frac{\textbf{24.4}}{\textbf{0.1191}}$$

Avg(Su) = 204.8

Average undrained shear strength for this borehole is greater than 100 kPa.

Therefore, the Site Class is conservatively taken as "Site Class D" based on the standard penetration resistance.

Seismic Site Classification for Seismic Site Response (Based on BH19-05)

Site Class for Cohesionless Layers (N_{60})

Elevation		**Soil**	**Layer Thickness** ***t***	**Corrected N-Value** N_{60}	t/N_{60}	
From	**To**					
(masl)	**(masl)**		**(m)**	**( )**		
65.6	64.8	Silty Clay				(1)
64.8	64.0					
64.0	63.3					
63.3	62.5					
62.5	61.8	Silt trace Gravel	0.7	5	0.1400	
61.8	61.0		0.8	5	0.1600	
61.0	58.2	Silty Sand (Till)	2.8	17	0.1647	
58.2	35.6	Bedrock	22.6	100	0.2260	(2)
		TOTAL =	**26.9**	Sum t/N_{60} =	**0.6907**	

NOTES:

(1) The founding elevation is assumed as 65.6 masl based on the fill depth in this borehole.

(2) The N-Value of bedrock is conservatively taken as 100.

The average standard penetration resistance is calculated using the following formula:

(as per OBC 2006 Table 4.1.8.4.A.):

$$\text{Avg}(N_{60}) = \frac{\text{Total Thickness of all Layers}}{\sum \frac{\text{Layer Thickness } (t)}{\text{Layer Corrected N-Value } (N_{60})}}$$

$$\textbf{Avg(N60) =} \ \frac{\textbf{26.9}}{\textbf{0.6907}}$$

Avg(N60) = 38.9

Average standard penetration resistance for the borehole is between 15 and 50.

Site Class for Cohesive Layers (Su)

Elevation		**Soil**	**Layer Thickness** ***t***	**Undrained Shear Strength** S_u	t/Su	
From	**To**					
(masl)	**(masl)**		**(m)**	**( )**		
65.6	64.8	Silty Clay	0.8	145.0	0.0055	(1)
64.8	64.0		0.8	98.0	0.0082	
64.0	63.3		0.7	50.0	0.0140	
63.3	62.5		0.8	96.0	0.0083	
62.5	61.8	Silt trace Gravel				
61.8	61.0					
16.0	58.2	Silty Sand (Till)				
58.2	35.6	Bedrock	22.6	250.0	0.0904	(2)
		TOTAL =	**25.7**	Sum t/Su =	**0.1264**	

NOTES:

(1) The founding elevationassumed as 65.2 masl based on the fill depth in this borehole.

(2) The shear strength of bedrock is conservatively taken as 250 kPa.

The average undrained shear strength is calculated using the following formula:

(as per OBC 2006 Table 4.1.8.4.A.):

$$\text{Avg(Su)} = \frac{\text{Total Thickness of all Layers}}{\sum \frac{\text{Layer Thickness } (t)}{\text{Layer Shear Strength } (Su)}}$$

$$\textbf{Avg(Su) =} \ \frac{\textbf{25.7}}{\textbf{0.1264}}$$

Avg(Su) = 203.3

Average undrained shear strength for this borehole is greater than 100 kPa.

Therefore, the Site Class is conservatively taken as "Site Class D" based on the standard penetration resistance.