



**SUPPLEMENTARY GEOTECHNICAL INVESTIGATION
HOPEWELL CROSSING SUBDIVISION DEVELOPMENT
TOWNSHIP OF WOOLWICH (BRESLAU), ONTARIO**

for

**ARISS GLEN DEVELOPMENT LIMITED
c/o THOMASFIELD HOMES LIMITED**

PETO MacCALLUM LTD.
16 FRANKLIN STREET SOUTH
KITCHENER, ONTARIO
N2C 1R4
PHONE: (519) 893-7500
FAX: (519) 893-0654
EMAIL: kitchener@petomaccallum.com

Distribution:
4 cc: Thomasfield Homes Limited
1 cc: PML Kitchener

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Mr. Tom McLaughlin
Ariss Glen Development Limited
c/o Thomasfield Homes Limited
295 Southgate Drive
P.O. Box 1112
Guelph, Ontario
N1H 6N3

Dear Mr. McLaughlin

**Supplementary Geotechnical Investigation
Hopewell Crossing Subdivision Development
Township of Woolwich (Breslau), Ontario**

Peto MacCallum Ltd. (PML) is pleased to present the findings of our recent supplementary geotechnical investigation for the project referenced above.

The project involves a planned residential / commercial subdivision development located on two parcels of land, east of Hopewell Creek and south of Highway 7 in the Township of Woolwich, Ontario. The proposed land use for the subdivision is to comprise of residential, commercial, and office / employment land use.

We trust this report has been completed within our terms of reference, and is sufficient for your immediate requirements. If you have any questions or require further information, please do not hesitate to contact our office.

Sincerely

Peto MacCallum Ltd.



William Loghrin, P.Eng.
Project Engineer, Geotechnical Services

WL:sh



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1. INTRODUCTION

Peto MacCallum Ltd. (PML) is pleased to report the findings of the supplementary geotechnical investigation recently completed for the above noted site. Authorization was received from Mr. T. McLaughlin in an email dated November 7, 2016.

1.1 Project Description

The project involves a planned residential / commercial subdivision development located on two parcels of land, east of Hopewell Creek and south of Highway 7 in the Township of Woolwich, Ontario. The westerly parcel, known formerly as 'Seagrams Land', is approximately 91.7 ha (226.6 acres) and is bound by Victoria Street North (Highway 7) to the north, Greenhouse Road to the east, a railway to the south, and the Hopewell Creek to the west. The easterly parcel, formerly known as 'Bisgould Land', is approximately 44.0 ha (108.6 acres) and is encompassed by Highway 7 to the north, a cemetery plot to the east, Kramp Road to the south and Greenhouse Road to the west.

PML conducted geotechnical and hydrogeological investigations for the site in 2010, for Thomasfield Homes Ltd. with the findings provided in the following reports:

1. Geotechnical Investigation
Proposed Mixed Use Development
Township of Woolwich (Breslau), Ontario
PML Ref. 10KF058, Report 1, April 8, 2011
2. Hydrogeological Site Assessment for
Seagrams and Bisgould Lands and Stormwater Management Ponds
Township of Woolwich, Ontario
PML Ref. 10KF058, Report 2, April 15, 2011
3. Supplementary Geotechnical Investigation
Proposed Mixed Use Development
Township of Woolwich (Breslau), Ontario
PML Ref. 10KF058, Report 3, December 17, 2010



The layout of the proposed development has changed somewhat from the date of the previous geotechnical and hydrogeological investigations. Specifically, the current proposed development plans include a sanitary pumping station located in Block 81 at the central portion of the site, along with a deep sanitary sewer along Streets D and Streets C and I connecting to Greenhouse Road, and four storm water management (SWM) ponds in Blocks 61, 63, 65 and 66.

1.2 Terms of Reference

The purpose of the supplementary geotechnical investigation was to further explore the subsurface soil and ground water conditions at the site, and based on the findings, provide geotechnical design and construction recommendations for the proposed pumping station, deep trunk sanitary and storm water management ponds. Specific considerations to be addressed in this report include:

- A description of the site and the field investigation procedure;
- A summary of the subsurface soil and ground water conditions encountered;
- Log of borehole sheets, a borehole location plan drawing, and geotechnical laboratory test results;
- Foundation design options for the sewage pumping station, including strip or spread footing bearing resistances and settlement projections, and seismic site classification;
- Below grade tanks, including lateral earth pressures and hydrostatic uplift considerations
- Excavation recommendations, including safe side slopes and dewatering requirements;
- Pipe bedding, cover and backfill requirements, including material and compaction requirements, suitability of excavated soils for reuse as backfill; and,
- Storm water pond design, including geotechnical recommendations for design of berms, liners, and under drain systems.



It must be noted that the detailed design of the proposed facilities have not been finalized at the time of this report. Accordingly, when final design details are available they should be submitted to PML for review to ensure the recommendations provided in this report are applicable.

2. INVESTIGATION PROCEDURES

2.1 Field Work

The field work for the supplementary investigation was carried out between December 6, 2016 and January 4, 2017, and consisted of 12 sampled boreholes (boreholes 101 to 106, and boreholes 110 to 115) at locations shown on the appended Borehole Location Plan, Drawing 1. The boreholes were advanced to depths of between 8.1 and 29.4 m below grade. Boreholes 103, 105, 113, 114 and 115 were fitted with 50 mm observation well installations for purpose of a hydrogeological investigation being carried out by Blue Metric.

The boreholes were advanced using a CME 75, track mounted drillrig equipped with continuous flight hollow stem augers, supplied and operated by a specialist drilling contractor.

The scope of work included three test pits excavated to observe the general subsurface conditions at the site. The test pits were excavated at the general locations shown on the Borehole Location Plan, Drawing 1.

Representative samples of the overburden were recovered at regular intervals throughout the depths explored. Standard penetration tests were carried out during sampling operations using conventional split spoon equipment.

The ground water conditions on the boreholes were assessed during drilling by visual examination of the soils, the sampler and drill rods as the samples were retrieved and of measurement of the water level, if any, in the open borehole.



The field work was supervised throughout by a member of our engineering staff who directed the drilling and sampling operation, prepared stratigraphic logs, monitored ground water conditions, and processed the recovered samples.

The supplementary drilling program was established through consultation with the client. The borehole locations were determined in the field by PML. The ground surface elevations at the boreholes were interpolated from topographic site plan drawings, provided by the client.

Upon completion of drilling the boreholes were sealed in accordance with O.Reg. 903. Monitoring wells were installed in Boreholes 103, 105, 113, 114 and 115 to permit measurements of the ground water levels and samples of the ground water. The monitoring wells comprised 50 mm diameter PVC schedule 40 pipe, filter sand, bentonite seals, and protective caps. Copies of the well records will be provided under separate cover.

In accordance with O.Reg 903/90, as amended, the owner of the wells is defined as the owner of the land upon which the wells are situated and the owner will have to decommission the current and previous wells when no longer needed. PML would be pleased to assist in this regard.

2.2 Laboratory Testing

All samples secured in the field were returned to our laboratory for detailed visual examination to confirm field soil classifications. The laboratory testing program included moisture content determinations on all recovered samples. Particle size distribution analyses were conducted on ten samples of the major soil types encountered. Atterburg limits testing was conducted on two samples of the native till soils.



3. SUMMARIZED FINDINGS

Reference is made to the appended Log of Borehole sheets for details of the subsurface conditions including soil descriptions, inferred stratigraphy, standard penetration N values, pocket penetrometer shear strengths, ground water observations during and upon completion of drilling, Atterburg limits, and natural moisture content determination test results. In general, the subsurface conditions encountered in the recent boreholes comprised topsoil overlying discontinuous layers of silt, sand and clay based soils and till deposits.

Due to the soil sampling procedures and the limited size of samples, the depth/elevation demarcations on the borehole logs must be viewed as "transitional" zones, and cannot be construed as exact geologic boundaries between layers.

3.1 Sewage Pumping Station, Block 81

Boreholes 101 to 106 were advanced in Block 81, the proposed location of the sewage pumping station.

The subsurface stratigraphy comprised surficial topsoil, over granular deposits comprising silt / sandy silt / silty sand / silty sand and gravel / sand / sand and gravel underlain by silt till and silty clay / clayey silt till.

The surficial topsoil was 0.20 to 0.25 m thick. The topsoil was moist to wet with laboratory moisture contents between 12 and 22%.

Granular deposits comprised of layers of silt / sandy silt / silty sand / silty sand and gravel / sand / sand and gravel were encountered in Boreholes 101 to 106 below the topsoil, and extended to between 2.1 and 11.1 m depth. Occasional to numerous cobbles were noted within the deposits. The relative density was loose to very dense with SPT N values ranging from 9 to greater than 50 blows per 0.3 m penetration of the split spoon sampler. The soils were damp to saturated with laboratory moisture contents between 8 and 20%.



Silt till / clayey silt till / silty clay till were encountered below upper granular soils and extended to the borehole termination depths. The till deposits contained occasional to numerous cobbles and boulders. Water bearing layers of silt, sand, sand and silt, sandy silt, silty sand, and sandy gravel, were within the till. The till was compact to very dense / very stiff to hard with SPT N values ranging from 15 to greater than 50 blows per 0.3 m penetration. Pocket penetrometer shear strengths of the clayey silt till and silty clay till ranged from 125 to greater than 225 kPa. The till deposits were moist to wet / drier than plastic limit (DTPL) to about plastic limit (APL), with moisture contents ranging between 6 and 32%.

Wet / saturated soils were observed below 0.2 to 2.2 m depth in Boreholes 101 to 106 during drilling (elevation 320.1 to 321.5).

Water level measurements in the wells installed in Boreholes 103 and 105, on January 18, 2017, were 1.5 and 1.2 m depth (elevation 320.0 and 320.5), respectively.

3.2 Trunk Sanitary Sewer

Boreholes 110 to 114 were located along the proposed route of the trunk sanitary sewer, along the proposed Street D, and Streets C and I near Greenhouse Road. The general subsurface stratigraphy encountered comprised topsoil over granular layers comprising silty sand / silt / sand and gravel / silty sand and gravel, underlain by sandy silt till, silt till and sandy gravel deposits.

The surficial topsoil in Boreholes 110 to 114 was 0.20 to 0.25 m thick. The topsoil was damp to moist with moisture contents between 10 and 15%.

Layers of silty sand / silt / sand and gravel / silty sand and gravel were encountered below the topsoil, and extended to between 2.1 to 4.0 m depths. These deposits contained occasional to numerous cobbles. The relative density was loose to very dense with SPT N values ranging from 8 to greater than 50 blows per 0.3 m penetration. The soils were damp to saturated with moisture contents between 4 and 18%.



Sandy silt till / silt till was encountered below the silty sand / silt / sand and gravel / silty sand and gravel, and extended to the borehole termination depths. The till deposits were occasionally interlayered with silt / silty sand and sandy gravel. The till deposits contained occasional to numerous cobbles, and boulders should be anticipated. The till was compact to very dense with STP N values ranging from 12 to greater than 50 blows per 0.3 m penetration. The till soils were moist to saturated with moisture contents between 6 and 20%.

Wet / saturated soils were generally found below 2.2 and 5.6 m depth (elevation 317.8 to 323.9) in Boreholes 110 to 114. Water level readings taken on January 18, 2017 in the monitoring wells installed in Boreholes 113 and 114 were at 1.4 and 0.9 m depth (elevation 320.6 and 322.5), respectively.

3.3 Storm Water Pond, Block 66

Borehole 115 was advanced at the proposed SWM pond area in Block 66. The subsurface stratigraphy comprised topsoil, silty sand, silty sand and gravel, and sandy silt till. Reference is given to our previous investigation reports for the stratigraphy at the Block 61, 63 and 65 SWM pond sites.

The surficial topsoil in Borehole 115 was 0.10 m thick.

Layers of native silty sand and silty sand and gravel were encountered below the surficial topsoil and extended to 2.9 m depth. These layers were loose to compact with SPT N values ranging from 7 to 25 blows per 0.3 m penetration. The soils were damp to moist with moisture contents between 8 and 12%.

Sandy silt till was encountered below the silty sand and gravel, and extended to the 8.1 m borehole termination depth. The till was dense to very dense with SPT N values ranging from 36 to greater than 50 blows per 0.3 m penetration. The till was moist to wet with moisture contents between 6 and 8%.



Wet soils were observed below 2.9 m depth in Borehole 115 during drilling. The water level measured on January 18, 2017 in the installed monitoring well was at 0.4 m depth (elevation 322.5).

3.4 Ground Water Observations

As noted previously, monitoring wells were installed in five of the boreholes to permit ground water monitoring at sampling of ground water, and for hydrogeological testing purposes.

Ground water observations carried out during and upon completion of drilling and in the observation wells are provided on the appended Log of Borehole Sheets. During drilling, wet / saturated soil conditions were encountered in the recent boreholes, between 0.2 to 5.6 m depth (about elevations 317.8 and 323.9). The water levels measured (by Blue Metric) in the monitoring wells was between 0.4 and 1.5 m depth (between about elevation 320.2 and 322.5), on January 18, 2017.

The relatively impermeable nature of the till deposits may contribute to the development of perched water conditions following seasonal and short term precipitation events.

In general, based on the ground water observations, moisture content profile and soil colouring, the ground water level appears to roughly mimic the topographic contour of the site, generally dropping off from the east to the west. It should be noted that ground water levels, particularly perched ground water levels, can vary significantly in response to seasonal weather conditions and weather events.



4. DISCUSSION AND RECOMMENDATIONS

The project involves a planned residential / commercial, mixed-use subdivision. As noted, earlier geotechnical and hydrogeological investigations were conducted by PML in 2010 and we make reference to our previous Reports 1, 2 and 3, PML Ref 10KF058, dated April 8, 2011, April 11, 2011 and December 17, 2010, respectively for general recommendations which remain applicable for the proposed development. Recommendations specific to the currently proposed sanitary pumping station, deep sanitary trunk sewer and storm water management ponds are provided herein.

It must be noted that at the time of this report, detailed design of the proposed facilities were not finalized. Accordingly when final design details are available, they should be submitted to PML for review to ensure the recommendations provided in this report are applicable.

4.1 Sewage Pumping Station

The proposed pumping station will be located in the general area near Boreholes 101 to 106 as identified on the Borehole Location Plan. The design details for the pumping station were not available at the time of this report, however, it is understood that the bottom of the pumping station will be located about 13 to 17 m below existing grade (around elevations 305 to 309).

4.1.1 Footings, Base Slabs

It is understood that monolithic precast or cast-in-place concrete bases will be used for the sewage pumping chamber structures. Further, the main base slab will be designed to act as a rigid raft foundation, without the need for isolated, interior column pads. Based on the boreholes, between elevation 305 to 309, the base slab for the pumping station structure will be founded on insitu very stiff to hard / dense to very dense till soils, sand, silt and/or sand and silt with a net bearing resistance of at least 200 to 400 kPa at SLS, and a factored bearing resistance of 300 to 600 kPa of ULS.



The table below provides a summary of the subgrade soils and available bearing resistance at various levels on a borehole by borehole basis:

BOREHOLE	ELEVATION 309	ELEVATION 308	ELEVATION 307	ELEVATION 306	ELEVATION 305
101	200 kPa SLS 300 kPa ULS Silty Clay Till	200 kPa SLS 300 kPa ULS Silty Clay Till	200 kPa SLS 300 kPa ULS Silty Clay Till	200 kPa SLS 300 kPa ULS Silty Clay Till	200 kPa SLS 300 kPa ULS Silty Clay Till
102	300 kPa SLS 450 kPa ULS Silt	400 kPa SLS 600 kPa ULS Silt	300 kPa SLS 450 kPa ULS Silt	200 kPa SLS 300 kPa ULS Silty Clay Till	200 kPa SLS 300 kPa ULS Silty Clay Till
103	300 kPa SLS 450 kPa ULS Silt	300 kPa SLS 450 kPa ULS Silt	200 kPa SLS 300 kPa ULS Silty Clay Till	300 kPa SLS 450 kPa ULS Silty Clay Till	400 kPa SLS 600 kPa ULS Silt Till
104	300 kPa SLS 450 kPa ULS Silty Sand	300 kPa SLS 450 kPa ULS Clayey Silt Till	300 kPa SLS 450 kPa ULS Clayey Silt Till	300 kPa SLS 450 kPa ULS Clayey Silt Till	300 kPa SLS 450 kPa ULS Clayey Silt Till
105	400 kPa SLS 600 kPa ULS Sand and Silt	400 kPa SLS 600 kPa ULS Sand and Silt	400 kPa SLS 600 kPa ULS Sand and Silt	300 kPa SLS 450 kPa ULS Clayey Silt Till	300 kPa SLS 450 kPa ULS Clayey Silt Till
106	400 kPa SLS 600 kPa ULS Silt	400 kPa SLS 600 kPa ULS Silt	300 kPa SLS 450 kPa ULS Clayey Silt Till	300 kPa SLS 450 kPa ULS Clayey Silt Till	400 kPa SLS 600 kPa ULS Silt Till

For the anticipated design, it is likely that the actual applied net load will be negative, as the weight of the structure and contents will not exceed the weight of soil removed.

Total settlement for foundations design for an SLS bearing pressures noted is not expected to exceed 25 mm with differential settlement of 50 % of this value.

Prior to placement of the bases, founding surfaces should be inspected by PML to ensure that no deleterious materials are present and the required bearing capacity is available. Any unstable or disturbed material identified should be subexcavated and replaced with compacted granular fill, or lean mix concrete. The base of the excavation will be located in silty or clayey soils, which are saturated and sensitive and will be easily disturbed by construction activities. Contractors must



be aware of these conditions and adopt equipment and means to minimize disturbance to the subgrade. It is recommended that following geotechnical review and approval, a 100 mm concrete working slab be placed to protect the subgrade and facilitate placement of reinforcing steel and structural concrete.

Design provisions for earthquake loading should also be applied. For the soil conditions at the site, a class D site category may be assumed, in accordance with the 2012 Ontario Building Code.

All founding surfaces are to be provided with at least 1.2 m of earth cover, and protected from seasonal freezing conditions.

Backfill should be brought up simultaneously on all sides of the structure to minimize the potential for movement and/or damage during construction. Furthermore, the use of heavy equipment adjacent to structures should be prohibited. Fill materials should comprise approved on site mineral soils or imported granular soil. The allowable lift thickness will depend on the compaction equipment selected and demonstrated compaction success in the field, but should not exceed 300 mm. The backfill should be compacted to a minimum of 95% standard Proctor maximum dry density (SPMDD) below pavements and other settlement sensitive structures, and 90% SPMDD in non-settlement sensitive (landscaped) areas.

4.1.2 Hydrostatic Uplift Design

Cognizant of the shallow ground water at this location, about 1.2 to 1.5 m below existing grade (Elevation 320.2 to 320.5) consideration must be given to hydrostatic uplift during the design of the pumping station. Uplift resistance will be provided by the weight of the structure, plus the buoyant weight of the backfill over the outside edge of the concrete footings, which could be extended laterally to provide additional uplift resistance if space constraints allow. Additional mechanical anchorage of the building may also be required to achieve sufficient uplift resistance, and if required, PML can provide further recommendations in this regard.



4.1.3 Lateral Earth Pressure

All below grade walls must be designed to resist the lateral earth pressure and water pressure, which may be determined from the following equation:

$$p = K [\gamma (h-d_w) + \gamma' d_w + q] + \gamma_w d_w$$

where

- p = total lateral pressure at depth h (m) below ground surface (kPa)
- K = lateral earth pressure coefficient of compacted backfill
= 0.5
- h = depth below grade (m) at which lateral pressure is calculated
- d_w = depth below ground water level at depth h below final exterior grade (m)
= h , when the water level is at the ground surface
- γ = unit weight of soil
= 21 kN/m³ for compacted sand and gravel at 95% SPMDD
- γ_w = unit weight of water
= 9.8 kN/m³
- γ' = buoyant unit weight of soil
= $\gamma - \gamma_w$
- q = vertical stress at depth h due to surcharge loads (kPa)

An appropriate factor of safety must be used in the design.

The wet well and below grade tanks must be designed to handle all load potential load combinations for backfilled or re-exposed walls under both empty and full conditions.



4.2 Trunk Sanitary Sewers

It is expected that the proposed sanitary sewer will be founded on competent native deposits. Therefore, bearing capacity or settlement problems are not anticipated.

Providing adequate ground water control is achieved, bearing problems are not anticipated for services founded on the native soils. It may be necessary to increase the bedding thickness or utilize clear stone bedding fully wrapped in filter cloth if excessively loose, soft or wet conditions are present at the pipe subgrade. The need for this is best determined during construction.

Conventional bedding and cover constructed in accordance with applicable Ontario Provincial Standard Drawings (OPSD) will be suitable. Material containing stones larger than 50 mm size should not be used in the bedding layer. The bedding and cover material should be placed in 150 mm lifts compacted to at least 95% SPMDD. Compaction should be provided beneath the pipe haunches to provide uniform support. Over-compaction should be avoided as damage to the pipe could result.

The trench backfill material should comprise approved material placed in uniform 200 mm thick lifts within 3% of the optimum moisture content and compacted to at least 95% SPMDD. It is anticipated that the excavated material will primarily comprise existing silty sand / silt / sand and gravel / silty sand and gravel and till deposits. The insitu moisture content of the insitu native mineral soils typically ranges from 4 to 20%.

In general, the site soils classified as damp/moist should be suitable for reuse as backfill and compactible to 95% SPMDD. Material described as 'wet' or 'saturated' on the appended borehole logs will generally not be compactible to 95% SPMDD. The excavated wet and saturated soils are considered suitable for reuse only if the work is carried out during the dry summer months and the construction schedule is flexible to permit air drying to reduce the moisture content closer to the optimum value. Otherwise, the wet and saturated soils should be segregated and allocated to areas where post construction settlement would not present a concern.



Material with an elevated moisture content could be used in the lower portion of the trench below a depth of 1.5 m from finished road grade provided it is placed and compacted in thin lifts to at least 90% SPMDD and some post construction settlement of the road surface is acceptable. If placement of the surface asphalt is delayed for at least one year, this settlement should have minimal impact on road performance.

It should also be noted that the insitu till materials will tend to retain a voided structure when placed as backfill. Sufficient compaction must be applied to breakdown all lumps / clods within the fill matrix to achieve a non-voided condition. Significant post construction settlement could otherwise result.

The trenching and backfilling operations should be carried out in a manner which minimizes the length of trench left open yet accommodates efficient pipe laying and compaction activities.

4.3 Excavation and Ground Water Control

Excavation for trenches and the pumping station will extend through a variety of cohesionless silts, sands, sand and gravel, and till and cohesive till, encountering water bearing layers at many or all locations.

The soils are classified as Type 3 soil as defined in the Occupational Health and Safety Act (OHSA). Provided adequate ground water control is achieved, trench excavations within Type 3 soil that are to be entered by workers, may not be steeper than one horizontal to one vertical (1H:1V) from the base of the excavation.

The pumping station excavations can be conducted using conventional open cut methods, however, braced excavation may be preferred to minimize the extent of excavation. In this regard, driven sheet piling may not be feasible given the very dense soils identified and the presence of cobbles and boulders. A secant caisson wall system is envisaged, although difficulty removing boulders, where encountered, should not be disregarded.



At the sewage pumping station, ground water levels measured in the monitoring wells were at 1.2 to 1.5 m below grade (elevation 320.2 to 320.5). Based on the proposed 13 to 17 m wet well depth and shallow water level in this area, both open-cut and braced excavations will likely require significant construction dewatering activities, including the use of vacuum well points or eductor wells.

Excavations for the trunk sanitary sewer are to extend to between 5 and 9 m depth (elevations 315 to 319). Excavations for the storm water ponds are generally expected to extend 3 to 8 m below existing grade (about elevations 315 to 319). Excavations for the trunk sewer and storm water ponds extend through granular over till deposits contains water bearing layers. It is anticipated that dewatering using high capacity sump pumps, keg wells, well points and eductor wells will be necessary for ground water control management.

The design of the dewatering system should be provided by the Contractor and the system should meet a performance specification to maintain and control ground water at least 0.5 m below the excavation base level, in order to provide a stable excavation base and safe side slopes throughout construction.

It should be noted that, under the Ontario Water Resources Act, the Water Taking and Transfer Regulation 387/04, a Permit to Take Water (PTTW) from the Ministry of Environment and Climate Change (MOECC) is required if the dewatering discharge is greater than 50,000 L/day. In accordance with the above noted regulatory requirements and in compliance with the MOECC's PTTW Manual (April 2005), and application should be filed to the MOECC for the subject property construction dewatering PTTW, if the dewatering discharge is greater than 400,000 L/day, or about 4.6 L/S. If the dewatering discharge is between 50,000 L/day (or about 0.6 L/S) and 400,000 L/day (or about 4.6 L/S) dewatering activities need to be registered on the Environmental Activity and Sector Registry (EASR). Open cut excavations for the proposed sewage pumping station are expected to have pumping rates that exceed 400,000 L/day and a PTTW will be necessary along with a supporting hydrogeological investigation. Alternatively, if a water tight perimeter cut off structure is provided for the pumping station then dewatering rates may not exceed 400,000 L/day but could exceed 50,000 L/day where an EASR would be required. It is



also anticipated that dewatering for the sewer installation and storm water pond construction will exceed 50,000 L/day.

It is recommended that test pits be carried out during the tendering stage of the project in order that prospective contractors may familiarize themselves with soil and ground water conditions. Also, the dewatering requirements should also be established by the contractor in the context of a performance specification.

4.3.1 Braced Excavations

Depending on space availability and/or to reduce the size of the excavation, consideration should be given to using an engineered system to support the excavation walls. The soil parameters presented in the following table may be used for the design. The parameters are based on the Rankin method of analysis which ignores wall friction. To provide adequate ground water control and to prevent piping in the base, the sheeting or secant wall should extend at least 0.5 m into the till, although greater penetration depths may be necessary to develop adequate passive resistance at the toe.

SOIL TYPE	NATIVE COHESSIONLESS SOILS	CLAYEY SILT / SILTY CLAY TILL
Coefficient of Active Earth Pressure K_a	0.36	0.36
Coefficient of Passive Earth Pressure K_p	2.77	2.77
Angle of Internal Friction ϕ	28	--
Cohesive Strength c_u	--	100
Unit Weight γ (kN/m ³)	21	21

It will also be important to ensure that the excavation does not undermine existing in-ground structures, roads and or other services in their proximity. The need for underpinning or braced excavation can be established according to criteria illustrated on the appended Figure 11. It should be noted that a trench liner box may not be relied on for bracing purposes. For the soil stratigraphy encountered, the bracing system may be designed using a triangular or rectangular stress distribution in accordance with methods outlined in the latest Canadian Foundation



Engineering Manual (CFEM), and summarized on Figures 12 and 13, appended. The system design should also consider load effects from the adjacent embankments, structures and construction equipment.

For the pumping station, the soils below the base of the excavation will comprise dense to very dense granular soil or very stiff to hard clayey soils. Basal heave in dense granular soil is not of concern where adequate ground water control and lateral support is provided. For clayey soils, a factor of safety of at least 2 should be provided, where an undrained shear strength of 100 kPa may be assumed for design.

The ground surface adjacent to a braced excavation is expected to experience some inward movement and vertical settlement. The magnitude of movements adjacent to a braced cut can be limited by proper selection of the lateral earth pressure coefficient provided good quality workmanship and construction practice is employed.

4.4 Storm Water Management and Soil Infiltration

Four storm water management (SWM) ponds are proposed in Blocks 61, 63, 65 and 66. General details of the SWM pond are presented in the following table.

BLOCK	POND NO.	HIGH GROUND WATER LEVEL*	PROPOSED BASE ELEVATION	PROPOSED PERMANENT POOL ELEVATION	TYPICAL SUBGRADE SOIL
61	2	318.0	316.0	317.0	Silt and Sand
63	3	322.5	320.0	321.0	Silt/Clayey Silt
65	1	321.0	320.5	321.5	Silty Sand and Gravel
66	4	322.5	321.4	322.6	Silty Sand/Silt/Sand and Silt Till

* Reference to Blue Metric report dated March 2017.



The high ground water level for the ponds in Blocks 61, 63, and 65 are above or close to the permanent pool levels. This must be considered in the design as it will reduce the storage capacity of the ponds. Also, the bases of the ponds are below the high ground water levels, and this will impact the design and construction. The typical subgrade soils at the pond sites are relatively permeable and seepage of ground water through the bases and sides will occur in unlined ponds.

As the four SWM ponds are to be watertight then the four ponds will have to be lined to prevent exfiltration of storm water and if infiltration of ground water. In this regard, it is noted that the design calls for a combined HDPE/Clay liner system. The earthen liner should comprise clayey silt or silty clay soils having a hydraulic conductivity of no more than 1×10^{-6} cm/s. The native clayey silt and silty clay found in the boreholes are considered relatively impermeable, however the clayey silt and silty clay are generally located several metres below the proposed SWM pond base elevations. Consequently, it will be necessary to use an approved onsite or imported soil to construct the earth liner.

For lined ponds an underdrain system would also be required to maintain stability of the pond base during maintenance. In order to prevent heave of the liner and unstable conditions during maintenance, the underdrain systems must extend under the entire lined area of the pond, and have the capacity to lower the ground water level under the ponds to dewater/depressurize the uplift forces caused by ground water conditions at the sites.

Construction at the SWM ponds will require dewatering to lower the ground water level to at least 0.5 m below the deepest point of the excavation, in order to safely accommodate excavation to the level of the underdrain subgrade. A non-woven geotextile should be placed over the subgrade base and pond side slopes. The underdrain system should comprise a minimum 0.5 m thick layer of free draining material such as Ontario Provincial Standard Specifications (OPSS) Granular B, with 150 mm diameter perforated pipes wrapped in filter fabric, placed at the base of the granular layer, and set at positive grade to sump pits. The granular layer should be covered with non-woven filter fabric to prevent migration of fines into the system.

Perimeter drains are also recommended around the ponds to maintain stability of the pond.



Fill used for earth liner and underdrain construction at the SWM facility, should be placed in lifts with a maximum 300 mm thickness, and compacted to at least 95% standard Proctor maximum dry density (SPMDD). General recommendations for fill subgrade preparation and engineered fill construction are provided in Appendix A.

Berms should be constructed using select soil placed in maximum 300 mm thick lifts compacted to 95% SPMDD. Finished slopes of the ponds should not be steeper than five horizontal to one vertical (5H:1V) for the interior. Slopes should be provided vegetation cover or other means for erosion protection.

Full-time inspection should be carried out by PML personnel to examine and approve backfill, fill placement operations, and to check the compaction by in situ density testing using nuclear gauges.

4.5 Geotechnical Review and Construction Inspection and Testing

When design details are available, is recommended that the design drawings be submitted to PML for geotechnical review for compatibility with site conditions and recommendations of this report.

Foundation construction and earthworks operations should be carried out under the supervision of PML to approve subgrade preparation, backfill materials, placement and compaction procedures, and verify the specified degree of compaction is achieved uniformly throughout fill materials.

The comments and recommendations provided in the report are based on the information revealed in the boreholes. Conditions away from and between boreholes may vary. Geotechnical review during construction should be on going to confirm the subsurface conditions are substantially similar to those encountered in the boreholes, which may otherwise require modification to the original recommendations.



We trust this report has been completed within our terms of reference, and is sufficient for your immediate requirements. If you have any questions or require further information, please do not hesitate to contact our office.

Peto MacCallum Ltd.



William Loghrin, P.Eng.
Project Engineer, Geotechnical Services

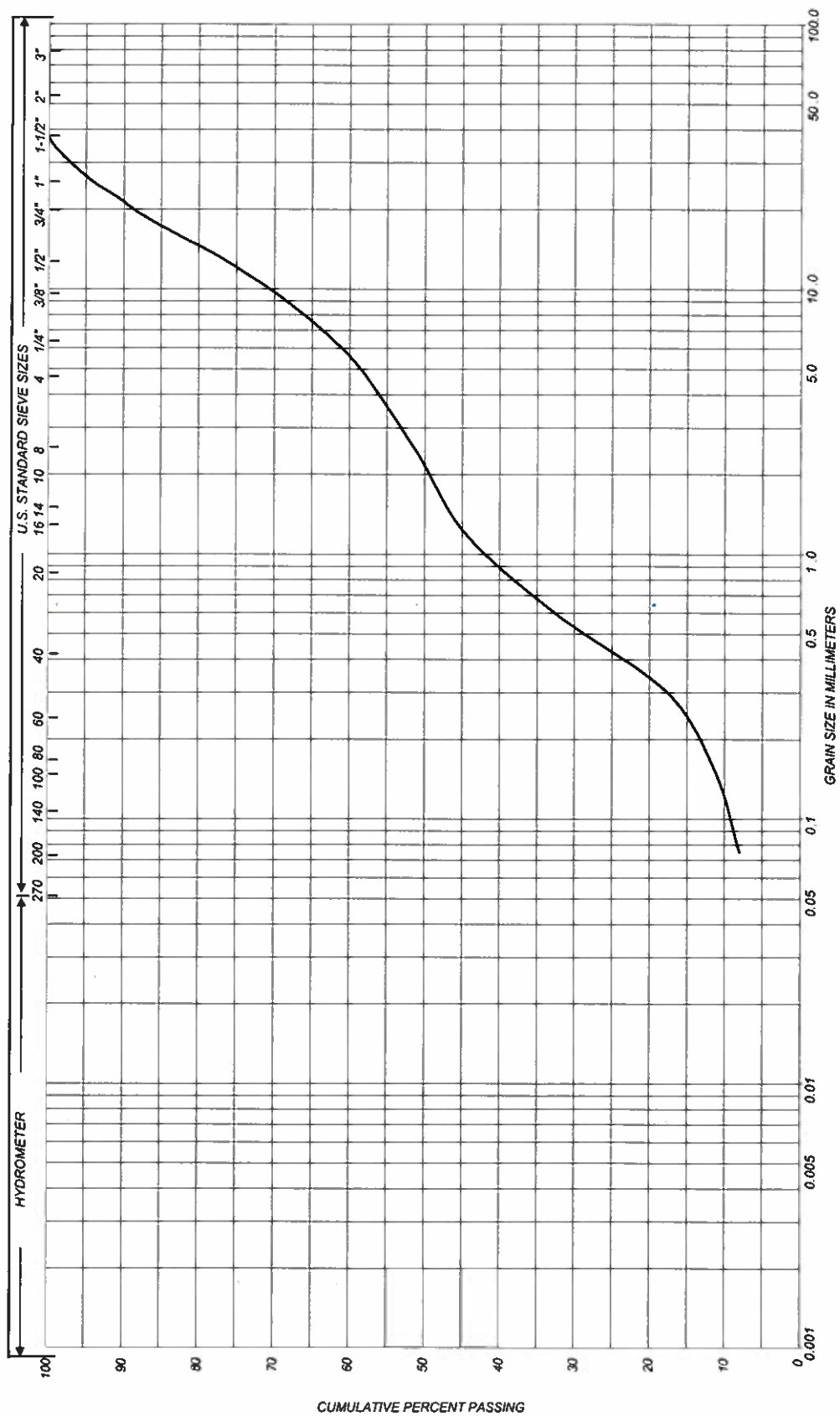


Gerry Mitchell, MEng, P.Eng.
Director
Principal Consultant

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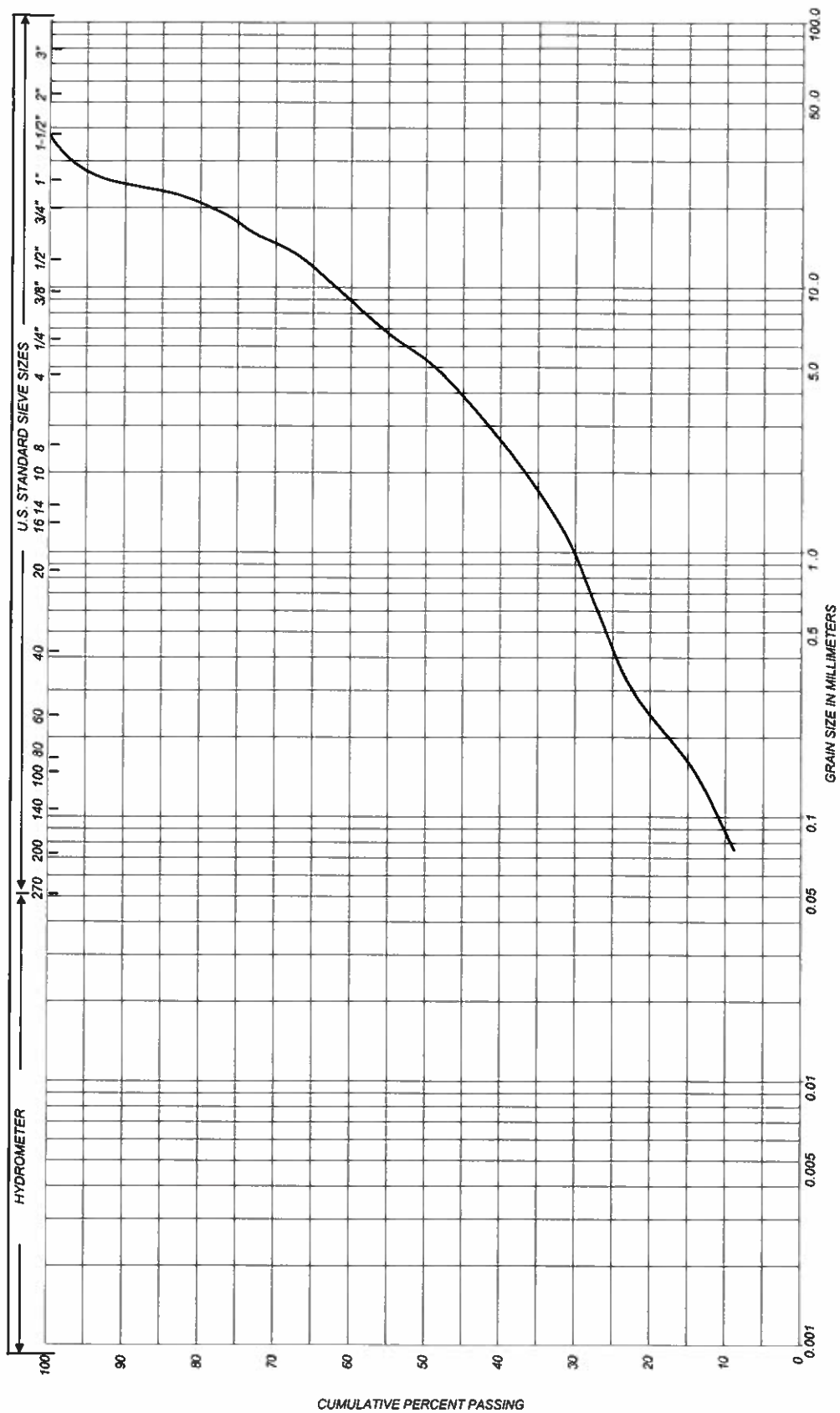
PML REF.	16KF069
FIGURE NO.	1

PARTICLE SIZE DISTRIBUTION CHART

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REMARKS	Borehole 112, Samples SS2 to SS4, Depth 0.8 to 2.7 m SAND AND GRAVEL
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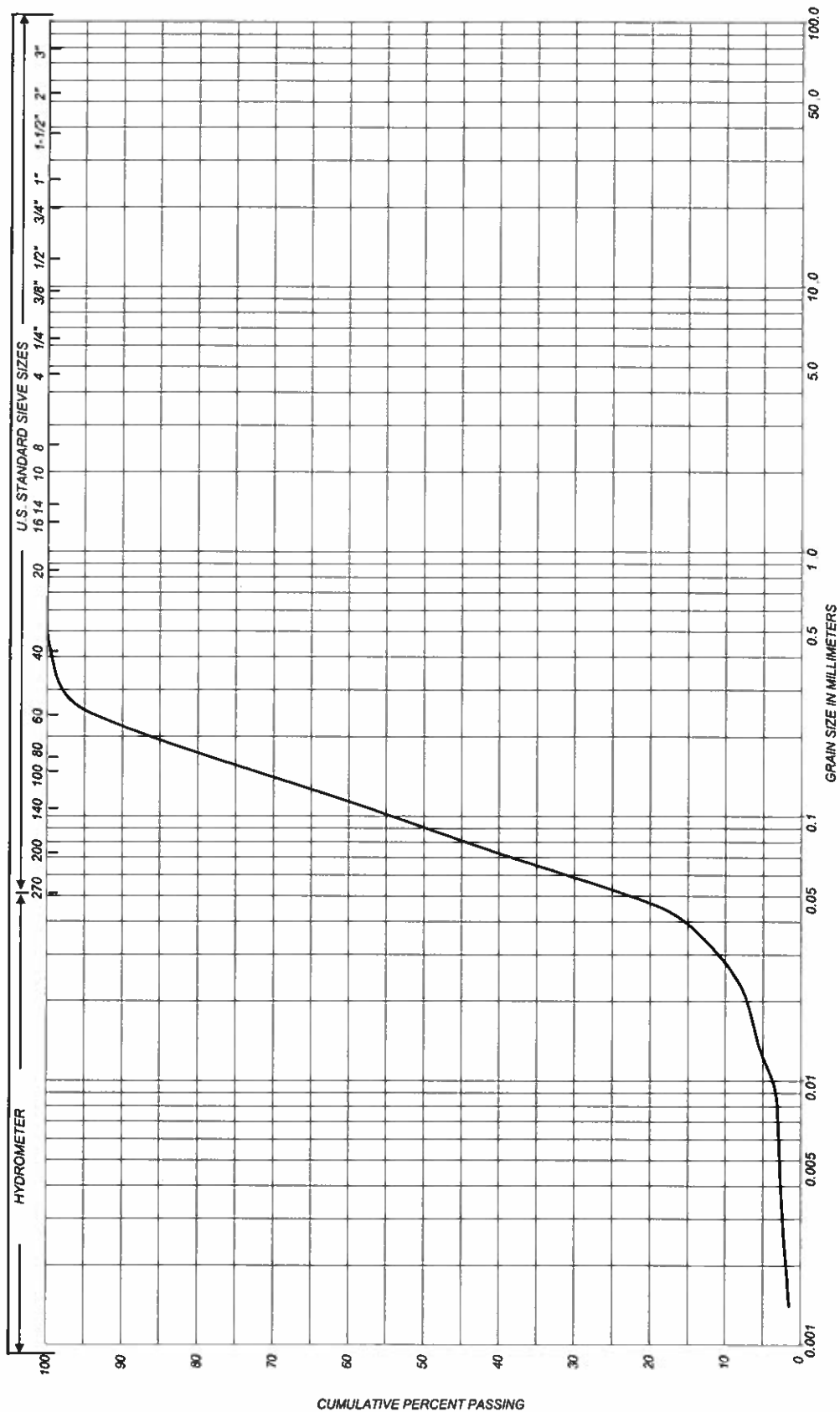
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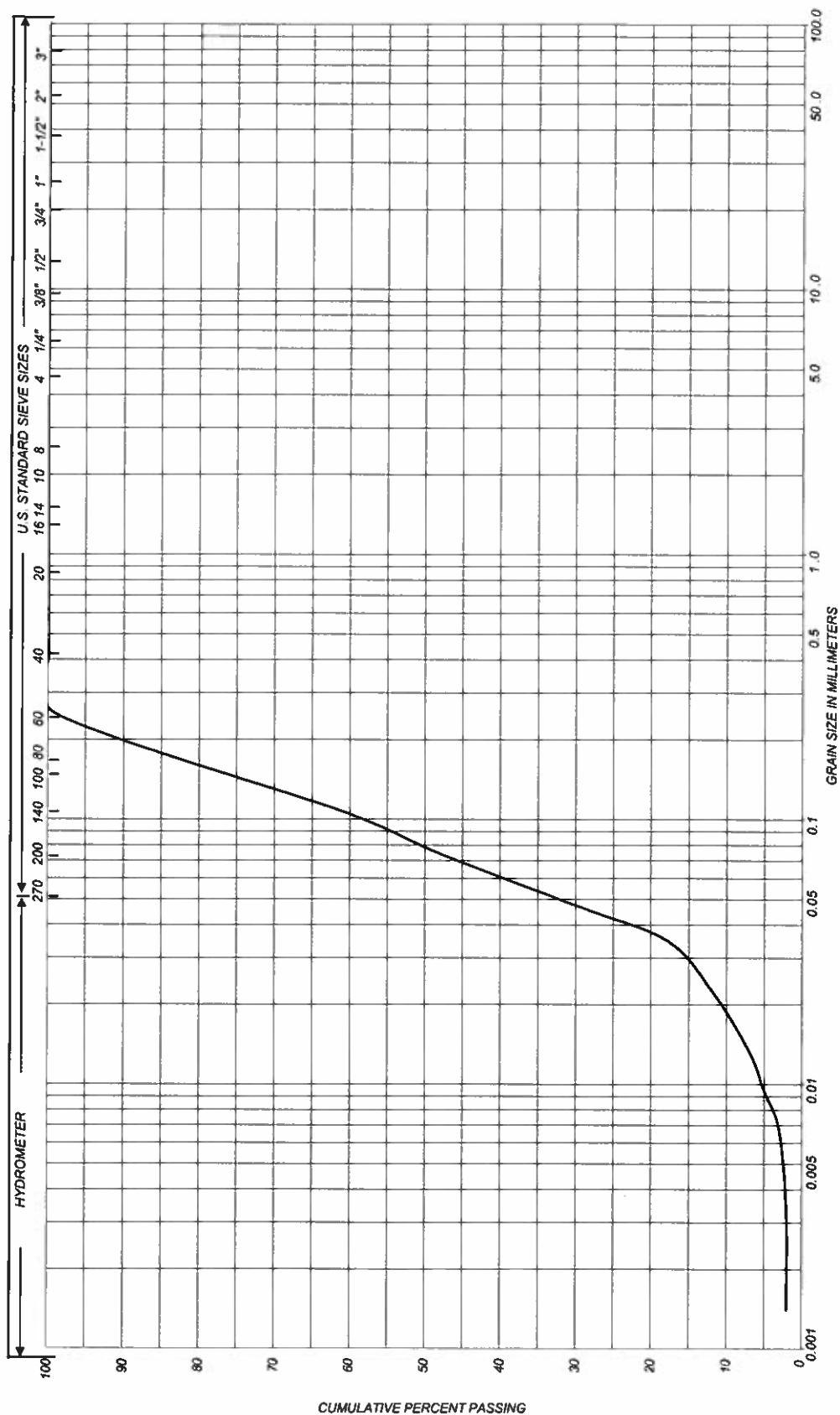


SILT & CLAY										COARSE SILTS		UNIFIED
CLAY	FINE	MEDIUM SILT	COARSE	FINE		MEDIUM		SAND		GRAVEL	COBBLES M.T.	
				VERY FINE	FINE	VERY FINE	FINE	VERY FINE	FINE			
CLAY										GRAVEL		
CLAY										GRAVEL		

REMARKS	Borehole 114, Sample SS7, Depth 6.1 to 6.5 m
	SANDY GRAVEL

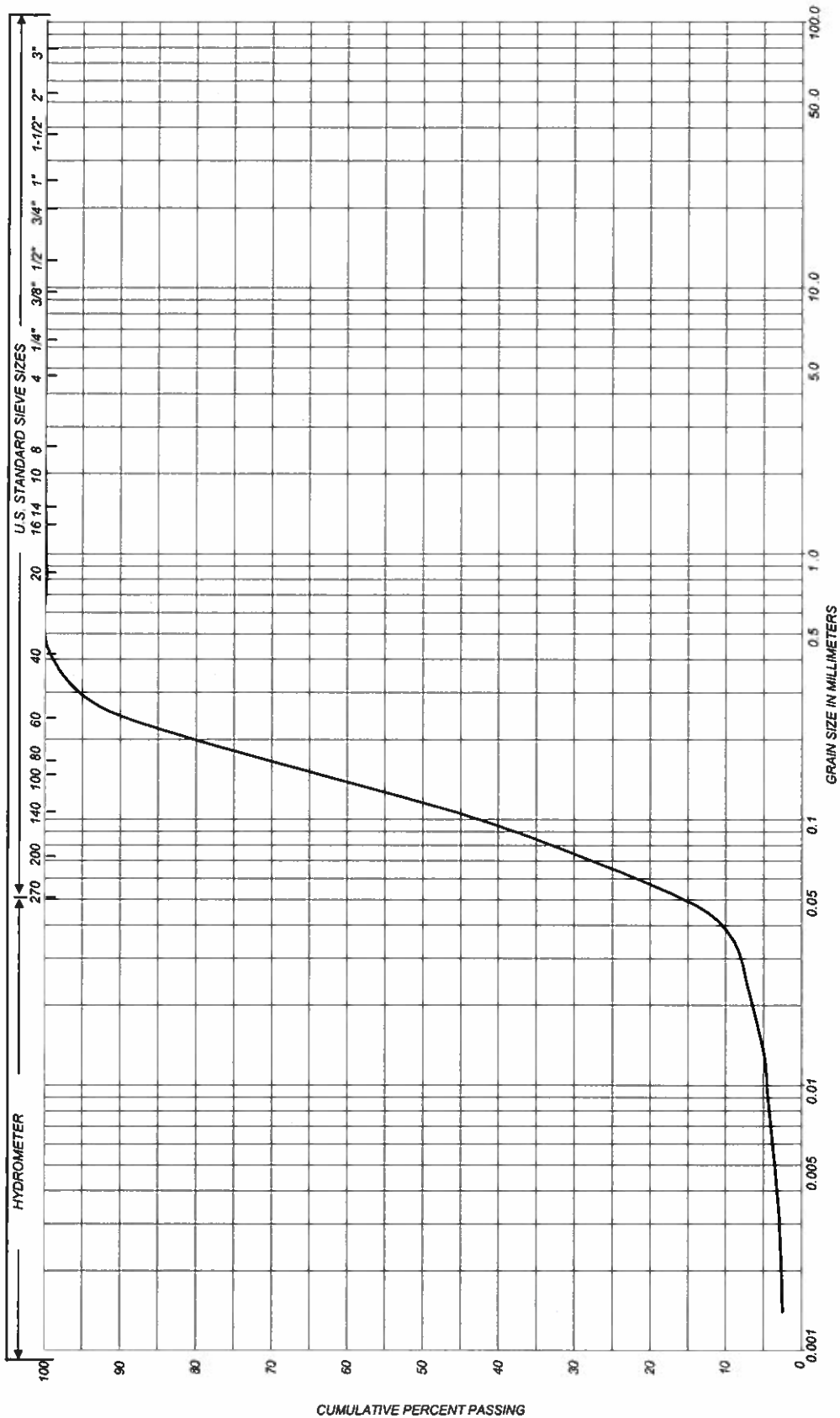
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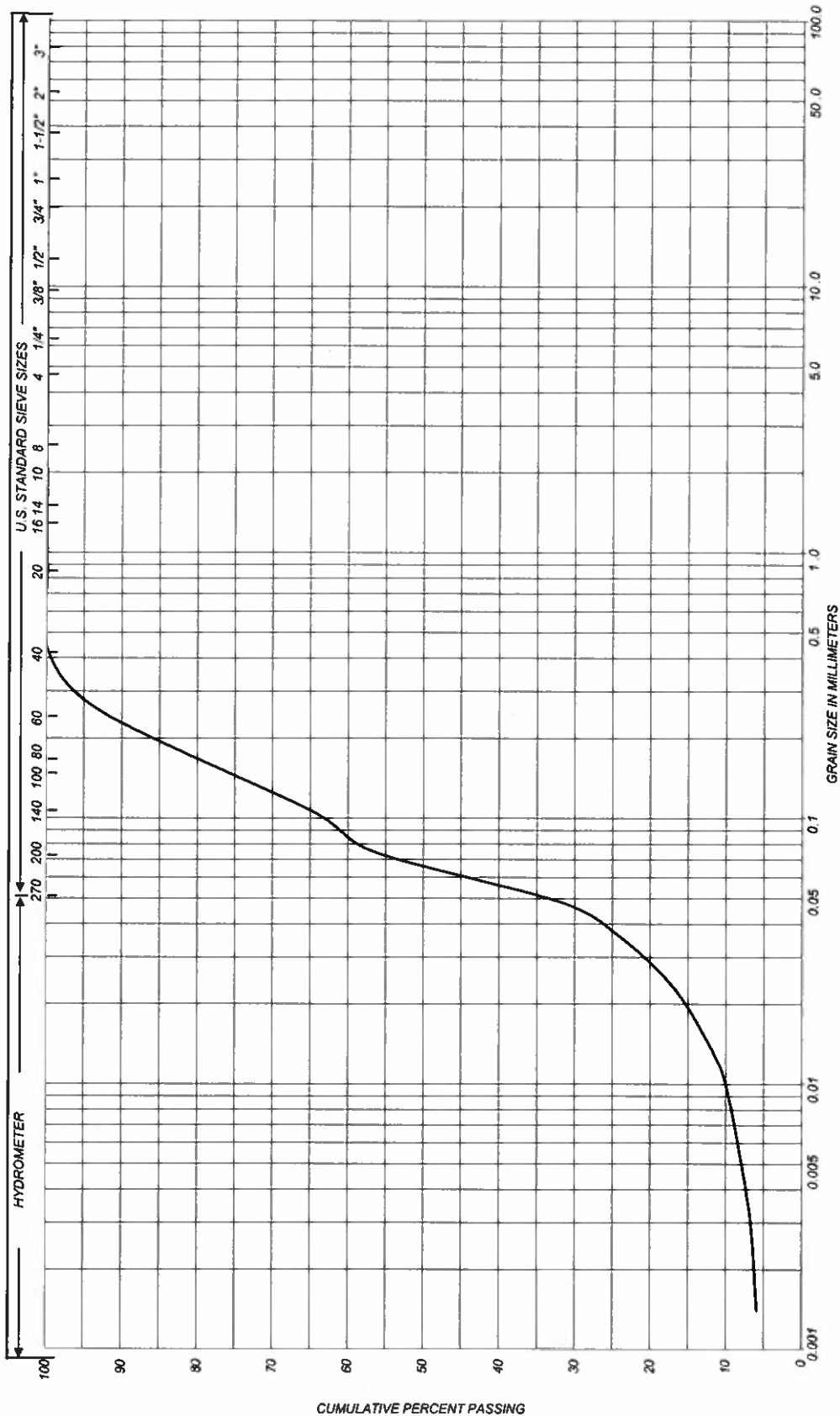
REMARKS	Borehole 103, Sampler SS7, Depth 6.1 to 6.6 m SAND AND SILT
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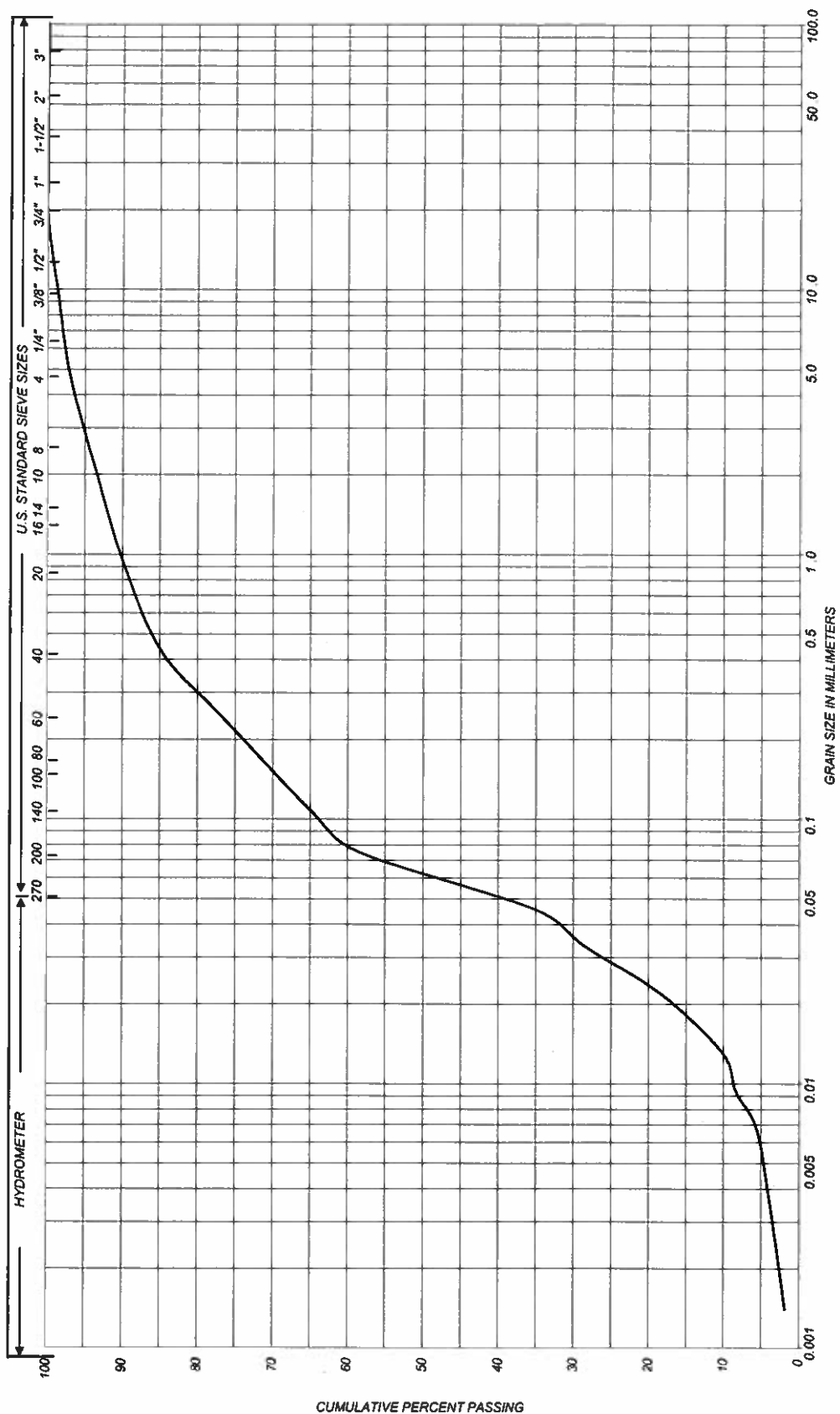


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CLAY		FINE		MEDIUM SILT		COARSE		FINE		MEDIUM SAND		COARSE		GRAVEL		COB. & S&LS		UNIFIED	
CLAY		FINE		MEDIUM SILT		COARSE		FINE		MEDIUM SAND									

PARTICLE SIZE DISTRIBUTION CHART

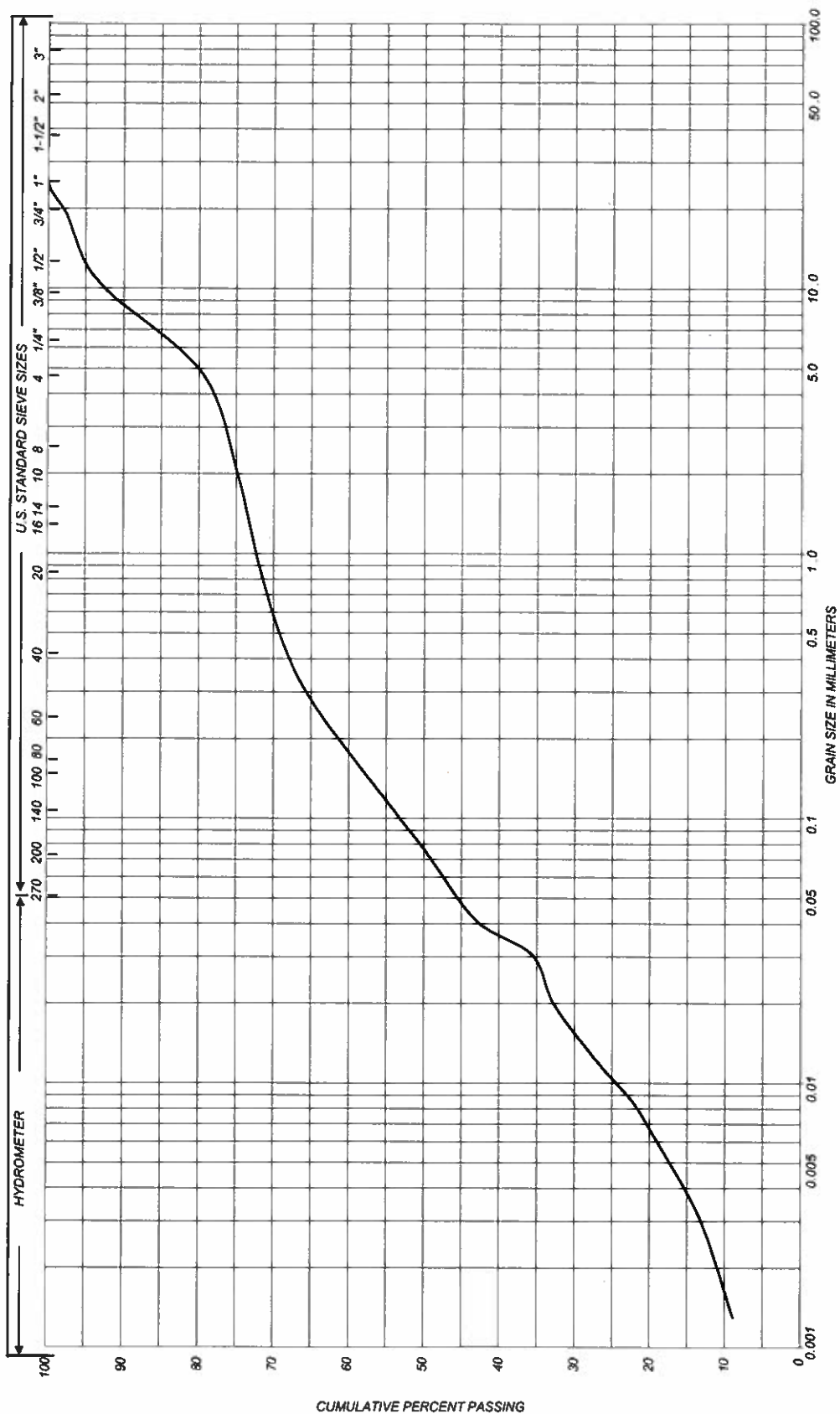


PARTICLE SIZE DISTRIBUTION CHART

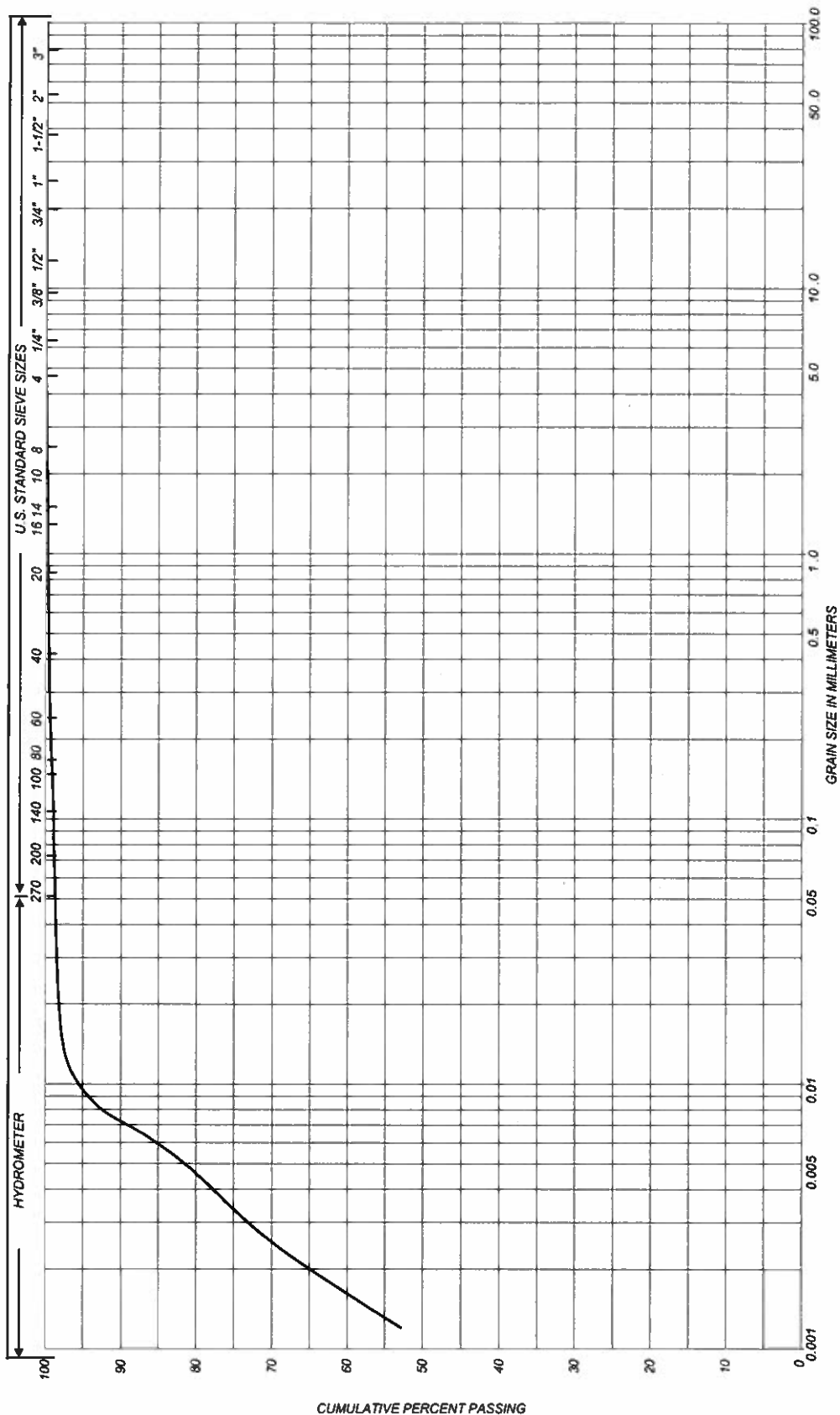
[illegible]

REMARKS	Borehole 110, Sampler SS7, Depth 6.1 to 6.6 m SANDY SILT
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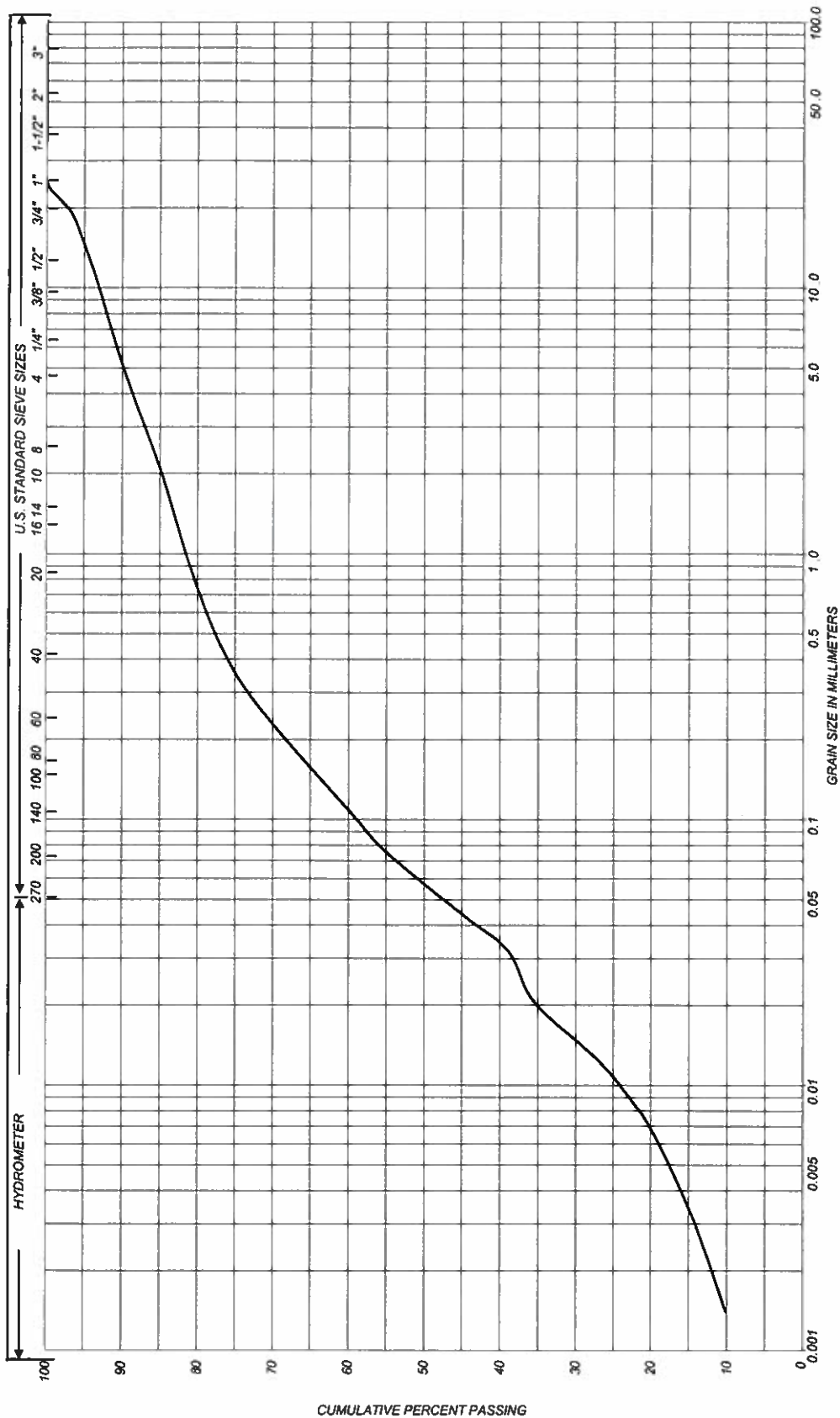
PARTICLE SIZE DISTRIBUTION CHART



PARTICLE SIZE DISTRIBUTION CHART



PARTICLE SIZE DISTRIBUTION CHART



SILT & CLAY		FINE		MEDIUM		COARSE		GRAVEL		COBBLES		UNIFIED	
CLAY	FINE	MEDIUM	SILT	VERY FINE	FINE	MEDIUM	SAND	COARSE	GRAVEL	COBBLES	M.I.T.	U.S. BUREAU	

REMARKS Borehole 104, Sample SS12, Depth 18.3 to 18.4 m
SANDY SILT TILL

NOTES

1. The need to underpin existing footings/utilities is dependent upon soil type, proximity of the existing facility to the face of the excavation, loads imposed on the foundation and permissible movements.

ZONE A:
Foundations of relatively heavy and/or settlement sensitive structures/utilities located in Zone A generally require underpinning.

ZONE B:
Foundations of structures located within Zone B generally do not require underpinning. Consideration should be given to underpinning of settlement sensitive utilities or heavy foundation units located in this zone.

ZONE C:
Utilities and foundations located within Zone C do not normally require underpinning.

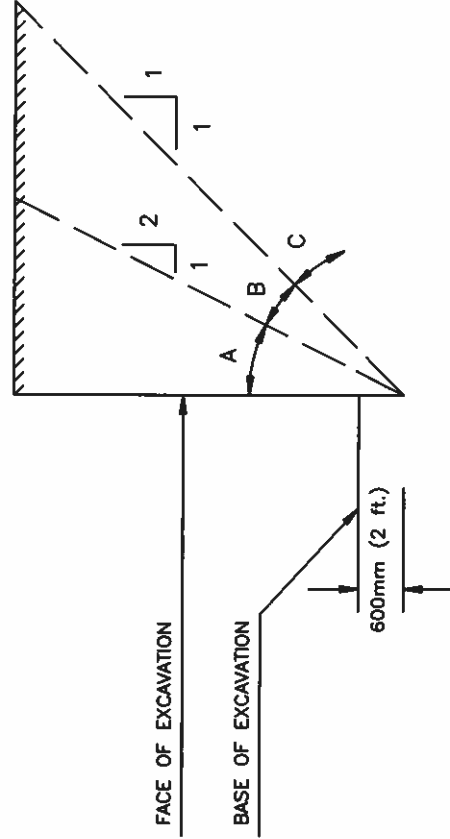
Underpinning of foundations located in Zones A and B should extend at least into Zone C.

2. As an alternative to underpinning, it may be possible to control movement of existing utilities and foundations by supporting the face of the excavation with bracing/tiebacks or a rigid (caisson) wall. Horizontal and vertical earth pressures imposed on the excavation wall by non-underpinned foundations must be considered in the design of the support system.

3. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction to monitor any movement which may occur.

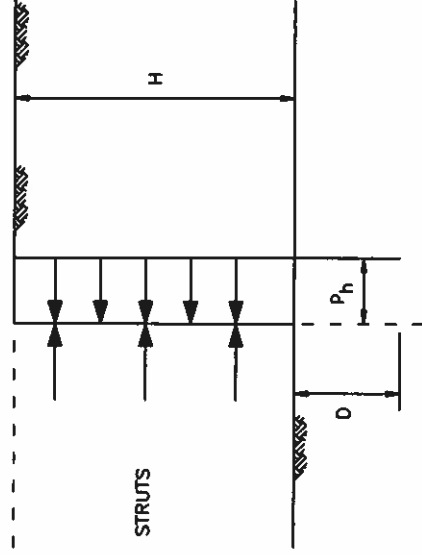
4. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.

5. This sheet is to be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.



NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.
2. Stability of base of excavation must be confirmed when bracing system design, excavation geometry and surcharge loads are established. If groundwater table is well above base of excavation and/or artesian conditions exist, local lowering of the groundwater level will be necessary to prevent bottom heave/piping of the base of the excavation.
3. Earth pressure diagram is applicable to maximum depth of cut of 12m (40 ft.).
4. Structural components of bracing system should be confirmed adequate for each level of excavation.
5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
6. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system are not included in earth pressure diagram.
7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
8. If settlement sensitive structures are located near the excavation, special measures should be undertaken to control settlements. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction.
9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, earth pressure diagram must be reviewed, and remedial works may be required.
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
11. Bracing system should be regularly examined for signs of distress.
12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

EARTH PRESSURE DIAGRAM

P_h = design lateral earth pressure
 $= 0.65 \text{ } \gamma H$

K = lateral earth pressure coefficient

γ = unit weight of soil

H = depth of excavation

D = depth of embedment of soldier piles (if used).

RECOMMENDED DESIGN PARAMETERS

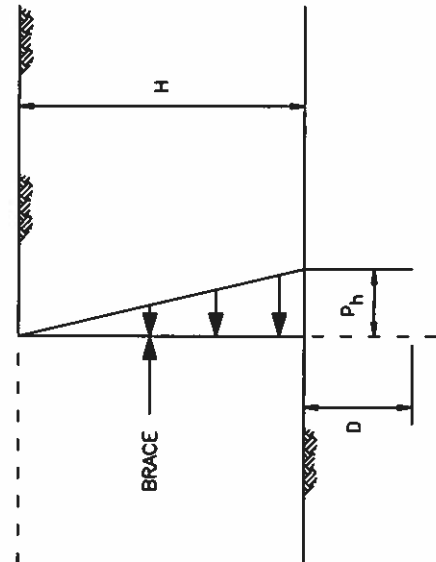
$\gamma = 21.0 \text{ kN/m}^3$

$K = 0.36$ (movement of retained soil acceptable)

0.50 (movement of adjacent structures/facilities unacceptable)

NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.
2. Stability of base of excavation must be confirmed when bracing system design, excavation geometry and surcharge loads are established. If groundwater table is well above base of excavation and/or artesian conditions exist, local lowering of the groundwater level will be necessary to prevent bottom heave/piping of the base of the excavation.
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4. Structural components of bracing system should be confirmed adequate for each level of excavation.
5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
6. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system are not included in earth pressure diagram.
7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
8. If settlement sensitive structures are located near the excavation, special measures should be undertaken to control settlements. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction.
9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, the earth pressure diagram must be reviewed, and remedial works may be required.
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
11. Bracing system should be regularly examined for signs of distress.
12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

EARTH PRESSURE DIAGRAM

P_h = design lateral earth pressure
 $= K\gamma H$

K = lateral earth pressure coefficient

γ = unit weight of soil

H = depth of excavation

D = depth of embedment of soldier piles (if used).

RECOMMENDED DESIGN PARAMETERS

γ = 21.0 kN/m³

K = 0.36 (movement of retained soil acceptable)

0.50 (movement of adjacent structures/facilities unacceptable)

LIST OF ABBREVIATIONS



PENETRATION RESISTANCE

Standard Penetration Resistance N: - The number of blows required to advance a standard split spoon sampler 0.3 m into the subsoil. - Driven by means of a 63.5 kg hammer falling freely a distance of 0.76 m.

Dynamic Penetration Resistance: The number of blows required to advance a 51 mm, 60 degree cone, fitted to the end of drill rods, 0.3 m into the subsoil. The driving energy being 475 J per blow.

DESCRIPTION OF SOIL

The consistency of cohesive soils and the relative density or denseness of cohesionless soils are described in the following terms:

<u>CONSISTENCY</u>	<u>N (blows/0.3 m)</u>	<u>c (kPa)</u>	<u>DENSENESS</u>	<u>N (blows/0.3 m)</u>
Very Soft	0 - 2	0 - 12	Very Loose	0 - 4
Soft	2 - 4	12 - 25	Loose	4 - 10
Firm	4 - 8	25 - 50	Compact	10 - 30
Stiff	8 - 15	50 - 100	Dense	30 - 50
Very Stiff	15 - 30	100 - 200	Very Dense	> 50
Hard	> 30	> 200		
WTPL	Wetter Than Plastic Limit			
APL	About Plastic Limit			
DTPL	Drier Than Plastic Limit			

TYPE OF SAMPLE

SS	Split Spoon	TW	Thinwall Open
WS	Washed Sample	TP	Thinwall Piston
SB	Scraper Bucket Sample	OS	Oesterberg Sample
AS	Auger Sample	FS	Foil Sample
CS	Chunk Sample	RC	Rock Core
ST	Slotted Tube Sample	USS	Undisturbed Shear Strength
PH	Sample Advanced Hydraulically	RSS	Remoulded Shear Strength
PM	Sample Advanced Manually		

SOIL TESTS

Qu	Unconfined Compression	LV	Laboratory Vane
Q	Undrained Triaxial	FV	Field Vane
Qcu	Consolidated Undrained Triaxial	C	Consolidation
Qd	Drained Triaxial		

LOG OF BOREHOLE NO. 101

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

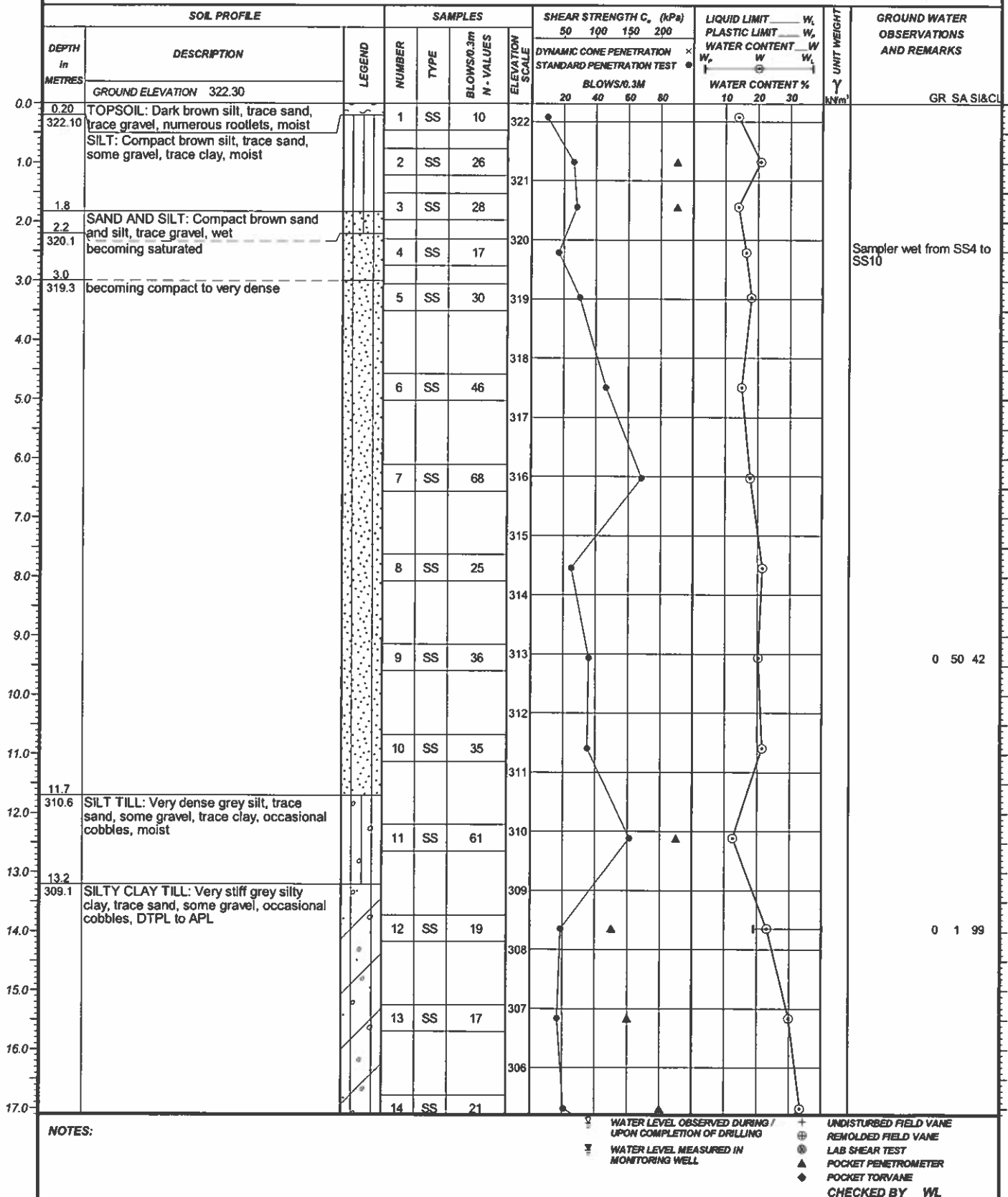
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2016 12 06

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary



LOG OF BOREHOLE NO. 101

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2016 12 06

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary

SOIL PROFILE			SAMPLES			SHEAR STRENGTH C_u (kPa)		LIQUID LIMIT W_L		UNIT WEIGHT γ kN/m ³	GROUND WATER OBSERVATIONS AND REMARKS	
DEPTH In METRES	DESCRIPTION	LEGEND	NUMBER	TYPE	BLOWS/0.3m N - VALUES	ELEVATION SCALE	50 100 150 200		PLASTIC LIMIT W_p			
							DYNAMIC CONE PENETRATION		WATER CONTENT W			
							STANDARD PENETRATION TEST		WATER CONTENT %			
							BLOWS/0.3M		10 20 30			
307.3	CONTINUED FROM PREVIOUS PAGE						20 40 60 80	10 20 30			GR SA SI&CL	
17.8						305						
18.0	304.5 CLAYEY SILT TILL: Hard grey clayey silt, trace sand, some gravel, occasional cobbles, DTPL		15	SS	50/75mm	304						
19.0						303						
20.0			16	SS	50/125mm	302						
21.0	21.0 numerous cobbles					301						
22.0						300						
23.0			17	SS	50/100mm	299						
24.0						298						
25.0						297						
26.0	26.0 occasional wet sand and gravel layers		18	SS	50/125mm	296					Sand heave to 19.8 m inside auger prior to SS18 flushed to clear	
27.0						295						
28.0						294						
29.0	29.4 BOREHOLE TERMINATED AT 29.4 m		19	SS	50/125mm	293						
292.9												
30.0												
31.0												
32.0												
33.0												
34.0												

NOTES:

WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING

WATER LEVEL MEASURED IN MONITORING WELL

UNDISTURBED FIELD VANE

REMOLDED FIELD VANE

LAB SHEAR TEST

POCKET PENETROMETER

POCKET TORVANE

CHECKED BY WL

NOTES:

- | | |
|---|--|
| <p>WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING</p> <p>WATER LEVEL MEASURED IN MONITORING WELL</p> | <p>UNDISTURBED FIELD VANE</p> <p>REMOLDED FIELD VANE</p> <p>LAB SHEAR TEST</p> <p>POCKET PENETROMETER</p> <p>POCKET TORVANE</p> <p>CHECKED BY WL</p> |
|---|--|

LOG OF BOREHOLE NO. 102

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

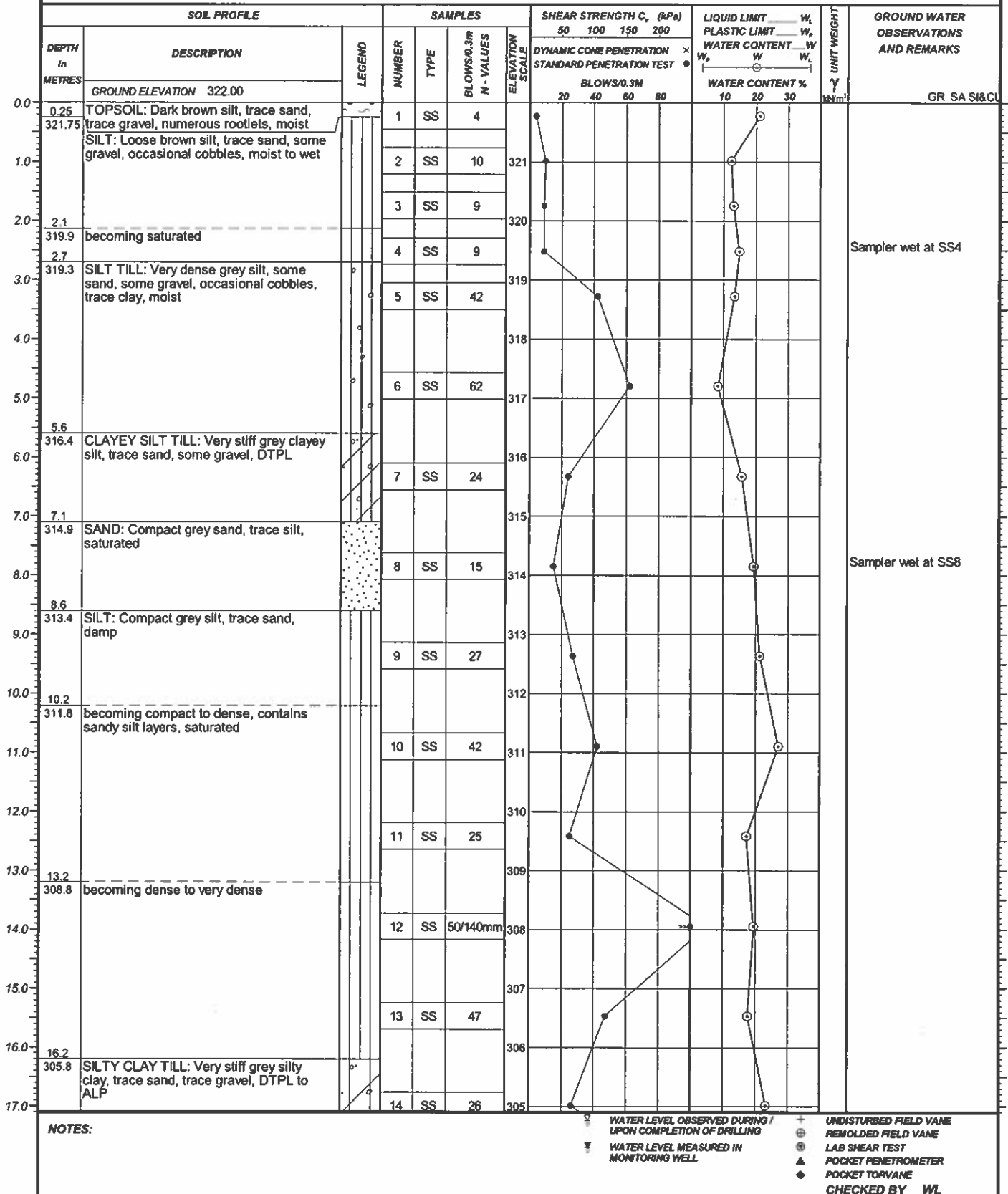
BORING DATE: 2017 01 03

BORING METHOD Continuous Flight Hollow Stem Augers

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary



LOG OF BOREHOLE NO. 102

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2017 01 03

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary

SOIL PROFILE			SAMPLES			SHEAR STRENGTH C_u (kPa)		LIQUID LIMIT W_L		UNIT WEIGHT γ (kN/m ³)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH In METRES	DESCRIPTION	LEGEND	NUMBER	TYPE	BLOWS/0.3m N - VALUES	ELEVATION SCALE	50 100 150 200	DYNAMIC CONE PENETRATION STANDARD PENETRATION TEST	PLASTIC LIMIT W_P		
307.0	CONTINUED FROM PREVIOUS PAGE										GR SA SI&CL
17.8											
18.0	304.2 SILT TILL: Very dense grey silt, some sand, some gravel, trace clay, occasional cobbles, moist		15	SS	50/130mm	304					
19.0						303					
20.0			16	SS	50/100mm	302					
20.3	BOREHOLE TERMINATED AT 20.3 m										Upon completion of augering Free water inside auger at 18.4 m
301.7											
21.0											
22.0											
23.0											
24.0											
25.0											
26.0											
27.0											
28.0											
29.0											
30.0											
31.0											
32.0											
33.0											
34.0											

NOTES:

- | | | |
|---|---|------------------------|
| WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING | + | UNDISTURBED FIELD VANE |
| WATER LEVEL MEASURED IN MONITORING WELL | ⊕ | REMOLDED FIELD VANE |
| | ⊗ | LAB SHEAR TEST |
| | ▲ | POCKET PENETROMETER |
| | ◆ | POCKET TORVANE |
| | | CHECKED BY WL |

LOG OF BOREHOLE NO. 103

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

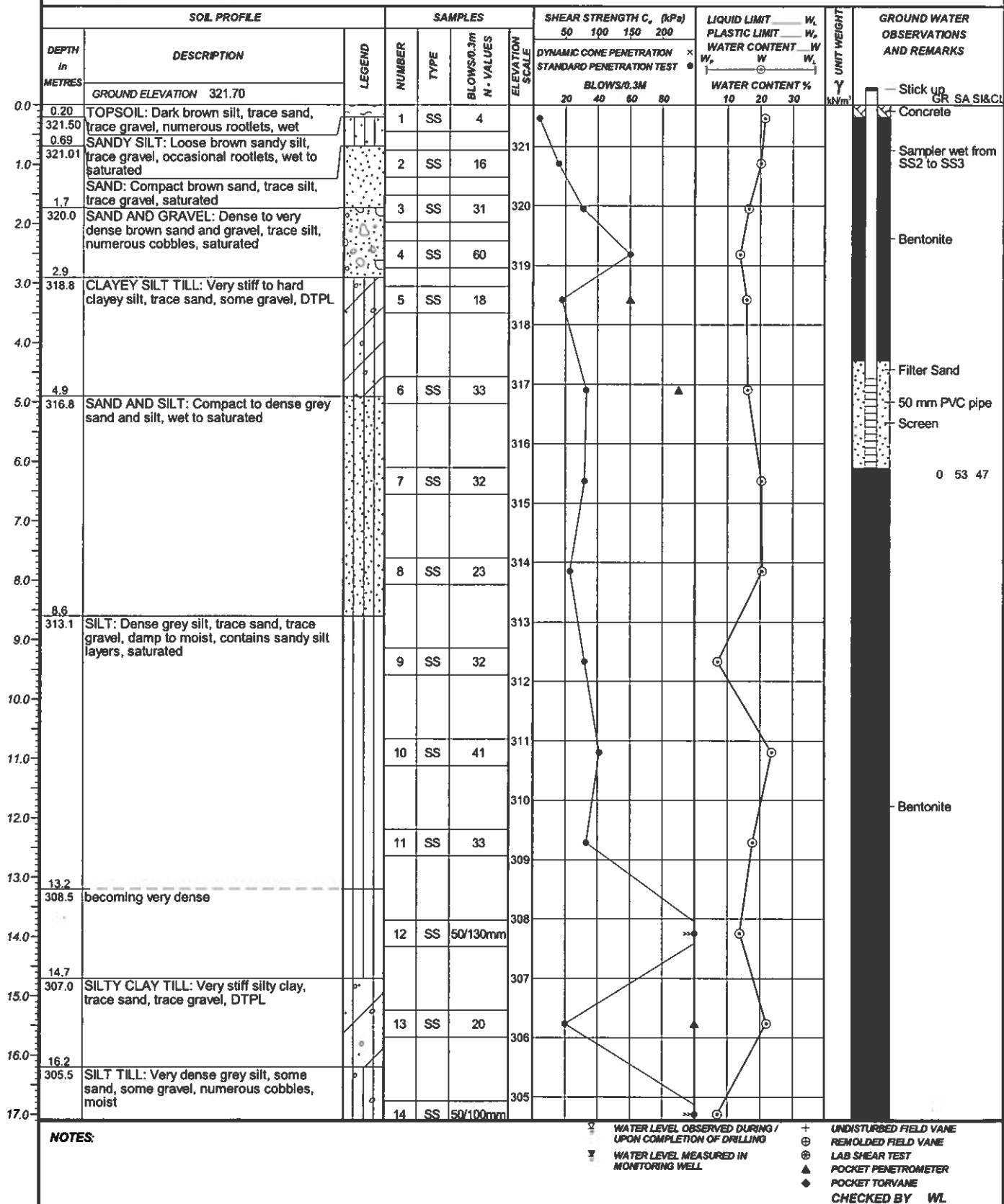
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2017 01 04

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary



LOG OF BOREHOLE NO. 103

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2017 01 04

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary

SOIL PROFILE			SAMPLES			SHEAR STRENGTH C_u (kPa)				LIQUID LIMIT W_L				UNIT WEIGHT γ kN/m ³	GROUND WATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	NUMBER	TYPE	BLOWS/0.3m N - VALUES	ELEVATION SCALE	DYNAMIC CONE PENETRATION STANDARD PENETRATION TEST				PLASTIC LIMIT W_P					
							BLOWS/0.3M				WATER CONTENT %					
							50	100	150	200	W _u	W _p	W _L			W _P
306.7	CONTINUED FROM PREVIOUS PAGE						20	40	60	80	10	20	30		GR SA SI&CL	
17.5	BOREHOLE TERMINATED AT 17.5 m NO FURTHER PROGRESS DUE TO AUGER REFUSAL ON PROBABLE BOULDER		15	SC	50/100mm										WATER LEVEL READING January 18, 2017: 1.5 m	
18.0																
19.0																
20.0																
21.0																
22.0																
23.0																
24.0																
25.0																
26.0																
27.0																
28.0																
29.0																
30.0																
31.0																
32.0																
33.0																
34.0																

NOTES:

WATER LEVEL OBSERVED DURING /
UPON COMPLETION OF DRILLING

WATER LEVEL MEASURED IN
MONITORING WELL

UNDISTURBED FIELD VANE

REMOLED FIELD VANE

LAB SHEAR TEST

POCKET PENETROMETER

POCKET TORVANE

CHECKED BY WL

LOG OF BOREHOLE NO. 104

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

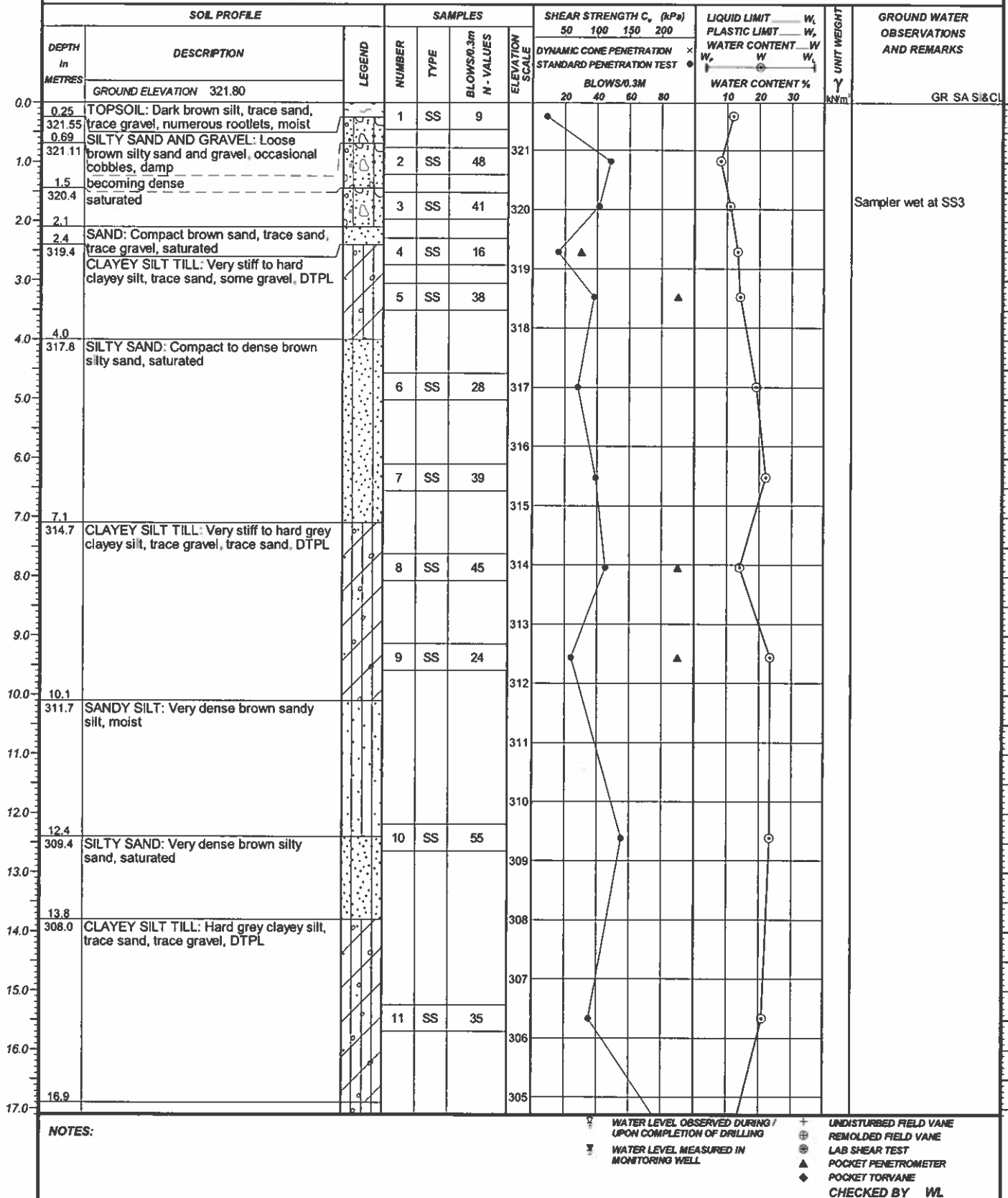
BORING DATE: 2016 12 07

PML REF.: 16KF069

ENGINEER W. Loghrin

BORING METHOD Continuous Flight Hollow Stem Augers

TECHNICIAN H. Shinwary



LOG OF BOREHOLE NO. 104

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

BORING DATE: 2016 12 07

PML REF.: 16KF069

ENGINEER W. Loghrin

BORING METHOD Continuous Flight Hollow Stem Augers

TECHNICIAN H. Shinwary

SOIL PROFILE		LEGEND		SAMPLES		SHEAR STRENGTH C_u (kPa)		LIQUID LIMIT W_L		PLASTIC LIMIT W_P		WATER CONTENT W		GROUND WATER	
DEPTH In METRES	DESCRIPTION			NUMBER	TYPE	BLOWS/0.3m N-VALUES	ELEVATION SCAL							OBSERVATIONS AND REMARKS	
306.8	CONTINUED FROM PREVIOUS PAGE														
304.9	SILT TILL: Very dense grey silt, trace sand, some gravel, occasional cobbles, moist (continued)														
18.0															
19.0				12	SS	50/100mm	304							Non-plastic	10 35 55
20.0							303								
21.0							302								
22.0							301								
23.0				13	SS	50/75mm	300								
24.0							299								
25.0							298								
26.0				14	SS	50/100mm	297								
27.0							296								
28.0							295								
26.5	SANDY GRAVEL: Very dense brown/grey sandy gravel, numerous cobbles, saturated						294								
295.3				15	SS	50/50mm									
28.4	BOREHOLE TERMINATED AT 28.4 m NO FURTHER PROGRESS DUE TO AUGER REFUSAL IN BOULDERS													Upon completion of augering Free water inside auger at 22.9 m.	
293.4															
29.0															
30.0															
31.0															
32.0															
33.0															
34.0															

NOTES:

- WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING
- WATER LEVEL MEASURED IN MONITORING WELL
- UNDISTURBED FIELD VANE
- REMOLDED FIELD VANE
- LAB SHEAR TEST
- POCKET PENETROMETER
- POCKET TORVANE
- CHECKED BY WL

LOG OF BOREHOLE NO. 105

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

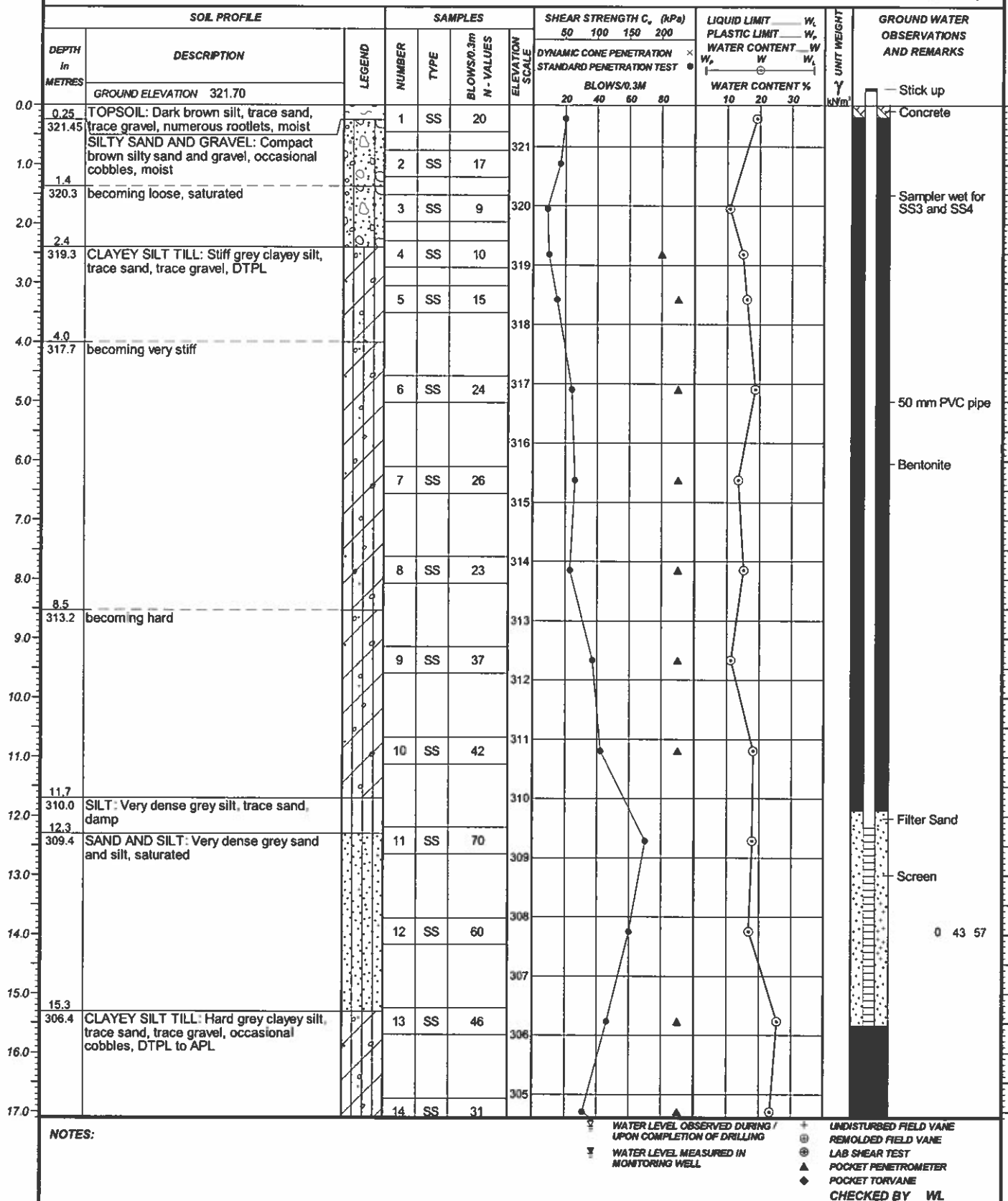
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2016 12 05

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary



LOG OF BOREHOLE NO. 105

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2016 12 05

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary

SOIL PROFILE			SAMPLES				SHEAR STRENGTH C_u (kPa)				LIQUID LIMIT W_L				UNIT WEIGHT γ kNm	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH In METRES	DESCRIPTION	LEGEND	NUMBER	TYPE	BLOWS/0.3m N - VALUES	ELEVATION SCALE	50 100 150 200				PLASTIC LIMIT W_p					
							DYNAMIC CONE PENETRATION $\times$				WATER CONTENT W					
							STANDARD PENETRATION TEST $\bullet$				W_p W W_L					
306.7	CONTINUED FROM PREVIOUS PAGE						BLOWS/0.3M				WATER CONTENT %					
							20	40	60	80	10	20	30			
17.8						304									Bentonite	
18.0	303.9 SILT TILL: Very dense grey silt, trace sand, some gravel, trace clay, occasional cobbles, moist		15	SS	50/150mm	303										
19.0						302										
20.0	20.3		16	SS	50/125mm											
20.3	301.4 BOREHOLE TERMINATED AT 20.3 m														WATER LEVEL READINGS: January 18, 2017: 1.2 m	
21.0																
22.0																
23.0																
24.0																
25.0																
26.0																
27.0																
28.0																
29.0																
30.0																
31.0																
32.0																
33.0																
34.0																

NOTES:

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WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING

WATER LEVEL MEASURED IN MONITORING WELL

UNDISTURBED FIELD VANE

REMOLDED FIELD VANE

LAB SHEAR TEST

POCKET PENETROMETER

POCKET TORVANE

CHECKED BY

WL

NOTES:

- WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING
- WATER LEVEL MEASURED IN MONITORING WELL
- UNDISTURBED FIELD VANE
- REMOLDED FIELD VANE
- LAB SHEAR TEST
- POCKET PENETROMETER
- POCKET TORVANE
- CHECKED BY WL

LOG OF BOREHOLE NO. 106

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

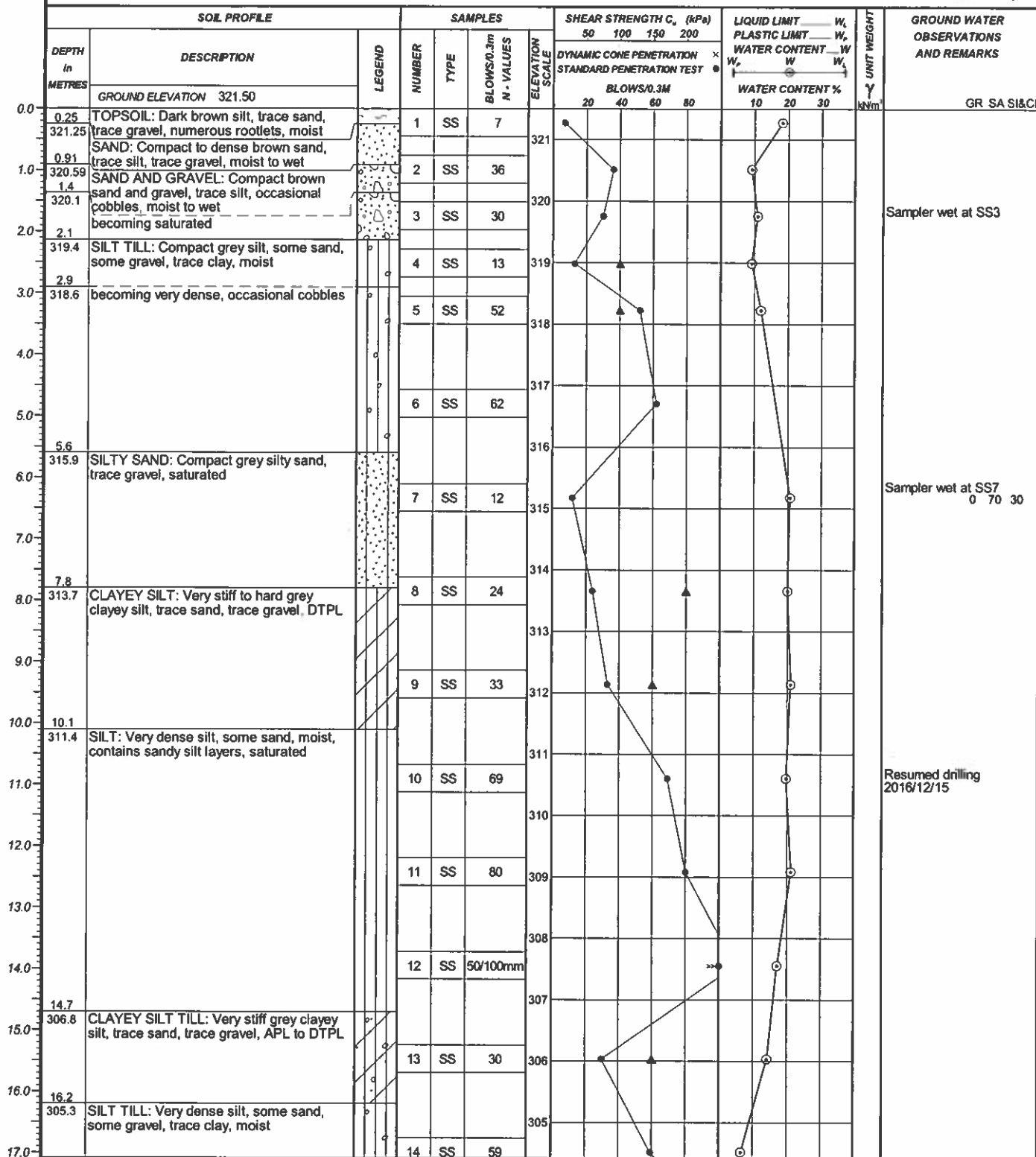
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2016 12 14

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary



NOTES:

- WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING
- WATER LEVEL MEASURED IN MONITORING WELL
- UNDISTURBED FIELD VANE
- REMOLDED FIELD VANE
- LAB SHEAR TEST
- POCKET PENETROMETER
- POCKET TORVANE
- CHECKED BY WL

LOG OF BOREHOLE NO. 106

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2016 12 14

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary

SOIL PROFILE		LEGEND	SAMPLES			SHEAR STRENGTH C_u (kPa)		LIQUID LIMIT W_L		UNIT WEIGHT	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION		NUMBER	TYPE	BLOWS/0.3m N - VALUES	ELEVATION SCALE	DYNAMIC CONE PENETRATION STANDARD PENETRATION TEST	PLASTIC LIMIT W_p	WATER CONTENT %		
306.5	CONTINUED FROM PREVIOUS PAGE										
17.8	SILT TILL: Very dense silt, some sand, some gravel, trace clay, moist (continued)					304					
303.7	occasional cobbles		15	SS	50/130mm	303					
20.3			16	SS	50/100mm	302					
301.2	BOREHOLE TERMINATED AT 20.3 m										Upon completion of augering Free water at 19.5 m inside augers
21.0											
22.0											
23.0											
24.0											
25.0											
26.0											
27.0											
28.0											
29.0											
30.0											
31.0											
32.0											
33.0											
34.0											

NOTES:

- | | | |
|---|---|------------------------|
| WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING | + | UNDISTURBED FIELD VANE |
| WATER LEVEL MEASURED IN MONITORING WELL | ⊗ | REMOLDED FIELD VANE |
| | ⊙ | LAB SHEAR TEST |
| | ▲ | POCKET PENETROMETER |
| | ◆ | POCKET TORVANE |
| | | CHECKED BY WL |

LOG OF BOREHOLE NO. 110

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

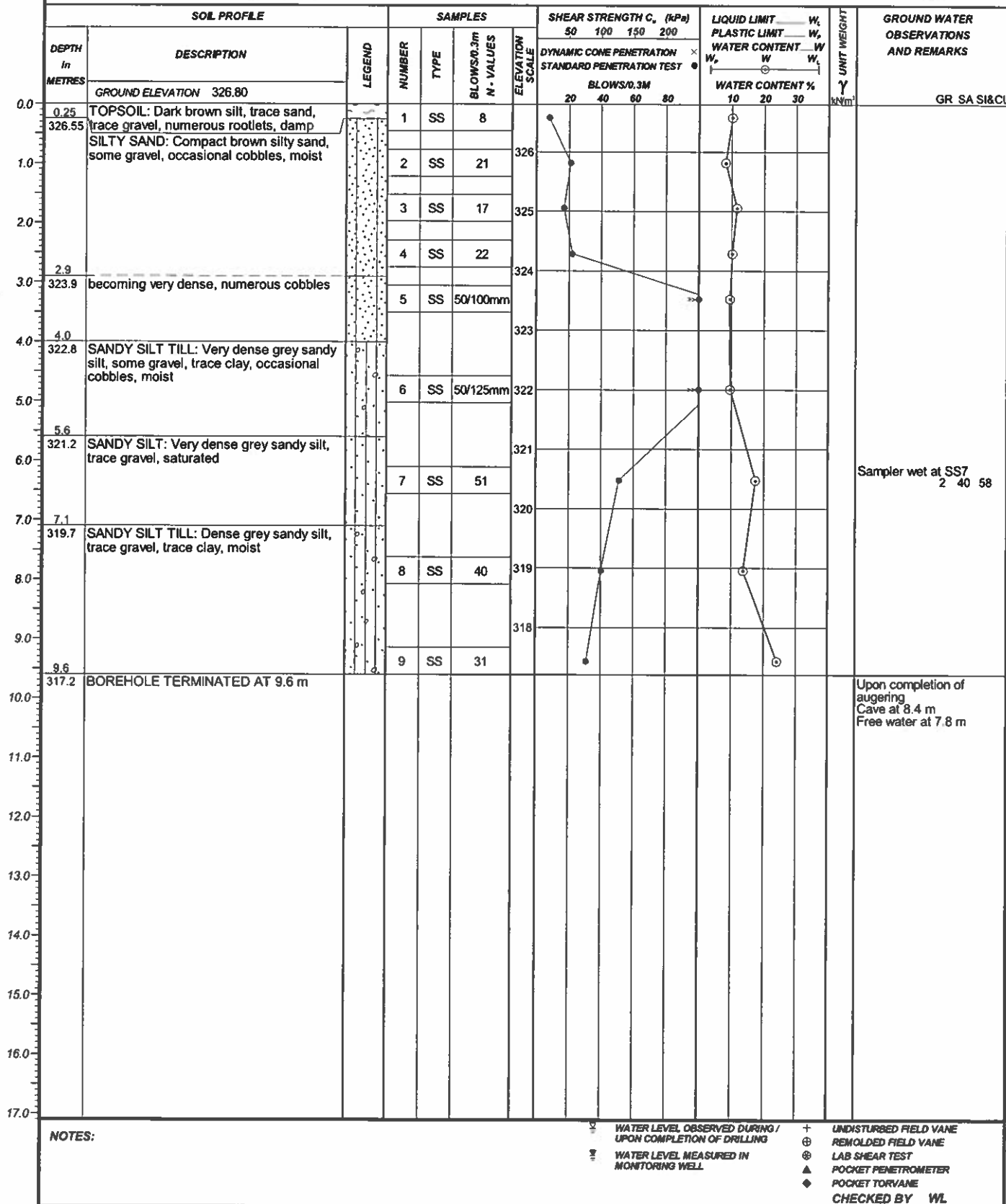
BORING DATE: 2016 12 13

PML REF.: 16KF069

ENGINEER W. Lohrin

BORING METHOD Continuous Flight Hollow Stem Augers

TECHNICIAN H. Shinwary



LOG OF BOREHOLE NO. 111

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

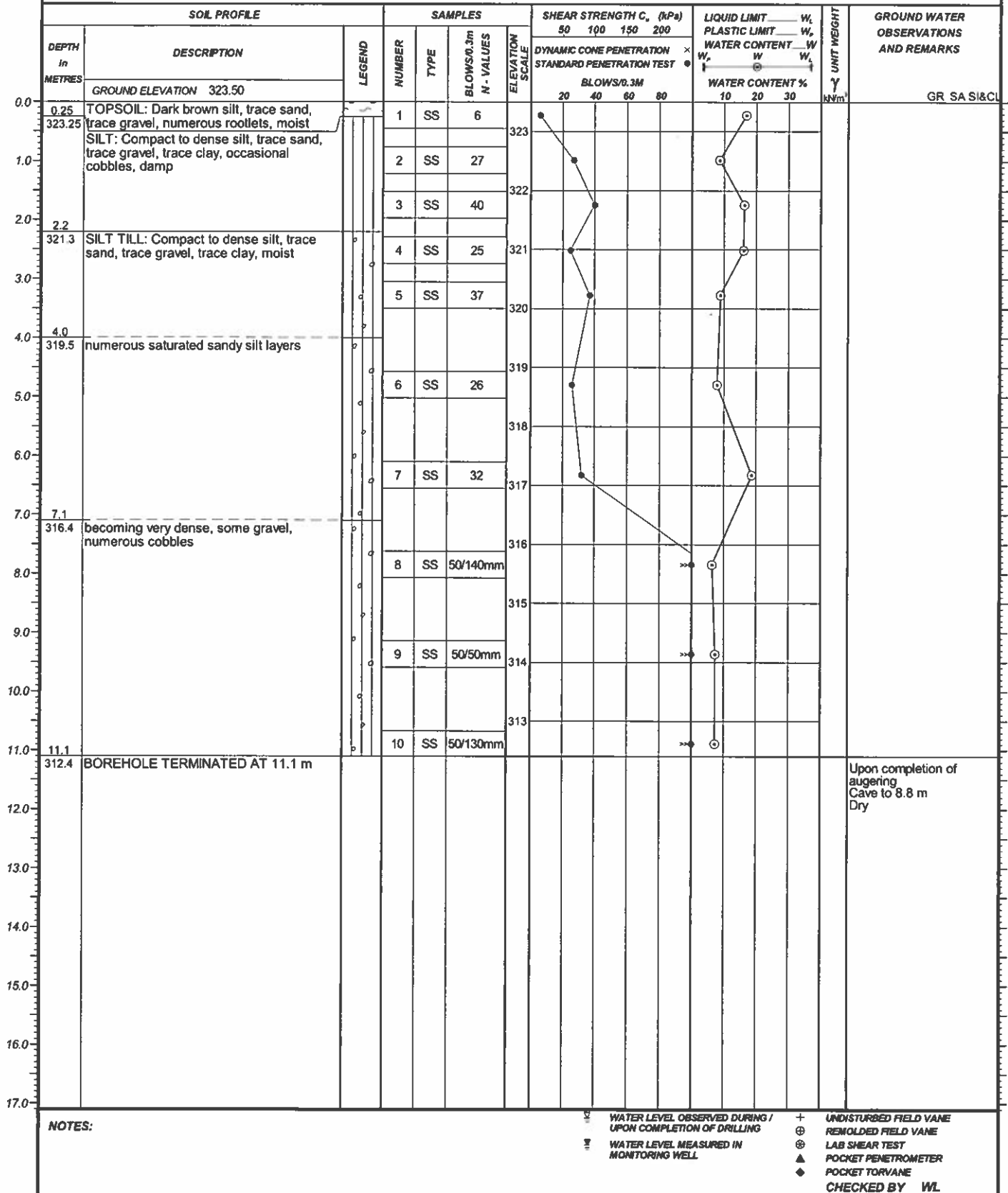
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2016 12 12

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary



LOG OF BOREHOLE NO. 112

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2016 12 12

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary

SOIL PROFILE			SAMPLES			SHEAR STRENGTH C_u (kPa)		LIQUID LIMIT W_L		UNIT WEIGHT γ (kN/m ³)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH In METRES	DESCRIPTION	LEGEND	NUMBER	TYPE	BLOWS/0.3m N - VALUES	ELEVATION SCALE	50 100 150 200	DYNAMIC CONE PENETRATION STANDARD PENETRATION TEST	WATER CONTENT W (%)		
	GROUND ELEVATION 322.10										GR SA SI&CL
0.0	0.25 TOPSOIL: Dark brown silt, trace sand, trace gravel, numerous rootlets, moist		1	SS	9	322					
0.25	321.85 SAND AND GRAVEL: Compact sand and gravel, trace silt, numerous cobbles, damp		2	SS	27	321					
1.0	1.4 becoming dense, moist to wet		3	SS	42	320					45 48 7
2.0	2.2 becoming compact, saturated		4	SS	14	319					Sampler wet at SS4
2.7	319.4 SANDY SILT TILL: Compact grey sandy silt, trace gravel, trace clay, saturated		5	SS	12	318					
3.0						317					
4.0						316					
5.0						315					
5.6	316.5 SILT: Dense grey silt, trace sand, trace gravel, damp with occasional silty sand layers, saturated		7	SS	37	314					
6.0						313					
7.0											
8.0											
8.6	313.5 SILTY SAND: Very dense grey silty sand, saturated		8	SS	48						
9.0											
9.6	312.5 BOREHOLE TERMINATED AT 9.6 m		9	SS	53						Sampler wet at SS9
10.0											Upon completion of augering Free water at 5.3 m inside augers, after removal of augers. Cave to 4.1 m Free water at 1.2 m
11.0											
12.0											
13.0											
14.0											
15.0											
16.0											
17.0											

NOTES:

- WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING
- WATER LEVEL MEASURED IN MONITORING WELL
- UNDISTURBED FIELD VANE
- REMOLED FIELD VANE
- LAB SHEAR TEST
- POCKET PENETROMETER
- POCKET TORVANE
- CHECKED BY WL

LOG OF BOREHOLE NO. 113

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

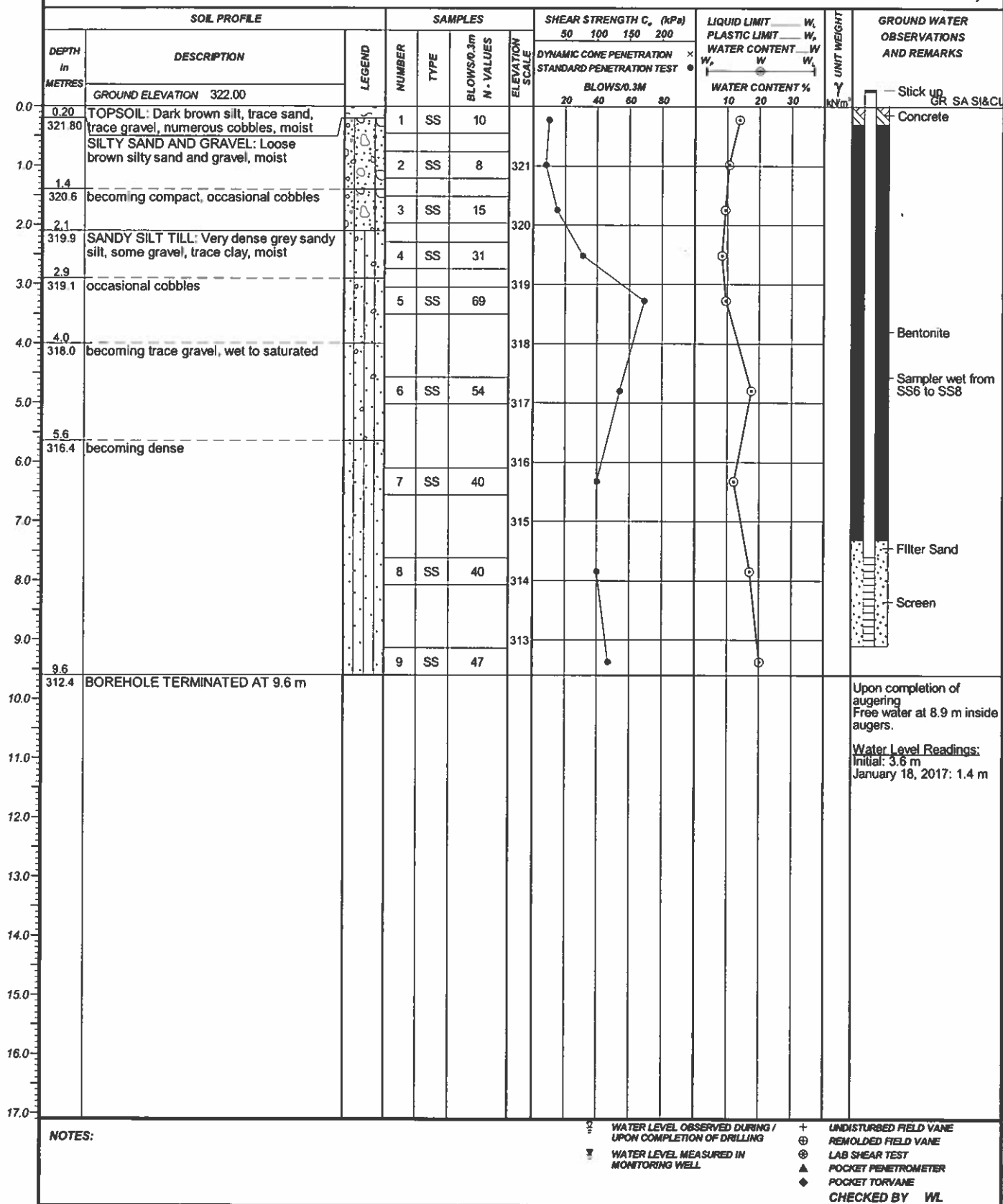
BORING DATE: 2016 12 14

BORING METHOD Continuous Flight Hollow Stem Augers

PML REF.: 16KF069

ENGINEER W. Lohrin

TECHNICIAN H. Shinwary



LOG OF BOREHOLE NO. 114

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

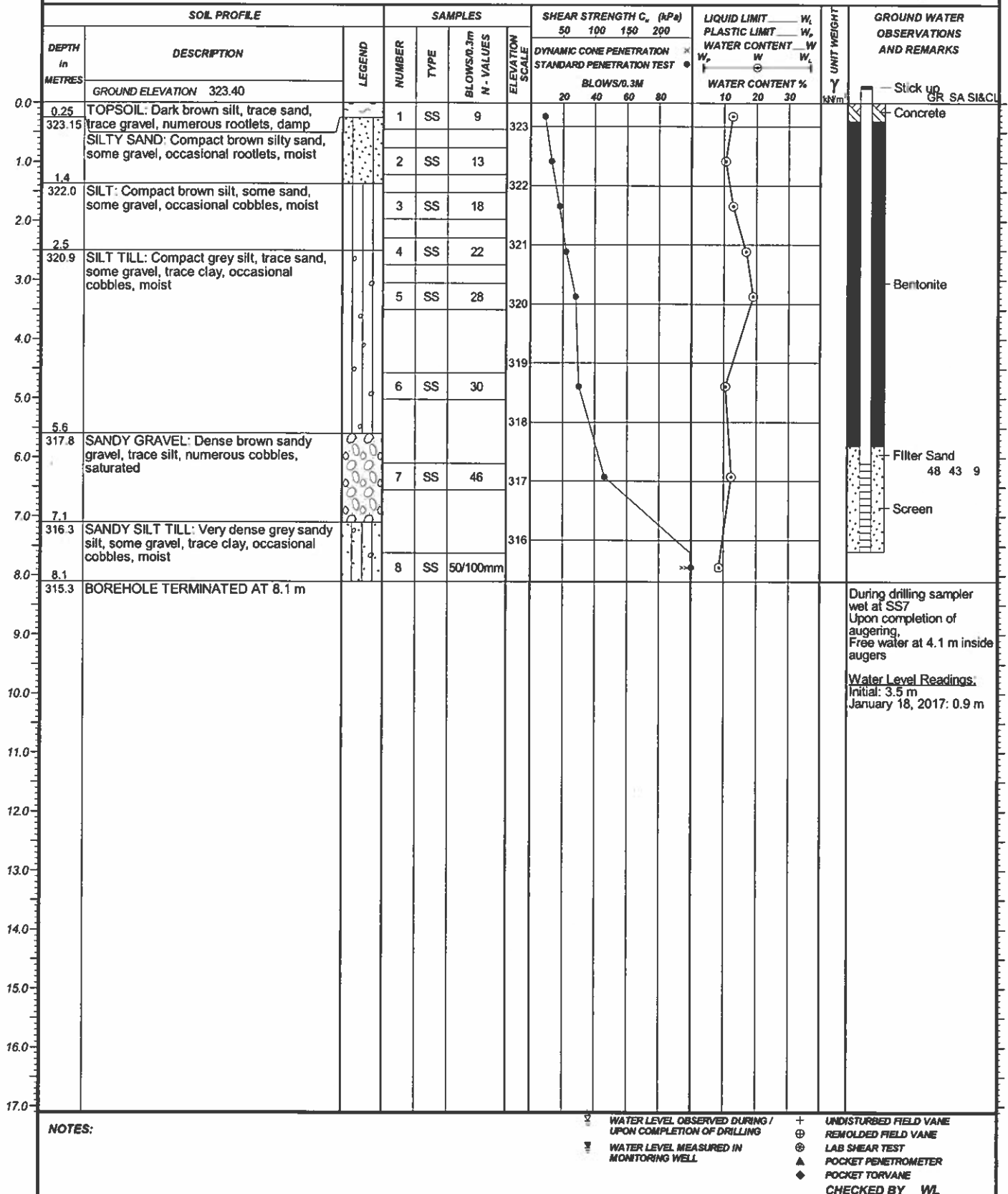
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2016 12 13

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary



LOG OF BOREHOLE NO. 115

PROJECT Supplementary Geotechnical Investigation - Hopewell Crossing Subdivision Development

LOCATION Township of Woolwich, Ontario

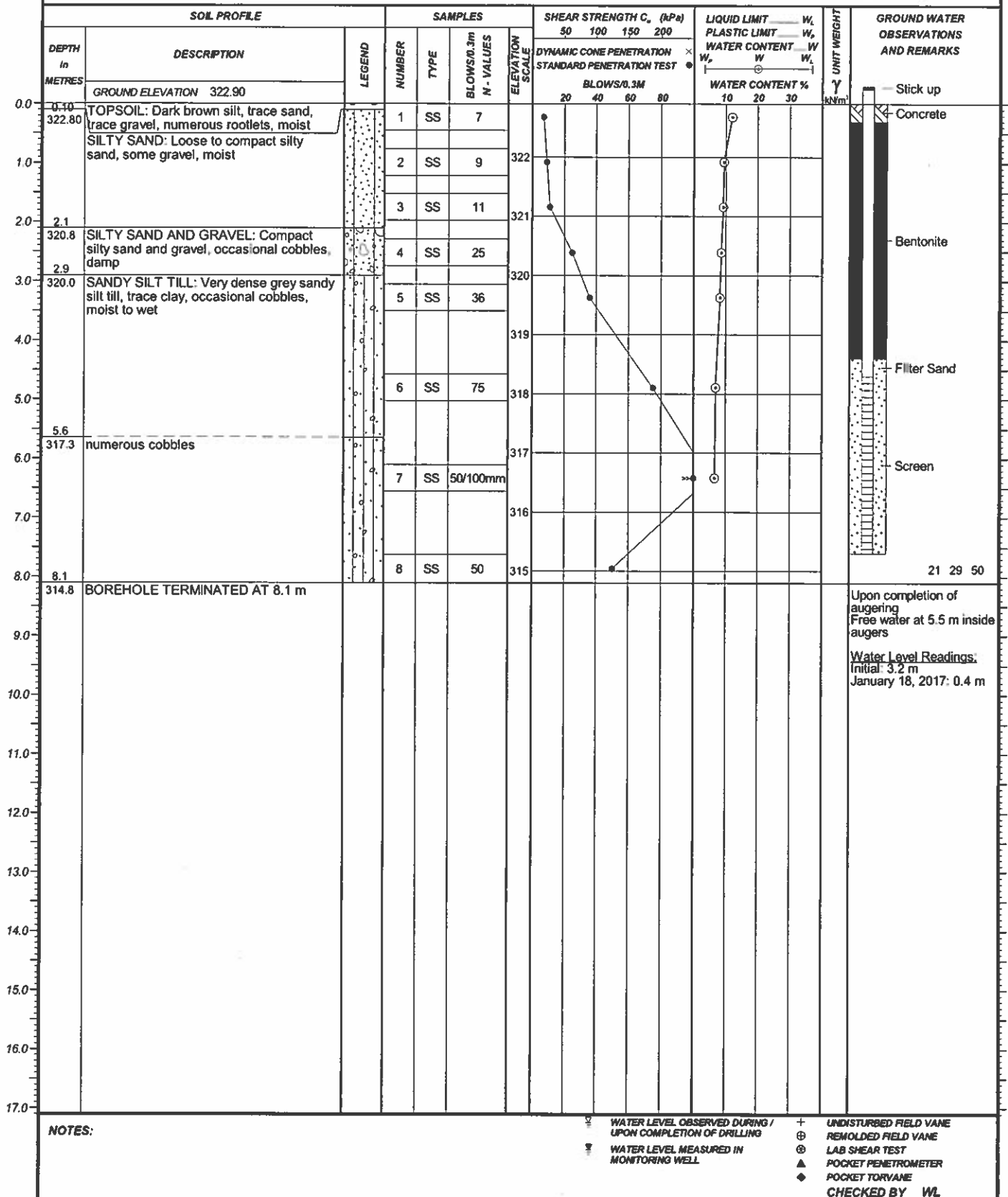
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE: 2016 12 16

PML REF.: 16KF069

ENGINEER W. Loghrin

TECHNICIAN H. Shinwary



LOG OF TEST PIT NO. 1

PROJECT Supplementary Geotechnical Investigaoin
LOCATION Hopewell Crossing Subdivision Development
DATE 2016 11 14

PML REF.: 16KF069
ENGINEER K. Hanes
TECHNICIAN K. Hanes

SOIL PROFILE			SAMPLE NUMBER	SHEAR STRENGTH C_u (kPa)				LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W W_P W W_L WATER CONTENT %			UNIT WEIGHT γ kN/m ³	GROUND WATER OBSERVATIONS AND REMARKS GRAIN SIZE DISTRIBUTION (%) GR SA SI&CL
DEPTH in METRES	DESCRIPTION	LEGEND										
	GROUND ELEVATION											
0.20	TOPSOIL: Dark brown silt topsoil, rootlets, moist											
1.00	SILT: Light brown silt, some sand, moist											
2.2	SAND AND GRAVEL: Light brown to grey sand and gravel, moist											
3.3	becoming saturated											
6.5	SILT TILL: Grey silt, some fine sand, trace to some gravel, moist											
6.5	TEST PIT TERMINATED AT 6.5 m											
7.0												Ground water seepage and cave between 2.2 and 3.3 m
7.0												Upon completion of excavation: Wet cave with ground water at 4.5 m

NOTES:

- ⊗ LAB SHEAR TEST
- ▲ POCKET PENETROMETER
- ◆ POCKET TORVANE
- ⊥ WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING

CHECKED BY KH

LOG OF TEST PIT NO. 2

PROJECT Supplementary Geotechnical Investigaoin
LOCATION Hopewell Crossing Subdivision Development
DATE 2016 11 14

PML REF.: 16KF069
ENGINEER K. Hanes
TECHNICIAN K. Hanes

SOIL PROFILE		LEGEND	SAMPLE NUMBER	SHEAR STRENGTH C_u (kPa)	LIQUID LIMIT W_L	PLASTIC LIMIT W_P	WATER CONTENT W	UNIT WEIGHT γ	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION								
	GROUND ELEVATION								
0.30	TOPSOIL: Dark brown sandy silt topsoil, rootlets, moist								
1.2	SANDY SILT: Brown sandy silt, moist								
6.5	CLAYEY SILT TILL: Brown to grey clayey silt, trace gravel, APL to DTPL								
7.0	becoming very hard, grey, some cobbles, DTPL								
	TEST PIT TERMINATED AT 7.0 m								Upon completion of excavation: Test Pit open No free water

NOTES:

⊗ LAB SHEAR TEST
 ▲ POCKET PENETROMETER
 ◆ POCKET TORVANE
 △ WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING

CHECKED BY KH

LOG OF TEST PIT NO. 3

PROJECT Supplementary Geotechnical Investigation
LOCATION Hopewell Crossing Subdivision Development
DATE 2016 11 14

PML REF.: 16KF069
ENGINEER K. Hanes
TECHNICIAN K. Hanes

SOIL PROFILE		LEGEND	SAMPLE NUMBER	SHEAR STRENGTH C_u (kPa)				LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W $W_L - W_P$ WATER CONTENT %				UNIT WEIGHT γ (kN/m ³)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION												
	GROUND ELEVATION			50	100	150	200	10	20	30			GRAIN SIZE DISTRIBUTION (%) GR SA SI&CL
0.30	TOPSOIL: Dark brown sandy silt topsoil, rootlets, moist												
1.00	SILT: Brown silt, some sand, trace gravel, occasional cobbles, moist to wet												
4.5	CLAYEY SILT TILL: Brown to grey clayey silt, some sand, trace gravel, occasional cobbles APL to DTPL												
5.0	wet to saturated sandy silt layer												Ground water seepage from 4.5 m
8.0	TEST PIT TERMINATED AT 8.0 m												Upon completion of excavation: Test Pit caved to 6.5 m No free water

NOTES:

- ⊗ LAB SHEAR TEST
- ▲ POCKET PENETROMETER
- ◆ POCKET TORVANE
- ||| WATER LEVEL OBSERVED DURING / UPON COMPLETION OF DRILLING

CHECKED BY KH