



**GEOTECHNICAL INVESTIGATION
PROPOSED WILMOT WOODS DEVELOPMENT
NEW HAMBURG, ONTARIO**

for

WILMOT WOODS DEVELOPMENT INC.

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Mr. Adam Belksy
Wilmot Woods Development Inc.
310 Fairway Road South, P.O. Box 45016
Kitchener, Ontario
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Dear Mr. Belksy

**Geotechnical Investigation
Proposed Wilmot Woods Development
New Hamburg, Ontario**

Peto MacCallum Ltd. (PML) is pleased to report the results of the geotechnical investigation recently completed at the above noted project site. Authorization to proceed with this assignment was provided by Mr. Galbraith in an email dated July 9, 2018.

In general, the project involves the proposed construction of a residential subdivision on a 37 Ha site located south of Waterloo Street and north of the railway in New Hamburg, Ontario. The proposed development is located on an existing agricultural property. Details of the proposed development have yet to be established, but in general it is understood that approximately 600 residential dwellings will be constructed at the site. It is anticipated that the proposed development will have full municipal servicing including watermain, storm and sanitary sewers, with typical invert depths expected to be to a maximum 3 m depth below existing grade. It is also understood that a trunk sewer will extend southward from the site, under the railway and connect to a proposed trunk sewer on the neighbouring property (the Wilmot Employment Lands). Two storm water ponds also are proposed for the development, one near the northwest corner of the site, and the second at the south side of the site.



The purpose of the current geotechnical investigation was to explore the subsurface soil and ground water conditions at the site and based on this information, to provide geotechnical recommendations for the proposed development. 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;
- Excavation and construction dewatering requirements;
- House foundation design, including bearing resistances, settlement projections and site class for seismic design;
- Slab on grade floors and below grade walls, including compaction requirements and geotechnical suitability of onsite soils for re-use;
- Site servicing (storm, sanitary, water and utilities) including pipe bedding requirements;
- Pavement structure design for new roadways; and,
- Suitability of native soils for infiltration of stormwater.

Recommendations for the proposed trenchless crossing of the railway will be provided under separate cover.

The comments and recommendations provided in this report are based on the site conditions at the time of the investigation, and are for the current project only. Any changes in plans will require review by PML to assess the applicability of the report, and may require modified recommendations, additional analysis and / or investigation. When the project design is complete, the general recommendations given in this report should be reviewed by PML to ensure their applicability.



Investigation Procedure

The field work for this geotechnical investigation was conducted between July 30 and October 15, 2018. The investigation program comprised a total of 22 boreholes (BH1 to BH12 and MW101 to MW110) advanced to between 6.5 and 12.6 m depth, with monitoring wells installed in ten of the boreholes (designated MW101 to MW110). The borehole and monitoring well locations are shown on the appended Borehole Location Plan, Drawing 1.

The borehole locations were established in the field by PML. Borehole locations were surveyed by PML using a Global Navigation Satellite System (GNSS). The survey equipment was provided by SOKKIA Canada, model GCX-2.

The boreholes were advanced using a Diedrich D-50 track mounted drillrig fitted with continuous flight solid and hollow stem augers and automatic hammer, supplied and operated by a specialist drilling contractor. The work was carried out under the full-time supervision of a PML engineering staff member who directed the drilling and sampling operations, documented the soil stratigraphy, monitored ground water conditions and processed the recovered samples.

Representative samples of the overburden were recovered at regular intervals throughout the depths explored. Standard penetration tests (SPT) were carried out during sampling operations in the boreholes using conventional split spoon equipment. Pocket penetrometer testing was carried out on the recovered samples to determine the undrained shear strength of the cohesive soils. Ground water observations were made in the boreholes during and upon completion of drilling. The boreholes were backfilled in accordance with O.Reg.903 upon completion of drilling.

Monitoring wells were installed in ten boreholes to more accurately measure ground water levels. The monitoring wells comprised 50 mm diameter PVC pipe, filter sand, bentonite seals, and protective casings. Subsequent water level measurements from the wells were conducted by PML.

All of the recovered samples were returned to PML's laboratory for detailed visual examination, classification, and routine moisture content determinations. The laboratory testing also included particle size distribution analyses on eleven samples of the major soil types encountered.



Summarized Subsurface Conditions

Reference is made to the appended Log of Borehole sheets for details of the field work including soil descriptions, inferred stratigraphy, standard penetration test (SPT) N values, pocket penetrometer test values, ground water observations and laboratory moisture content determinations.

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.

In general, the soil stratigraphy encountered comprised surficial topsoil and localized fill, underlain by an extensive clayey silt deposit containing localized discontinuous cohesionless silt, sandy silt, silty sand and sand layers.

Surficial topsoil was contacted in all of the boreholes, with the exception of Borehole 104. The topsoil was between 180 and 850 mm thick, with an average of 294 mm.

Surficial fill was encountered locally in Borehole 104, and extended 0.68 m depth.

An extensive clayey silt deposit was encountered below the surficial topsoil and fill deposits, in all of the boreholes, and extended to the 6.5 to 12.6 m borehole termination depths. The cohesive clayey silt deposit was generally firm to very stiff based on SPT N values between 4 and 30 blows per 0.3 m penetration of the split spoon sampler. Pocket penetrometer shear strengths of the clayey silt ranged between 50 and 225 kPa. Moisture content ranged between 8 and 31% indicating drier than plastic limit (DTPL) to wetter than limit (WPL) conditions in the cohesive clayey silt soils.

Reference is given to the appended Figures 1 to 6 for the results of the particle size analyses conducted on samples of the clayey silt.

Discontinuous localized layers of cohesionless silt, sandy silt, silty sand and sand were encountered above, below and within the clayey silt, in Boreholes BH5, BH6, BH9, MW101, MW102. These layers were loose to compact based on SPT N values between 5 to 28 blows per 0.3 m penetration of the split spoon sampler. Moisture contents ranged between 5 and 22% indicating damp to saturated conditions in the cohesionless soils. Reference is given to Figures 6 to 11 for the results of particle size distribution analyses conducted on samples of the silt, sandy silt, silty sand, and sand soils.



Ground Water Conditions

Ground water observations carried out during and upon completion of drilling are presented on the appended Log of Borehole Sheets.

During drilling, wet and saturated conditions were generally encountered in the silt, sandy silt, silty sand and sand layers below 1.0 to 2.9 m depth (Elevation 338.1 to 340.3). Free water was observed upon completion of drilling, in Boreholes 2 and 9, below 2.0 to 2.3 m depth (Elevation 339.4 to 340.9).

On November 26, 2018 water level measurements from the monitoring wells installed in Boreholes 101 to 110 ranged between 0.1 to 5.4 m depth below existing grade (about Elevation 338.1 to 344.1).

The ground water levels at the site are subject to seasonal fluctuations and precipitation patterns. It should be noted that the relatively impermeable nature of the clayey silt deposits could contribute to the development of perched water conditions following short term and seasonal participation events.

Discussion and Recommendations

In general, the project involves the proposed construction of a residential subdivision on a 37 Ha site located south of Waterloo Street and north of the railway in New Hamburg, Ontario. It is anticipated that the proposed development will have full municipal servicing including watermain, storm and sanitary sewers, with typical invert depths expected to be to a maximum 3 m depth below existing grade. It is also understood that a trunk sewer will extend southward from the site, under the railway and connect to a proposed trunk sewer on the neighbouring property (the Wilmot Employment Lands). Two storm water ponds also are proposed for the development.

Foundations and Earthworks Grading

Details of the buildings in the proposed residential subdivision have yet to be established. We have provided the following preliminary foundation design recommendations and earthworks grading recommendations for the development.



The site is generally underlain by firm to very stiff clayey silt. It is feasible to support buildings on conventional spread or strip footings founded in the native firm to very stiff clayey silt. Based on the investigation findings, footings founded a minimum 0.3 m into the firm to very stiff native clayey silt deposits, below any surficial fill and topsoil and local surficial soft or loose zones, may be designed for a net bearing resistance of 150 kPa at the serviceability limit state (SLS) and a factored bearing resistance of 225 kPa at the ultimate limit state (ULS).

Alternatively, in areas where grades are to be raised footings may be placed at higher elevations on engineered structural fill. The existing topsoil and fill must be excavated to the levels of competent native clayey silt deposits in advance of engineered structural fill placement. Engineered structural fill used to establish footing founding subgrade levels should comprise an approved compactable inorganic soil, placed in lifts with a maximum thickness of 300 mm and be compacted to at least 98% standard Proctor maximum dry density (SPMDD). Additional generic recommendations for engineered fill construction are provided in Appendix A. Footings supported on approved engineered structural fill may also be designed using the values for a net factored resistance of 150 at SLS and 225 kPa at ULS. Full time inspection of any structural fill placement by PML personnel is recommended to approve subgrade conditions, fill materials and to verify that the specified compaction levels are being achieved.

The excavated soils likely to be used for site grading will comprise primarily clayey silt soils, in DTPL to WTPL condition, with moisture contents between 8 to 31%. Material described as 'wet' or 'WTPL' on the appended borehole will generally not be compactible to 98% SPMDD and should be segregated and allocated to areas where post construction settlement would not present a concern. The bulk of the clayey silt soil to be excavated would be too wet for immediate reuse given the observed moisture conditions.

Therefore, the excavated 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.

It should also be noted that the in-situ clayey silt 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 maximum total settlement of foundations designed for the net SLS bearing pressures noted above are not expected to exceed 25 mm. Differential settlements of around 50 to 75% of the total settlement should be anticipated.

All founding surfaces should be examined by PML personnel prior to concrete placement, to check that all loose, frozen, organic or otherwise deleterious materials have been satisfactorily removed and the required bearing capacity is available throughout.

All exterior footings and all footings exposed to seasonal freezing conditions must be provided with frost protection. The minimum frost protection should be 1.2 m of earth cover or the thermal insulation equivalent.

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.

Slab-on-Grade Floors

Preparation of the floor slab subgrade should include stripping of the surficial, topsoil, and other deleterious material, placement and compaction of engineered fill, if necessary, followed by proof rolling of the exposed subgrade with a heavy roller to ensure uniform adequate support. Excessively loose, soft or compressible materials revealed during the proofrolling operations should be subexcavated and replaced with well compacted approved material.

Engineered fill placed under the floor slab to achieve finished subgrade levels or as foundation wall backfill should comprise approved inorganic material having a moisture content within 3% of the optimum value, placed in maximum 200 mm thick lifts, and compacted to at least 95% SPMDD. Reference is given to Appendix A for additional engineered fill construction recommendations.

A minimum 150 mm thick layer of free draining Granular A type material compacted to 98% SPMDD is recommended directly beneath the slab-on-grade. A polyethylene vapour barrier should be placed on the surface of the granular base if a moisture sensitive finish is to be placed on the floor. Joints should be saw cut into concrete floor immediately after initial set of the concrete to control potential cracking of the slab.



Below Grade Walls

Below grade walls and basement walls should be designed as retaining walls to resist the unbalanced horizontal earth pressure imposed by the backfill adjacent to the wall. The unfactored lateral earth pressure, p , may be computed using the following equation, assuming a triangular pressure distribution:

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

where K = lateral earth pressure coefficient
= 0.5 for wall restrained at both
top and bottom

γ = unit weight of free-draining
granular material
= 21 kN/m³

h = depth below final grade (m)

q = surcharge load (kPa), if present

The excavation adjacent to the basement walls should be backfilled with free-draining granular material satisfying the OPS Granular B gradation specification and a weeping tile system installed to minimize the build-up of hydrostatic pressure behind the wall.

The weeping tiles should be surrounded by a properly designed graded granular filter or wrapped with approved geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free sump or outlet.



Excavation and Ground Water Control

It is generally envisaged that excavations for the earthworks and site servicing will extend to a maximum 5 m depth within the proposed development.

Excavations for service installations are expected to extend up to about 3 m depth through topsoil and into the native clayey silt deposits containing localized layers of silt, sandy silt, silty sand and sand, which are classified as Type 3 materials as defined in the OHSA. Subject to inspection and providing adequate ground water control is achieved, excavations within Type 3 soils that are to be entered by workers should be inclined from the base of the excavation at one horizontal to one vertical (1H:1V) or flatter.

It is anticipated that ground water seepage or surface water entering the excavations will be handled readily by conventional sump pumping. The actual dewatering methods should be established at the contractor's discretion within the context of a performance specification for the project. Regardless of the dewatering method chosen, the hydraulic head and ground water inflow must be properly controlled to ensure a stable and safe excavation and to facilitate construction. The design of the dewatering system should be specified to maintain and control ground water at least 0.3 m below the excavation base level, in order to provide a stable excavation base 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 Conservation and Parks (MECP) is required if the dewatering discharge is greater than 400,000 L/day. In accordance with the above noted regulatory requirements and in compliance with the MECP's PTTW Manual (April 2005), and application should be filed to the MECP 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). PML would be pleased to assist with this process, if required. The depth of excavations for site grading and site servicing are expected to extend to a maximum 3 m depth into clayey silt deposits with wet to saturated layers of silt, sand, sandy silt, and silty sand and it is generally envisaged that sump pumping from within trenching excavating will have dewatering rates less than 50,000 L/day, and a PTTW or EASR should not be required.



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 to be contacted. Also, as noted above, the dewatering requirements should be established by the contractor in the context of a performance specification.

Pipe Bedding and Cover

It is expected that the proposed water and sewer pipes will be founded on competent native deposits, or engineered fill. Providing adequate ground water control is achieved, bearing problems are not anticipated for conduits founded on the native mineral soils or engineered fill. It may be necessary to increase the bedding thickness 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.

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 clayey silt. The in-situ moisture content of the clayey silt typically ranges from 8 to 31%. Based on our experience with similar types of material, the upper limit of placement moisture content compatible with efficient compaction is expected to be about 15%. Therefore, the excavated clayey silt containing 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.



Excavated materials intended for backfilling purposes should not be exposed to the elements for prolonged time periods, as they might be rendered unsuitable for reuse. Organic soil, topsoil, deleterious or excessively wet material should not be used as backfill. Should construction start during the winter season, particular attention must be given to ensure that frozen material is not used as backfill for service trenches. Topsoil may be reused for landscape purposes only.

It should also be noted that the in-situ clayey silt 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.

Soil Infiltration

Two storm water management (SWM) ponds are proposed for the site. The first SWM pond will be near Boreholes BH2, MW102 and MW103, while the second will be near MW108 and MW109. Design details of the ponds have yet to be finalized. Typical soils at the pond site comprise clayey silt with occasional to numerous silt layers and deposits of sand and sandy silt. Although the clayey silt is considered to be relatively impermeable, the silt, sandy silt and sand layers which are interlayered with the clayey silt are more permeable. Therefore, it will be necessary to line the base of the ponds to maintain the permanent pool water levels.

The earthen liner should comprise clayey silt soils having a hydraulic conductivity of no more than 1×10^{-6} cm/s. The native clayey silt has a permeability less than 1×10^{-6} cm/sec, however, inspection and testing during construction will be required to confirm if excavated materials are suitable for reuse as the earth liner.

Fill used for earth liner construction at the pond, should be placed in lifts with a maximum 300 mm thickness, and compacted to at least 95%SPMDD. General recommendations for fill subgrade preparation and engineered fill construction are provided in Appendix A.



Where berms are required, they 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 5H:1V for the interior. Slopes should be provided with 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.

It is understood that onsite storm water infiltration parameters are required. The following table provides hydraulic conductivity and infiltration design parameters for the major onsite soils encountered. An appropriate factor of safety should also be used for design.

SOIL	HYDRAULIC CONDUCTIVITY (cm/s)	INFILTRATION RATE (mm/hr)
Clayey Silt/Silt	Less than 1×10^{-6}	Less than 0.1
Silt/Sandy Silt/Silty Sand	1×10^{-4} to 1×10^{-5}	2
Sand	1×10^{-3}	40

Cognizant of the low permeability and infiltration rates and considering the limited extent and high groundwater level within the silt/sandy silt/silty sand/sand layers, the amount of onsite infiltration is expected to be negligible.



Pavement Design

Based on the proposed pavement usage, frost susceptibility, and strength of the expected subgrade soils, the following minimum pavement component thicknesses are considered suitable for the proposed residential subdivision roadways.

PAVEMENT COMPONENT	THICKNESS (mm)
Asphalt	80
Granular A Base	150
Granular B Subbase	600

The pavement design considers that construction will be carried out during the drier time of the year and that the subgrade is stable, as determined by proofrolling and inspection by PML personnel. If the subgrade is wet and unstable, subexcavation and placement of additional granular subbase material will be required.

In areas where the subgrade is sensitive to disturbance or construction is to occur outside of the drier time of year, then consideration can be given to thickening the granular subbase or using a geotextile separator between the pavement structure and subbase, in lieu of additional granular subbase. The geotextile separator envisaged should provide reinforcement, filtration and separation of the granular subbase from the anticipated clayey silt / clayey silt fill subgrade soils, and a woven geotextile such as Terrafix's 200 W (or equivalent) is envisaged.

The pavement materials should conform to current OPS and municipal specifications. The Granular A base and Granular B subbase courses should be placed in thin lifts and be compacted to a minimum of 100% SPMDD, and asphalt should be placed to a minimum of 92% of the material's maximum relative density (MRD) and reference is made to OPS Specification 310.

During construction, testing should be conducted to confirm the gradation and compactibility characteristics of the granular base materials and the mix design properties of the asphalt.



Proofrolling procedures and the placement and compaction of all the granular materials and asphalt for the pavement construction should be inspected on a continuous basis by PML personnel.

The pavement subgrade materials will lose strength to support traffic loads if allowed to become wet. Moreover, the silty clay subgrade soils are considered frost susceptible and the roadway may heave during freezing and thawing periods. Drainage of the pavement structure is essential to maintain structural integrity and limit frost heave. In this regard, installation of longitudinal subdrains is recommended. The longitudinal subdrains should comprise a minimum 100 mm diameter perforated plastic pipe, set below the subbase level, and outlet to ditching, or catch basins. Subdrain pipes should be surrounded by appropriate filter media such as clear stone wrapped in geotextile, or alternatively the pipes should be wrapped in filter cloth and surrounded by concrete sand.

Geotechnical Review and Construction Inspection and Testing

When development design is complete, it is recommended that the design drawings be submitted to PML for general geotechnical review for compatibility with site conditions and recommendations of this report.

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, particularly where service trenches exist. 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.

This report is subject to the Statement of Limitations that is included in Appendix B, which must be read in conjunction with the report



Closure

We trust the information presented in this report 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.
Manager Engineering Services



Gerry Mitchell, MEng, P.Eng.
Senior Consultant

WL/GM:tm

Enclosures:

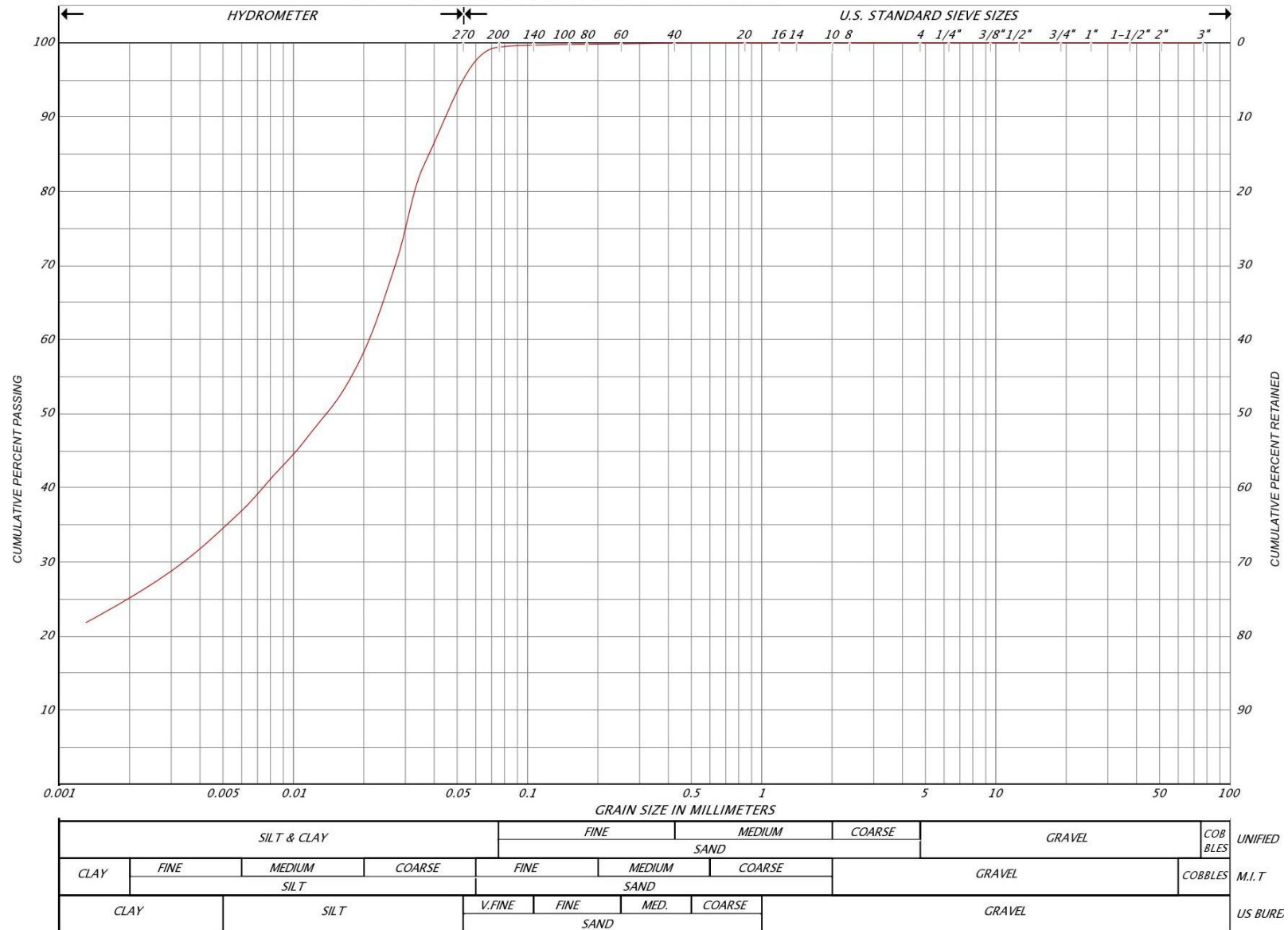
Figures 1 to 11 - Particle Size Distribution Charts
List of Abbreviations
Log of Boreholes BH1 to BH12 and BH/MW101 to BH/MW110
Drawing 1 - Borehole Location Plan
Appendix A - Engineered Fill
Appendix B - Statement of Limitations

REMARKS Borehole MW101, Sample SS7, Depth 6.1 to 6.5 m
CLAYEY SILT

PARTICLE SIZE DISTRIBUTION CHART

PML REF. 18KF031

FIGURE NO. 2

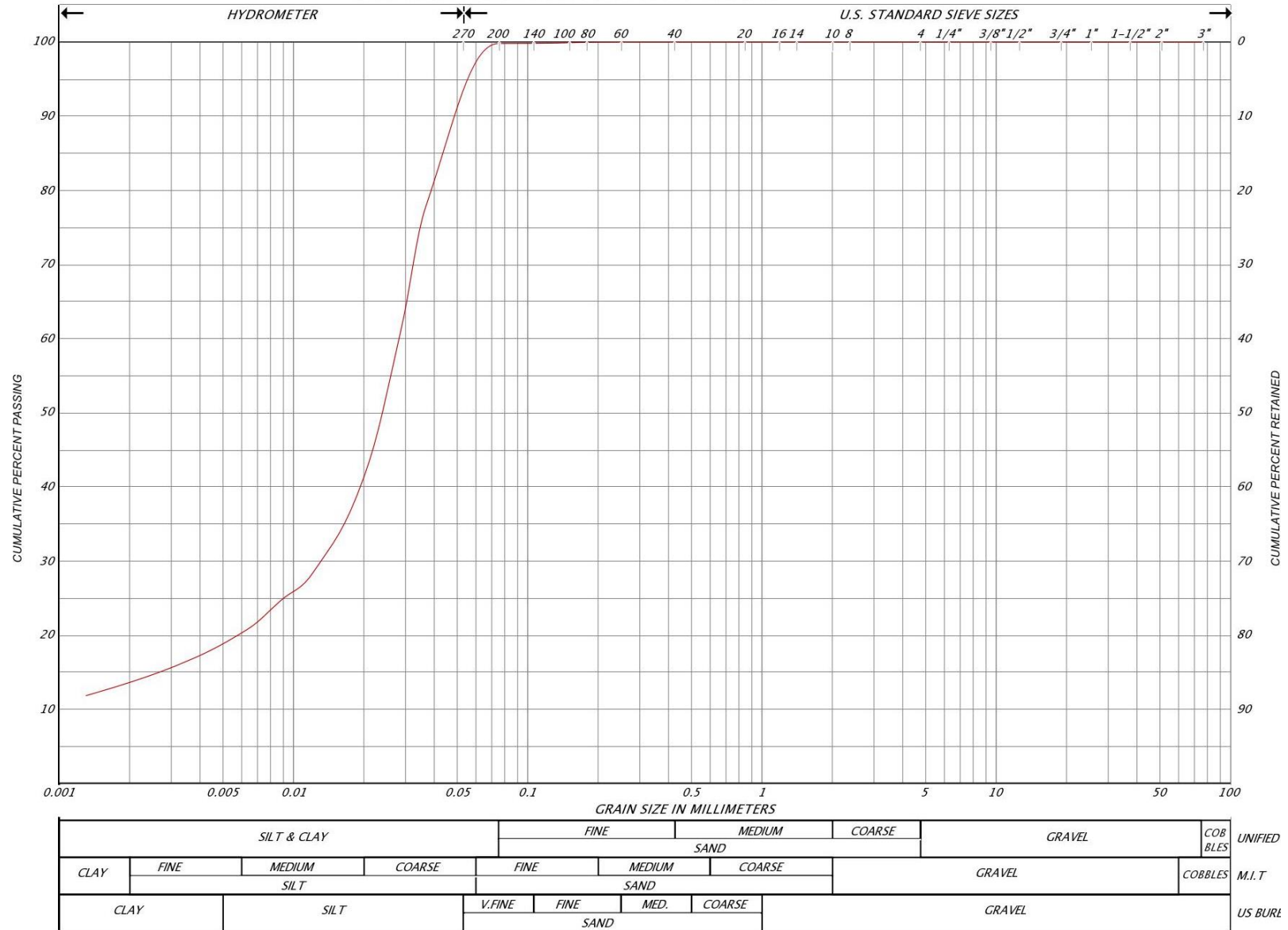


REMARKS Borehole MW104, Sample SS6, Depth 4.6 to 5.0 m

CLAYEY SILT

PARTICLE SIZE DISTRIBUTION CHART

PML REF. 18KF031
FIGURE NO. 3

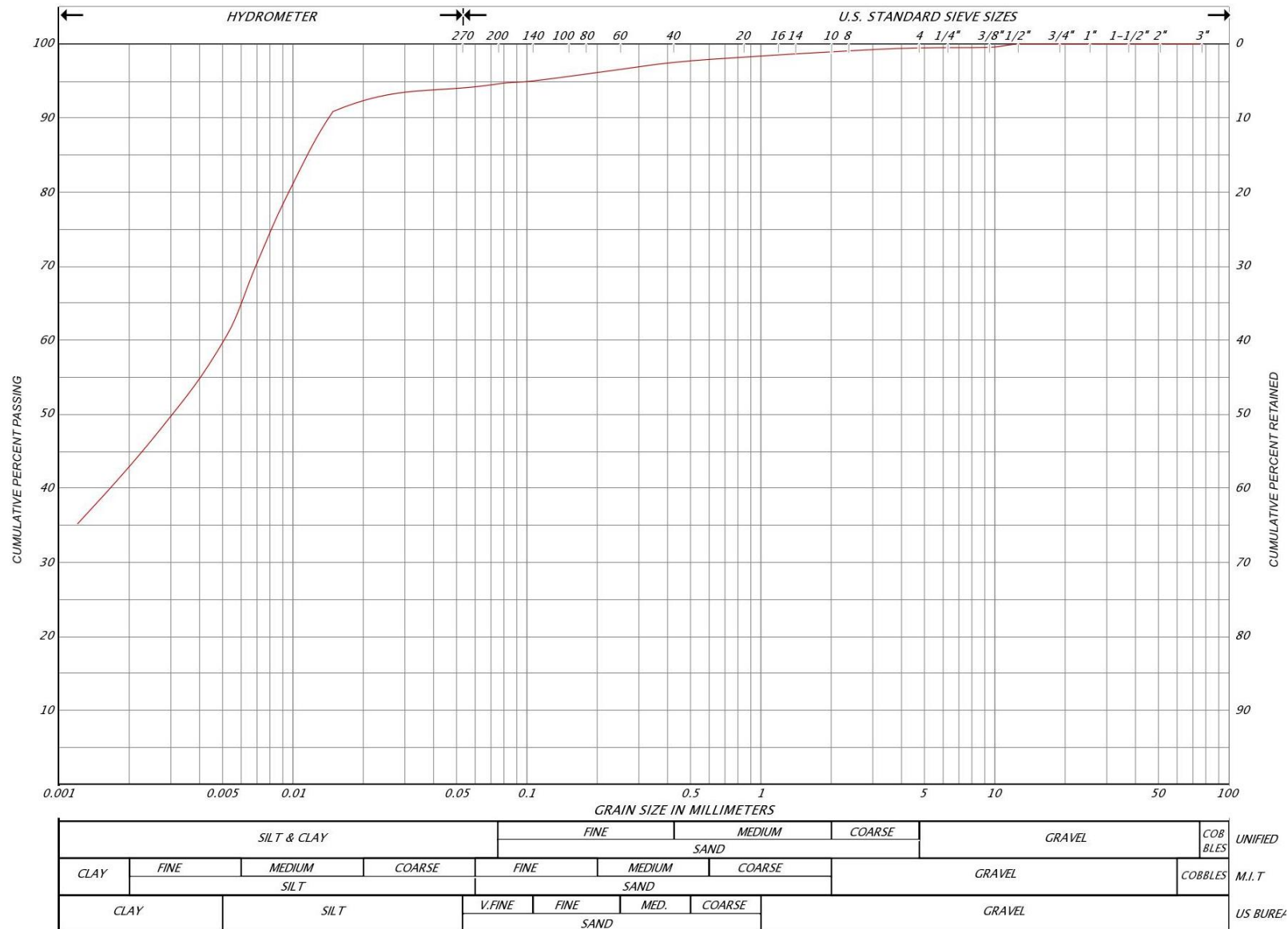


REMARKS Borehole MW105, Sample SS6, Depth 4.5 to 5.0 m
CLAYEY SILT

PARTICLE SIZE DISTRIBUTION CHART

PML REF. 18KF031

FIGURE NO. 4

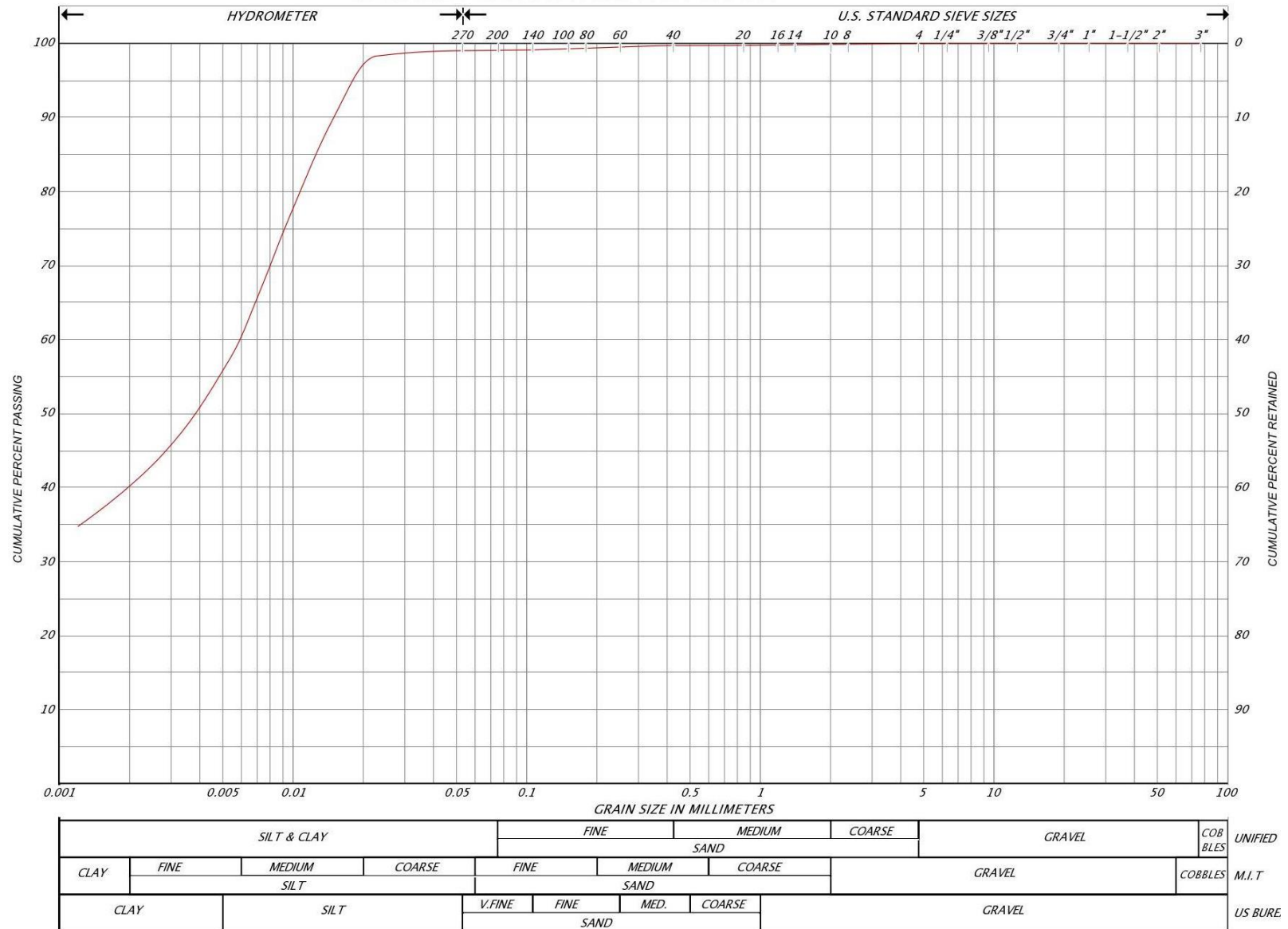


REMARKS Borehole MW107, Sample SS3, Depth 1.5 to 2.0 m

CLAYEY SILT

PARTICLE SIZE DISTRIBUTION CHART

PML REF. 18KF031
FIGURE NO. 5

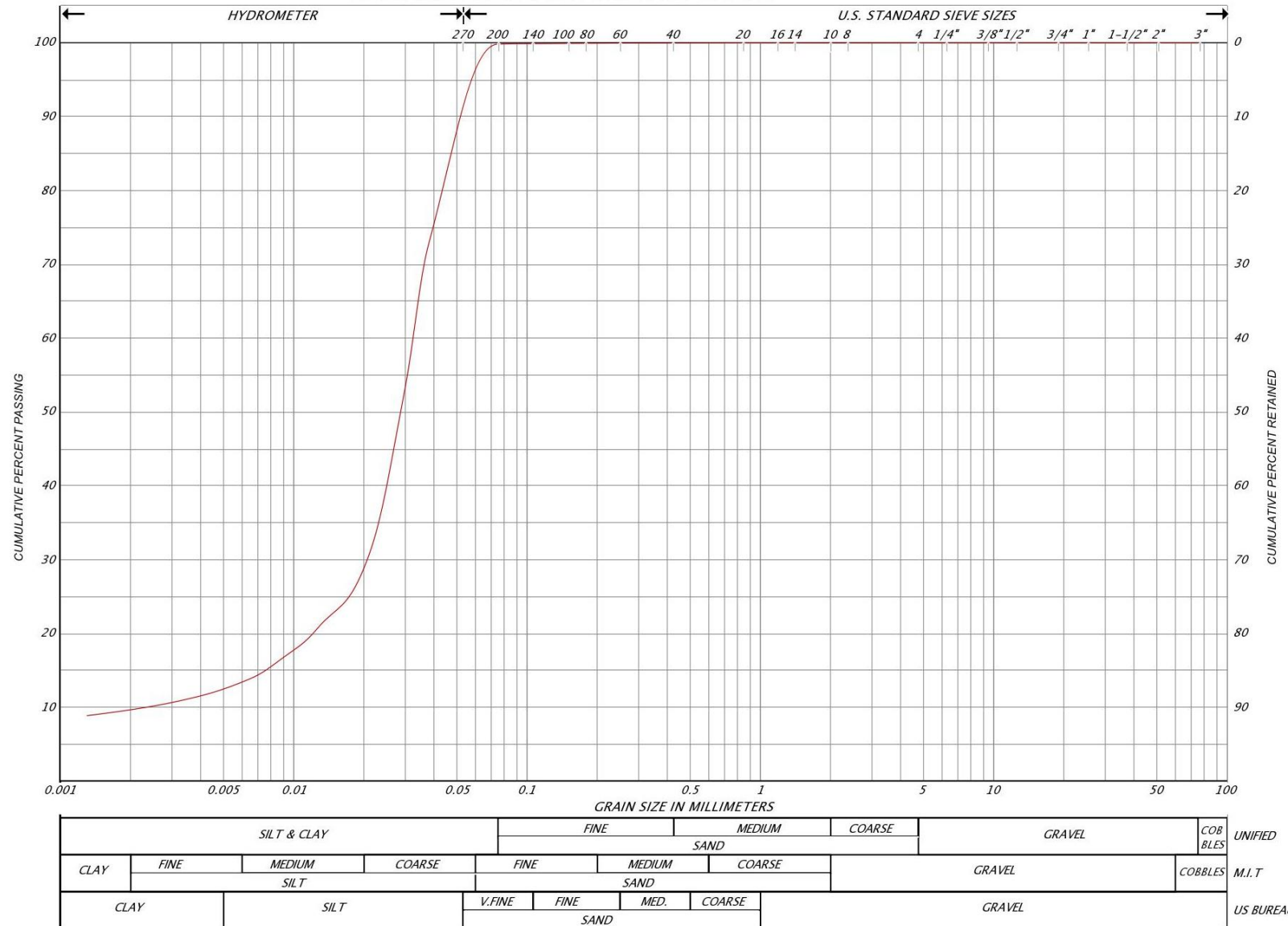


REMARKS Borehole MW107, Sample SS6, Depth 4.6 to 5.0 m

CLAYEY SILT

PARTICLE SIZE DISTRIBUTION CHART

PML REF. 18KF031
FIGURE NO. 6



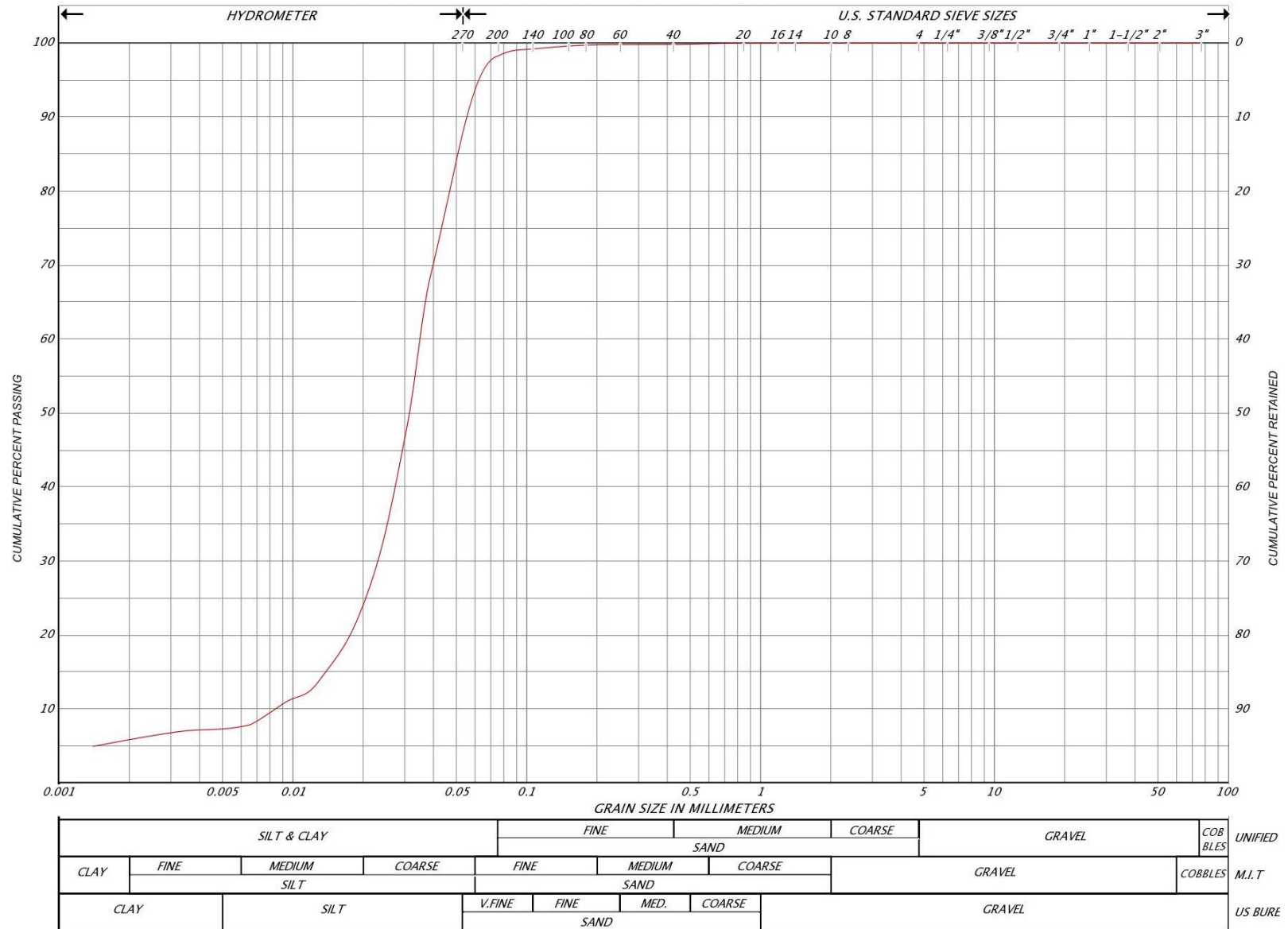
REMARKS Borehole MW109, Sample SS11, Depth 9.1 to 9.6 m

CLAYEY SILT

PARTICLE SIZE DISTRIBUTION CHART

PML REF. 18KF031

FIGURE NO. 7



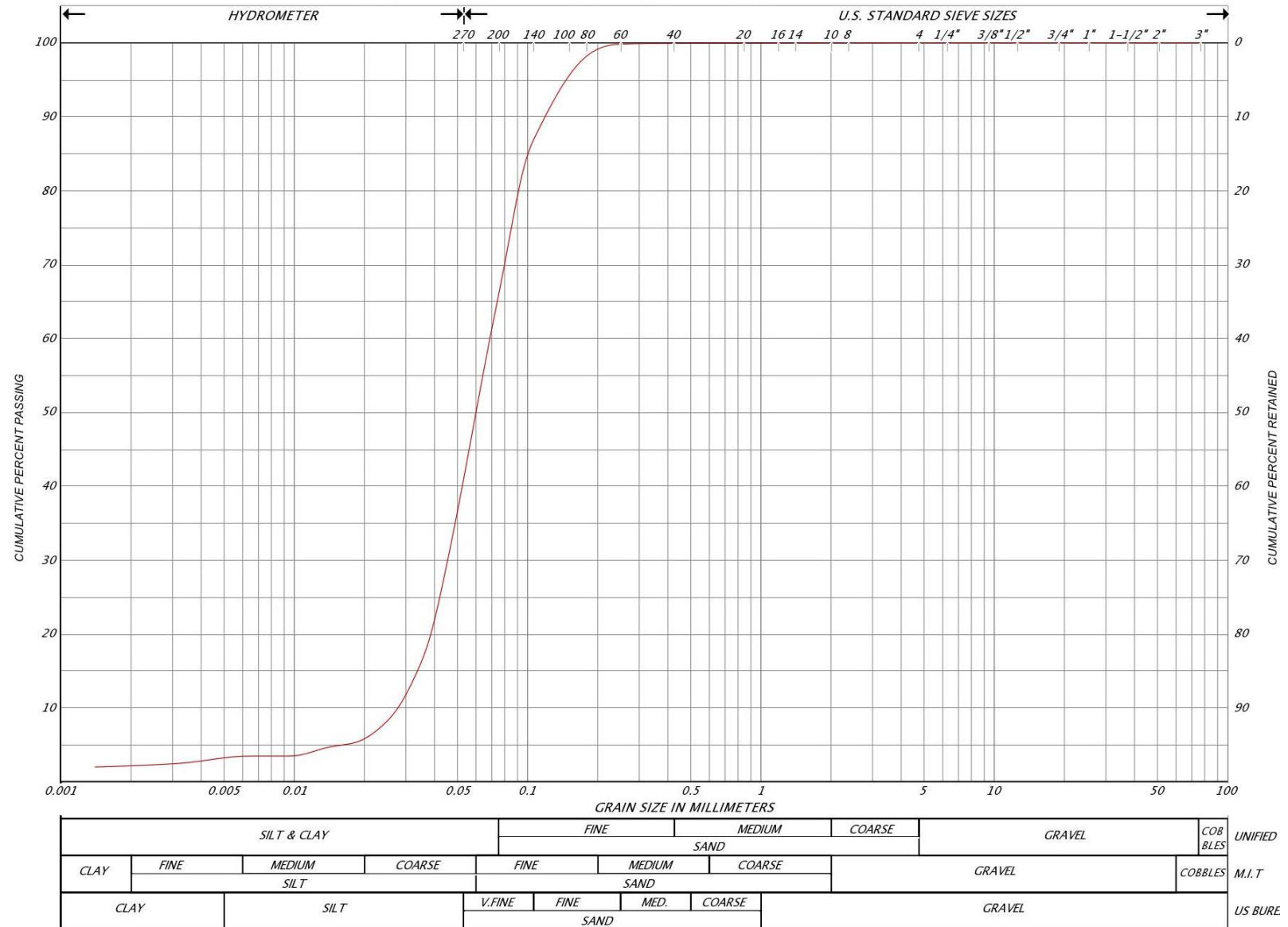
REMARKS Borehole BH9, Sample SS3, Depth 1.5 to 2.0 m

SILT

PARTICLE SIZE DISTRIBUTION CHART

PML REF. 18KF031

FIGURE NO. 8



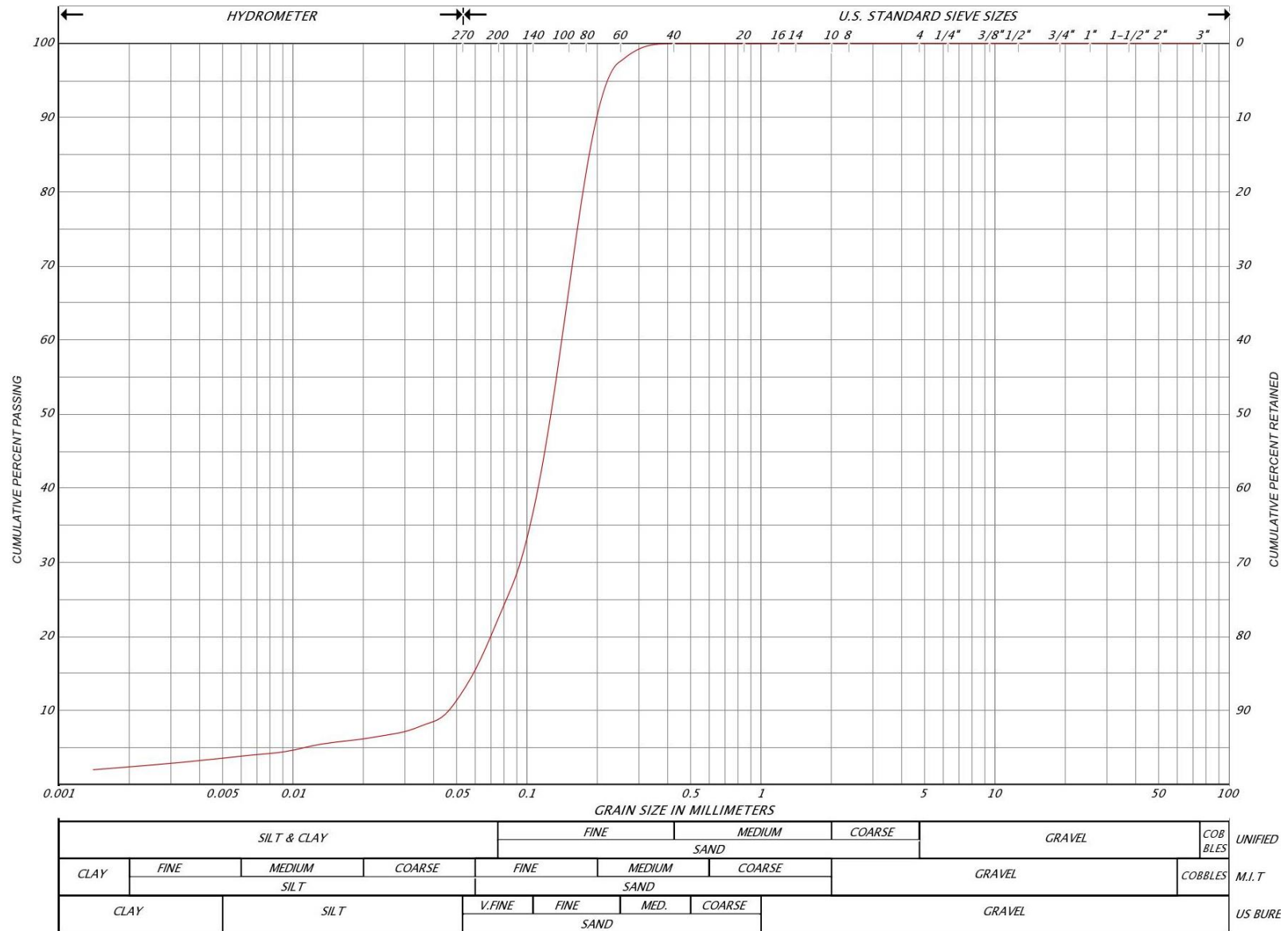
REMARKS Borehole MW102, Sample SS6, Depth 4.6 to 5.0 m

SANDY SILT

PARTICLE SIZE DISTRIBUTION CHART

PML REF. 18KF031

FIGURE NO. 9

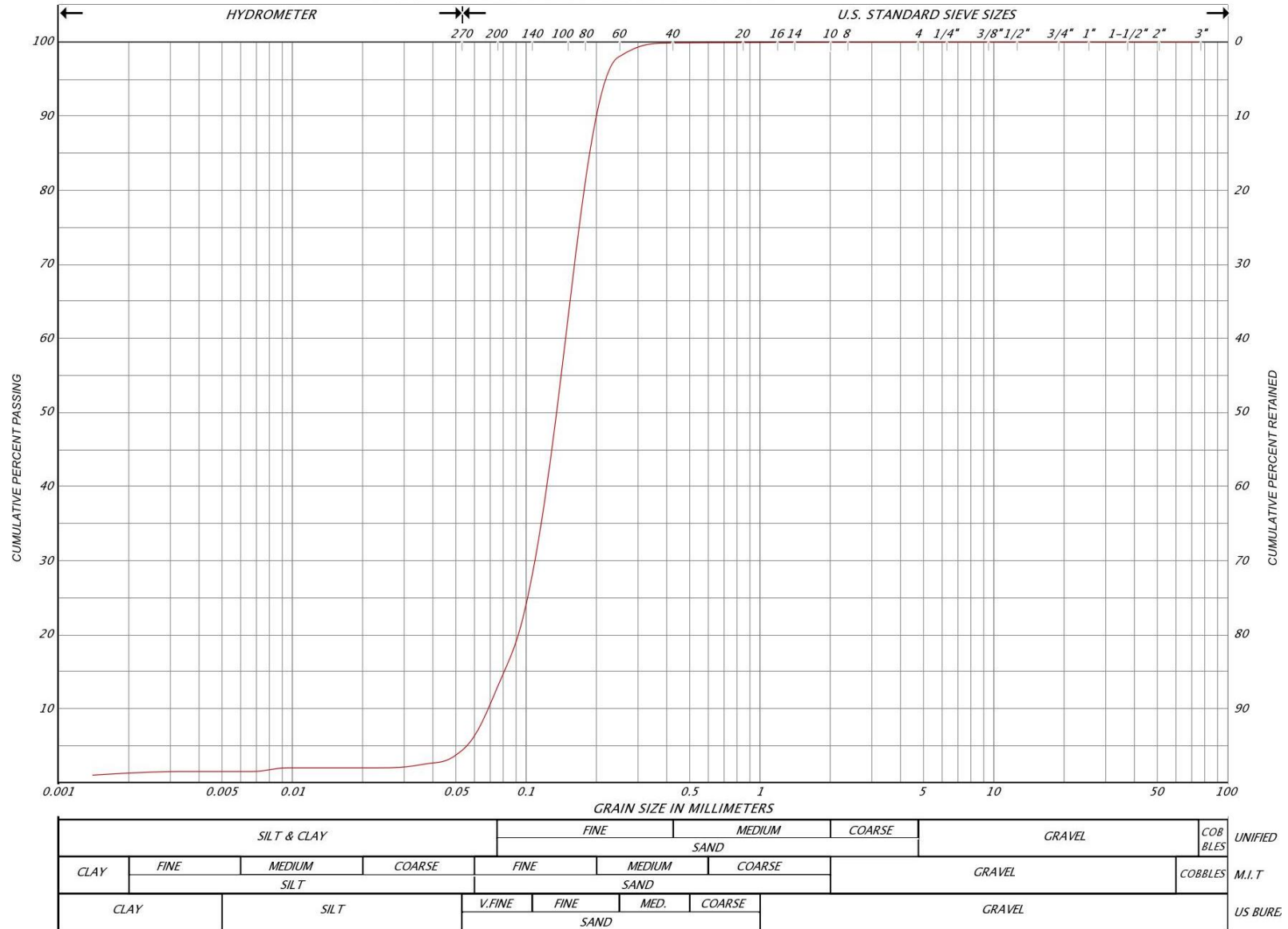


REMARKS Borehole BH9, Sample SS5, Depth 3.1 to 3.5 m

SILTY SAND

PARTICLE SIZE DISTRIBUTION CHART

PML REF. 18KF031
FIGURE NO. 10



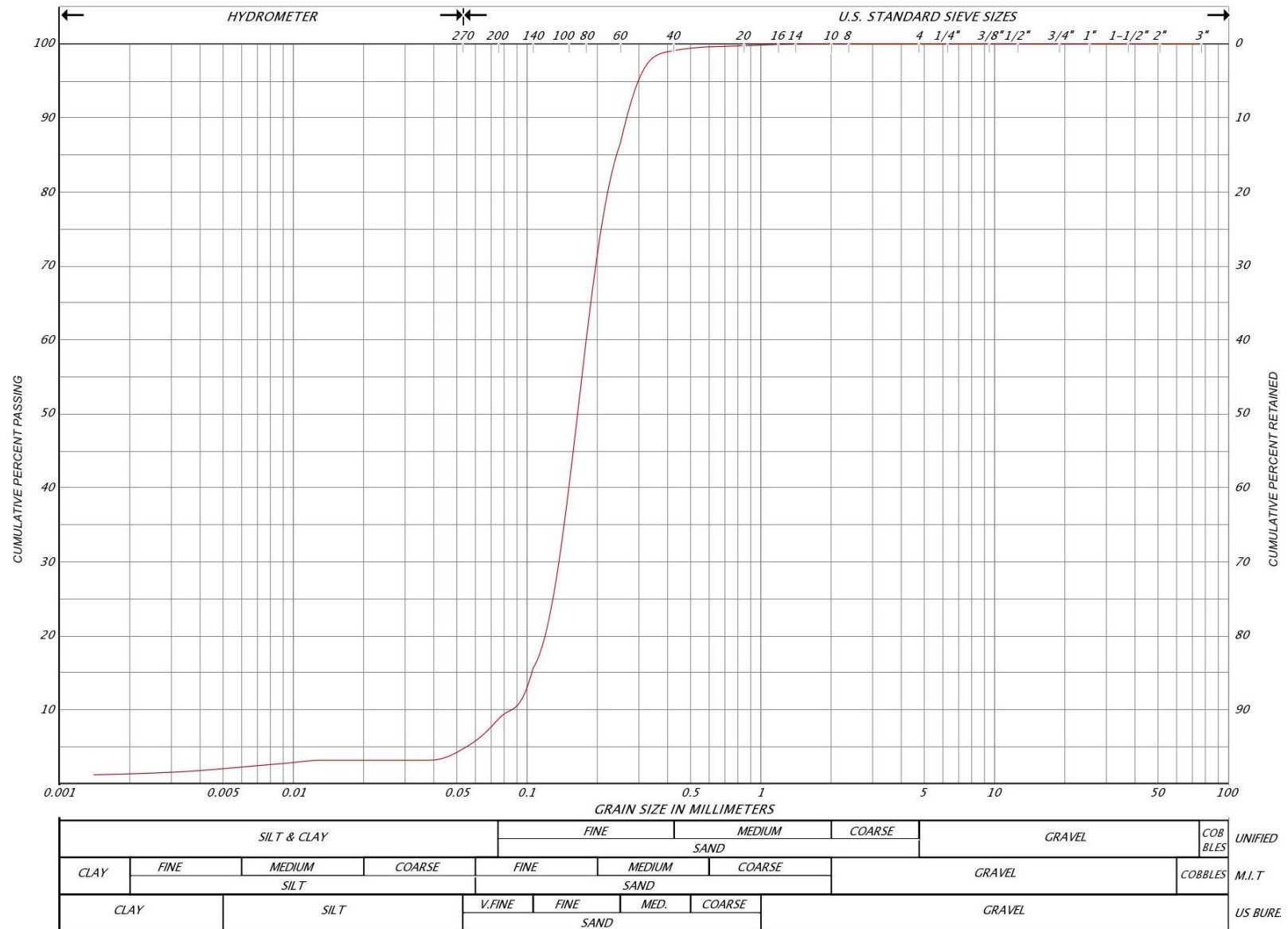
REMARKS Borehole BH6, Sample SS5, Depth 3.1 to 3.5 m

SAND

PARTICLE SIZE DISTRIBUTION CHART

PML REF. 18KF031

FIGURE NO. 11



REMARKS Borehole MW102, Sample SS4, Depth 2.3 to 2.7 m

SAND

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		

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN M. Rapsey

NOTES

LOG OF BOREHOLE NO. 2

17T 523879E 4804339N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE September 24, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN M. Rapsey

SOIL PROFILE			SAMPLES			ELEVATION SCALE	SHEAR STRENGTH (kPa)				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT kN/m ³	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH ELEV (metres)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		+ FIELD VANE Δ TORVANE ○ Qu								
							▲ POCKET PENETROMETER ○ Q								
						DYNAMIC CONE PENETRATION STANDARD PENETRATION TEST ×				WATER CONTENT (%)					
						20 40 60 80				10 20 30 40					
0.0	SURFACE ELEVATION 341.42														
0.18	TOPSOIL: Dark brown sandy silt, trace clay, occasional rootlets, damp														
341.24															
0.70	SAND: Loose brown sand, some silt, damp														
340.72															
1.0	SANDY SILT: Loose brown sandy silt, moist														
1.4															
340.0	becoming compact, saturated														

NOTES

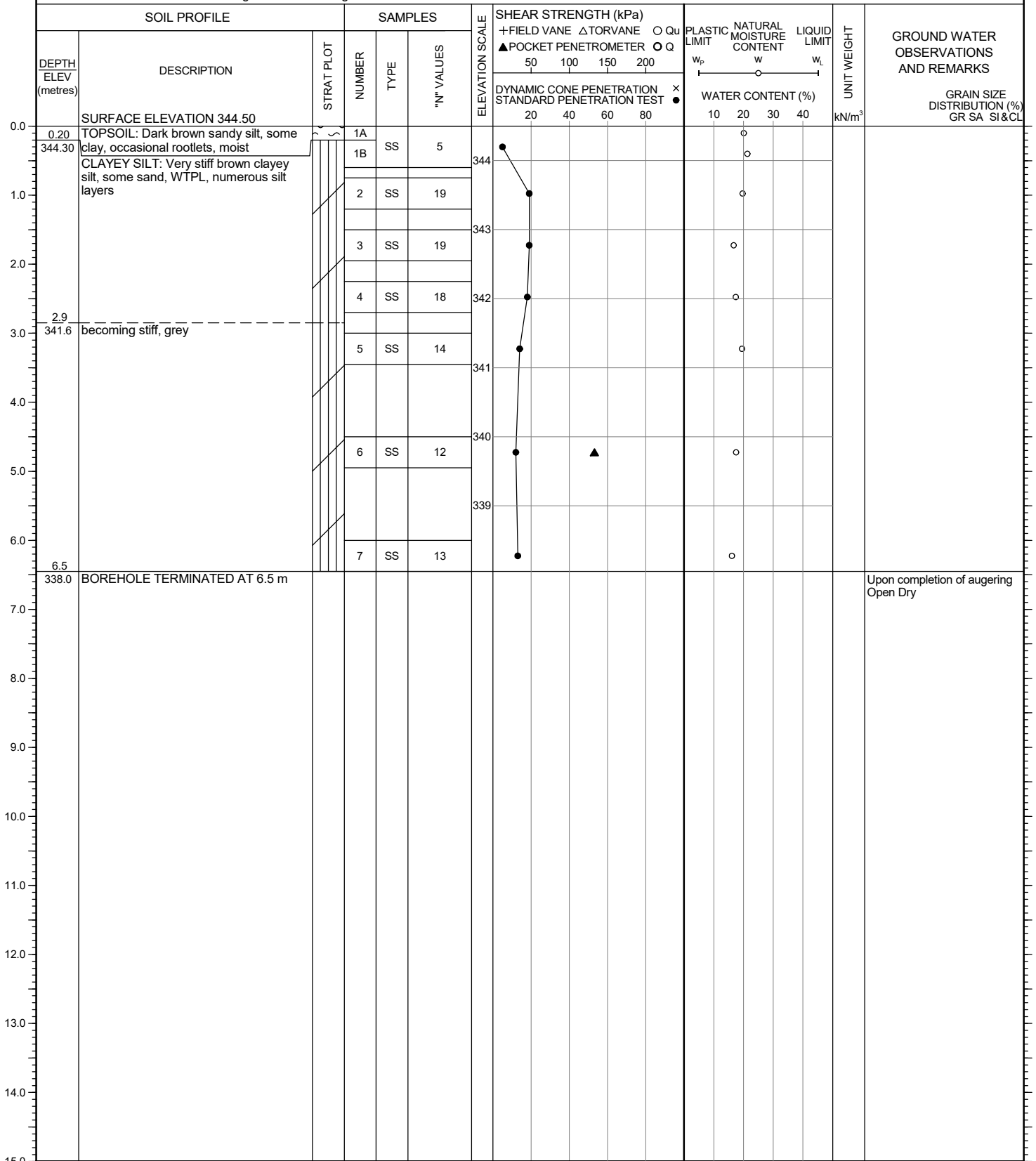
LOG OF BOREHOLE NO. 3

17T 524026E 4804185N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE July 31, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES

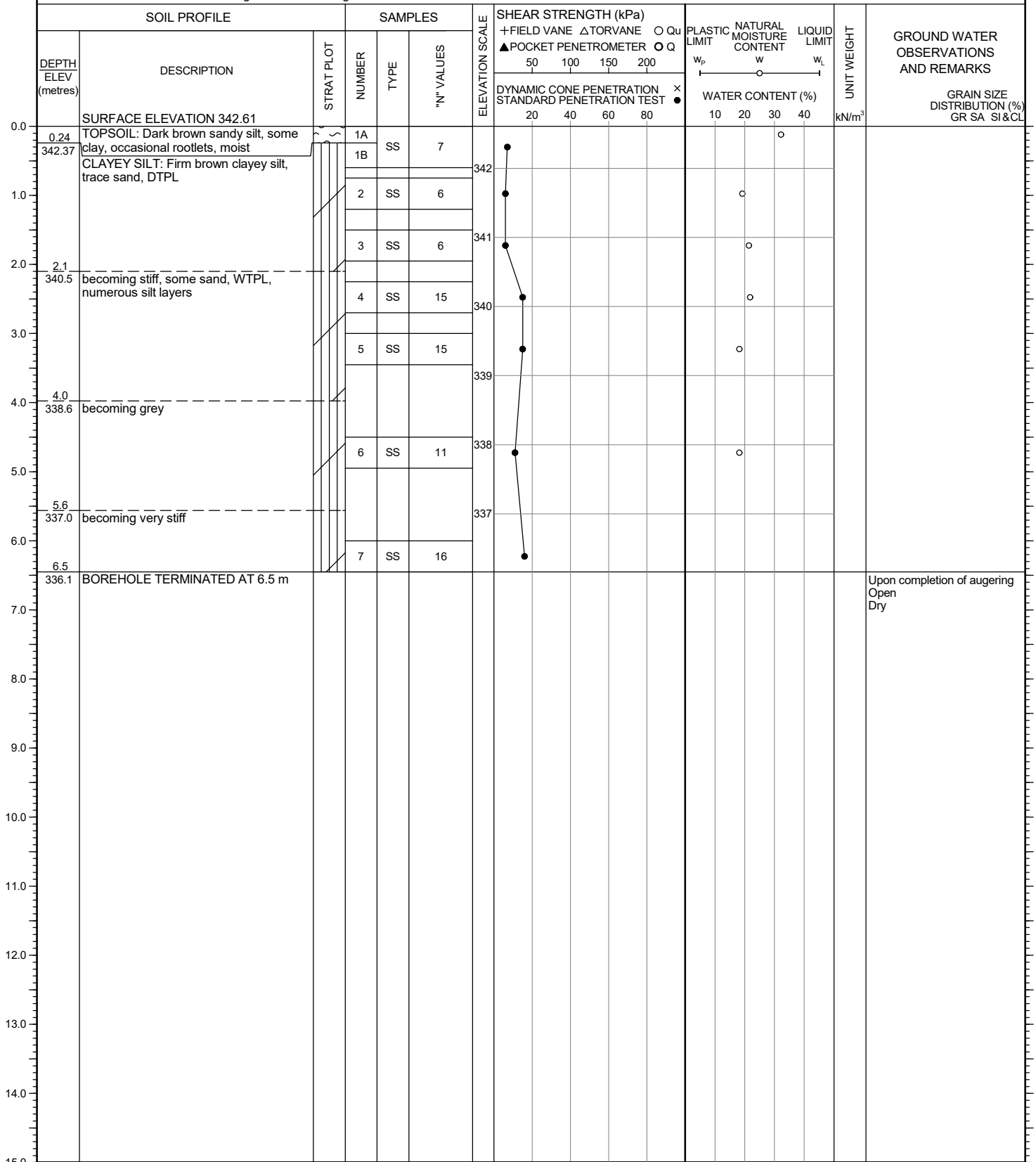
LOG OF BOREHOLE NO. 4

17T 524118E 4804016N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE July 30, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES

LOG OF BOREHOLE NO. 5

17T 523995E 4804385N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE July 31, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt

SOIL PROFILE			SAMPLES			ELEVATION SCALE	SHEAR STRENGTH (kPa)				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT kN/m ³	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH ELEV (metres)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		+ FIELD VANE	Δ TORVANE	○ Qu	▲ POCKET PENETROMETER	○ Q				
0.0	SURFACE ELEVATION 341.78														
0.31 341.47	TOPSOIL: Dark brown sandy silt, some clay, occasional rootlets, moist		1A	SS	13										
	CLAYEY SILT: Stiff brown clayey silt, some sand, DTPL		1B												
1.0			2	SS	11										
1.4 340.4	numerous silt layers														
2.0			3	SS	13										
2.6 339.2	becoming grey, WTPL		4	SS	14										
3.0			5	SS	9										
4.0															
5.0			6	SS	11										
6.0															
6.5 335.3	BOREHOLE TERMINATED AT 6.5 m		7	SS	13										
7.0															Upon completion of augering Open Dry
8.0															
9.0															
10.0															
11.0															
12.0															
13.0															
14.0															
15.0															

NOTES

LOG OF BOREHOLE NO. 6

17T 524153E 4804198N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE July 31, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt

SOIL PROFILE			SAMPLES			ELEVATION SCALE	SHEAR STRENGTH (kPa)				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT kN/m ³	GROUND WATER OBSERVATIONS AND REMARKS	
DEPTH ELEV (metres)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		DYNAMIC CONE PENETRATION STANDARD PENETRATION TEST									WATER CONTENT (%)
							+ FIELD VANE	Δ TORVANE	○ Qu	▲ POCKET PENETROMETER						
0.0	SURFACE ELEVATION 343.61					20	40	60	80	10	20	30	40			
0.25	TOPSOIL: Dark brown sandy silt, some clay, occasional rootlets, moist		1A	SS	7											
343.36	SILTY SAND: Compact brown silty sand, trace gravel, damp		1B			343										
1.0			2A	SS	17											
1.1	SILT: Compact brown silt, trace clay, trace sand, wet		2B													
1.4																
342.2	SAND: Compact brown sand, some silt, wet		3	SS	20	342										
2.0																
2.1	becoming saturated															
341.5			4	SS	19	341										
3.0			5	SS	22	340										
4.0																
5.0			6	SS	28	339										
5.5																
338.1	CLAYEY SILT: Very stiff grey clayey silt, WTPL, numerous silt layers					338										
6.0																
6.5			7	SS	23											
337.1	BOREHOLE TERMINATED AT 6.5 m														Upon completion of augering Cave at 2.1 m Dry	
7.0																
8.0																
9.0																
10.0																
11.0																
12.0																
13.0																
14.0																
15.0																

NOTES

LOG OF BOREHOLE NO. 7

17T 524277E 4804038N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE July 30, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt

SOIL PROFILE			SAMPLES			ELEVATION SCALE	SHEAR STRENGTH (kPa)			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT kN/m ³	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH ELEV (metres)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		+ FIELD VANE Δ TORVANE ○ Qu							
							▲ POCKET PENETROMETER ○ Q							
							DYNAMIC CONE PENETRATION STANDARD PENETRATION TEST x							
						WATER CONTENT (%)								
0.0	0.22	SURFACE ELEVATION 341.57				20	40	60	80	10	20	30	40	
341.35		TOPSOIL: Dark brown sandy silt, trace clay, occasional rootlets, moist	1A	SS	10						○			
		CLAYEY SILT: Stiff to very stiff brown clayey silt, some sand, trace gravel, DTPL	1B											
1.0			2	SS	16						○			
1.4														
340.2		numerous silt layers	3	SS	14						○			
2.0														
2.1			4	SS	19						○			
339.5		becoming grey, WTPL												
			5	SS	16						○			
3.0														
4.0														
5.0			6	SS	9						○			
6.0														
6.5			7	SS	20						○			
335.1		BOREHOLE TERMINATED AT 6.5 m												
7.0														Upon completion of augering Open Dry
8.0														
9.0														
10.0														
11.0														
12.0														
13.0														
14.0														
15.0														

NOTES

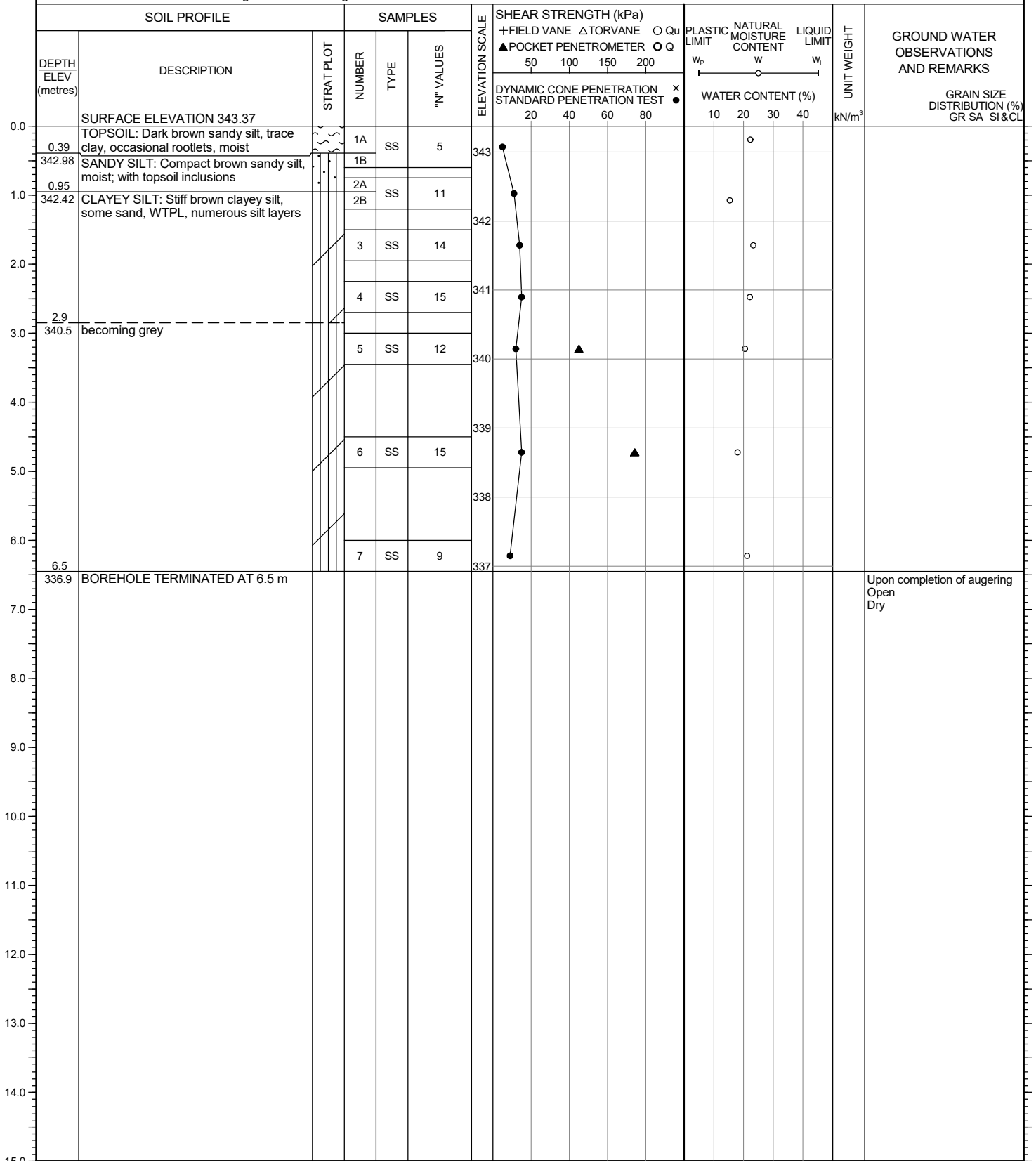
LOG OF BOREHOLE NO. 8

17T 524205E 4804333N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE August 1, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES

LOG OF BOREHOLE NO. 9

17T 524349E 4804152N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE August 1, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt

SOIL PROFILE			SAMPLES			ELEVATION SCALE	SHEAR STRENGTH (kPa)				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT kN/m ³	GROUND WATER OBSERVATIONS AND REMARKS	
DEPTH ELEV (metres)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		+ FIELD VANE Δ TORVANE ○ Qu ▲ POCKET PENETROMETER ○ Q									
							DYNAMIC CONE PENETRATION STANDARD PENETRATION TEST x									
							WATER CONTENT (%)									
							20	40	60	80	10	20	30	40	GRAIN SIZE DISTRIBUTION (%) GR SA Si & CL	
0.0	SURFACE ELEVATION 343.24															
0.27	TOPSOIL: Dark brown sandy silt, some clay, occasional rootlets, moist		1A	SS	5	343										
342.97			1B													
0.68	SILTY SAND: Compact brown silty sand, damp															
342.56																
1.0	SILT: Compact brown silt, trace sand, trace clay, moist to wet		2	SS	16											
						342										
			3	SS	17											
			4	SS	14											
2.9						341										
340.3	SILTY SAND: Compact brown silty sand, wet		5	SS	22	340										
4.0	becoming saturated															
339.2						339										
			6	SS	16											
5.5						338										
337.7	SANDY SILT: Compact grey sandy silt, trace clay, saturated															
6.0			7	SS	26	337										
6.5	BOREHOLE TERMINATED AT 6.5 m															
336.7																
7.0															Upon completion of augering	
															Free water at 2.3 m	
															Cave at 2.4 m	

NOTES

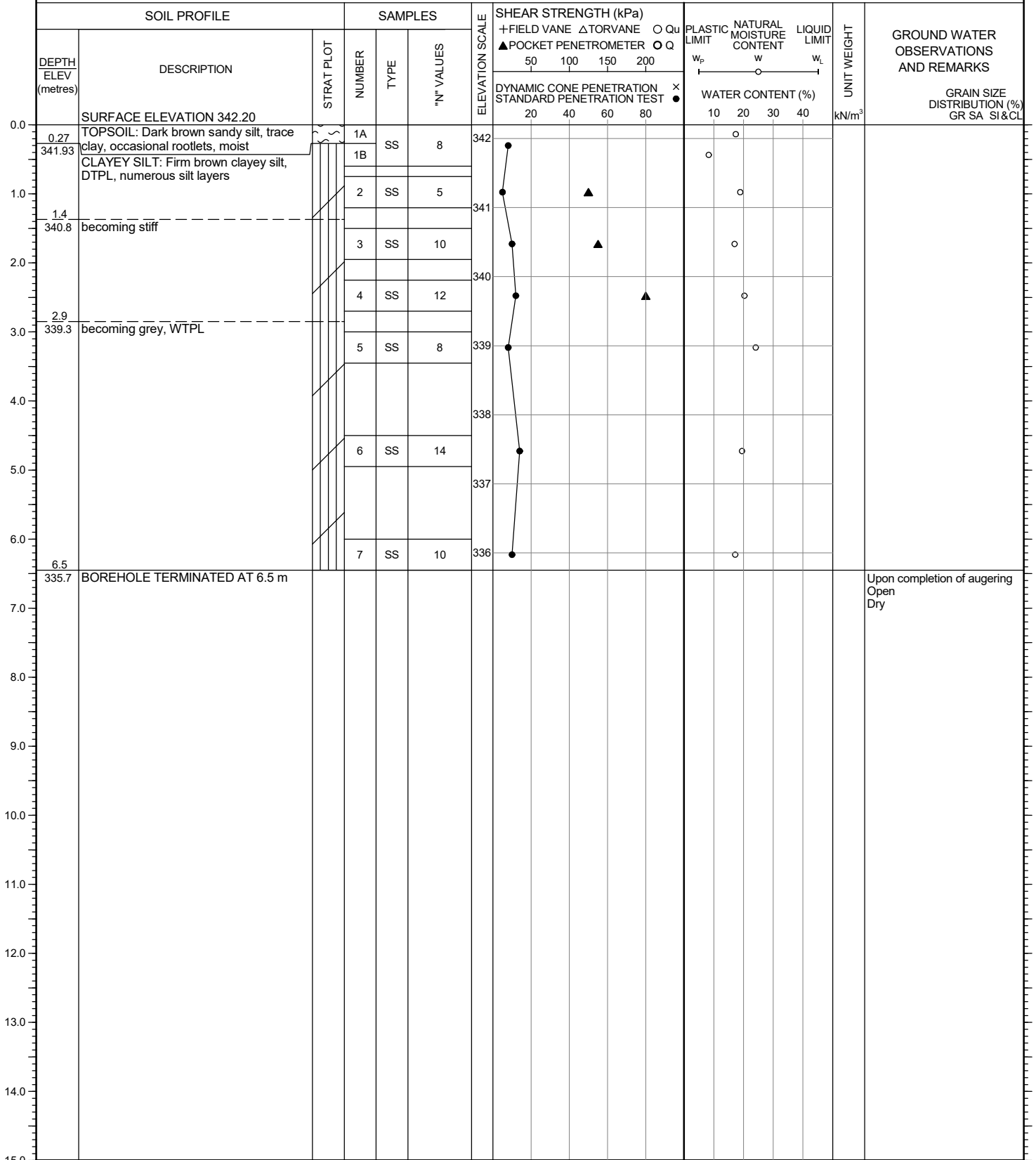
LOG OF BOREHOLE NO. 10

17T 524328E 4804366N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE August 2, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES

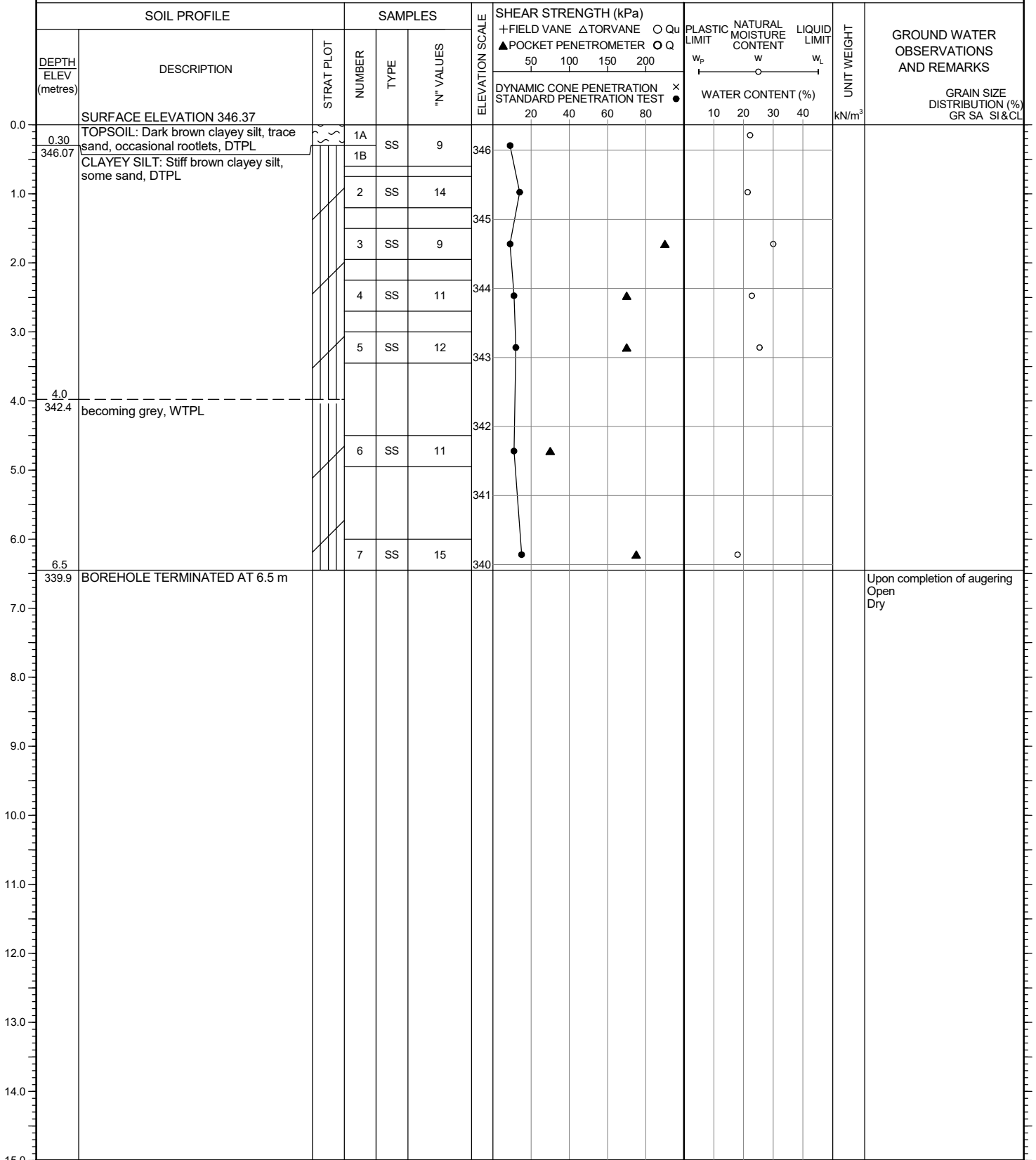
LOG OF BOREHOLE NO. 11

17T 524447E 4804190N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE August 2, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES

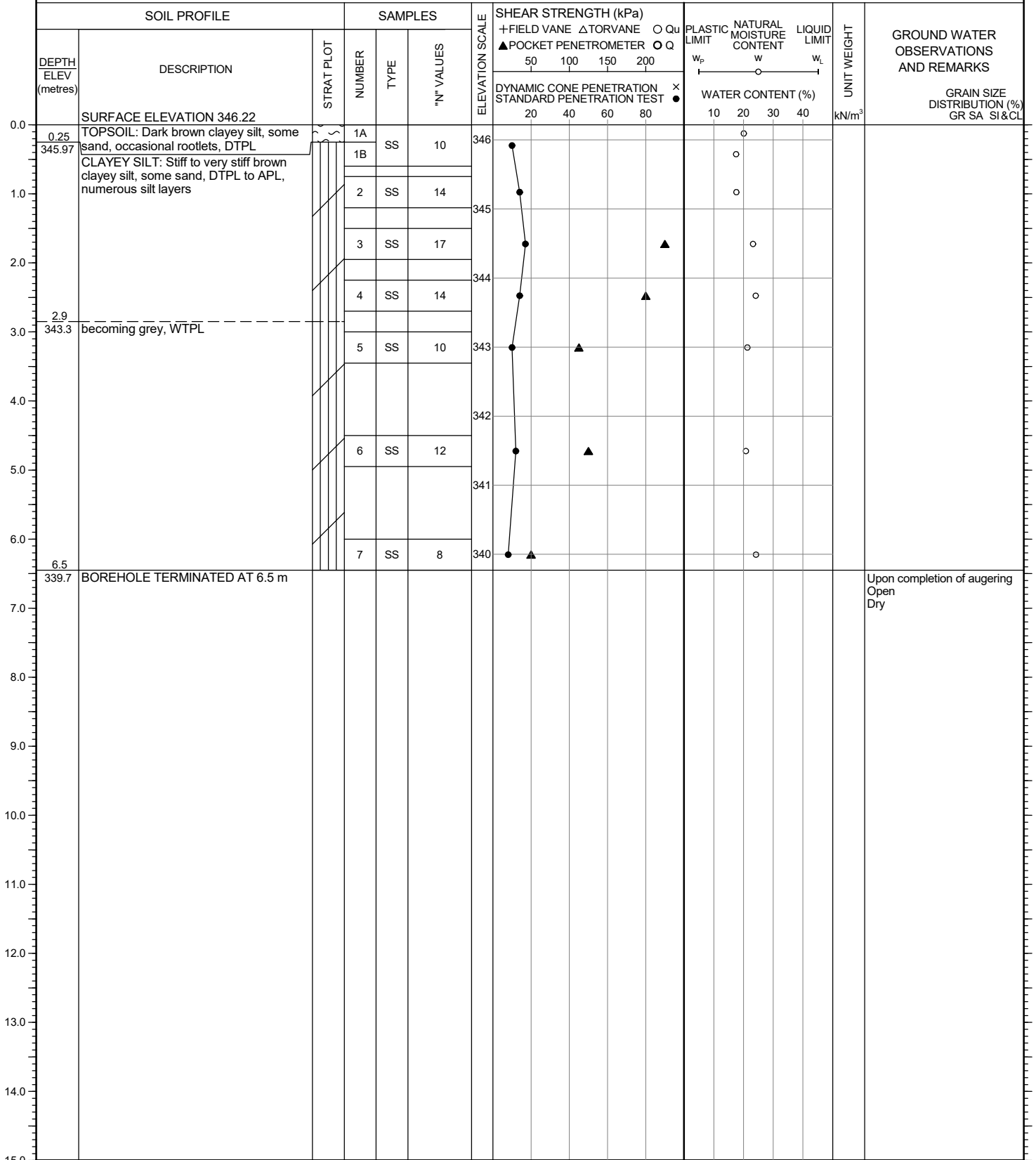
LOG OF BOREHOLE NO. 12

17T 524560E 4804228N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE August 2, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES

LOG OF BOREHOLE/MONITORING WELL NO. 101

17T 523684.7E 4804597N

PROJECT Wilmot Woods Development

LOCATION New Hamburg, Ontario

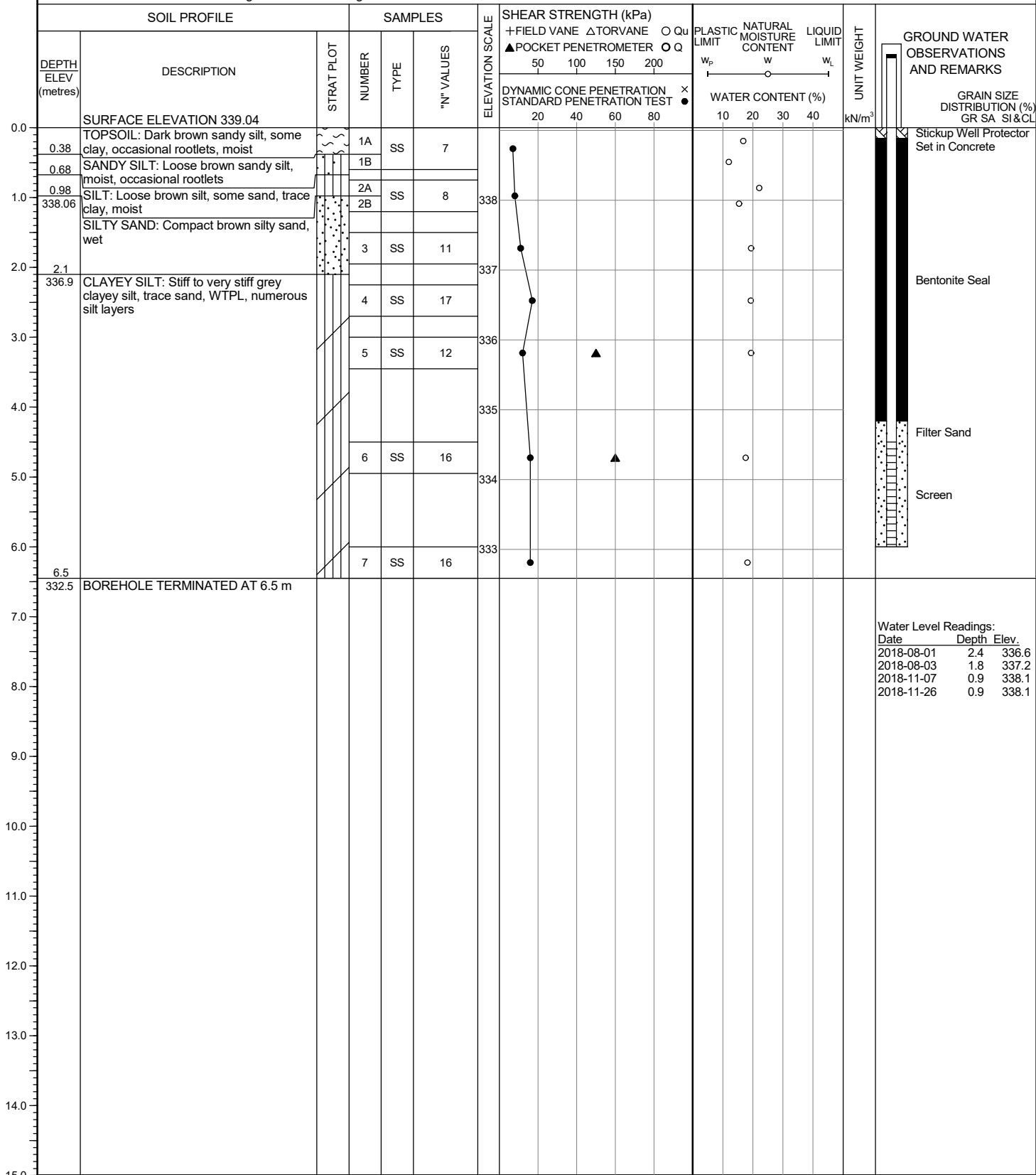
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE August 1, 2018

PML REF. 18KF031

ENGINEER H. Shinwary

TECHNICIAN K. Pettitt



Water Level Readings:

Date	Depth	Elev.
2018-08-01	2.4	336.6
2018-08-03	1.8	337.2
2018-11-07	0.9	338.1
2018-11-26	0.9	338.1

NOTES

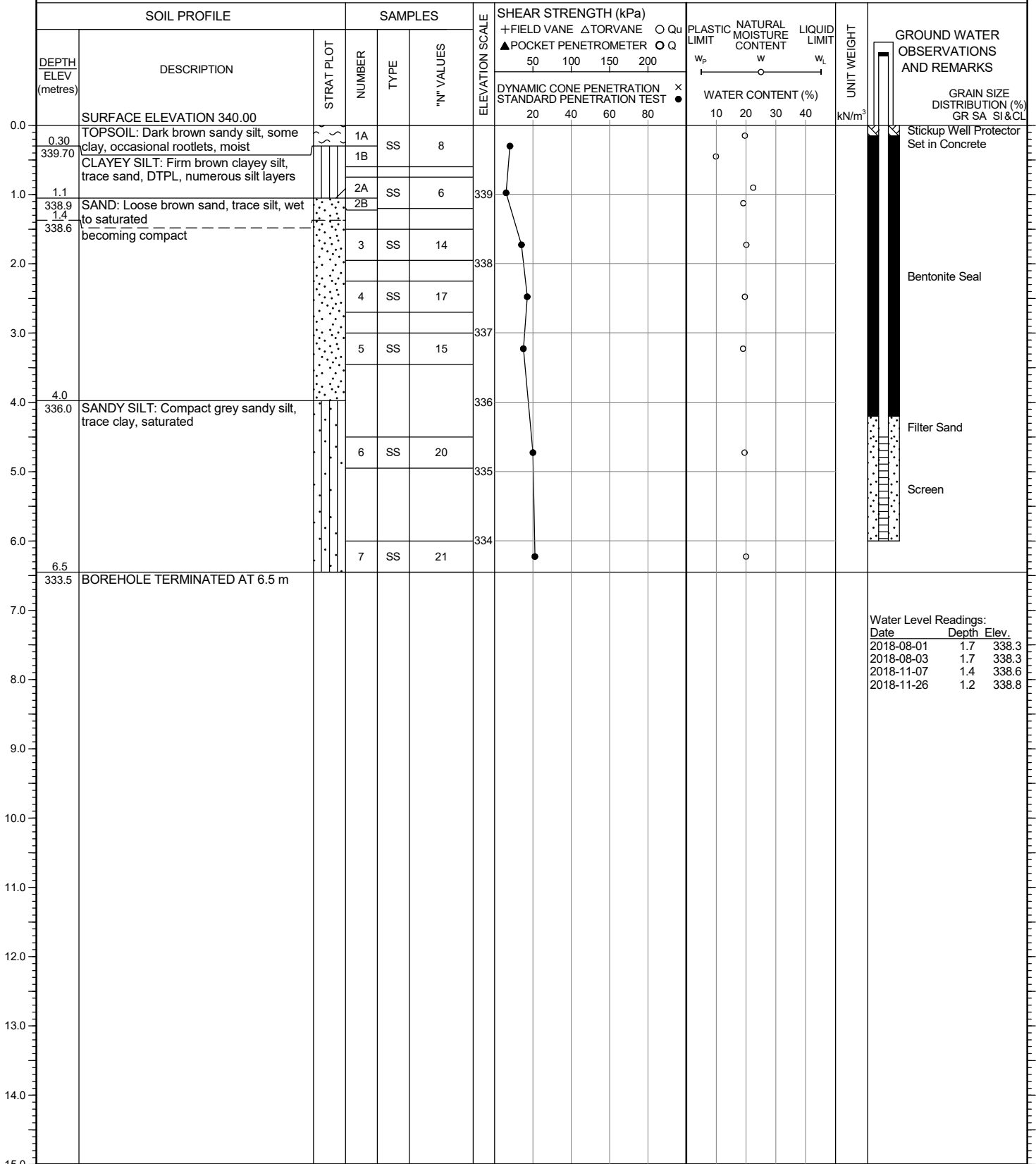
LOG OF BOREHOLE/MONITORING WELL NO. 102

17T 523788.7E 4804396N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE August 1, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES

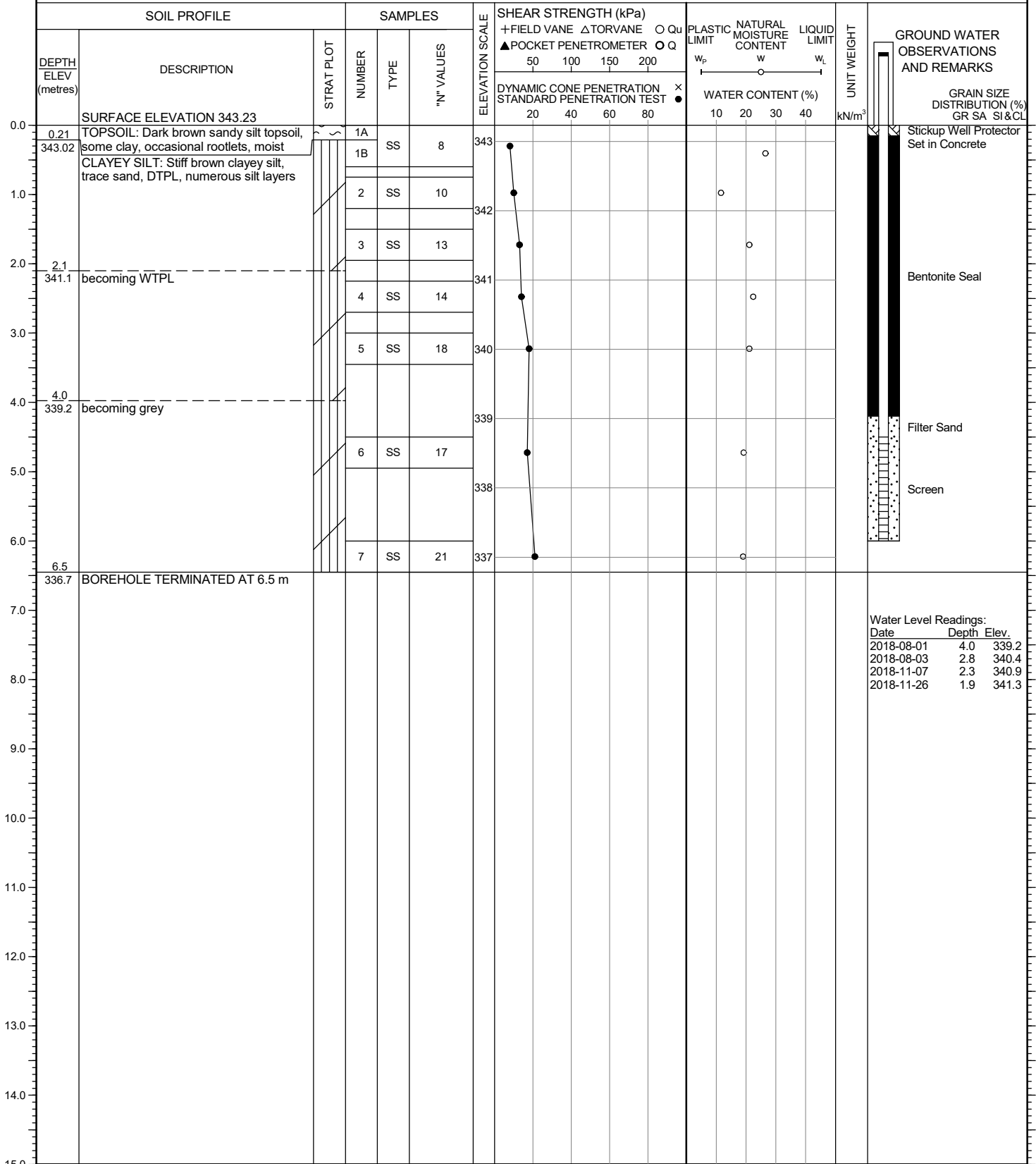
LOG OF BOREHOLE/MONITORING WELL NO. 103

17T 523910.3E 4804249N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE August 1, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES

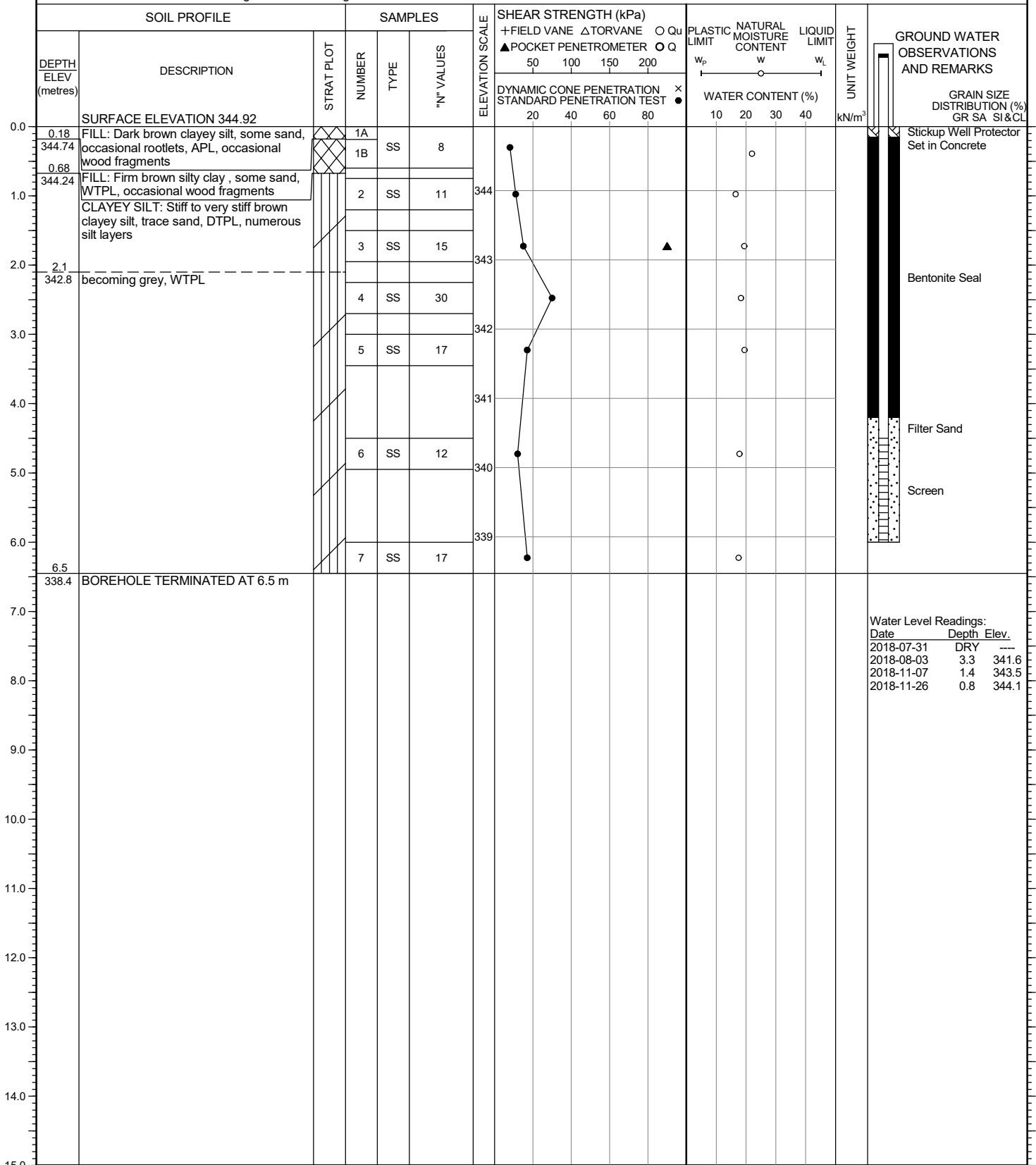
LOG OF BOREHOLE/MONITORING WELL NO. 104

17T 524180E 4803929N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE July 30, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES

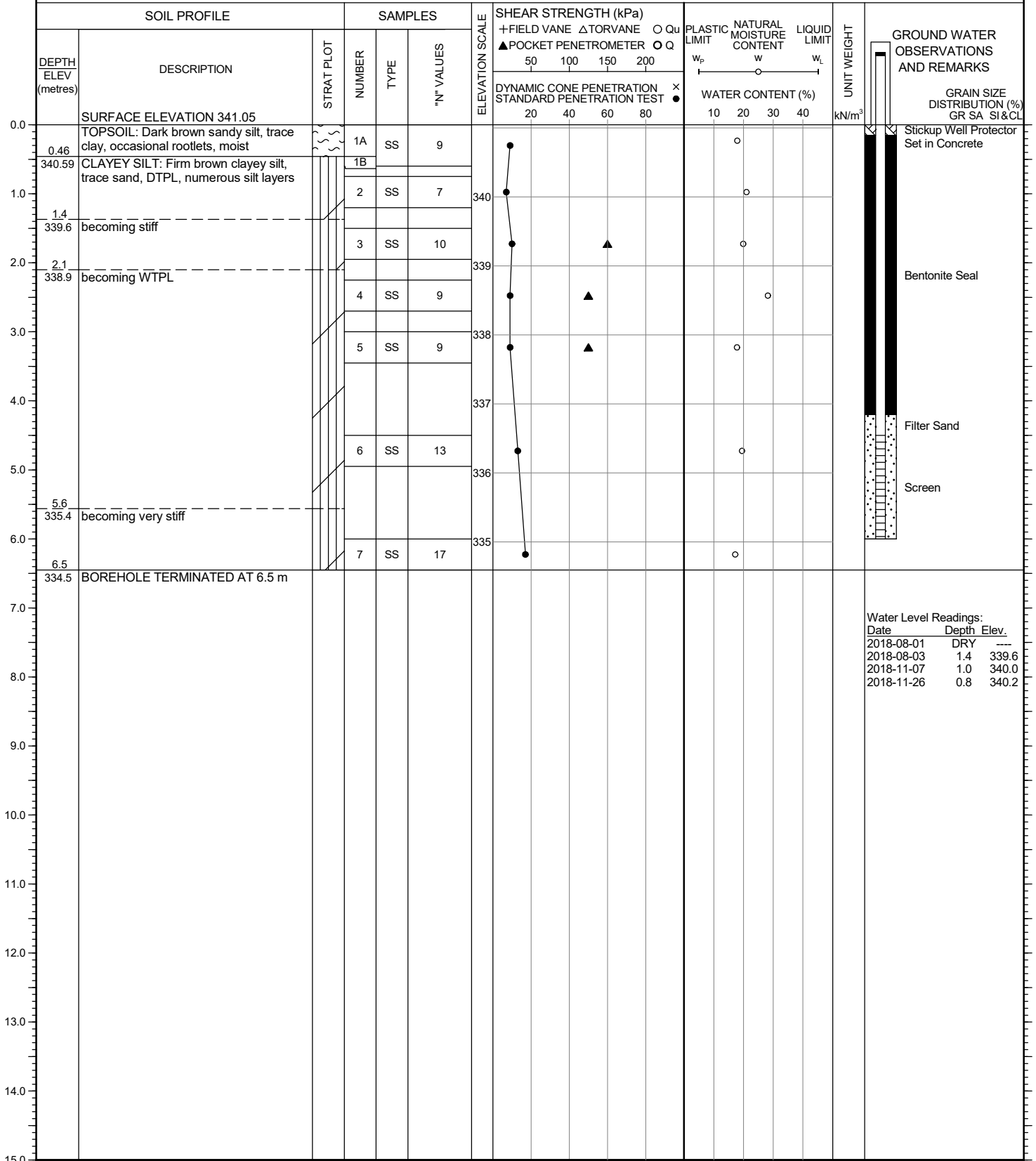
LOG OF BOREHOLE/MONITORING WELL NO. 105

17T 524125.6E 4804437N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE August 1, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES

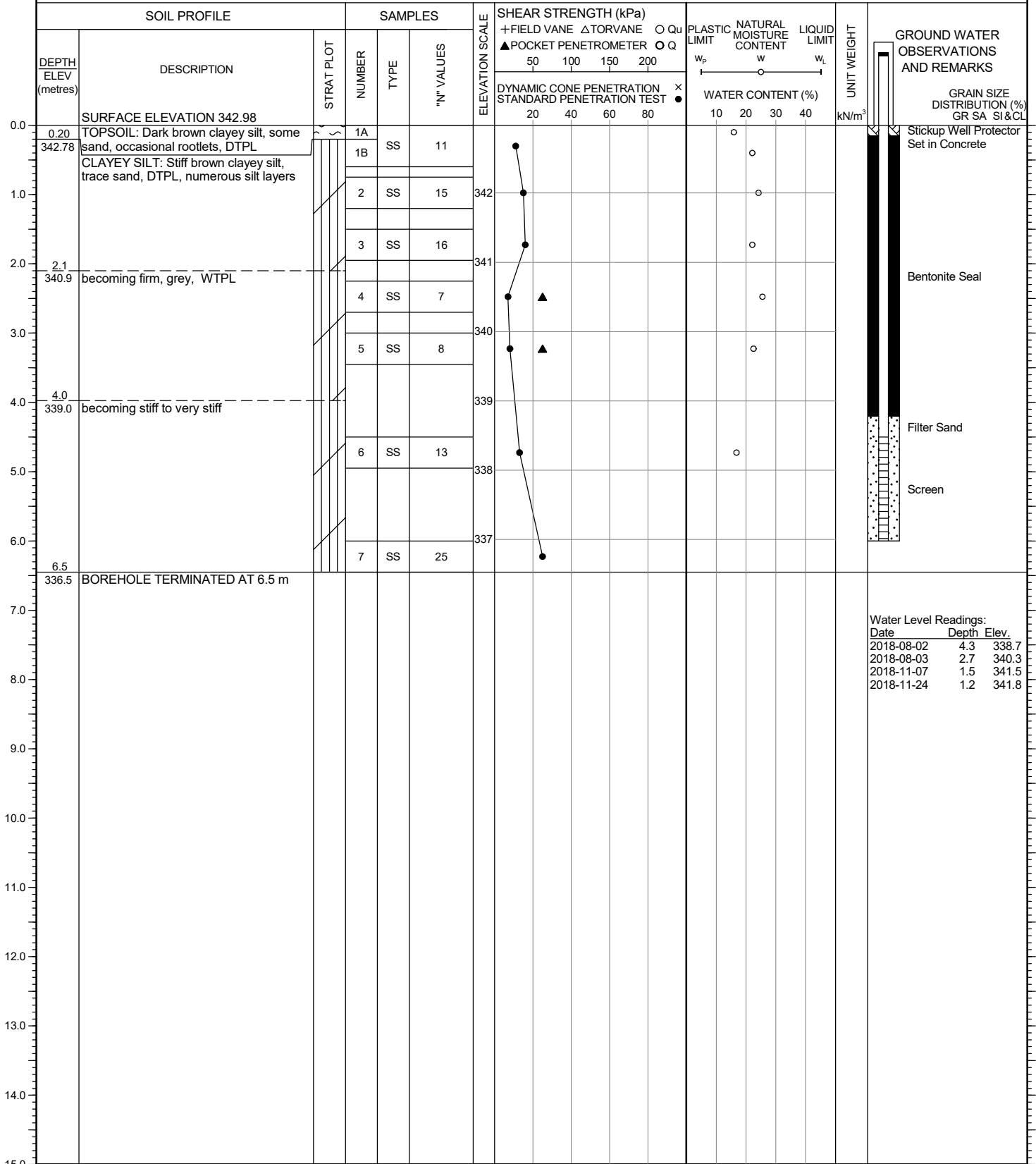
LOG OF BOREHOLE/MONITORING WELL NO. 106

17T 524544.9E 4804378N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE August 2, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



Water Level Readings:

Date	Depth	Elev.
2018-08-02	4.3	338.7
2018-08-03	2.7	340.3
2018-11-07	1.5	341.5
2018-11-24	1.2	341.8

NOTES

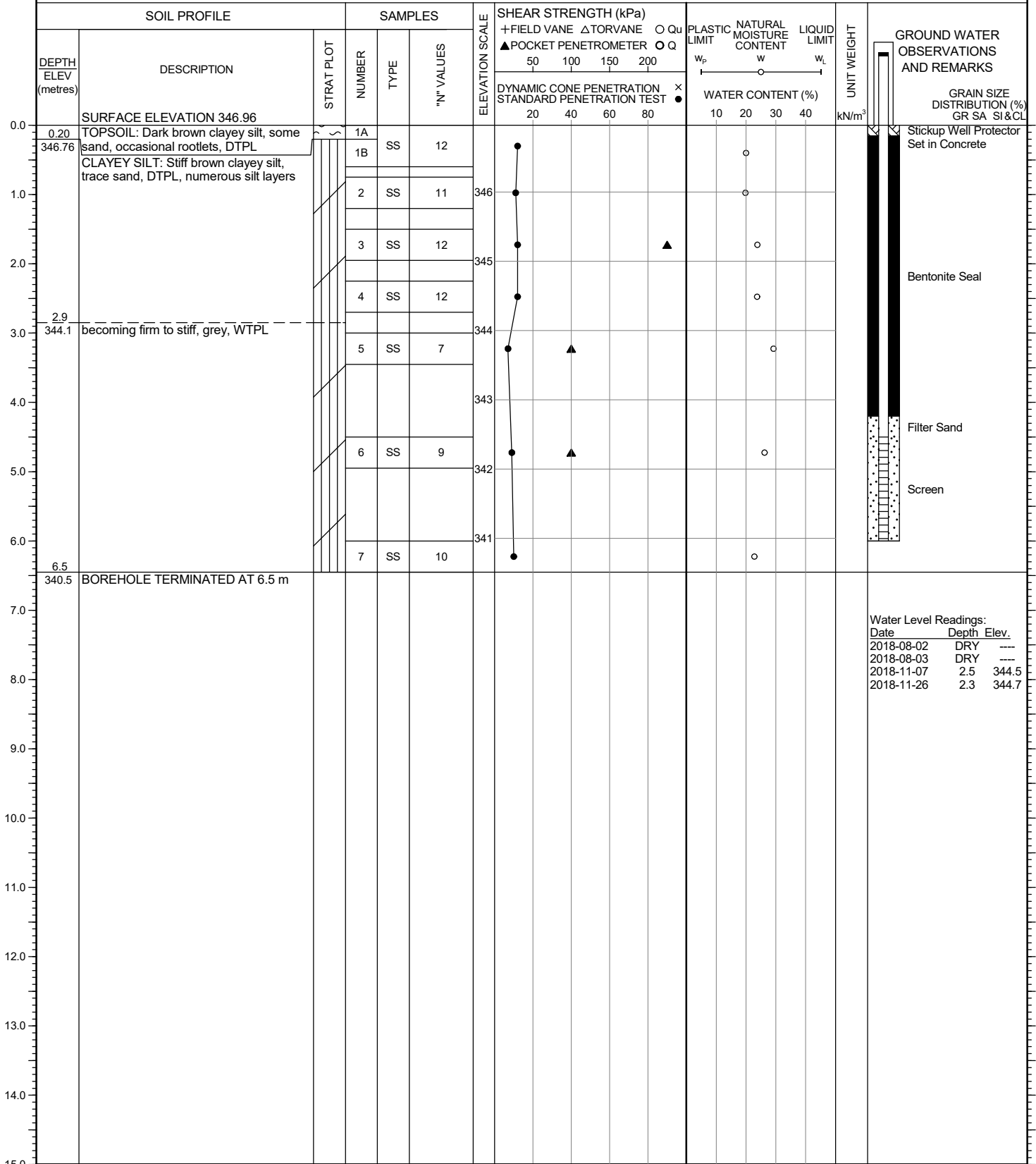
LOG OF BOREHOLE/MONITORING WELL NO. 107

17T 524733E 4804139N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE August 2, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES

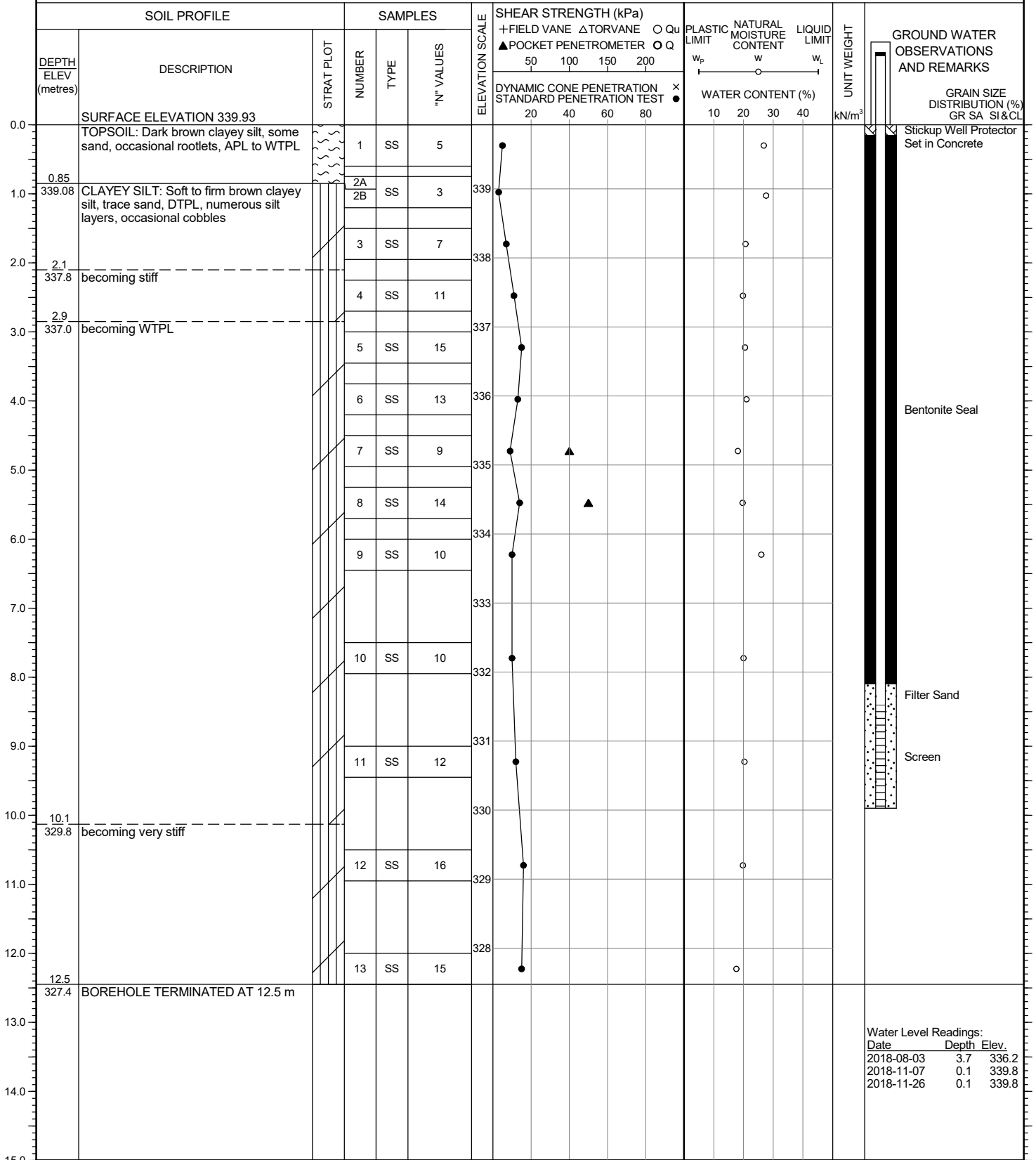
LOG OF BOREHOLE/MONITORING WELL NO. 109

17T 524441.1E 4803985N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

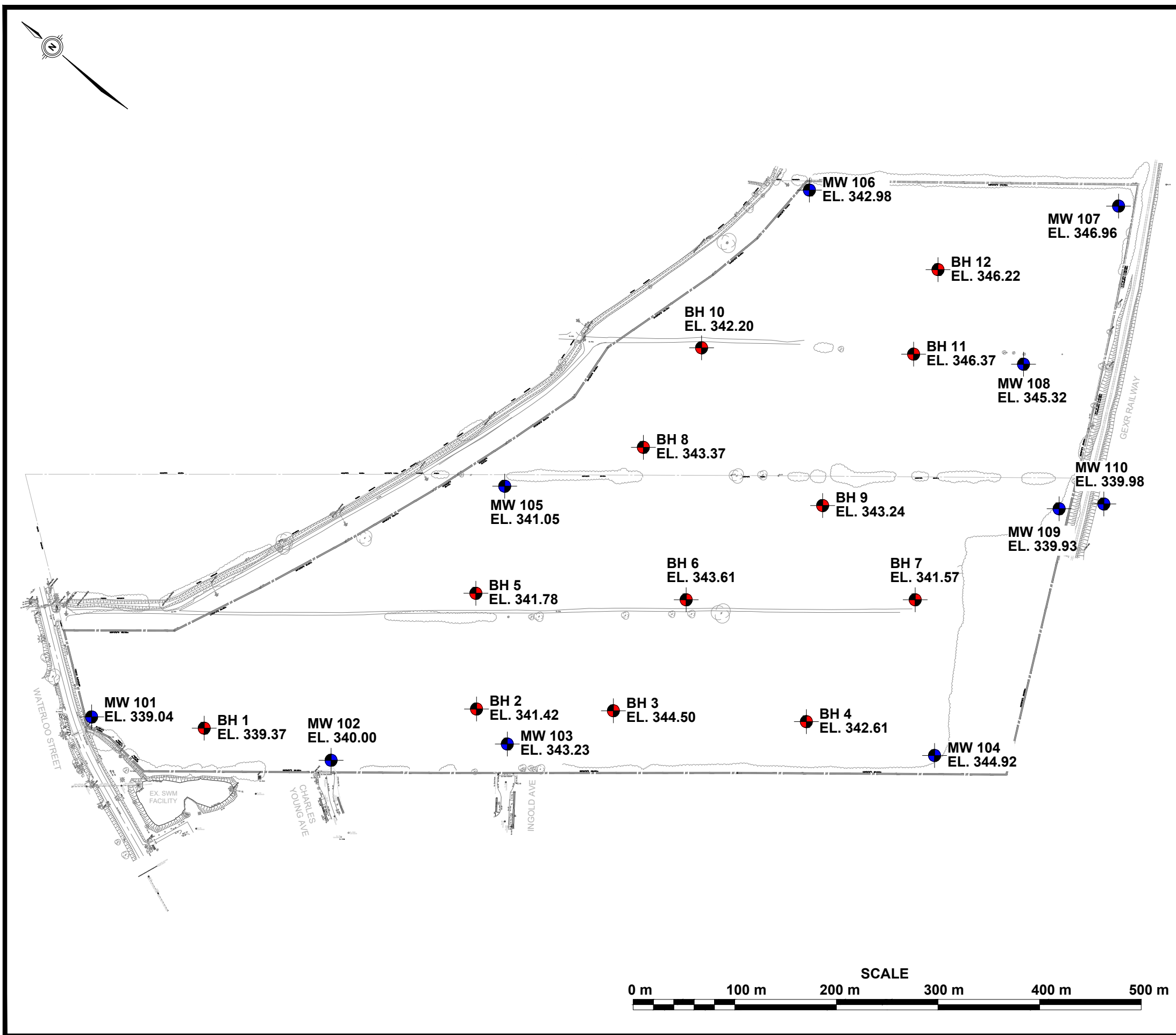
BORING DATE August 2, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



Water Level Readings:
Date Depth Elev.
2018-08-03 3.7 336.2
2018-11-07 0.1 339.8
2018-11-26 0.1 339.8

NOTES



LEGEND:

- BOREHOLE LOCATION
- MONITORING WELL LOCATION

REFERENCE:

BOREHOLE LOCATION PLAN REPRODUCED FROM DRAWING SUPPLIED BY CLIENT.

NOTE:

THE INFERRED STRATIGRAPHY REFERRED TO IN THE REPORT IS BASED ON THE DATA FROM THESE BOREHOLES SUPPLEMENTED BY GEOLOGICAL EVIDENCE. THE ACTUAL STRATIGRAPHY BETWEEN THE BOREHOLES MAY VARY.

WILMOT WOODS DEVELOPMENT INC.

PROPOSED WILMOT WOODS DEVELOPMENT

NEW HAMBURG

NEW HAMBURG, ONTARIO

BOREHOLE LOCATION PLAN

Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN	D. BRICE	DATE	SCALE	PML REF.	DWG. NO.
CHECKED	W. LOGHRIN	FEBRUARY 2022	AS SHOWN	18KF031	1
APPROVED	G. MITCHELL				



APPENDIX A

ENGINEERED FILL

The information presented in this appendix is intended for general guidance only. Site specific conditions and prevailing weather may require modification of compaction standards, backfill type or procedures. Each site must be discussed, and procedures agreed with Peto MacCallum Ltd. prior to the start of the earthworks and must be subject to ongoing review during construction. This appendix is not intended to apply to embankments. Steeply sloping ravine residential lots require special consideration.

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

1. Purpose

The site specific purpose of the engineered fill must be recognized. In advance of construction, all parties should discuss the project and its requirements and agree on an appropriate set of standards and procedures.

2. Minimum Extent

The engineered fill envelope must extend beyond the footprint of the structure to be supported. The minimum extent of the envelope should be defined from a geotechnical perspective by:

- at founding level, extend a minimum 1.0 m beyond the outer edge of the foundations, greater if adequate layout has not yet been completed as noted below; and
- extend downward and outward at a slope no greater than 45° to meet the subgrade

All fill within the envelope established above must meet the requirements of engineered fill in order to support the structure safely. Other considerations such as survey control, or construction methods may require an envelope that is larger, as noted in the following sections.

Once the minimum envelope has been established, structures must not be moved or extended without consultation with Peto MacCallum Ltd. Similarly, Peto MacCallum Ltd. should be consulted prior to any excavation within the minimum envelope.

3. Survey Control

Accurate survey control is essential to the success of an engineered fill project. The boundaries of the engineered fill must be laid out by a surveyor in consultation with engineering staff from Peto MacCallum Ltd. Careful consideration of the maximum building envelope is required.

During construction it is necessary to have a qualified surveyor provide total station control on the three dimensional extent of filling.

4. Subsurface Preparation

Prior to placement of fill, the subgrade must be prepared to the satisfaction of Peto MacCallum Ltd. All deleterious material must be removed and in some cases, excavation of native mineral soils may be required.

Particular attention must be paid to wet subgrades and possible additional measures required to achieve sufficient compaction. Where fill is placed against a slope, benching may be necessary and natural drainage paths must not be blocked.

5. Suitable Fill Materials

All material to be used as fill must be approved by Peto MacCallum Ltd. Such approval will be influenced by many factors and must be site and project specific. External fill sources must be sampled, tested and approved prior to material being hauled to site.

6. Test Section

In advance of the start of construction of the engineered fill pad, the Contractor should conduct a test section. The compaction criterion will be assessed in consultation with Peto MacCallum Ltd. for the various fill material types using different lift thicknesses and number of passes for the compaction equipment proposed by the Contractor.

Additional test sections may be required throughout the course of the project to reflect changes in fill sources, natural moisture content of the material and weather conditions.

The Contractor should be particularly aware of changes in the moisture content of fill material. Site review by Peto MacCallum Ltd. is required to ensure the desired lift thickness is maintained and that each lift is systematically compacted, tested and approved before a subsequent lift is commenced.

7. Inspection and Testing

Uniform, thorough compaction is crucial to the performance of the engineered fill and the supported structure. Hence, all subgrade preparation, filling and compacting must be carried out under the full time inspection by Peto MacCallum Ltd.

All founding surfaces for all buildings and residential dwellings or any part thereof (including but not limited to footings and floor slabs) on structural fill or native soils must be inspected and approved by PML engineering personnel prior to placement of the base/subbase granular material and/or concrete. The purpose of the inspection is to ensure the subgrade soils are capable of supporting the building/house foundation and floor slab loads and to confirm the building/house envelope does not extend beyond the limits of any structural fill pads.

8. Protection of Fill

Fill is generally more susceptible to the effects of weather than natural soil. Fill placed and approved to the level at which structural support is required must be protected from excessive wetting, drying, erosion or freezing. Where adequate protection has not been provided, it may be necessary to provide deeper footings or to strip and recompact some of the fill.

9. Construction Delay Time Considerations

The integrity of the fill pad can deteriorate due to the harsh effects of our Canadian weather. Hence, particular care must be taken if the fill pad is constructed over a long time period.

It is necessary therefore, that all fill sources are tested to ensure the material compactability prior to the soil arriving at site. When there has been a lengthy delay between construction periods of the fill pad, it is necessary to conduct subgrade proof rolling, test pits or boreholes to verify the adequacy of the exposed subgrade to accept new fill material.

When the fill pad will be constructed over a lengthy period of time, a field survey should be completed at the end of each construction season to verify the areal extent and the level at which the compacted fill has been brought up to, tested and approved.

In the following spring, subexcavation may be necessary if the fill pad has been softened attributable to ponded surface water or freeze/thaw cycles.

A new survey is required at the beginning of the next construction season to verify that random dumping and/or spreading of fill has not been carried out at the site.

10. Approved Fill Pad Surveillance

It should be appreciated that once the fill pad has been brought to final grade and documented by field survey, there must be ongoing surveillance to ensure that the integrity of the fill pad is not threatened.

Grading operations adjacent to fill pads can often take place several months or years after completion of the fill pad.

It is imperative that all site management and supervision staff, the staff of Contractors and earthwork operators be fully aware of the boundaries of all approved engineered fill pads.

Excavation into an approved engineered fill pad should never be contemplated without the full knowledge, approval and documentation by the geotechnical consultant.

If the fill pad is knowingly built several years in advance of ultimate construction, the areal limits of the fill pad should be substantially overbuilt laterally to allow for changes in possible structure location and elevation and other earthwork operations and competing interests on the site. The overbuilt distance required is project and/or site specified.

Iron bars should be placed at the corner/intermediate points of the fill pad as a permanent record of the approved limits of the work for record keeping purposes.

11. Unusual Working Conditions

Construction of fill pads may at times take place at night and/or during periods of freezing weather conditions because of the requirements of the project schedule. It should be appreciated therefore, that both situations present more difficult working conditions. The Owner, Contractor, Design Consultant and Geotechnical Engineer must be willing to work together to revise site construction procedures, enhance field testing and surveillance, and incorporate design modifications as necessary to suit site conditions.

When working at night there must be sufficient artificial light to properly illuminate the fill pad and borrow areas.

Placement of material to form an engineered fill pad during winter and freezing temperatures has its own special conditions that must be addressed. It is imperative that each day prior to placement of new fill, the exposed subgrade must be inspected and any overnight snow or frozen material removed. Particular attention should be given to the borrow source inspection to ensure only nonfrozen fill is brought to the site.

The Contractor must continually assess the work program and have the necessary spreading and compacting equipment to ensure that densification of the fill material takes place in a minimum amount of time. Changes may be required to the spreading methods, lift thickness, and compaction techniques to ensure the desired compaction is achieved uniformly throughout each fill lift.

The Contractor should adequately protect the subgrade at the end of each shift to minimize frost penetration overnight. Since water cannot be added to the fill material to facilitate compaction, it is imperative that densification of the fill be achieved by additional compaction effort and an appropriate reduced lift thickness. Once the fill pad has been completed, it must be properly protected from freezing temperatures and ponding of water during the spring thaw period.

If the pad is unusually thick or if the fill thickness varies dramatically across the width or length of the fill pad, Peto MacCallum Ltd. should be consulted for additional recommendations. In this case, alternative special provisions may be recommended, such as providing a surcharge preload for a limited time or increase the degree of compaction of the fill.



APPENDIX B

STATEMENT OF LIMITATIONS

STATEMENT OF LIMITATIONS



This report is prepared for and made available for the sole use of the client named. Peto MacCallum Ltd. (PML) hereby disclaims any liability or responsibility to any person or entity, other than those for whom this report is specifically issued, for any loss, damage, expenses, or penalties that may arise or result from the use of any information or recommendations contained in this report. The contents of this report may not be used or relied upon by any other person without the express written consent and authorization of PML.

This report shall not be relied upon for any purpose other than as agreed with the client named without the written consent of PML. It shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. A portion of this report may not be used as a separate entity: that is to say the report is to be read in its entirety at all times.

The report is based solely on the scope of services which are specifically referred to in this report. No physical or intrusive testing has been performed, except as specifically referenced in this report. This report is not a certification of compliance with past or present regulations, codes, guidelines and policies.

The scope of services carried out by PML is based on details of the proposed development and land use to address certain issues, purposes and objectives with respect to the specific site as identified by the client. Services not expressly set forth in writing are expressly excluded from the services provided by PML. In other words, PML has not performed any observations, investigations, study analysis, engineering evaluation or testing that is not specifically listed in the scope of services in this report. PML assumes no responsibility or duty to the client for any such services and shall not be liable for failing to discover any condition, whose discovery would require the performance of services not specifically referred to in this report.

The findings and comments made by PML in this report are based on the conditions observed at the time of PML's site reconnaissance. No assurances can be made and no assurances are given with respect to any potential changes in site conditions following the time of completion of PML's field work. Furthermore, regulations, codes and guidelines may change at any time subsequent to the date of this report and these changes may effect the validity of the findings and recommendations given in this report.

STATEMENT OF LIMITATIONS



The results and conclusions with respect to site conditions are therefore in no way intended to be taken as a guarantee or representation, expressed or implied, that the site is free from any contaminants from past or current land use activities or that the conditions in all areas of the site and beneath or within structures are the same as those areas specifically sampled.

Any investigation, examination, measurements or sampling explorations at a particular location may not be representative of conditions between sampled locations. Soil, ground water, surface water, or building material conditions between and beyond the sampled locations may differ from those encountered at the sampling locations and conditions may become apparent during construction which could not be detected or anticipated at the time of the intrusive sampling investigation.

Budget estimates contained in this report are to be viewed as an engineering estimate of probable costs and provided solely for the purposes of assisting the client in its budgeting process. It is understood and agreed that PML will not in any way be held liable as a result of any budget figures provided by it.

The Client expressly waives its right to withhold PML's fees, either in whole or in part, or to make any claim or commence any action or bring any other proceedings, whether in contract, tort, or otherwise against PML in anyway connected with advice or information given by PML relating to the cost estimate or Environmental Remediation/Cleanup and Restoration or Soil and Ground Water Management Plan Cost Estimate.



**GEOTECHNICAL INVESTIGATION
PROPOSED WILMOT WOODS DEVELOPMENT
CN RAILWAY CROSSING
NEW HAMBURG, ONTARIO**

for

WILMOT WOODS DEVELOPMENT INC.

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PML Ref.: 18KF031
Report: 2
February 24, 2022

February 24, 2022

PML Ref.: 18KF031
Report: 2

Mr. Adam Belksy
Wilmot Woods Development Inc.
310 Fairway Road South, P.O. Box 45016
Kitchener, Ontario
N2C 2R6

Dear Mr. Belksy

Geotechnical Investigation
Proposed Wilmot Woods Development
CN Railway Crossing
New Hamburg, Ontario

Peto MacCallum Ltd. (PML) is pleased to report the results of the geotechnical investigation recently completed at the above noted project site. Authorization to proceed with this assignment was provided by Mr. Galbraith in email dated July 9, 2018.

In general, the project involves the proposed installation of a proposed trunk sanitary sewer extending from the proposed Pfenning Farm development southward under the railway, and will connect to a proposed trunk sewer on the neighboring property (the Wilmot employment Lands). The railway which runs along the south side of the Pfenning Farm development site is located on top of an embankment at about Elevation 345. It is understood that a trenchless installation using jack and bore or pipe ramming methods is being considered for the railway crossing. Detailed design of the crossing has not been provided. However, in general, sewer crossing under the railway will comprise a 400 mm diameter pipe with inverts at about Elevation 335.7, about 9 m below the railway and 4 to 5 m depth below existing grade beyond the railway.

The purpose of this geotechnical report is to assess the existing subsurface soil and ground water information, and provide preliminary geotechnical engineering recommendations for the proposed trenchless installation of the trunk sanitary sewer under the railway.

Recommendations for the proposed construction of a residential subdivision are provided under separate cover, PML Ref. 18KF031, Report 1.

The comments and recommendations provided in this report are based on the site conditions at the time of the investigation, and are for the current project only. Any changes in plans will require review by PML to assess the applicability of the report, and may require modified recommendations, additional analysis and / or investigation. When the project design is complete, the general recommendations given in this report should be reviewed by PML to ensure their applicability.



Investigation

The field work for this geotechnical investigation was conducted on August 2, 2018 and October 15, 2018 as part of the investigation for the entire development. The investigation program for the railway crossing comprised two boreholes (MW109 and MW110) advanced to between 12.5 and 12.6 m depth, with monitoring wells installed in both of the boreholes, at the locations shown on the appended Drawing 1. Details of the geotechnical investigation are presented under separate cover in our previous report PML Ref 18KF031 Report 1. Reference is given to Appendix A for the Log of Borehole Sheets, and particle size analyses applicable to the current Railway crossing.

The borehole locations were established in the field by PML. Borehole locations were surveyed by PML using Global Navigation Satellite System (GNSS). The survey equipment was provided by SOKKIA Canada, model GCX-2.

The boreholes were advanced using a Diedrich D-50 track mounted drillrig fitted with continuous flight solid and hollow stem augers and automatic hammer, supplied and operated by a specialist drilling contractor. The work was carried out under the full-time supervision of a PML engineering staff member who directed the drilling and sampling operations, documented the soil stratigraphy, monitored ground water conditions and processed the recovered samples.

Representative samples of the overburden were recovered at regular intervals throughout the depths explored. Standard penetration tests (SPT) were carried out during sampling operations of the boreholes using conventional split spoon equipment. Pocket penetrometer testing was carried out on the recovered samples to determine the undrained shear strength of the cohesive soils. Ground water observations were made in the boreholes during and upon completion of drilling. The boreholes were backfilled and compacted in accordance with O.Reg.903 upon completion of drilling.

Monitoring wells were installed in Boreholes MW109 and MW110 to more accurately measure ground water levels. The monitoring wells comprised 50 mm diameter PVC pipe, filter sand,

All of the recovered samples were returned to PML's laboratory for detailed visual examination, classification, and routine moisture content determinations. The laboratory testing also included particle size distribution analyses on samples of the major soil type encountered.



Summarized Subsurface Conditions

Reference is made to the appended Log of Borehole sheets for details of the field work including soil descriptions, inferred stratigraphy, standard penetration test (SPT) N values, dynamic cone penetration test values, ground water observations and laboratory moisture content determinations.

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.

In general, the soil stratigraphy encountered comprised surficial topsoil, underlain by an extensive clayey silt deposit containing numerous cohesionless silt layers.

Surficial topsoil was contacted in both boreholes and was between 850 mm and 360 mm thick in Boreholes MW109 and MW110, respectively.

An extensive clayey silt deposit was encountered below the surficial topsoil, in both boreholes, and extended to the 12.5 to 12.6 m borehole termination depths. The cohesive clayey silt deposit was generally firm to very stiff based on standard penetration N values between 7 and 18 blows per 0.3 m penetration of the split spoon sampler. Pocket penetrometer shear strengths of the clayey silt ranged between 75 and 125 kPa. Moisture content ranged between 18 and 28% indicating drier than plastic limit (DTPL) to wetter than limit (WTPL) conditions in the cohesive clayey silt soils. Occasional cobbles and boulders are present in the clayey silt deposit.

Ground Water Conditions

Ground water observations carried out during and upon completion of drilling are presented on the appended Log of Borehole Sheets.

During drilling, WTPL conditions were generally encountered in the clayey below 2.9 m depth (Elevation 337.0 and 337.1).

On November 26, 2018 water level measurements from the monitoring wells installed in Boreholes 109 and 110 ranged between 0.1 to 0.2 m depth below existing grade (about Elevation 339.8).



The ground water levels at the site are subject to seasonal fluctuations and precipitation patterns. It should be noted that the relatively impermeable nature of the clayey silt deposits could contribute to the development of perched water conditions following short term and seasonal participation events.

Discussion and Recommendations

As noted previously, the project involves the proposed installation of a trunk sewer below the railway tracks which will extend southward from the Pfenning Farm to the Wilmot employment Lands. The crossing will house a 400 mm diameter pipe inside a steel casing with proposed inverts about Elevations 335.7. The sewer pipe and casing invert will be about 9 m below the railway tracks and 4 to 5 m below the existing grades on each side of the railway. For discussion purposes a maximum 900 mm diameter steel casing is envisioned. Therefore, the top of the steel pipe crossing will be about 4 m below ground on the lands beside the railway, and about 8 m below the railway tracks.

In general, the soil stratigraphy encountered comprised surficial topsoil, underlain by an extensive clayey silt deposit containing numerous cohesionless silt layers and occasional cobbles. The cohesive clayey silt deposit was generally firm to very stiff. The ground water level is at about Elevation 339.8, and above the proposed trunk sewer depth. Considering the subsurface conditions encountered and details proposed for the crossing, both jack and bore and pipe ramming methods were evaluated, and further details on both methods are provided in the sections below.

Regardless of the method used, it is recommended that the contractor prepare a plan in advance of construction outlining the details of the installation to provide instructions for the construction crews and provide a possible contingency action plan should difficulties occur during the tunnelling operations. The plan should also be reviewed by the project proponent and the railway authorities prior to construction. Upon request, PML can assist in reviewing the plan to check that assumptions regarding soil and ground water conditions are appropriate.

Reference is also given to Ontario Provincial Standard Specifications (OPSS) 415, Construction Specifications for Pipeline and Utility Installation by Tunnelling.



Jack and Bore

Jack and bore (or cased auger boring) typically involves the simultaneous advancement of a continuous flight auger and conduit pipe. The auger is used to excavate soil in advance of the casing and transport cuttings back to the receiving pit where they are removed. Rotary power to auger and pushing force is provided by a drill rig located within a jacking pit. Jack and bore is a common method of trenchless installation and in appropriate site and soil conditions may be preferable from a cost perspective.

Jack and bore installation(s) should be conducted in accordance with OPSS 416, Construction Specifications for Pipeline and Utility Installation by Jacking and Boring.

A jack and bore installation is technically feasible for the installations provided the work is carried out by contractors experienced with tunneling in clayey silt with layered silt conditions, and provided adequate measures are used to prevent loss of ground.

The sanitary sewer installation will be below the ground water level in the clayey silt, and ground water conditions could hinder or prevent a jack and bore installation. In wet soils there is potential for ground surface subsidence due to wet soil flowing into the bore, which could result in voids. Dewatering the clayey silt with numerous silt layers is not likely to be effective, therefore the risk of ravelling soil cannot be entirely eliminated.

The jacking forces should be estimated to select the suitable jacks. Suitable jacking head and suitable bracing between jacks and jacking head should be used to assure that pressure will be applied to the pipe uniformly around the ring of the pipe. Utilizing an effective lubrication system may minimize pipe friction during advance. A suitable face pressure or soil plug should be maintained to minimize loss of ground during advance. Any over-cut during augering and pipe advance which potentially create soil disturbance, space or voids outside the pipe should be grouted or filled to avoid potential ground movements.

The proposed crossing may encounter cobbles and boulders during installation. Contractors working on this assignment should provide measures to break up or remove cobbles and boulders if they are encountered during the jack and bore installation.



Pipe Ramming

Pipe ramming installation is analogous to driving an open-ended tube pile horizontally. Impact forces from a percussive hammer (mole) are used to advance a conduit pipe from an entry pit to a receiving pit. During the advance, most of the soil being penetrated fills the conduit rather than being excavated. The rammed conduit is terminated in a receiving pit at which point the soil contained in the pipe is removed.

When the driving has been completed, soil within the pipe can be removed via augering or a pipe shovel. Augering is expected to be the preferred method, however if soil within the pipe cannot be augered, use of a pipe shovel will be necessary. A pipe shovel is essentially a special scoop made from a pipe which fits inside the liner. Excavation via pipe shovel involves advancing the shovel into the soil plug using impact hammer (mole), then pulling the shovel and its contents out with a chain or cable, the process is repeated as required.

Reference is given to the following section for recommendations pertaining to the construction of entry and receiving pits. Dewatering measures will be required at the entry and receiving pits. Ideally, dewatering measures for the staging works should be established to such that the zone of influence includes as much of the tunnel area as possible as this will generally reduce potential ground water seepage through the tunnel during construction.

A pipe ramming installation is technically feasible for the sanitary sewer installation provided the work is carried out by contractors experienced with tunneling in clayey silt conditions, and adequate ground water control is achieved.

It is noted that grade adjustments cannot be completed once pipe ramming has started. It will be necessary to increase the slope of the crossing due to the potential for uncontrolled deflection during pipe ramming installation.

The proposed crossing may encounter cobbles and boulders during installation. Contractors working on this assignment should provide measures to break up cobbles and boulders if they are encountered during the pipe ramming installation.

The effects of the percussive forces on the railway must also be considered and assessed.



In summary, jack and bore is technically feasible for the proposed crossings, however pipe ramming is generally recommended as an alternative given the ground water conditions within the proposed tunneling depths.

Entry and Receiving Pit Excavation and Ground Water Control

Provided adequate ground water control is achieved, the subsurface soils encountered at the borehole locations are classified as Type 3 soils as defined in the Occupational Health and Safety Act (OHSA). For temporary unsupported excavations within Type 3 soils that are to be entered by workers, the excavation side slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V) from the base of shallow excavations. Workers should not enter an unprotected excavation if there is evidence of ongoing seepage in the banks or base of the excavation. An adequate temporary support system (i.e., trench boxes) should be provided at all locations where space limitations prevent construction of sufficiently shallow slopes.

It will also be important to ensure that the excavations do not undermine possible existing in-ground structures, roads and or other services in their proximity. The need for underpinning and utility support can be established according to criteria illustrated on the appended Figure 1. It should be noted that a trench liner box may not be relied on for bracing purposes.

The envisaged excavations for the sending and receiving pits of the sanitary sewer crossing will extend about 6.5 m into clayey silt with numerous wet silt seams. Excavations are anticipated to require dewatering to control of ground water seepage or surface water entering the excavation, and sump pumps along with a more sophisticated dewatering system such as well points, or eductor wells, or water tight sheet pile or secant caisson cut-off may be required.

Regardless of the dewatering method chosen, the hydraulic head and ground water inflow must be properly controlled to ensure stable and safe excavation and to facilitate construction. The design of the dewatering system should be left to the contractor's discretion, and the system should meet a performance specification to maintain and control ground water at least 0.30 m below the excavation base level, in order to provide a stable excavation base 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 Conservation and Parks (MECP) 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 MECP's PTTW Manual (April 2005), and application should be filed to the MECP 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). PML would be happy to assist with this process, if required.

The depth of excavations for trunk sewer installation are expected to extend to a maximum 6.5 m depth into clayey silt deposits with wet to saturated layers of silt. Due to the presence of numerous water bearing silt layers in the native deposits, excavations for the sending and receiving pits might have pumping rates that exceed 50,000 L/day, and a PTTW or EASR may be required, along with a hydrogeological investigation in support of the application. The estimated construction dewatering volumes should be reassessed once more detailed design information has been established. Upon request, PML would be please to provide further recommendations in this regard.

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 to be expected. Also, the dewatering requirements should be established by the contractor in the context of a performance specification.



Braced Excavations

If space is not available for an enlarged unsupported open cut excavation then consideration should be given to using an engineered system (timber lagging, sheet piling or caisson wall) to support the excavation walls. The soil parameters presented in the following table may be used for the design of braced excavation systems. The parameters are based on the Rankin method of analysis which ignores wall friction.

SOIL TYPE	CLAYEY SILT
Coefficient of Active Earth Pressure K_a	0.33
Coefficient of Passive Earth Pressure K_p	3.00
Angle of Internal Friction ϕ	30
Cohesive Strength c_u	50 kPa
Unit Weight γ (kN/m ³)	21

For the soil stratigraphy encountered, the bracing system may be designed as a multi-braced system using a rectangular stress distribution in accordance with methods outlined in the latest Canadian Foundation Engineering Manual (CFEM) and summarized on the appended Figure 2, or as a singly-braced system using a triangular stress distribution and summarized on the appended Figure 3. The system design should also consider load effects from ground water, the adjacent embankments, structures and construction equipment.

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 practices are employed.

The maximum allowable movements of braced excavations must be established in a performance specification, and should be determined through agreement with the railway authority.



Backfill

In general, conventional bedding and cover will be suitable for sewer installation and backfilling of the sending and receiving pits. Material containing stones larger than 50 mm size should not be used in the bedding or cover layers. The bedding should be constructed in accordance with applicable Ontario Provincial Standard Drawings (OPSD) and / or local standards. The bedding and cover material should be placed in 150 mm lifts and compacted to at least 95% standard Proctor maximum dry density (SPMDD).

Backfill materials should comprise approved imported granular soil. The backfill materials should be placed in 300 mm maximum lifts and compacted to a minimum of 95% SPMDD.

Should construction extend to the winter months, care must be taken to ensure that frozen material is not used as backfill.

Excavated materials intended for backfilling purposes should not be exposed to the elements for prolonged time periods, as they might be rendered unsuitable for reuse.

Monitoring

The ground surface over the tunnel route may become distorted and distressed by tunnelling. The most common type of distress is settlement caused by loss of ground around the tunnel. Heave of the ground surface is also possible depending on the type of installation. Monitoring of lateral movement of ground near excavations may also be necessary. Mitigation of the distress or distortion on the railway would be a major inconvenience and possibly a safety issue.

Distress at the ground surface is generally prevented or minimized by good construction practices and proper planning. In this regard, preparation of an installation plan as noted above is recommended.



It is also recommended that the project proponent implement a monitoring program to check the condition of the ground over the tunnel before, during and upon completion of construction. The monitoring program should be carried out by a qualified geotechnical consulting firm that is familiar with the proposed work and subsurface conditions. The monitoring period should begin prior to tunnelling, extend throughout the duration and continue at least 2 weeks after completion of tunnelling. More specific monitoring procedures may also be required by the railway authorities.

Monitoring points should be marked using a method approved by the railway authority. Monitoring points should also be functional throughout the monitoring period and should not deteriorate because of traffic, maintenance activities, and weather conditions.

If distress is observed during construction, the contractor should be informed and corrective action should be undertaken immediately. Specific corrective action will be dependent on the nature of the distress and type of installation. Regardless, the process should be outlined in the monitoring program and be part of the contingency actions in the contractor's installation plan.

In general, settlement or heave of the railway from a properly constructed jack and bore or pipe ramming installation carried out in accordance with the recommendations noted in this report, should be less than 10 mm. If settlement or heave of the ground surface exceeds 10 mm, the construction process should be reviewed and adjusted to mitigate further disturbances for the remainder of the tunnelling work.

If total settlement or heave exceeds 15 mm, tunnelling operations should be terminated, the site secured against further deterioration and mitigative action should be undertaken immediately to reinstate the railway and existing utilities

All actions to prevent, secure, or mitigate destruction or damage to the highway and associated features should be done in accordance with, and approved by the railway authorities.



Geotechnical Review and Construction Inspection and Testing

When development design is complete, it is recommended that the design drawings be submitted to PML for general geotechnical review for compatibility with site conditions and recommendations of this report.

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, particularly where service trenches exist. 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.

This report is subject to the Statement of Limitations that is included in Appendix B, which must be read in conjunction with the report.



Closure

We trust the information presented in this report 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.
Manager Engineering Service



Gerry Mitchell, MEng, P.Eng.
Senior Consultant

WL/GM:tm

Enclosures:

- Figure 1 – General Recommendations Regarding Underpinning
- Figure 2 – Lateral Earth Pressure Distribution Multi-Braced Cuts in Cohesionless Soils
- Figure 3 – Lateral Earth Pressure Distribution Singly-Braced Cuts in Cohesionless Soils
- Drawing 1 - Borehole Location Plan
- Appendix A – Figure 6 and Borehole Logs BH/MW109 and BH/MW110
- Appendix B – Statement of Limitations



Closure

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Figure 1 – General Recommendations Regarding Underpinning
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Appendix A – Figure 6 and Borehole Logs BH/MW109 and BH/MW110
Appendix B – Statement of Limitations

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:

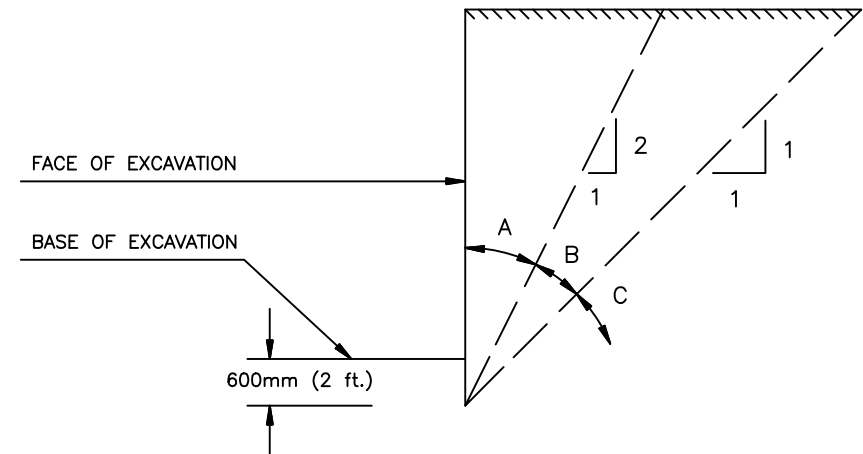
Foundations of structures located within Zone B generally do not require underpinning. Consideration should be given to underpinning of settlement sensitive utilities or heavy foundation units located in this zone.

ZONE C:

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

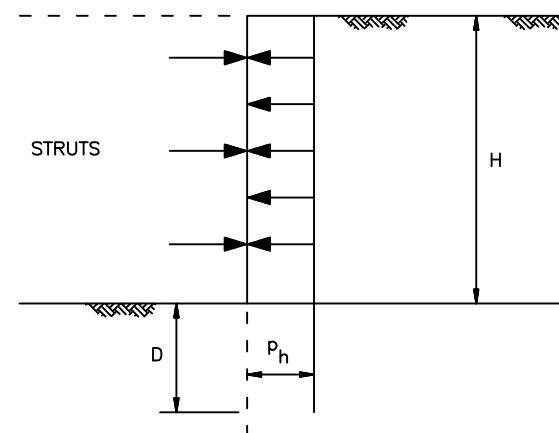
Underpinning of foundations located in Zones A and B should extend at least into Zone C.

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

$$p_h = \text{design lateral earth pressure} \\ = 0.65 \quad K\gamma H$$

$$K = \text{lateral earth pressure coefficient}$$

$$\gamma = \text{unit weight of soil}$$

$$H = \text{depth of excavation}$$

$$D = \text{depth of embedment of soldier piles (if used).}$$

RECOMMENDED DESIGN PARAMETERS

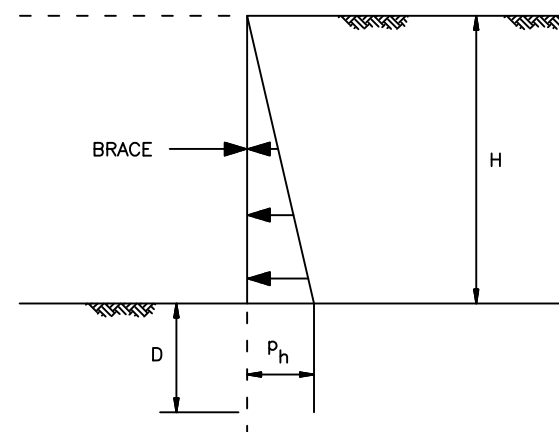
$$\gamma = 21.0 \text{ kN/m}^3$$

$$K = 0.33 \quad (\text{movement of retained soil acceptable})$$

$$0.50 \quad (\text{movement of adjacent structures/facilities unacceptable})$$

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.
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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

$$p_h = \text{design lateral earth pressure} \\ = K\gamma H$$

$$K = \text{lateral earth pressure coefficient}$$

$$\gamma = \text{unit weight of soil}$$

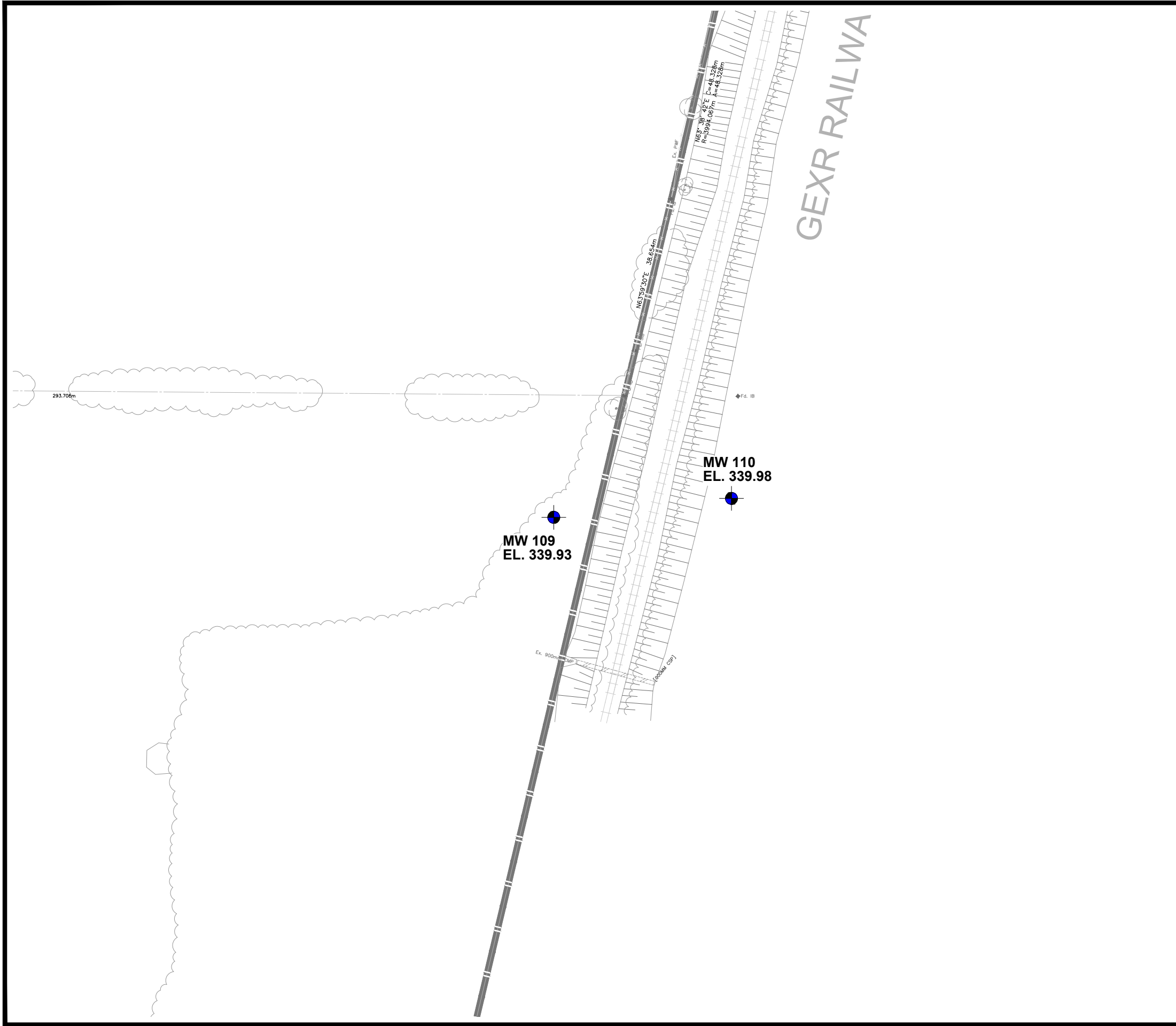
$$H = \text{depth of excavation}$$

$$D = \text{depth of embedment of soldier piles (if used).}$$

RECOMMENDED DESIGN PARAMETERS

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

$$K = \begin{array}{l} 0.33 \text{ (movement of retained soil acceptable)} \\ 0.50 \text{ (movement of adjacent structures/facilities unacceptable)} \end{array}$$



LEGEND:

- BOREHOLE LOCATION
- MONITORING WELL LOCATION

REFERENCE:

BOREHOLE LOCATION PLAN REPRODUCED FROM DRAWING SUPPLIED BY CLIENT.

NOTE:

THE INFERRED STRATIGRAPHY REFERRED TO IN THE REPORT IS BASED ON THE DATA FROM THESE BOREHOLES SUPPLEMENTED BY GEOLOGICAL EVIDENCE. THE ACTUAL STRATIGRAPHY BETWEEN THE BOREHOLES MAY VARY.

WILMOT WOODS DEVELOPMENT INC.

PROPOSED WILMOT WOODS DEVELOPMENT
NEW HAMBURG
NEW HAMBURG, ONTARIO

BOREHOLE LOCATION PLAN

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN	D. BRICE	DATE	SCALE	PML REF.	DWG. NO.
CHECKED	W. LOGHRIN	FEBRUARY 2022	AS SHOWN	18KF031	1
APPROVED	G. MITCHELL				

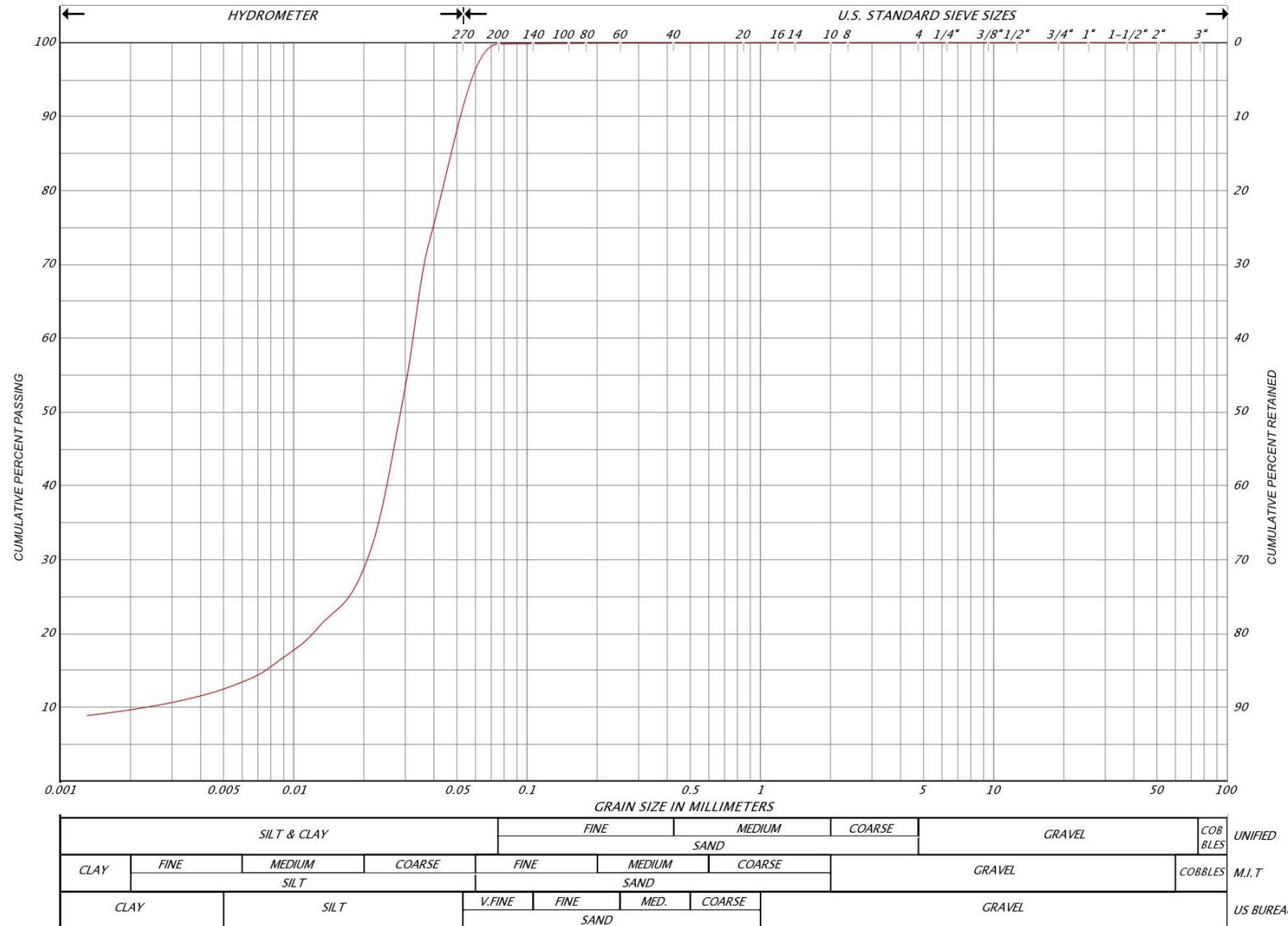


APPENDIX A

FIGURE 6 AND BOREHOLE LOGS 109 AND 110
FROM PREVIOUS REPORT

PARTICLE SIZE DISTRIBUTION CHART

PML REF. 18KF031
FIGURE NO. 6



REMARKS Borehole MW109, Sample SS11, Depth 9.1 to 9.6 m

CLAYEY SILT

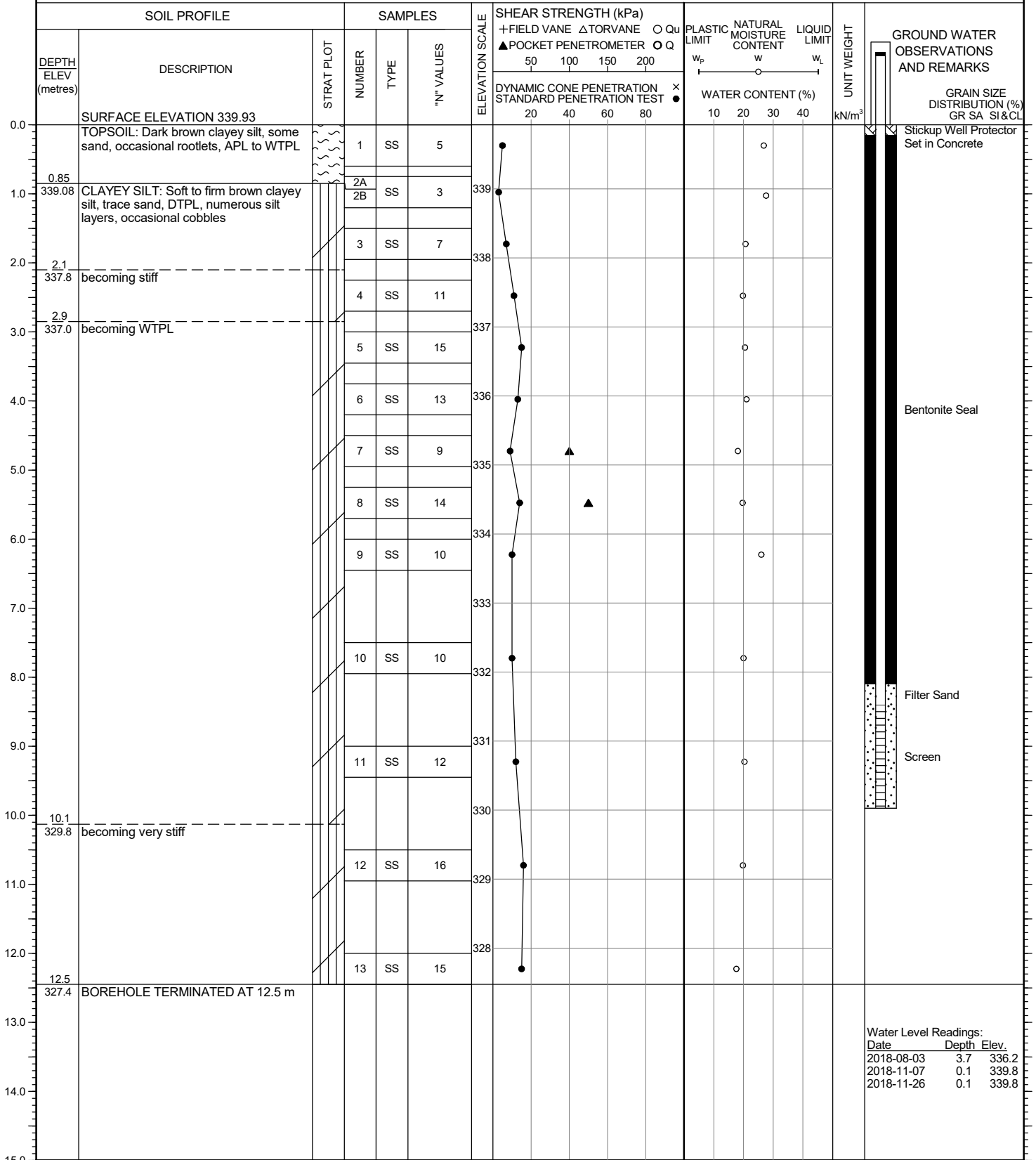
LOG OF BOREHOLE/MONITORING WELL NO. 109

17T 524441.1E 4803985N

PROJECT Wilmot Woods Development
LOCATION New Hamburg, Ontario
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE August 2, 2018

PML REF. 18KF031
ENGINEER H. Shinwary
TECHNICIAN K. Pettitt



NOTES



APPENDIX B

STATEMENT OF LIMITATIONS

STATEMENT OF LIMITATIONS



This report is prepared for and made available for the sole use of the client named. Peto MacCallum Ltd. (PML) hereby disclaims any liability or responsibility to any person or entity, other than those for whom this report is specifically issued, for any loss, damage, expenses, or penalties that may arise or result from the use of any information or recommendations contained in this report. The contents of this report may not be used or relied upon by any other person without the express written consent and authorization of PML.

This report shall not be relied upon for any purpose other than as agreed with the client named without the written consent of PML. It shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. A portion of this report may not be used as a separate entity: that is to say the report is to be read in its entirety at all times.

The report is based solely on the scope of services which are specifically referred to in this report. No physical or intrusive testing has been performed, except as specifically referenced in this report. This report is not a certification of compliance with past or present regulations, codes, guidelines and policies.

The scope of services carried out by PML is based on details of the proposed development and land use to address certain issues, purposes and objectives with respect to the specific site as identified by the client. Services not expressly set forth in writing are expressly excluded from the services provided by PML. In other words, PML has not performed any observations, investigations, study analysis, engineering evaluation or testing that is not specifically listed in the scope of services in this report. PML assumes no responsibility or duty to the client for any such services and shall not be liable for failing to discover any condition, whose discovery would require the performance of services not specifically referred to in this report.

The findings and comments made by PML in this report are based on the conditions observed at the time of PML's site reconnaissance. No assurances can be made and no assurances are given with respect to any potential changes in site conditions following the time of completion of PML's field work. Furthermore, regulations, codes and guidelines may change at any time subsequent to the date of this report and these changes may effect the validity of the findings and recommendations given in this report.

STATEMENT OF LIMITATIONS



The results and conclusions with respect to site conditions are therefore in no way intended to be taken as a guarantee or representation, expressed or implied, that the site is free from any contaminants from past or current land use activities or that the conditions in all areas of the site and beneath or within structures are the same as those areas specifically sampled.

Any investigation, examination, measurements or sampling explorations at a particular location may not be representative of conditions between sampled locations. Soil, ground water, surface water, or building material conditions between and beyond the sampled locations may differ from those encountered at the sampling locations and conditions may become apparent during construction which could not be detected or anticipated at the time of the intrusive sampling investigation.

Budget estimates contained in this report are to be viewed as an engineering estimate of probable costs and provided solely for the purposes of assisting the client in its budgeting process. It is understood and agreed that PML will not in any way be held liable as a result of any budget figures provided by it.

The Client expressly waives its right to withhold PML's fees, either in whole or in part, or to make any claim or commence any action or bring any other proceedings, whether in contract, tort, or otherwise against PML in anyway connected with advice or information given by PML relating to the cost estimate or Environmental Remediation/Cleanup and Restoration or Soil and Ground Water Management Plan Cost Estimate.

February 24, 2022

PML Ref.: 18KF031
Report: 3 Revised

Mr. Adam Belksy
Wilmot Woods Development Inc.
310 Fairway Road South, P.O. Box 45016
Kitchener, Ontario
N2C 2R6

Dear Mr. Belksy

**Monitoring Well Water Level Reading
Proposed Wilmot Woods Development
New Hamburg, Ontario**

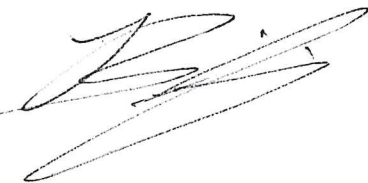
Peto MacCallum Ltd. is pleased to submit our report regarding the ground water monitoring program of Proposed Wilmot Woods Development, New Hamburg, Ontario.

A record of water monitoring completed since September 2018 to February 2021 is presented in Table 1, attached.

We trust that 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.
Manager, Engineering Services

WL:tm

Enclosure:

Table 1: Ground Water Level Readings in Monitoring Wells

Distribution:

1 cc: Wilmot Woods Development Inc.
1 cc: MTE Consultants Inc. (+ email – jcabral@mte85.com).
1 cc: PML Kitchener



TABLE 1
GROUND WATER LEVEL READINGS IN MONITORING WELLS

MONITORING WELL No.	GROUND SURFACE ELEVATION ⁽¹⁾	MID-SCREEN ELEVATION ⁽²⁾ (DEPH, m)	HYDROSTATIC GROUND WATER LEVEL ELEVATION (DEPTH, m) ⁽²⁾								
			SEPTEMBER 19, 2018	OCTOBER 10, 2018	NOVEMBER 7, 2018	NOVEMBER 26, 2018	FEBRUARY 8, 2019	NOVEMBER 19, 2020	DECEMBER 18, 2020	JANUARY 30, 2021	FEBRUARY 26, 2021
101	339.0	333.8 (5.2)	337.2 (1.80)	337.7 (1.34)	338.2 (0.85)	337.8 (1.24)	338.1 (0.96)	337.572 (1.46)	337.832 (1.20)	337.172 (1.86)	337.152 (1.88)
102	340.0	334.8 (5.2)	338.3 (1.71)	338.5 (1.48)	338.8 (1.23)	338.8 (1.24)	338.6 (1.41)	338.404 (1.69)	338.594 (1.50)	338.484 (1.61)	338.794 (1.30)
103	343.2	338.0 (5.2)	340.0 (3.22)	340.6 (2.66)	340.9 (2.33)	341.3 (1.91)	341.5 (1.70)	340.326 (3.00)	340.666 (2.66)	341.466 (1.86)	340.746 (2.58)
104	344.9	339.7 (5.2)	341.8 (3.17)	342.5 (2.45)	343.5 (1.41)	344.1 (0.83)	343.9 (1.03)	341.708 (3.23)	343.168 (1.77)	343.678 (1.26)	343.358 (1.58)
105	341.1	335.9 (5.2)	339.7 (1.31)	340.0 (1.08)	340.1 (0.91)	340.3 (0.77)	340.2 (0.84)	339.874 (1.26)	340.024 (1.11)	340.554 (0.58)	339.284 (1.85)
106	343.0	337.8 (5.2)	340.9 (2.13)	341.7 (1.25)	341.7 (1.31)	341.8 (1.15)	341.3 (1.65)	339.523 (3.47)	341.303 (1.69)	341.753 (1.24)	342.283 (0.71)
107	347.0	341.8 (5.2)	343.9 (3.07)	-	344.7 (2.30)	344.6 (2.33)	344.8 (2.20)	341.475 (5.52)	343.175 (3.82)	346.415 (0.58)	346.245 (0.75)
108	345.3	340.1 (5.2)	339.4 (5.94)	-	339.6 (5.68)	339.9 (5.40)	339.7 (5.63)	339.387 (6.00)	Dry	344.877 (0.51)	344.647 (0.74)
109	339.9	330.8 (9.1)	339.4 (0.53)	339.4 (0.53)	339.7 (0.19)	339.9 (0.04)	339.7 (0.20)	327.355 (0.48)	327.495 (0.34)	327.185 (0.65)	327.375 (0.46)
110	340.0	328.4 (11.6)	-	-	339.8 (0.43)	339.8 (0.17)	339.7 (0.26)	332.442 (0.84)	332.282 (1.00)	332.652 (0.63)	332.782 (0.50)

Notes:

- (1) Ground surface elevations at the monitoring well locations were surveyed by PML and are geodetic.
(2) Water levels measured using a Solinst flat tape water level reader.