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Geotechnical Investigation
Proposed Development of the White Church Lands

White Church Road and Upper James Street
Hamilton, Ontario

Prepared for:

White Church Landowners Group Inc.
% SCS Consulting Group
30 Centurian Drive, #100
Markham, Ontario
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■ GEOTECHNICAL INVESTIGATIONS ■ ENVIRONMENTAL SITE ASSESSMENTS & CLEANUP ■ GROUNDWATER STUDIES ■ SLOPE STABILITY STUDIES
■ ASPHALT TECHNOLOGY ■ ASPHALT MIX DESIGNS ■ PAVEMENT PERFORMANCE ANALYSIS ■ CONSTRUCTION MATERIALS TESTING & INSPECTION
■ ANALYSIS OF SOIL CORROSION POTENTIAL ■ PAVEMENT REHABILITATION & TENDER SPECIFICATIONS ■ CONCRETE QUALITY ASSURANCE TESTING
■ ROOF/STEEL INSPECTIONS ■ HYDROGEOLOGICAL ASSESSMENTS ■ FAILURE ANALYSES & EXPERT WITNESS SERVICES ■ AGGREGATE EVALUATION

EXECUTIVE SUMMARY

SCOPE OF SERVICES

Proposed Development	It is understood that any future development to be undertaken at the site is likely to comprise of single-detached, townhouse and residential condominium development for low density zones, low- to mid-rise towers and stacked townhouses in medium-density zones and high-rise towers in high-density zones. The development is also expected to include for community parks, institutional and community centre blocks, woodland lots and Storm Water Management ponds.
Report Deliverables	The Preliminary Geotechnical Investigation Report is required to provide an understanding of the subsurface conditions underlying the site and to provide preliminary design and construction recommendations for the proposed new residential development.

SITE DETAILS AND SETTING

Coordinates	589650, 4777630	**Geodetic Elevation**	220 m to 232 m

Site Description	The development area is situated along both White Church Road and Airport Road, is approximately 3,644,000 m^2 (364.4 hectares) in plan area and is semi-rectangular in shape. The site is of a generally agricultural use, with some small-scale commercial use and limited areas of rural, residential use also noted. The topography of the development area is generally of an undulating, glacial horizon.
Geology	Organic soil was encountered at the ground surface. Interbedded deposits of silt, clayey silt/silty clay and till deposits were encountered underlying the organic material in all boreholes and extends to the maximum dill depths of between 6.0 m and 12.6 m below the ground surface.
Groundwater	Groundwater or water seepages were not encountered during drilling, with all boreholes remaining open and dry to completion, though wet soils, particularly the silt till and deeper clayey silt till, were noted at variable depth across the development area. It should be noted that groundwater conditions are expected to vary according to the time of the year and seasonal precipitation levels.

GENERAL ENGINEERING CONSIDERATIONS

Foundations	Based on the ground conditions observed at the borehole locations and though there are no designs are available for the property at this time, it is considered by Landtek that the anticipated lightly and moderately loaded structures of low to moderate intensity development may be supported by the native soils underlying the site using conventional, concrete strip or pads foundations.
Settlements	The general limiting of the total settlement to 25 mm and the differential settlement to 19 mm by the recommended geotechnical reaction at the SLS is considered appropriate for foundations.
Earthquake Considerations	Based on the soil conditions encountered, and in accordance with Table 4.1.8.4.A. of the current Ontario Building Code (*OBC*), the site is considered to be a 'D' Site Class.
Damp Proofing and Waterproofing	Any future, at-grade will not require damp proofing or waterproofing, though any associated service or elevator pits should be damp proofed as a minimum. Where habitable basement or parking lot levels are proposed, the subsurface areas (i.e., basement walls and floor slabs etc.) should be damp proofed where above the groundwater levels provided by Landtek's Hydrogeological Assessment, and appropriately waterproofed, where below groundwater. Municipal approval will be required for long-term (permanent) groundwater dewatering.

GENERAL CONSTRUCTION CONSIDERATIONS

Excavations	The subsurface soils to be encountered during excavation at the site are expected to behave as "Type 2" materials according to the OHSA classification in Part III. It should be possible to excavate the overburden soils with a hydraulic backhoe. Moist Type 2 soils are expected to remain stable for 'short' construction periods at battered slopes of 45°, per OHSA requirements.
Short-Term (Construction) Dewatering	Elements of the development are expected to include multiple levels of basement. As such, for short-term dewatering, groundwater is expected to be encountered within basement excavations, particularly where two or more basement levels are proposed. Considerations and parameters regarding construction dewatering, including the "*seasonally highest groundwater level*", are provided by Landtek's Hydrogeological Assessment for the site, as reported under separate cover.

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1.0 INTRODUCTION

Landtek Limited (herein “*Landtek*”) is pleased to submit this Preliminary Geotechnical Investigation report for the proposed development located at the site identified as White Church Lands at White Church Road and Airport Road in Hamilton, Ontario. Authorization to proceed with the work was received from Mr. Nicholas McIntosh, P. Eng., of SCS Consulting Group Ltd. (herein “*SCS*”) on August 28, 2023, acting on behalf of the White Church Landowners Group Inc.

At the time of issue of this report, Landtek understands that no designs are available for the development area other than the preliminary layout of low- medium- and high-density zoning. It is understood however, that any development to be undertaken at the site is likely to comprise of single-detached, townhouse and residential condominium development for low density zones, low- to mid-rise towers and stacked townhouses in medium-density zones and high-rise towers in high-density zones.

The development is also expected to include for community parks, institutional and community centre blocks, woodland lots and Storm Water Management (herein “*SWM*”) ponds. New municipal and private road pavement structures and services are also anticipated.

Given the absence of concise development plan, this investigation is to be considered preliminary until such time that a development concept is available for each development parcel and an appropriate, more detailed investigation is completed to compliment the development plan. On this basis, the primary objectives of this investigation are:

- To provide an outline understanding of the subsurface soil and groundwater conditions for foundation design and construction;
- Provide outline and generalized design and construction recommendations with regards to building foundations, at-grade floor slabs, pavement structures, and subsurface drainage and utilities using trenched and trenchless excavation methodologies; and,
- Assess the characteristics, from a geotechnical perspective, of the soils to be excavated and their potential impact on excavatability, reuse and shoring systems.

This Geotechnical Investigation report has been prepared for the Client, the nominated engineers, designers, and project managers pertaining to the proposed development site identified as the “*White Church Lands*”, located in Hamilton, Ontario. Reliance on this report is also extended to Municipalities and Regulatory Authorities but is limited to the intended purpose of the report only.

Any further dissemination of this report outside of those parties previously detailed is not permitted without Landtek’s prior written approval. Further details of the limitations of this report are presented in Appendix A.

2.0 SITE SETTING

2.1 Site Location and Description

The development site is located in Hamilton, Ontario, and is centered at approximate grid reference 589650, 4777630 (UTM 17T coordinates). The Geodetic elevation of the ground surface at the site is approximately 220 m to 232 m.

The site location is shown in Figure 2.1.1 below.

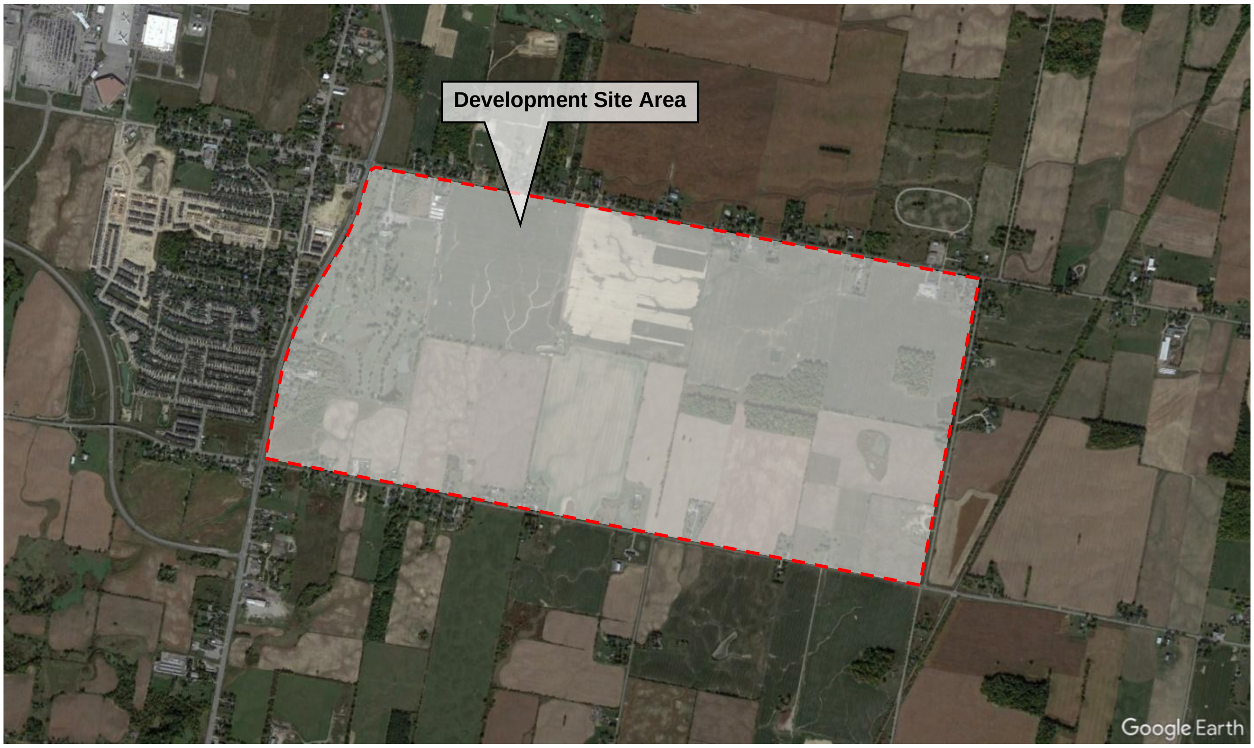


Figure 2.1.1: *Development Site Area and Surrounding*

The development area is situated along both White Church Road and Airport Road, is approximately 3,644,000 m^2 (364.4 hectares) in plan area and is semi-rectangular in shape. The site is of a generally agricultural use, with some small-scale commercial use and limited areas of rural, residential use also noted.

The development area is situated to the north of White Church Road and is bound to the north by Airport Road, to the east by Miles Road, and to the west by Upper James Street.The topography of the development area is generally of an undulating, glacial horizon, with a range in Geodetic elevation between approximately 232 m in the north and 220 in the south. The general trend of slope in topography is towards south and southwest.

2.2 Published Geology

Based on previous geotechnical experience for the area and a review of the existing geological publications for the site area, Ontario Geological Survey (herein "*OGS*") Map P. 993 "*Quaternary Geology of the Grimsby Area*", the site is underlain by deposits of glaciolacustrine clay and silt, and clay and silt tills of the Halton Till formation.

The Ontario Department of Mines (herein "*ODM*") Map 2343 "*Paleozoic Geology of the Grimsby*

Area” indicates that the superficial geology is underlain by brown or tan dolostone of the Guelph Formation.

Information provided by historical borehole records from within the vicinity of the site, and held by the OGS, generally confirms the anticipated geological conditions beneath the site. Based on the data from records for Borehole ID 853160, located approximately 1 km west of the site, the soil profile comprises of a veneer of clay and silt deposits to a depth of 23.3 m.

2.3 Published Hydrology and Hydrogeology

Based on publicly available information held by both Hamilton and Niagara Peninsula Conservation Authorities (herein “*HCA*” and “*NPCA*”, respectively), the nearest surface water features are Three Mile Creek and Twenty Mile Creek, the tributaries of which are noted to transect the site. Localized ponds and wetlands are also noted within the development area.

According to the OGS, static groundwater levels in the vicinity of the site are generally associated with the deeper till deposits and strata of the Guelph Formation bedrock. Publicly available documentation for groundwater levels in the area report variable groundwater levels, but generally within the range of 10.6 m to 18.3 m below existing ground level.

The groundwater data is also supported by previous, intrusive investigations completed by Landtek and others in the vicinity of the property. Historical reporting identifies groundwater levels at approximately 2.5 m to 11.0 m depth and have been attributed to both locally perched groundwaters and site-wide groundwater regimes.

3.0 FIELDWORK AND INVESTIGATION METHODOLOGY

Fieldwork undertaken at the site by Landtek included clearance of underground services, borehole layout, borehole drilling and soil sampling, and field supervision. A total of twenty-one boreholes, identified as boreholes BH1 to BH24 (excluding BH14, BH15 and BH21) were drilled in phases between March 11 and August 8, 2024.

A subsequent investigation was undertaken within an agricultural area in the central north of the site, and boreholes BH22, BH23 and BH24 were drilled for this additional phase of investigation on January 6, 2025.

All boreholes were logged using those standard symbols and terms defined in Appendix B. The Exploratory Hole Location Plan, Drawing 23354-01, and associated borehole logs are provided in Appendix C. The boreholes were drilled using a Dietrich D-50 track mounted drill rig equipped with continuous flight, solid stem augers to a maximum depth of between approximately 6.0 m and 12.1 m. Full time supervision of drilling and soil sampling operations was carried out by a representative of Landtek. Standard Penetration Tests (SPT's) and split spoon samples were taken during drilling at selected depths. Boreholes encountering auger refusal on bedrock were extended to prove target depth using NQ-gauge, rotary coring methodologies.

Fifteen (15) boreholes were completed as monitoring wells and re-identified as boreholes BH/MW3S/D (nested), BH/MW4, BH/MW6, BH/MW8, BH/MW9, BH/MW10, BH/MW11, BH/MW12, BH/MW16, BH/MW17, BH/MW18, BH/MW19S/D (nested), BH/MW20, BH/MW22 and BH/MW24. The monitoring wells consisted of new/sealed 50 mm polyvinyl chloride (PVC) screen with No.10 slots threaded onto a matching riser. The screens and risers were pre-threaded including o-ring seals such that no glues or solvents were used to connect the pipe sections. The annular space between the PVC well and the borehole was backfilled to approximately 0.3 m above the top of the screen section with sand pack, and then with bentonite to existing ground level. A J-Plug lockable air-tight cap was installed on the riser. The monitoring well installation details are presented on the respective borehole logs.

All soil samples were transported to the Landtek’s in-house, Canadian Council of Independent Laboratories (CCIL) certified laboratory and visually examined to determine their textural classification. Moisture content testing was carried out on all samples. Twelve selected, composite soil samples were submitted to Paracel Laboratories (herein “*Paracel*”) for Soil Corrosivity parameter testing. No further chemical testing was proposed for the Geotechnical Investigation element.

The borehole locations were established by Landtek relative to site measurements and existing site features. All depth-related remarks relative to topographical survey information available for the site, drawing reference 365466-T, as completed by A. T. McLaren Ltd.

4.0 SUBSURFACE CONDITIONS

4.1 Overview

The borehole information is generally consistent with the geological data identified in Section 2.2, with the predominant soils comprising of glaciolacustrine clays, silts and tills.

The detailed borehole logs are presented in Appendix C, with the ground conditions encountered by the boreholes discussed in the following sections.

4.2 Organic Material

An approximately 50 mm to 200 mm thick layer of organic soil was encountered in all boreholes.

Organic soil thicknesses may vary across the site, particularly in areas of wetland or agricultural land where ploughing has occurred. As such, the thicknesses measured at the borehole locations should be taken as indicative and may not be representative of sitewide organic soil depths.

4.3 Silt

Silt deposits were encountered in boreholes BH/MW6, BH/MW8, BH/MW22, BH23 and BH/MW24 underlying the organic material and clayey silt deposits at a depth of 1.5 m to 7.6 m below ground level. The silt deposits encountered are primarily brown, and grey at depth in colour and include trace fractions of grey clay seams and iron staining.

SPT ‘‘N’’ values ranging from 3 to 29 were reported, indicating the silt to be of a loose to compact, but generally compact consistency. Moisture contents in the silt deposits were 14 % to 23 %, which is representative of a moist to wet soil with silt as the primary constituent. The moisture content testing results are presented on the borehole logs in Appendix C.

4.4 Clayey Silt to Silty Clay

Clayey silt to silty clay deposits were encountered in all boreholes except boreholes BH1, BH23 and BH/MW24 below the organic material, and range in depth between approximately 0.1 m to 6.0 m below the ground surface. The clayey silt to silty clay deposits encountered are primarily brown, and grey at depth in colour, and includes variable fractions of gravel, iron staining, red shale fragments, grey clay seams, and sand.

SPT ‘‘N’’ values ranging from 4 to 55 were reported, indicating the clayey silt to silty clay to be of a soft to hard, but generally very stiff consistency. Moisture contents in the clayey silt to silty clay deposits range between 13 % and 37 %, which are representative of a moist to wet soil with silt and clay as primary constituents. The moisture content testing results are presented on the borehole logs in Appendix C.

4.5 Silt Till

Silt till deposits were encountered in boreholes BH1, nested boreholes BH/MW3S/D, BH23 and BH/MW24 underlying the silt, clayey silt and clayey silt to silty clay till deposits, ranging in depth between approximately 0.7 m to 8.1 m below ground level. The silt till deposits encountered are primarily grey in colour and include variable fractions of clay, iron staining and gravel.

SPT ‘‘N’’ values ranging from 10 to 45 were reported, indicating the silt till to be of a loose to dense, but generally compact consistency. Moisture contents in the silt till deposits range between

10 % and 19 %, which are representative of a moist to wet soil with silt as the primary constituent. The moisture content testing results are presented on the borehole logs in Appendix C.

4.6 Silty Clay to Clayey Silt Till

Silty clay to clayey silt till deposits were encountered in all boreholes except BH23 and BH/MW24 below the silty clay to clayey silt deposits and organic material, and range in depth between approximately 0.7 m to the maximum drill depth of approximately 12.6 m below the ground surface. The till deposits encountered are primarily brown, and grey at depth in colour and include variable fractions of gravel, iron staining, cobbles, grey clay seams and red shale fragments.

SPT ‘‘N’’ values ranging from 9 to 54 were reported, indicating the till to be of a stiff to hard, but generally very stiff consistency. Moisture contents in the till deposits range between 13 % and 25 %, which are representative of a moist to wet soil with silt and clay as primary constituents. The moisture content testing results are presented on the borehole logs in Appendix C.

4.7 Bedrock

Bedrock was not encountered during this investigation.

4.8 Groundwater

Groundwater or water seepages were not encountered during drilling, with all boreholes remaining open and dry to completion though wet soils, particularly the silt till and deeper clayey silt till, were noted at variable depth across the development area.

Groundwater monitoring well visits are bring completed at the site as part of Landtek’s ongoing Hydrogeological Investigation for the development area. The preliminary results of the groundwater monitoring are presented in Table 4.8.1 following.

Table 4.8.1: Summary of Water Level Measurements

MW ID	Monitoring Well Details			Groundwater Monitoring Results (m)							
	Surface Elevation	Screen Depth	Wet Soils	19-Jul-24		16-Aug-24		28-Aug-24		18-Sep-24	
				Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.
BH/MW3S	-	1.5 m – 3.0 m	2.5 m	0.89	-	1.06	-	1.28	-	2.42	-
BH/MW3D	-	3.0 m – 6.0 m	2.5 m	0.71	-	1.17	-	1.39	-	4.63	-
BH/MW4	-	3.0 m – 6.0 m	5.5 m	0.21	-	0.78	-	1.99	-	3.44	-
BH/MW6	-	3.0 m – 6.0 m	-	0.4	-	0.88	-	1.06	-	5.61	-
BH/MW8	-	3.0 m – 6.0 m	-	0.48	-	1.18	-	1.45	-	2.08	-
BH/MW9	-	6.0 m – 9.0 m	-	7.44	-	5.75	-	6.12	-	3.97	-
BH/MW10	-	3.0 m – 6.0 m	-	0.43	-	0.50	-	0.57	-	0.68	-
BH/MW11	-	3.0 m – 6.0 m	-	0.78	-	1.17	-	1.35	-	1.69	-
BH/MW12	-	3.0 m – 6.0 m	-	-	-	0.98	-	1.68	-	1.73	-
BH/MW16	-	3.0 m – 6.0 m	-	-	-	1.00	-	1.17	-	1.49	-
BH/MW17	-	3.0 m – 6.0 m	-	-	-	5.29	-	4.39	-	5.15	-
BH/MW18	-	5.4 m – 8.4 m	-	-	-	1.77	-	1.03	-	1.31	-
BH/MW19S	-	1.5 m – 3.0 m	2.8 m	-	-	1.31	-	1.44	-	1.67	-
BH/MW19D	-	3.0 m – 6.0 m	3.0 m	-	-	1.38	-	1.47	-	1.69	-
BH/MW20	-	3.0 m – 6.0 m	-	-	-	1.23	-	1.54	-	2.18	-
BH/MW22	-	4.5 m – 7.6 m	-	-	-	-	-	-	-	-	-
BH/MW24	-	4.5 m – 7.6 m	3.0 m	-	-	-	-	-	-	-	-

It should be noted that groundwater conditions and surface water flow conditions are expected to vary according to the time of the year and seasonal precipitation levels. Water seepage is also expected from soil fissures and fractures above the water table.

Further information pertaining to groundwater conditions is provided by Landtek's Hydrogeological Assessment for the site, as reported under separate cover.

5.0 FOUNDATION DESIGN CONSIDERATIONS

The recommended limit state bearing capacities provided in this report are based on the preliminary dataset compiled by this investigation paired with publicly available borehole data and Landtek's knowledge of the geotechnical and geological history of the area.

On this basis, the recommendations and considerations are provided on the understanding that more detailed investigations will be undertaken once specific development concepts and site layouts are developed.

5.1 Shallow Foundation Considerations

5.1.1 Foundations in Native Soils

Based on the ground conditions observed at the borehole locations and though there are no designs are available for the property at this time, it is considered by Landtek that the anticipated lightly and moderately loaded structures of low to moderate intensity development (i.e., townhomes, low- to mid-rise towers etc.) may be supported by the native soils underlying the site using conventional, concrete strip or pads foundations.

Table 5.1.1.1 summarizes preliminary, recommended geotechnical reactions at the Serviceability Limit State (herein "*SLS*") and factored geotechnical resistances at the Ultimate Limit State (herein "*ULS*") for the native soils expected to be encountered at founding depths. It should be noted that the design parameters have been determined by Landtek for the preliminary design stage only. It is also important to note that, where the bearing levels of the footings are at different design elevations, the footing base levels should be stepped along a line of 7V:10H, drawn upwards from the lowest footing, to avoid overlapping stresses.

In accordance with the Ontario Building Code (herein "*OBC*"), 9.12.2.2 (5), and based on local experience, the shallowing of exterior and interior footings to 0.9 m and 0.6 m depth below the basement finished floor level respectively, may be adopted for the development. Such shallowing of foundations is to be limited to only those areas where a minimum of one basement level is to be included.

Table 5.1.1.1: Preliminary Limit State Foundation Design Values

General Founding Depth Ranges	Founding Stratum	Foundation Design Value	
		SLS [1 2]	ULS [3 4]
1.5 m – 2.5 m	Clayey Silt/Silty Clay/Silt Till/Clayey Silt Till/Silty Clay Till	200 kPa	300 kPa
2.5 m – 6.0 m	Silt/Clayey Silt/Silty Clay/Silt Till/Clayey Silt Till/Silty Clay Till	200 kPa	300 kPa
6.0 m – 7.0 m	Clayey Silt Till/Silty Clay Till/ Silt Till	300 kPa	500 kPa

Notes:
1. The National Building Code general safety criterion for the serviceability limit states is: SLS resistance ≥ effect of service loads.
2. Recommended SLS bearing values conform to Estimated Values based on soil types given in Tables K-8 and K-9 of the National Building Codes User's Guide.
3. The ULS resistance factor for shallow foundations is 0.5, as given in Table K-1 of the National Building Code User's Guide.
4. The National Building Code general safety criterion for the ultimate limit states is: factored ULS resistance ≥ effect of factored loads.

Subsurface conditions can vary over relatively short distances, and the subsurface conditions revealed at the borehole locations may not be representative of subsurface conditions across the site. As such, a further, more detailed Geotechnical Investigation will be required once a development concept plan for the site has been established.

Design factors related to structural loads will determine the most cost-effective foundation system for the proposed development. The impact on foundation size and soil bearing pressure is illustrated in Figure 5.1.1.1 and emphasizes that foundation design sizes, bearing pressures, and bearing levels must be taken into account to avoid excessive consolidation settlements.

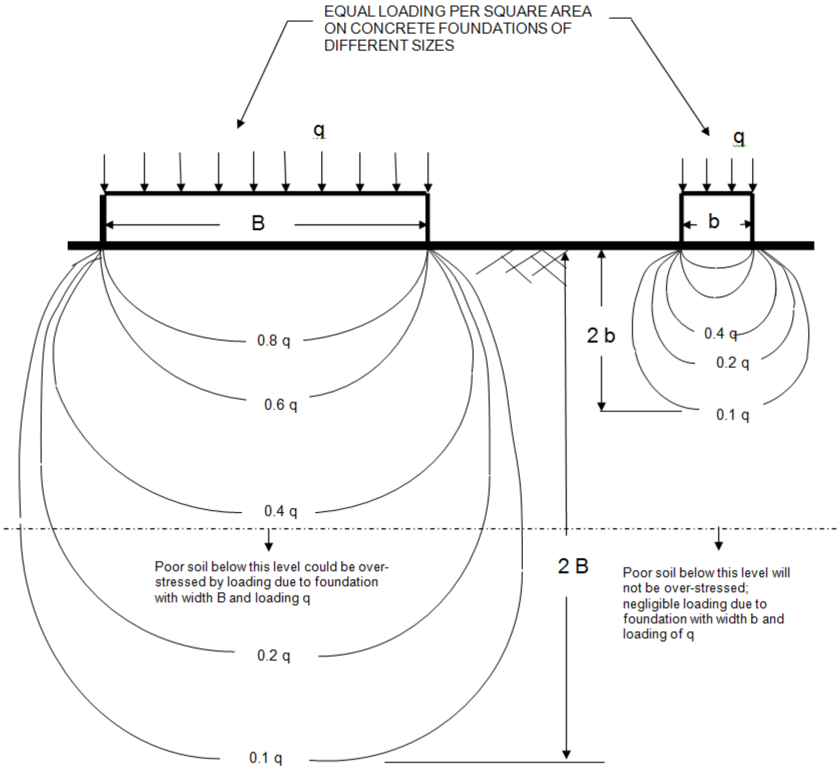


Figure 5.1.1.1: Illustration of Load Distribution below Variable Size Foundations with the Same Applied Loading

Footing foundations may be considered an appropriate option, though the acceptability of footings will depend upon design issues such as the elevation of the lowest floor level and the structural loading. If the footing design criteria provided in this report cannot be satisfied then an alternative solution may be considered, such as a piled solution, particularly if the proposed structures are of a generally high loading than anticipated.

5.1.2 Foundations on Engineered Fill

If engineered fill is required to support founding elements of the development, it is considered by Landtek that relatively lightly loaded structures can be adequately supported by conventional strip or pad footings founded on the engineered fill for a geotechnical reaction at the SLS of 100 kPa, and a factored geotechnical resistance at the ULS of 150 kPa.

It should be noted however, that this is very much dependent upon the nature and condition of the fill placed, the condition of the sub-grade upon which it is being placed, and the methods adopted for the placement and compaction of the fill materials. The engineered fill must be selected with care, then placed and compacted under strictly controlled conditions.

The following recommendations are provided to address the selection of fill material as well as the placement and compaction of engineered fill:

- Processed imported granular material or consistent quality imported clean earth fill, can be considered for engineered fill provided the soil moisture content is within about 2 % of the optimum value of the material. Imported fill should meet the environmental requirements established for the site;
- Engineered fill should only be placed in an area that has been satisfactorily prepared by stripping existing fill and organic soils, and proof rolling the native exposed soil with at least five passes of a minimum 10-ton static pad-foot steel drum type roller;
- Engineered fill should be placed in maximum 300 mm, loose lifts and compacted to a target value of 100 % Standard Proctor Maximum Dry Density (herein "*SPMDD*"). The placement and compaction of each lift should be monitored full time by Landtek, with in-place compaction determined using nuclear moisture/density testing equipment;
- Fill layers that do not meet the compaction requirements, or become wet or frozen, should not be approved for the placement of additional material;
- For engineered fill placement over large areas of varying elevation, the locations of quality control density tests should be recorded by total station survey; and,
- As a precautionary measure and to mitigate cracking, it is recommended that reinforcing steel be provided in footings on engineered fill, and at the top of poured concrete foundation walls. Two 15M bars (continuous) are recommended as a minimum for footing placement. The Structural Engineer should be consulted to confirm the design of such reinforcement.

5.2 Raft Foundation Considerations

For foundations for higher loaded structures than those detailed in Section 5.1, the soil conditions encountered indicate that a raft foundation may be considered an appropriate, shallow-founded alternative to strip or spread foundations.

Design values for the modulus of subgrade reaction generally decrease when the size of the loaded plate (or footing) is larger than 0.3 m by 0.3 m. For granular soils, if the loaded area on the soil is a width of b, the modulus of subgrade reaction can be taken as:

$$K_{\mathrm{vb}} = K_{\mathrm{v1}}\left(\frac{b+0.3}{2b}\right)^{2}$$

where:

k_{v1} = modulus of subgrade reaction for a loaded plate of dimensions 0.3 m x 0.3 m;
= 25 MPa/m, considered representative of the predominant soil bearing conditions at depth across the site;
b = raft foundation width in metres;
k_{vb} = modulus of subgrade reaction in MPa/m for actual foundation dimension b

For cohesive soils, if the loaded area on the soil is a width of b and a length (as a ratio to b) of mb, the modulus of subgrade reaction can be taken as:

$$K_{\mathrm{vb}} = \left(\frac{K_{\mathrm{v1}}}{b}\right)\left(\frac{m+0.15}{1.5m}\right)$$

where:

k_{v1} = modulus of subgrade reaction for a loaded plate of dimensions 0.3 m x 0.3 m;
= 30 MPa/m, considered representative of the predominant soil bearing conditions at depth across the site;
b = raft foundation width in metres;
m = ratio of foundation length to width where length, L, = mb
k_{vb} = modulus of subgrade reaction in MPa/m for actual foundation dimension b

The soil parameters to be used in the raft foundation design process include the modulus of subgrade reaction, corrected for the building footprint size, and the limiting average pressure at the underside of the raft foundation. The net average bearing pressure at the SLS acting on the underside of the raft is expected to be in the order of 150 kPa to 250 kPa for the native soils underlying the site at depths of approximately 3.0 m to 7.0 m below existing ground level.

5.3 Deep Foundation Considerations

5.3.1 Piled Foundations

If higher bearing capacities are required to support the building loads, then an alternative, deeper founding solution may be required, such as the following:

- "*Cast in Place*" concrete caissons, which could be constructed without any unexpected difficulties but based on the conditions of deeper groundwaters, should incorporate the use of liners. It is anticipated that a dewatering system will not be required provided that liners are used appropriately to control the piezometric water level conditions encountered at depth; or,
- Continuous Flight Auger (CFA) piles.

For piles seated within the silt and clay deposits, the point resistance at the bottom is expected to range between 200 kPa and 300 kPa at the SLS. The frictional resistance (skin friction) developed in the drilled shaft should be calculated as follows:

$$Q_s = 0.42D_s\,[100L_1]$$

where:
D_s = Diameter of drilled shaft
L_1 = Length of pile within the clayey silt to clay
Q_s = value in kN

Alternatively, the piles may be extended to bedrock, though the depths to bedrock are quite significant and in excess of this preliminary investigation. Based on publicly available information, dolostone bedrock is anticipated at depths of approximately 18 m to 25 m below ground level at its shallowest.

Based on generalised rock strength parameter testing, the dolostone bedrock underlying the site may be capable of supporting a factored geotechnical resistance of 2.0 MPa at the ULS as a minimum. This is on condition that any piled foundation is seated at a depth to provide a minimum 0.5 m rock socket (i.e., founded at a minimum of 0.5 m penetration depth into the weathered bedrock). This given however, the bedrock is expected to be capable of supporting more significant loads and further investigation will determine the site-specific rock strength parameters.

The following parameters may be applied for the bedrock when considering lateral pressures on loaded piles:

$$K_p = \text{Rankine passive pressure coefficient} = \tan^2(45 + \phi/2)$$

For the weathered dolostone:

- Internal angle of friction (ϕ) should be taken as 26°; and,
- Bulk unit weight (ϒ) should be taken as 24 kN/m^3.

For the competent dolostone:

- Internal angle of friction (ϕ) should be taken as 26°; and,
- Bulk unit weight (ϒ) should be taken as 26.5 kN/m^3.

This given however, that the bedrock is expected to be capable of supporting more significant loads and that further investigation will be required to determine the site-specific geotechnical resistances for the bedrock at depth.

In addition, the final design and seating depths for any piled foundation solution is to be based on the findings of the additional investigation required and specific pile-driving and pile load tests undertaken at the site prior to construction.

5.3.2 Settlement Considerations for Piled Foundations

For competent bedrock, the SLS condition will not govern the foundation design as the stress required to induce 25 mm of movement (typical settlement criteria for SLS) is anticipated to exceed the ULS. Therefore, any anticipated settlements for foundations seated within dolostone bedrock underlying the site should be considered negligible (i.e., less than 15 mm).

5.4 Piled Raft Foundation Considerations

If the option of a raft alone cannot be satisfied or a deeper founding solution is not viable, another alternative to consider is a "*piled raft foundation*". In the design, the piles act as "*settlement reducers*" and the reduction of the length of piles can be achieved as the raft resistance is also considered in the design.

Tables 5.4.1 and 5.4.2 below provide estimated ultimate load carrying capacities for drilled shafts with the base of the shaft seated within silt and clay till horizons. Pile displacement may be conservatively set at 20 mm for preliminary consideration, compared with the allowable foundation settlement of 25 mm.

Table 5.4.1: Estimate of Ultimate Load Capacity: 1.2 m Diameter Pile

Length of Drilled Shaft (m)	Estimated Ultimate Load Capacity (kN)
5	900
10	1,800
15	2,600
20	3,400
25	4,300

Table 5.4.2: Estimate of Ultimate Load Capacity: 1.6 m Diameter Pile

Length of Drilled Shaft (m)	Estimated Ultimate Load Capacity (kN)
5	1,500
10	2,800
15	4,000
20	5,200
25	6,500

5.5 Frost Susceptibility

The shallow soils encountered across the site are considered sensitive to water and frost, and their physical and mechanical properties are dependent on in-situ moisture content. As such, the founding soils at the site are considered to have a moderate to high frost susceptibility, being classified as Frost Group "*F4*" (Table 13.1 of the "*Canadian Foundation Engineering Manual*", 4th Edition). However, the indicative depths given for foundations in Sections 5.1.1 and 5.1.2 are considered below the maximum extents of influence from frost penetration in the Hamilton area.

Should any re-grading be proposed as part of the development and is situated adjacent to new or existing structures, it will be important to ensure that the associated exterior footings will have a minimum of 1.2 m of soil cover, or equivalent suitable insulation, for frost protection.

5.6 Settlement Considerations

Based on the outline information provided for the nature of the proposed redevelopment of the site, it is anticipated that the loads to be applied to the ground by any such structure will be generally low to moderate intensity.

As such, associated settlements are not expected to be large. Therefore, the general limiting of the total settlement to 25 mm and the differential settlement to 19 mm by the recommended geotechnical reaction at the SLS is considered appropriate.

5.7 Existing Building Demolition

It is expected that all existing structures and associated infrastructure, including pavements and services, will be removed prior to development. Excavations created by the demolition of existing structures will require backfilling with engineered fill prior to commencing development.

Material controls and placement requirements for such fill materials are provided in Sections 5.1.2 and 10.0 of this report.

5.8 Seismic Design Considerations

Based on the soil conditions encountered, and in accordance with Table 4.1.8.4.A. of the current Ontario Building Code (herein "*OBC*"), the site is generally indicated to be a 'C' Site Class. The acceleration and velocity-based site coefficients, F_a and F_v, should be determined from Tables 4.1.8.4.B. and 4.1.8.4.C. respectively of the OBC for the above recommended Site Class. The seismic design data given in Table 1.2 of Supplementary Standard SB-1 in Volume 2 of the OBC, for selected Municipal locations, should be used to complete the seismic analysis.

Should a higher classification be required (i.e., Class B or higher), then Shear Wave Velocity Testing should be undertaken for each specific development parcel using Multichannel Analysis of Surface Waves (MASW) methodologies. However, this assessment will not necessarily guarantee a change of classification, as it is wholly dependent on the ground conditions beneath the site being assessed.

5.9 Damp Proofing and Waterproofing Considerations

For any future structures that are to be constructed at-grade, no damp proofing or waterproofing to foundation walls is required. This given however, any subsurface areas such as service or elevator pits associated with the at-grade structure should be damp proofed as a minimum.

Where habitable basement or parking lot levels are proposed, the subsurface areas (i.e., basement walls and floor slabs etc.) above established groundwater levels should be damp proofed and comply with the OBC requirements. As a minimum it is recommended that the damp proofing system include a Delta Drainage Board or MiraDrain 2000 series product, or an approved alternative, along with an asphalt-based spray-on wall coating.

Should habitable basement or parking lot levels or any associated subsurface areas such as service or elevator pits be seated below the groundwater levels provided by Landtek's

Hydrogeological Assessment, as reported under separate cover, then such structures are to be appropriately waterproofed. The waterproofing should include for the required buffer zone (nominally 1.0 m to 1.5 m) above the stabilized or highest recorded groundwater level.

6.0 FLOOR SLAB AND PERIMETER DRAINAGE CONSIDERATIONS

Based on the borehole soil conditions and information provided to Landtek, it should be possible to construct conventional, at-grade and basement floor slabs using slab-on-grade methods. The subgrade support conditions are anticipated to be clays, silts and tills, or a combination thereof, which should provide competent conditions for placing the vapour barrier material.

After the subgrade has been prepared to the underfloor design elevation it is recommended that the area be proof-rolled with a loaded tandem axle dump truck to delineate if there are soft or unstable ground conditions that require repair. This operation should be completed before the underfloor vapour barrier granular material is placed.

It is recommended that a minimum 200 mm layer of clear, 19 mm crushed quarried stone be used as the vapour barrier under the floor slab. The vapour barrier stone should meet the requirements of Ontario Provincial Standard Specifications (herein “*OPSS*”) 1004 for 19 mm Type II clear stone. If a graded crushed stone is substituted for clear stone, the material should be limited to a maximum of 5 % fines (passing the 0.075 mm sieve). The floor slab thickness should meet the specifications of the project based on anticipated floor loadings.

The finished exterior ground surface should be sloped away from the buildings at a grade in the order of 2 %.

The concrete properties should meet the requirements of OPSS 1350. Contraction and isolation jointing practices should be in accordance with current Portland Cement Association recommendations, as given in the engineering bulletin "*Concrete Floors on Ground*”, second edition, by R. E. Spears, and W. C. Panarese.

The design of concrete slabs on native soils may be made on the basis of a value of modulus of subgrade reaction of 25 MPa/m for native silt and clay subgrade soils.

Perimeter drainage should be provided around all subsurface floor areas where water may accumulate unless the proposed structures are to be waterproofed as prescribed in Section 5.9. This, however, is subject to the Municipal approval allowing for the discharge of groundwater into the Municipal storm system where the perimeter drainage is going to be installed at a depth below the established groundwater level.

Underfloor drains may be also required depending on the provision of waterproofing, or excavation and groundwater seepage conditions, particularly where below the groundwater level. Groundwater should be anticipated within excavation profiles for structures that include two or more levels of basement, though groundwater levels may be locally shallower.

Drainage systems should comply with the current OBC and associated amendments. Further details pertaining to perimeter and underfloor drainage systems are provided in Drawings 23354-02 and 23354-03 respectively, in Appendix D.

7.0 EARTH PRESSURE CONSIDERATIONS FOR SUBSURFACE WALLS

7.1 General Earth Pressure Considerations

The earth pressure, p, acting on subsurface walls at any depth, h, in metres below the ground surface assumes an equivalent triangular fluid pressure distribution and may be calculated using the expression below. It is assumed that granular material is used as backfill. Allowances for pressure due to compaction operations should be included in the earth pressure determinations and a value of 12 kPa is applicable for a vibratory compactor and granular material.

If the structure retaining soil can move slightly, the active earth pressure case can be used in determining the lateral earth pressure. For restrained structures and no yielding an “at rest” earth pressure condition should be used. The determination of the earth pressures should be based on the following expression:

$$P_1 = K (\delta h + q)$$

where:
P_1 = the pressure in kPa acting against any subsurface wall at depth, h, in metres (feet) below the ground surface;
K = the at rest earth pressure coefficient considered appropriate for subsurface walls; OPSS 1010 Granular B Type 1 (pit-run sand and gravel) material has an effective angle of friction estimated to be 32° with a corresponding at rest earth pressure coefficient, K_o, of 0.45; and,
δ = the moist bulk unit weight of the retained backfill; 21.5 kN/m^3.

and,
q = the value for any adjacent surcharge in kPa, which may be acting close to the wall; and,
h = the depth, in m, at which the pressure is calculated

Backfill materials required for behind the retaining structure is assumed to meet an OPSS 1010 Granular B Type 1 pit-run sand and gravel material or OPSS 1010 Granular A. The granular fill should be compacted to a minimum of 98 % of the material’s SPMDD, or to the levels and backfilling procedures specified. Table 7.1 below provides those lateral earth pressure parameters for the predominant soils anticipated at the site.

Table 7.1: Recommended Lateral Pressure Parameters

Parameter	Site Soils (Generalized)	OPSS 1010 Granular A	OPSS 1010 Granular B Type I
Angle of Internal Friction, ϕ	38°	35°	32°
Unit Weight (KN/m^3)	19.5	23	22
Passive Earth Pressure Coefficient, K_p	4.20	3.70	3.25
At-Rest Earth Pressure Coefficient, K_o	0.38	0.43	0.47
Active Earth Pressure Coefficient, K_a	0.24	0.27	0.31

7.2 Hydrostatic Pressure Considerations

For waterproofed, subsurface walls below the established groundwater level, the pressure distribution on the wall should include the hydrostatic pressure. The determination of hydrostatic pressure should be based on the following expression:

$$P_2 = \delta_w h_w$$

where:
P_2 = hydrostatic pressure;
δ_w = unit weight of water; 9.8 kN/m^3; and,
h_w = depth of wall, below reported water level.

8.0 EXCAVATION AND BACKFILL CONSIDERATIONS FOR CONSTRUCTION

8.1 General Excavation Considerations for Soils

All temporary excavations and unbraced side slopes in the soils should conform to standards set out in the Occupational Health and Safety Act, Ontario Regulation 213/91 "*Construction Projects*" (herein "*OHSA*"). The subsurface soils to be encountered during excavation at the site are expected to behave as "*Type 2*" materials according to the OHSA classification in Part III. Type 2 soils are characteristic of the "*clayey silt to silty clay, silt till, and clayey silt to silty clay till*" deposits encountered beneath the site.

It should be possible to excavate the overburden soils with a hydraulic backhoe. Moist Type 2 soils are expected to be stable for short construction periods at slopes of approximately 45° to the horizontal (i.e., 1V:1H). According to the OHSA the excavation slope should be cut and shaped to meet the OHSA requirements for the soil with the highest classification number.

Excavations for new foundations will be required to satisfy the criteria given in the example shown in Figure 8.1.1. This is to avoid overlapping stresses and minimize the risk of undermining existing adjacent structures, including utilities, and/or triggering additional settlements of the existing structures due to soil disturbance.

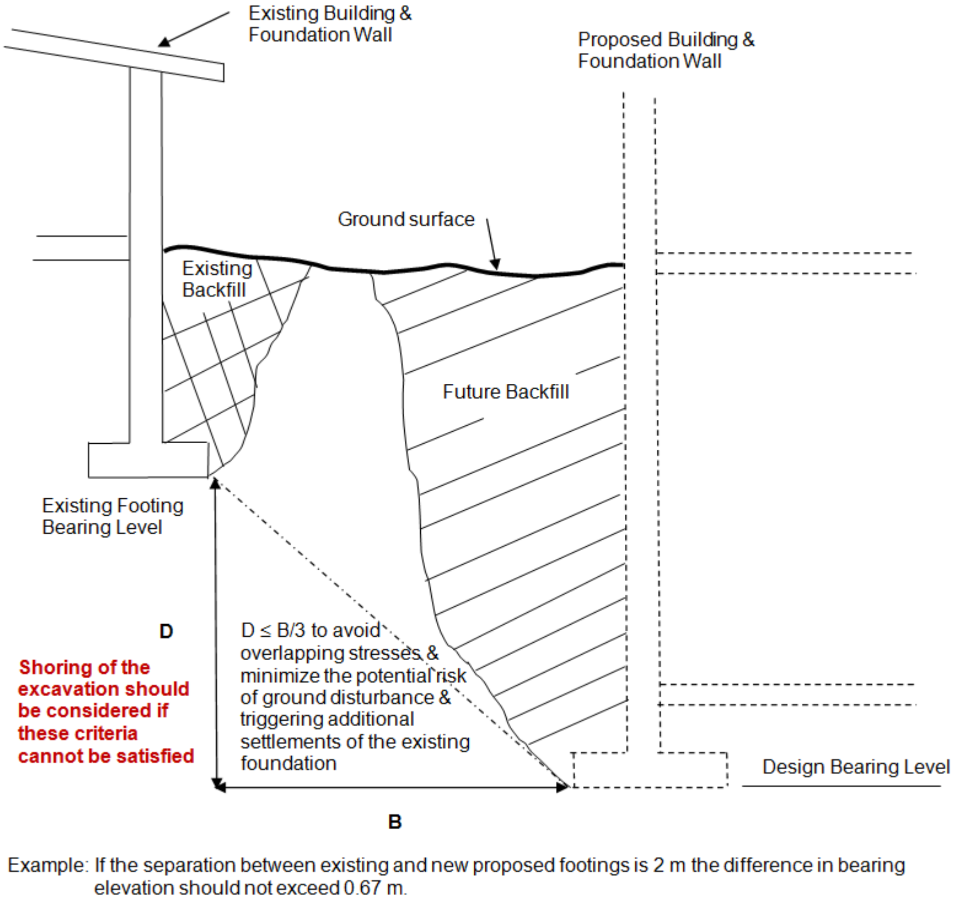


Figure 8.1.1: Criteria for Assessing Excavation Shoring Requirements (Not to Scale)

Consideration should be given to any existing trench excavations and associated backfill that may be present directly behind cut slopes within native soils that may appear to be stable on first excavation. In these circumstances, slopes can suddenly slough or collapse due to the effects of the adjacent backfill.

Consequently, for excavation conditions that cannot satisfy the OHSA requirements for unbraced 1H:1V side slopes, a trench box system should be used, or temporary shoring should be installed to maintain safe working conditions. Outline considerations for temporary shoring are provided in Section 8.4 of this report. In any event, the shoring design should be based on the procedures outlined in the latest edition of the "*Canadian Foundation Engineering Manual*".

8.2 Short-Term (Construction) Dewatering Considerations

Though no conceptual development plans have been provided at the time of issue of this report, elements of the site development are expected to include multiple levels of basement. As such, groundwater is expected to be encountered within basement excavations, particularly where two or more basement levels are proposed.

Considerations and parameters regarding dewatering, including the "*seasonally highest groundwater level*", are provided by Landtek's Hydrogeological Assessment for the White Church Road development, as reported under separate cover.

8.3 General Backfill Considerations

Backfill next to foundation walls should be selected to be compactable in narrow trench conditions. The native soils encountered at the site are expected to be reusable as trench backfill and backfill around the proposed structures on the site. Any variation in the moisture contents of the soils encountered may require selective separation of material to avoid the use of wet soil.

During inclement weather the native soils may become too wet to achieve satisfactory compaction. If construction is proposed for late in the year, a reduced level of trench compaction with a higher risk of future settlements is to be anticipated, and it is recommended that provisional contract quantities be established for the supply and placement of imported granular fill under such circumstances. The imported granular should meet the requirements of OPSS 1010 for Granular B Type I material as a minimum requirement.

8.4 Temporary Shoring Considerations

The installation of temporary shoring is also recommended to maintain safe working conditions and eliminate the possibility of loss of ground and damage to nearby structures and buried utilities on the adjacent road allowances during excavation for basement construction.

The requirement and application of shoring to support excavation side slopes will be dependent on the required excavation depth and the proximity of existing or newly constructed infrastructure adjacent to the excavation.

The preferred method of shoring for deeper excavation is expected to consist of a concrete caisson wall, though timber lagging may be considered for shallower basement excavations (i.e., one to two basement levels). This type of system is expected to provide the additional benefit of sealing the excavation from water penetration and loss of soil fines into the open excavation. Soldier piles and timber lagging may be considered as an option for a shoring system, though this type of system may require measures to prevent the loss of soil between the spaces of lagging boards where a wet or flowing soil layer may be present.

The shoring methods may provide lateral restraining force through the use of rakers or tieback anchors. Tieback anchors provide additional advantage since they do not protrude into the excavations as rakers would. However, the use of tieback anchors is also dependent upon whether permission is needed or whether it is physically possible to extend the anchors to the required distance into neighbouring properties.

Consideration should be also given to lateral and vertical movement of shoring systems being monitored during construction to ensure that movements are within the acceptable range.

It should be noted that the design of any temporary shoring system is the responsibility of the Contractor. Therefore, a specialist shoring contractor should be consulted to provide the most appropriate shoring type method and associated installation procedures. In any event, the shoring design should be based on the procedures outlined in the latest edition of the Canadian Foundation Engineering Manual. It is also recommended that lateral and vertical movement of the shoring system be monitored during construction to ensure that movements are within the acceptable range.

9.0 UTILITIES AND SERVICING CONSIDERATIONS

9.1 Service Installation Using Trenchless Methodologies

9.1.1 General Background

It is anticipated that deeper, truck services will be installed using trenchless methodologies. A brief summary of tunnelling methodology options is provided in Table 9.1.1.1, though it is anticipated that "*Jack and Bore*" (horizontal auger boring) methodologies will be the preferred. A specialist Tunneling Contractor should, however, be consulted to determine the most appropriate methodology.

Table 9.1.1.1: Summary of Tunneling Options

Method	Comments	Recommendations
Jack and Bore	• Dewatering may be required depending on the long-term groundwater conditions. • Requires installation of the launch and reception shafts and the thrust block. • No active control of ground loss at the face.	• May be a suitable option but does not allow active control of ground loss. • Boulders and cobbles pose considerable challenge for the method.
Horizontal Directional Drilling (HDD)	• Angle of entrance and exiting may be too steep, but not impossible.	• This method can be used for most ground conditions except for the presence of obstructions such as cobbles and boulders. • HDD may be deemed appropriate for poorer soil conditions, as per OPSS 450.
Pipe Ramming	• Dewatering may be required depending on long term groundwater condition.	• Minimizes the face ground loss but may cause unacceptable levels of vibrations.
Tunnel Boring Machine (TBM)	• Active control of face pressure and ground loss. • Requires installation of the launch and reception shafts and the thrust block. • Large cobbles may pose a challenge.	• May be a suitable option. Cost could be a consideration.
Pipe Jacking with TBM	• Considered uneconomical.	• May be objectionable based on cost.
Micro-Tunneling	• Active control of face pressure and ground loss. • Requires installation of the launch and reception shafts and the thrust block. • Remote control requires highly specialised contractor. • Large cobbles may pose a challenge.	• May be a suitable option. • Cost could be a consideration.

9.1.2 Subsurface Conditions along the Tunnel Alignments

Based on the profiles provided and the ground conditions encountered, the proposed tunnel at the site will be driven primarily through stiff and very stiff, silty clay and clayey silt deposits, though locally sandy deposits are also expected. The expected soil behaviour is such that excessive settlements during and post tunnelling are not anticipated (i.e., not greater than 5 mm).

The investigation identified groundwater within the screened native soils and therefore, groundwater within the tunnel alignment should be anticipated.

9.1.3 Tunnel Support

The design of any required waterproof primary liner will be the responsibility of the nominated Contractor. In the selection of the type of support, consideration shall be given to the presence of water within the silty and clayey strata, the stabilized groundwater levels reported along the proposed tunnel alignment and the need to prevent the infiltration of any fines into the tunnel opening, as this may result in the loss of ground support and the eventual overstressing or even the collapse of the primary liner system.

The design of the flexible primary tunnel support is to consider the following loading conditions:

- Ring loads caused by uniformly distributed radial earth pressure assumed to be equal to the full vertical earth pressure at the spring line of the tunnel. A unit weight of 20.5 kN/m^3 is to be assumed for the native soils overlying the spring line. Below the groundwater table the submerged unit weight should be used but the full piezometric groundwater pressure should be added to the earth pressure. In addition, loads from any existing underground utilities and structures that may cause stresses on the tunnel liner should be included;
- Bending and shear stresses caused by the anticipated distortion of the flexible liner. A diametral distortion of not less than 0.5 % of the tunnel diameter is to be assumed, though this could be larger if the contact between the soil and tunnel support around the tunnel is not uniform. This may result from over excavation or the loss of lateral support, particularly where any variability in soil strength is exposed within the tunnel (i.e., locally limited sand or silt seams etc.); and,
- Adequate provision shall be made in the design to prevent buckling by assuring uniform filling and grouting of the annular space behind the liner.

The service being installed should be designed for the full vertical pressure measured at spring line and for a horizontal earth pressure equal to 75 % of the full vertical pressure.

9.1.4 Dewatering

It is anticipated that the primary liner of the tunnels will be watertight. Therefore, dewatering will not be required. However, if the tunnel liners are not to be watertight, then the dewatering requirements provided by the Hydrogeological Assessment report should be applied.

The external water head acting on the shield shall be taken to be equal to the difference between the groundwater elevation measured in the vicinity of the particular section of tunnel and the elevation of the tunnel invert.

9.1.5 Temporary Access Shafts

Anticipated Ground Conditions

Superficial deposits anticipated at shaft locations should be readily excavatable using a suitably sized, hydraulic excavator or a clam shell.

Groundwater conditions are expected to be variable, but generally in the order of approximately 4.0 m to 6.0 m below ground level. Limited piezometric groundwater conditions are also anticipated.

Material Stockpile Management

Exposed, excavated soil stockpiles that are to be re-used as fill on site, should be temporarily covered during wet weather to help maintain their original moisture content. Such stockpiles are prone to wet weather exposure and, as such, the increased moisture contents will make these materials too wet to achieve the required levels of compaction.

Shaft Backfill

Access and egress shafts may be backfilled with on site, native, inorganic materials which have moisture content within ±1 % above and ±2 % below the optimum and are environmentally acceptable. Alternatively, imported granular materials can be used. If long term settlements are to be avoided, then the backfill materials should be placed in maximum 300 mm loose lifts and compacted to a minimum 98 % SPMDD. As an alternative, high performance bedding stone (HBP) or unshrinkable fill (U-fill) could be used.

9.1.6 Construction Instrumentation and Monitoring

Settlement Monitoring

Ground movements and deformations of the existing ground surface within the zone of influence (i.e., settlement trough) of the service pipe should be closely monitored during construction by installing surface monitoring points at ground surface either on or immediately beside any existing structures or underground utilities. Settlement monitoring points should be also installed near the launching shaft in order to estimate from these the expected movements of the structures and/or existing service pipes ahead of undertaking the tunnelling work.

All monitoring points will require installation at a time such that monitoring can be completed for a period of at least seven days before any tunnelling work is commenced. The monitoring of the settlement points will require completion on a daily basis by an Ontario Licenced Surveyor and will be reported in writing to the Geotechnical Engineer within one hour of survey completion.

Monitoring is to continue throughout the duration of the tunnelling works and for a period of two weeks after installation completion, maintaining the same monitoring frequency. If little or no settlement is reported during the post-installation monitoring period then the monitoring frequency is to be reduced to once every four weeks for 12 weeks.

Suggested settlement limits and alert levels that may be applied are provided in Table 9.1.6.1 following.

Table 9.1.6.1: Limits of Tunnelling Settlements

Measured Level of Movement	Alert Level
Review (notify CA Project Manager immediately, proceed with caution, monitor hourly for 3 hours)	5 mm to 9 mm
lert (stop work, notify CA Project Manager immediately, determine resolution before recommencing work)	10 mm or greater

Vibration Monitoring

Full time vibration monitoring is recommended during the shaft and tunnel excavation to protect the existing service and road infrastructure, and adjacent residential properties from the adverse impacts of vibration.

The following 9.1.6.2 provides vibration criteria that are to be applied for any neighbouring structure only.

Table 9.1.6.2: Limits of Vibrations

Frequency (Hz)	Peak particle Velocity (PPV) (mm/s)
Less than 4	8
From 4 to 10	15
More than 10	25

The criteria for "*annoyance*" are more stringent than for those that may result in structural damage. The recommended cautionary vibration criteria are summarized in the following table, Table 9.1.6.3.

Table 9.1.6.3: Suggested Cautionary Vibration Criteria

Structure	Peak Particle Velocity (PPV) (mm/s)	Frequency (Hz)
Residential and Commercial Buildings	8	All frequencies
Buried Services	8	All frequencies

Additional Monitoring Requirements

In addition to the monitoring requirements described in the preceding sections, the following should also be monitored:

- Shaft wall deflection by the installation and monitoring of inclinometers and convergence points;
- Groundwater pumping rates and groundwater levels to prevent excessive groundwater drawdown;
- Removed soil volumes per meter of tunnel excavated and grout volumes to monitor over-excavation; and,
- The soil types encountered at the tunnel face.

9.2 Service Installation By Trench Excavation

All temporary, open-cut service excavations and unbraced side slopes in the soils should conform to standards set out in the Occupational Health and Safety Act (herein "*OHSA*"). The subsurface soils to be encountered during excavation at the site are expected to behave as "*Type 2*" materials according to the OHSA classification in Part III. Type 2 soils are characteristic of the "*clayey silt to silty clay, silt till, and clayey silt to silty clay till*" deposits encountered beneath the site.

It should be possible to excavate service trenches through the overburden soils using a hydraulic backhoe. Moist Type 2 and Type 3 soils are expected to be stable for short construction periods at slopes of approximately 45° to the horizontal (i.e., 1V:1H). However, there may be service trenches and backfill situated directly behind cut slopes that appear to be stable. In these cases, slopes can suddenly slough or collapse due to the adjacent backfill. Consequently, for trench conditions that cannot satisfy the OHSA requirements for unbraced 1H:1V side slopes, a trench box system should be used to maintain safe working conditions.

Based on the findings of each borehole location and the proposed service installation depths, significant ground vibrations resulting from open-trench, excavation works are not expected other than those associated with normal construction activities.

Considerations regarding trench excavation dewatering are provided in Landtek's Hydrogeological Assessment report for the site, as reported under separate cover.

As required by the Corporation of the City of Hamilton (herein "*City of Hamilton*"), the trench is to be backfilled with either selected, approved excavated native soil or OPSS 1010.MUNI Granular "A" or "B" Type II material, though maximising the re-use of excavated native soils is preferred and can be managed based on the findings of Landtek's Soil Classification Report, as provided under separate cover.

The trench backfill should be uniformly compacted to a density that minimizes the risk of long-term settlements. The target compaction specification for trench backfill is 95 % Standard Proctor Maximum Dry Density (herein "*SPMDD*").

The excavated native soil should generally be considered to be re-usable from a geotechnical perspective, though may subject to any required moisture conditioning. Where used, and during inclement weather, the excavated soils may become too wet to achieve satisfactory compaction. If construction is proposed for late in the year, a reduced level of compaction with a higher risk of future settlements is to be anticipated. Therefore, it is advised that the fill placement and compaction protocol be discussed and agreed upon at a preconstruction meeting to minimize the risk of settlements.

9.3 Municipal Sewer Pipe Installation

9.3.1 Pipe Installation Considerations

It is expected that new storm sewer infrastructure will be installed below the minimum cover depth of 1.2 m below existing pavement surface and new sanitary sewer infrastructure below the minimum cover depth of 2.75 m below existing pavement surface, as per City of Hamilton Engineering Standards requirements. The subgrade support conditions under the sewer pipes are anticipated to be primarily of native silty and clayey deposits. It is considered that the native soils generally present favorable support conditions for sewer installation.

Should soft or very loose soils be encountered during construction, such soft areas should be sub-excavated and replaced with suitably compacted, engineered fill and approved by a Geotechnical Engineer to redevelop the required subgrade. A Geotechnical Engineer should be engaged during construction to examine the exposed sub-soil quality and condition, and confirm the subsurface conditions are consistent with design assumptions. This is in compliance with field review requirements in the National Building Code, Volume 1, Clause 4.2.2.3.

9.3.2 Foundation Considerations for Associated Infrastructure

Founding Subgrade Considerations

It is expected that any proposed access or connection chambers associated with the proposed sewers installations, can be founded in the undisturbed, native soils for a geotechnical reaction of 100 kPa at the SLS, and for a factored geotechnical resistance of 150 kPa at the ULS.

Subsurface conditions can vary over relatively short distances, and the subsurface conditions revealed at the test locations may not be representative of subsurface conditions across the site. Therefore, a Geotechnical Engineer should be engaged during construction to examine the exposed sub-soil quality and condition, and confirm the subsurface conditions are consistent with design assumptions. This is in compliance with field review requirements in the National Building Code, Volume 1, Clause 4.2.2.3.

Settlement Considerations

It is anticipated that the loads to be applied to the ground by any such structures will be generally very low in intensity. As such, associated settlements are not expected to be large. Therefore, the general limiting of the total settlement to 25 mm and the differential settlement to 19 mm by the recommended geotechnical reaction at the SLS is considered appropriate.

Seismic Design Considerations

In accordance with Table 4.1.8.4.A. of the current Ontario Building Code (herein "*OBC*") the subject property is considered to be a "D" Site Class. The acceleration and velocity-based site coefficients, F_a and F_v, should be determined from Tables 4.1.8.4.B. and 4.1.8.4.C. respectively of the OBC for the above recommended Site Class. The seismic design data given in Table 1.2 of Supplementary Standard SB-1 in Volume 2 of the OBC, for selected Municipal locations, should be used to complete the seismic analysis.

9.3.3 Bedding Cover and Backfill

There is no indication that special pipe bedding materials or procedures are required for the installation of rigid sewer pipes. All bedding cover and backfill materials should be selected in accordance with OPSS 1010 Aggregates – Base, Subbase, Select Subgrade, and Backfill Material, or City of Hamilton requirements, whichever is more stringent.

The pipes should be placed with a minimum bedding thickness in conformance of OPSD 802.010 series (typical 150 mm for rigid pipes, OPSD 802.010, 802.013 and 802.014). The use of normal Class B type bedding is applicable for the pipe.

Bedding material shall be placed in layers not exceeding 300 mm in thickness, loose measurement, and compacted to 100 % of the SPMDD before a subsequent layer is placed. Bedding on each side of the pipe shall be completed simultaneously. At no time shall the fill levels on each side of the storm and sanitary sewer pipe differ by more than one, 300 mm uncompacted layer.

9.4 Municipal Watermain Installation

9.4.1 Watermain Installation Considerations

As is expected that new watermain will be installed such that the top of pipe will be at depths of greater than 1.6 m below existing pavement surface, per City of Hamilton Engineering Standards requirement. At this depth, it is expected that native silty and clayey soils will be encountered. It is considered that the native soils generally present favorable support conditions for watermain installation and thrust block design and construction. Where fill materials are encountered at subgrade levels, inspection and localized remediation works may be required to overcome any potential for differential settlements to the service installation.

When backfilling the trench excavation, consideration should be also given to the requirement of clay seals or "*water stops*", as defined by OPSD 802.095. Clay seals prevent erosive run-off velocities from developing in the trench and are typically constructed of geotextile socks filled with less pervious, organic-free soils (i.e., soil permeability $k< 10^{-8}$ m/s).

The spacing of clay seals is to be selected based on a detailed Hydraulic Assessment, but 50 m to 100 m spacing is generally used for preliminary design purposes. In general, clay seals may not be required for fall gradients of less than 0.5 %. It should be noted however, that clay seals are required at all watercourse crossings, regardless of the fall gradient. It should be also noted that clay seal design is beyond the scope of geotechnical design.

In addition to clay seals and for proposed watermain installations, concrete thrust blocks should be installed against competent native soils, as per the requirements of the OPSD 1101 Series. It is recommended that the thrust blocks bear against native undisturbed soils and be designed for an average allowable resistance bearing pressure of 75 kPa.

Disturbed soil is subject to compression upon loading and therefore does not present favourable bearing conditions to support the proposed watermain installation. Therefore, should localized fill or other previously disturbed soil conditions be encountered during installation, alternative pipe restraint methods should be used, such as a mechanical joint pipe. Any areas of softer soils that yield notable deflection should be sub-excavated and replaced with suitably compacted, engineered fill and approved by a Geotechnical Engineer.

9.4.2 Foundation Considerations for Associated Infrastructure

Founding Subgrade Considerations

Based on the findings of the investigation, it is considered by Landtek that any proposed access chambers or valve boxes associated with the proposed service installations, can be founded in the undisturbed, native soils for a geotechnical reaction of 100 kPa at the SLS, and for a factored geotechnical resistance of 150 kPa at the ULS.

Subsurface conditions can vary over relatively short distances and the subsurface conditions revealed at the test locations may not be representative of subsurface conditions across the site. Therefore, a Geotechnical Engineer should be engaged during construction to examine the exposed sub-soil quality and condition, and confirm the subsurface conditions are consistent with design assumptions. This is in compliance with field review requirements in the National Building Code, Volume 1, Clause 4.2.2.3.

Settlement Considerations

It is anticipated that the loads to be applied to the ground by any such structures will be generally very low in intensity. As such, associated settlements in soils are not expected to be large. Therefore, the general limiting of the total settlement to 25 mm and the differential settlement to 19 mm by the recommended geotechnical reaction at the SLS is considered appropriate.

Seismic Design Considerations

In accordance with Table 4.1.8.4.A. of the current OBC the subject property is considered to be a "*D*" Site Class. The acceleration and velocity-based site coefficients, F_a and F_v, should be determined from Tables 4.1.8.4.B. and 4.1.8.4.C. respectively of the OBC for the above recommended Site Class. The seismic design data given in Table 1.2 of Supplementary Standard SB-1 in Volume 2 of the OBC, for selected Municipal locations, should be used to complete the seismic analysis.

9.4.3 Watermain Bedding and Cover

Watermain bedding and cover material shall be placed in accordance with the City of Hamilton specification for the installation of watermains.
All bedding cover and backfill materials should be selected in accordance with OPSS.MUNI 1010 Aggregates – Base, Subbase, Select Sub-grade, and Backfill Material, with bedding consisting of

Granular “A” material per City of Hamilton requirements. Bedding and cover for small diameter water services shall be Granular “D” material.

Bedding material shall be placed in layers not exceeding 300 mm in thickness, loose measurement, and compacted to 100 % of the SPMDD before a subsequent layer is placed. Bedding on each side of the pipe shall be completed simultaneously. At no time shall the fill levels on each side of the watermain pipe differ by more than one, 300 mm uncompacted layer.

9.5 Private Servicing Considerations

There is no indication that special pipe bedding materials or procedures are required for the installation of private services. All bedding cover and backfill materials should be selected in accordance with OPSS 1010 Aggregates – Base, Subbase, Select Subgrade, and Backfill Material.

Service pipes and conduits should be placed with a minimum bedding thickness in conformance of Ontario Provincial Standard Drawing (herein “*OPSD*”) 802.010, 802.013 and 802.014 for flexible pipe and OPSD 802.030, 031, 032, 033 and 034 for rigid pipes. The type of bedding shall be selected to suit the applicable pipe strength and site conditions.

Bedding material shall be placed in layers not exceeding 300 mm in thickness, loose measurement, and compacted to 95 % of the SPMDD before a subsequent layer is placed. Site servicing trench backfill should be uniformly compacted to a density that minimizes the risk of long-term settlements. Bedding on each side of the pipe shall be completed simultaneously. At no time should the levels on each side differ by more than the 300 mm uncompacted layer. The remainder of the trench should be backfilled as per the requirements defined in Sections 5.1.2 and 8.0 of this report.

It is assumed all private services will have a minimum of 1.2 m of soil cover for frost protection. For services installed at shallower depths, suitable insulation for frost protection is recommended.

9.6 Stormwater Management Pond Considerations

At the time of issue of this report, it is understood that seven Storm Water Management (herein “*SWM*”) ponds are proposed across the White Church Road development site area. It is expected that the pond designs will be of a pond with a permanent level of water retention and will be constructed by excavation into native soils.

In accordance with the City of Hamilton document “*City of Hamilton Criteria and Guidelines for Stormwater Infrastructure Design*”, dated April 16, 2009, the requirements for new Stormwater Management Pond design include for the side slopes to be of an angle no greater than 4H:1V.

It is anticipated that outfalls of the ponds will be such that the ponds will be retaining water during rainfall or snow melt events and will be in the order of 1.5 m to 2.0 m above the pond base. The high-water (100-year ponding) level of the ponds will be in the order of 3.0 m to 3.5 m above the pond base. On this basis and based on the findings of the investigation completed at the site, particularly the absence of groundwater within the anticipated SWM pond profile, it is anticipated that the pond base will be above any static or piezometric groundwater regime beneath the site and thus will not require any considerations towards hydraulic uplift.

It is considered that pond construction will only require the inclusion of a ‘standard’ liner to reduce any potential communication between any deeper groundwater system and the stormwater retained by the pond. This is in accordance with the “*City of Hamilton Criteria and Guidelines for Stormwater Infrastructure Design*” and will be required for each SWM pond location. The following recommendations and general comments are provided for consideration for the SWM pond liner design:

- Clay liner materials required should be of high clay-containing soils of low permeability; in the order of 1 x 10^{-6} to 1 x 10^{-7} cm/s to prevent water permeation and maintain their nominal density. There is potential for such native materials to be available from within the development site area, particularly where silty clay non-till soils are present;
- A minimum clay liner thickness of 300 m is considered appropriate at this preliminary stage for pond liner structures, though may be increased if groundwater is present at shallow depths;
- A geo-synthetic liner may be considered as an alternative to the clay liner material if grading or excavation for the required pond liner subgrade presents any issues, groundwater is present at shallow depth, or to ensure total separation of the water retained in the pond from the local groundwater regime. If this alternative is considered then a Bentofix SNRWL Series product is recommended, specifically a Thermal Lock ® Geosynthetic Clay Liner (GCL), consisting of 90% montmorillonite clay as a minimum, with reinforced geotextile upper and lower layers; and,
- Pond side slopes of 4H:1V should be protected from erosion by an appropriate vegetative cover.

10.0 SOIL CORROSIVITY AND SUBSURFACE CONCRETE

10.1 Soil Corrosivity

Twelve selected, composite soil samples were obtained from the boreholes associated with the proposed development and submitted to Paracel Laboratories for analysis of pH, soil conductivity, resistivity and concentrations of sulphates, and chlorides (Soil Corrosivity).

The American Water Works Association (AWWA) document, "*Polyethylene Encasement for Ductile-Iron Pipe Systems*" *ANSI/AWWA C105/A21.5-18,* dated December 1, 2018, uses a 10-point scoring method to determine the soil corrosivity potential. For each given soil sample, points were assigned to the different parameters to evaluate their contribution towards the corrosivity of soil.

The test results are provided in Appendix E and are summarized in Table 10.1.1.

Table 10.1.1: Results of Soil Corrosivity Testing

Borehole and Sample ID	Chloride (µg/g)	Sulphate (µg/g)	pH (pH units)	Resistivity (ohm.cm)	Moisture (%)	Total ANSI/AWWA Points
BH1 - SS4 and SS5	<10	199	7.78	3530	18.1	1
BH3 - SS4 and SS5	<10	962	7.78	1270	23.5	3
BH4 - SS3 and SS5	<10	199	7.78	3530	18.1	1
BH6 - SS4 and SS5	<10	962	7.78	1270	23.5	3
BH8 - SS4 and SS5	<10	199	7.78	3530	18.1	1
BH9 - SS3 and SS5	<10	962	7.78	1270	23.5	3
BH10 - SS3 and SS5	<10	199	7.78	3530	18.1	1
BH11 - SS3 and SS5	<10	962	7.78	1270	23.5	3
BH13 - SS3 and SS5	<10	199	7.78	3530	18.1	1
BH16 - SS3 and SS5	<10	962	7.78	1270	23.5	3
BH17 - SS6 and SS7	<10	199	7.78	3530	18.1	1
BH20 - SS6 and SS7	<10	962	7.78	1270	23.5	3

Corrosion protection for buried ductile-iron pipes is recommended, when a score of 10 points or greater is reported. Based on the total ANSI/AWWA values above of 1 to 3, ductile-iron pipes used at the site will not require corrosion protective measures such as cathodic protection. It should be noted that the analytical results only provide an indication of the potential for corrosion.

The contribution of chloride ions to soil corrosivity towards buried metallic improvements or steel structures is very significant. According to the Corrosion Guidelines (Caltrans, January 2015, version 2.1), a site is considered corrosive if, "*chloride concentration is 500 ppm or greater, sulphate concentration is 2,000 ppm or greater, or the pH is 5.5 or less*. "

In addition, the Canadian Standards Association (CSA) A23.1-14 "*Concrete materials and methods of concrete construction*", Table 3, "*Additional requirements for concrete subjected to sulphate attack*", states that design requirements for sulphate resistant concrete are only necessary when the water-soluble sulphate content of the soil in which the concrete is to be embedded is greater than 0.1 % (1,000 µg/g).

The representative soil samples at the site are reported to contain chloride ion concentrations of <10 µg/g (<0.01 %), and sulphate concentrations between 199 µg/g (0.0199 %) and 962 µg/g (0.0962 %). These equate to an average of <10 µg/g and 581 µg/g, respectively, and indicate a very limited, local potential (i.e., "*low risk*") of sulphate attack on buried reinforced concrete structures.

10.2 Concrete Class Considerations

The requirements for subsurface concrete subject to a sulphate and chloride environment are presented in Canadian Standards Association specification, CSA A23.1-14 *"Concrete Materials and Methods of Concrete Construction, Tables 1-4"*. Experience in the area indicates that the native soils generally have a mild sulphate environment and a low chloride concentration. It is recommended that subsurface concrete at the site have the characteristics for normal (GU) Portland cement.

For parking garage decks and ramps where proposed, it is recommended that the concrete exposure class be C-1 and the concrete have the following minimum properties:

- minimum 56-day compressive strength: 35 MPa;
- maximum water to cement ratio: 0.40;
- chloride ion penetrability requirement: < 1500 coulombs (within 91 days)
- cementing materials: GU (general use hydraulic cement) or GUb (blended general use)
- air content: as per CSA A23.1-14 Table 4, air content category 1 (freeze-thaw environment)

The concrete should be placed without segregation and should be consolidated to achieve a uniform dense mass.

10.3 Methods for Specifying Concrete

Alternative methods of specifying concrete for a project are outlined in CSA A23.1-14 and allow for "*Performance*" or "*Prescription*" based methods. Each method attaches different levels of responsibility to the owner, the contractor, and the concrete supplier. The pros and cons of each method should be examined prior to completion of the specifications for the project.

11.0 SOIL MANAGEMENT CONSIDERATIONS

It is anticipated that the various parcels of development at the site will involve some element of cut and fill operations. From a geotechnical perspective, and in order to optimize the use of the on-site soils, a Soil Management Plan should be established in accordance with the requirements of Ontario Regulation (herein "*O. Reg.*") 406/19 for excess soils and O. Reg. 153/04 for soil stockpiles.

The plan objective should be to achieve a self-sustainable development with respect to excavated materials and control the placement of organic soils so that there is negligible impact on the settlement performance of the compacted fill material. The soil management criteria should be per the following sections, as a minimum:

11.1 Organic and Deleterious Materials

Surface vegetation, topsoil and organic soils should not be placed within the proposed roadways, below finished subgrade level for pavement construction or building limits. These materials should be placed in landscaped areas where settlements are not critical.

11.2 Materials Reuse Management

11.2.1 Fill Compaction Requirements

Excavated soils for structural fill in pavement areas and building floor slab areas, which do not have topsoil or organic matter and are compactable with moisture contents within 2 % to 3 % of the optimum value, should be placed and compacted to a target density of 97 % of the SPMDD with no individual test result below 95 % SPMDD.

If engineered fill is required to support building foundations:

- the engineered fill should be placed and compacted in lifts to a target density of 100 % SPMDD with no individual tests below 98 % SPMDD; and,
- the soil should be placed in a loose lift thickness not exceeding 250 mm and should be compacted using a large (10 ton or larger) pad-foot type roller with vibratory capability.

If engineered fill to support building foundations is being considered, it is recommended that a pre-construction meeting be scheduled to review the proposed fill materials, fill placement and compaction procedures, and the testing and inspection requirements.

Soils to be placed in landscaped areas where settlements are not critical should receive nominal compaction effort in order to achieve at least 90 % of the SPMDD.

11.2.2 Structural Fill Subgrades

Prior to the placement of any structural fill materials, the exposed subgrade soil should be inspected and proof-rolled using a loaded tandem axle truck and traversing the exposed subgrade for full coverage. The proof-rolling should be monitored by a geotechnical representative of this office to delineate any soft areas which may require repair.

12.0 PAVEMENT CONSIDERATIONS

12.1 Private At-Grade Asphalt Pavement Design Considerations

Though no design plans have been provided to Landtek at the time of issue of this report, the proposed development is anticipated to include both Municipally adopted and private pavement structures. Private pavements are expected to include new access routes, condominium road and deck pavements.

Recommended pavement structure layer thicknesses for private pavements are provided in Table 12.1.1. The recommended pavement design section considers the accepted design practice that the total pavement structure thickness should meet or exceed one-half the anticipated depth of frost penetration for the geographical area (i.e., approximately 1.2 m) or as close as practicable.

Table 12.1.1: Recommended Private Asphalt Pavement Structure Layer Thicknesses

Pavement Layer	Light Duty Pavement Areas	Access and Fire Routes
Surface Course Asphalt OPSS HL 3	40 mm	40 mm
Binder Course Asphalt OPSS HL 8	50 mm	60 mm
Granular Base OPSS Granular A	150 mm	150 mm
Granular Subbase OPSS Granular B, Type II	300 mm[1]	350 mm[1]
Total Thickness	540 mm	600 mm

Notes:
1. If construction proceeds late in the year (i.e., November and December), the design thickness of pavement granular materials may have to be increased to address potential problems with subgrade instability and facilitate construction vehicle and truck access.

12.2 Municipal At-Grade Asphalt Pavement Design Considerations

It is anticipated that Municipally adopted pavements to be constructed for the development will comprise primarily of ‘residential local’ or ‘residential collector’ road pavement classifications.

The full-depth pavement structure designs presented in Table 12.2.1 are the standard designs presented by the City of Hamilton’s document “*Pavement Design and Rehabilitation Criteria*”, dated 2023.

Table 12.2.1: Recommended Municipal Pavement Structure Layer Thicknesses

Pavement Layer	Pavement Material	City of Hamilton Pavement Class	
		Residential Local	Residential Collector
Surface Course	SP12.5 (Traffic Category C)	40	40
Binder Course	SP19.0 (Traffic Category C)	80	100
Base Course	OPSS Granular A	150	150
Subbase Course	OPSS Granular B Type II	300	300
	Total Thickness	±570 mm	±590 mm

12.3 Sub-grade Preparation and Drainage

The overall performance of the pavement structure will greatly depend upon the support provided by the developed subgrade. A number of factors should be considered at the construction stages to ensure that an acceptable subgrade condition is developed and maintained:

- Sub-drains should be installed and should be 100 mm diameter perforated plastic pipe, with outfalls to catch basins at a continuous and uniform grade. The sub-drains and associated connections are to be installed in accordance with the City of Hamilton's Engineering Standards or OPSD 216.01;
- Any soft areas of notable deflection to the subgrade should be sub-excavated and replaced with a suitable backfill material approved by a qualified Geotechnical Engineer and compacted to 98 % of its SPMDD;
- The subgrade should be properly shaped, crowned and then proof-rolled under the full-time observation of a geotechnical representative of this office to delineate any soft areas which may require repair before placing the granular materials; and,
- Surface water should not be allowed to pond on the surface of or adjacent to the outside edges of any developed subgrade.

Should pavements proposed for the development be constructed as a two-stage paving operation it will be important to ensure that the following is undertaken to develop the surface of the binder course being used as a "*temporary*" surface during the construction phase:

- The surface is thoroughly cleaned and power washed to remove all residual contaminants;
- All deficiencies are corrected to meet the required design specifications; and,
- A suitable tack coat is appropriately applied immediately prior to the placement of the upper asphaltic concrete course(s).

Such preparatory works are to be completed in accordance with the appropriate OPSS, as required.

12.4 Deck Pavement Design Considerations

It is understood that the proposed development will include for medium-and high-rise structures and are likely to include for multiple level of basement parking that cover the structure footprint in full. Pavements for such structured are anticipated to be deck structures rather than standalone or at-grade pavements.

Such deck pavements should comprise a minimum 50 mm cover of OPSS HL 3 asphalt. The bedding or grading material to be placed between the concrete deck and the asphalt pavement surface should comprise either blinding sand or OPSS Granular A material, depending on the thickness of the layer required.

12.5 Pavement Materials

12.5.1 Granular Base Course

If the option with granular base material is used, the granular base course material should meet OPSS Granular "A" specifications. Quarried 20 mm limestone crushed to Granular "A" gradation specifications is recommended.

12.5.2 Hot Mix Asphalt

The surface and binder course asphalt of private pavement structures should meet current specifications for HL 3 and HL 8, respectively, as prescribed by the City of Hamilton or, alternatively, OPSS 1150.

For Municipal pavement structures, the binder course and surface course asphalt should meet current specifications for SP19.0 Traffic Category C and SP12.5 Traffic Category C, respectively per the City of Hamilton's Engineering Standards Form 800.

The standard asphalt binder grade for the climate conditions in Hamilton is PG 58-28. Given the anticipated low volume of commercial truck traffic it is considered that there is no requirement for a bump up to a higher PG grade of asphalt cement.

12.5.3 Material Placement and Compaction

The placing, spreading and rolling of the asphalt should be in accordance with current provincial standards or the City of Hamilton's Engineering Standards Form 800.

Granular base course and subbase course fill material should be compacted to 100 % SPMDD. Hot mix asphalt should be compacted to the criteria set out by the City of Hamilton's Engineering Standards Form 800.

Connections and tie-ins to existing pavement structures should be completed in accordance with OPSS.MUNI.310.

12.6 Sidewalk Considerations

Sidewalk and Multi-Use Pavement Considerations

The design and construction of concrete sidewalks should be completed to the satisfaction of the City of Hamilton's Engineering Standards, and as detailed in Table 12.6.1. The concrete and aggregates should be produced and placed to meet those standards also stipulated by the City of Hamilton's Engineering Standards.

Table 12.6.1: Recommended Minimum Concrete Sidewalk Specifications

Materials	Compaction Requirements	Layer Thickness
Normal Portland GU (32 MPa) (CAN3-CSA A23.1) - Class C-2	N/A	125 mm
Granular "A" Base	95 % SPMDD*	150 mm

* Standard Proctor Maximum Dry Density

Construction joints in concrete sidewalks should be properly sealed (e.g., bitumen filler) to minimize the water migration

It should be noted that the concrete sidewalk design specified in Table 12.6.1 addresses a use by pedestrian traffic only and does not include for use by vehicular traffic. For multi-use sidewalk pavements (i.e., where both pedestrian and bicycle traffic is to be accommodated), the following Table 12.6.2 provides the recommended pavement structure design.

Table 12.6.2: Recommended Multi-Use Sidewalk Pavement Specifications

Pavement Layer	Pavement Material	Recommended Layer Thickness
Surface Course	SP12.5 (Traffic Category C)	80 mm
Granular Base	OPSS Granular "A"	400 mm

The subgrade conditions and bearing strength may be variable along the sidewalk section and some subgrade improvements should be anticipated. It is recommended that prior to the placement of pavement granular fill, the exposed subgrade soil should be inspected and proof-rolled using a loaded tandem axle truck to traverse the exposed subgrade and provide for full coverage. The proof-rolling should be monitored by a geotechnical representative of this office to delineate any soft areas which may require repair. Repairs should be undertaken to avoid creating "*bathtub*" conditions in the subgrade within the pavement structure.

Where finished sidewalks are on level ground, and to ensure that they remain free of ponding water, a final slope/gradient of the sidewalk surface of at least 2 % should be maintained.

13.0 CLOSURE

The Limitations of Report, as stated in Appendix A, are an integral part of this report.

Soil samples will be retained and stored by Landtek for a period of three months after the report is issued. The samples will be disposed of at the end of the three-month period unless a written request from the client to extend the storage period is received.

We trust this report will be of assistance with the design and construction of the proposed development. Should you have any questions, please do not hesitate to contact our office.

Yours sincerely,

LANDTEK LIMITED

LICENSED PROFESSIONAL ENGINEER
R. DI CIENZO
Feb. 4/25
PROVINCE OF ONTARIO

James Dann, B. Eng. (Hons.) ACSM
Manager, Geotechnical Projects

Ralph Di Cienzo, P. Eng.
Consulting Engineer

APPENDIX A
LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the borehole locations. Subsurface and ground water conditions between and beyond the Boreholes may be different from those encountered at the borehole locations, and conditions may become apparent during construction that could not be detected or anticipated at the time of the geotechnical investigation. It is recommended practice that Landtek be retained during construction to confirm that the subsurface conditions throughout the site are consistent with the conditions encountered in the Boreholes.

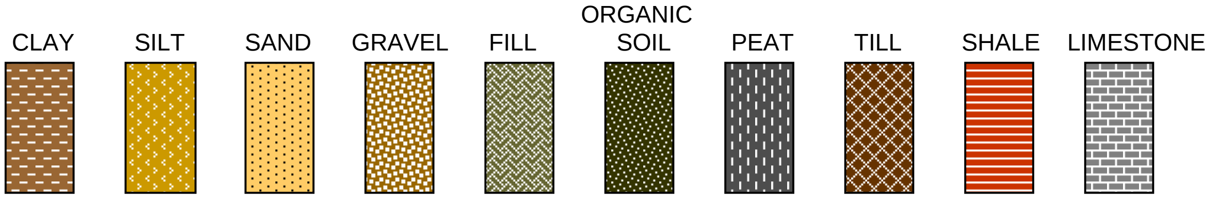
The comments made in this report on potential construction problems and possible remedial methods are intended only for the guidance of the designer. The number of Boreholes may not be sufficient to determine all the factors that may influence construction methods and costs. For example, the thickness and quality of surficial topsoil or fill layers may vary markedly and unpredictably. Additionally, bedrock contact depths throughout the site may vary significantly from what was encountered at the exact borehole locations. Contractors bidding on the project, or undertaking construction on the site should make their own interpretation of the factual borehole information, and establish their own conclusions as to how the subsurface conditions may affect their work.

The survey elevations in the report were obtained by Landtek Limited or others, and are strictly for use by Landtek in the preparation of the geotechnical report. The elevations should not be used by any other parties for any other purpose.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Landtek Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this report.

This report does not reflect environmental issues or concerns related to the property unless otherwise stated in the report. The design recommendations given in the report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, it is recommended that Landtek Limited be retained during the final design stage to verify that the design is consistent with the report recommendations, and that the assumptions made in the report are still valid.

APPENDIX B
SYMBOLS AND TERMS USED IN THE REPORT



RELATIVE PROPORTIONS

Term	**Range**
Trace	0 - 5%
A Little	5 – 15%
Some	15 – 30%
With	30 – 50%

CLASSIFICATION BY PARTICLE SIZE

Boulder -------------------- > 200 mm
Cobble --------------------- 80 mm – 200 mm
Gravel -
  Coarse ---------- 19 mm – 80 mm
  Fine ------------- 4.75 mm – 19 mm
Sand -
  Coarse ---------- 4.75 mm – 2 mm
  Medium -------- 2 mm – 0.425 mm
  Fine ------------- 0.425 mm – 0.75 mm
Silt ------------------------- 0.075 mm – 0.002 mm
Clay ------------------------ < 0.002 mm

DENSITY OF NON-COHESIVE SOILS

Descriptive Term	**Relative Density**	**Standard Penetration Test**
Very Loose	0 – 15%	0 – 4 Blows Per 300 mm Penetration
Loose	15 – 35%	4 – 10 Blows Per 300 mm Penetration
Compact	35 – 65%	10 – 30 Blows Per 300 mm Penetration
Dense	65 – 85%	30 – 50 Blows Per 300 mm Penetration
Very Dense	85 – 100%	Over 50 Blows Per 300 mm Penetration

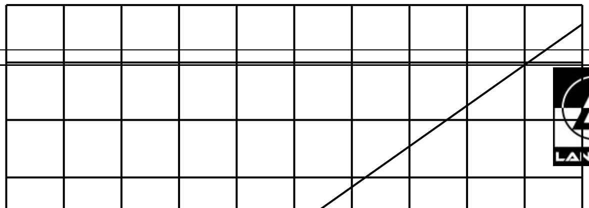
CONSISTENCY OF COHESIVE SOILS

Descriptive Term	**Undrained Shear Strength kPa (psf)**	**N Value Standard Penetration Test**	**Remarks**
Very Soft	< 12 (< 250)	< 2	Can penetrate with fist
Soft	12 – 25 (250 – 500)	2 – 4	Can indent with fist
Firm	25 – 50 (500 –1000)	4 – 8	Can penetrate with thumb
Stiff	50 – 100 (1000 – 2000)	8 – 15	Can indent with thumb
Very Stiff	100 – 200 (2000 – 4000)	15 – 30	Can indent with thumb-nail
Hard	> 200 (> 4000)	> 30	Can indent with thumb-nail

Notes: 1. Relative density determined by standard laboratory tests.
2. N value – blows/300 mm penetration of a 623 N (140 Lb.) hammer falling 760 mm (30 in.) on a 50 mm O.D. split spoon soil sampler. The split spoon sampler is driven 450 mm (18 in.) or 610 mm (24 in.). The “N” value is the Standard Penetration Test (SPT) value and is normally taken as the number of blows to advance the sampler the last 300 mm.

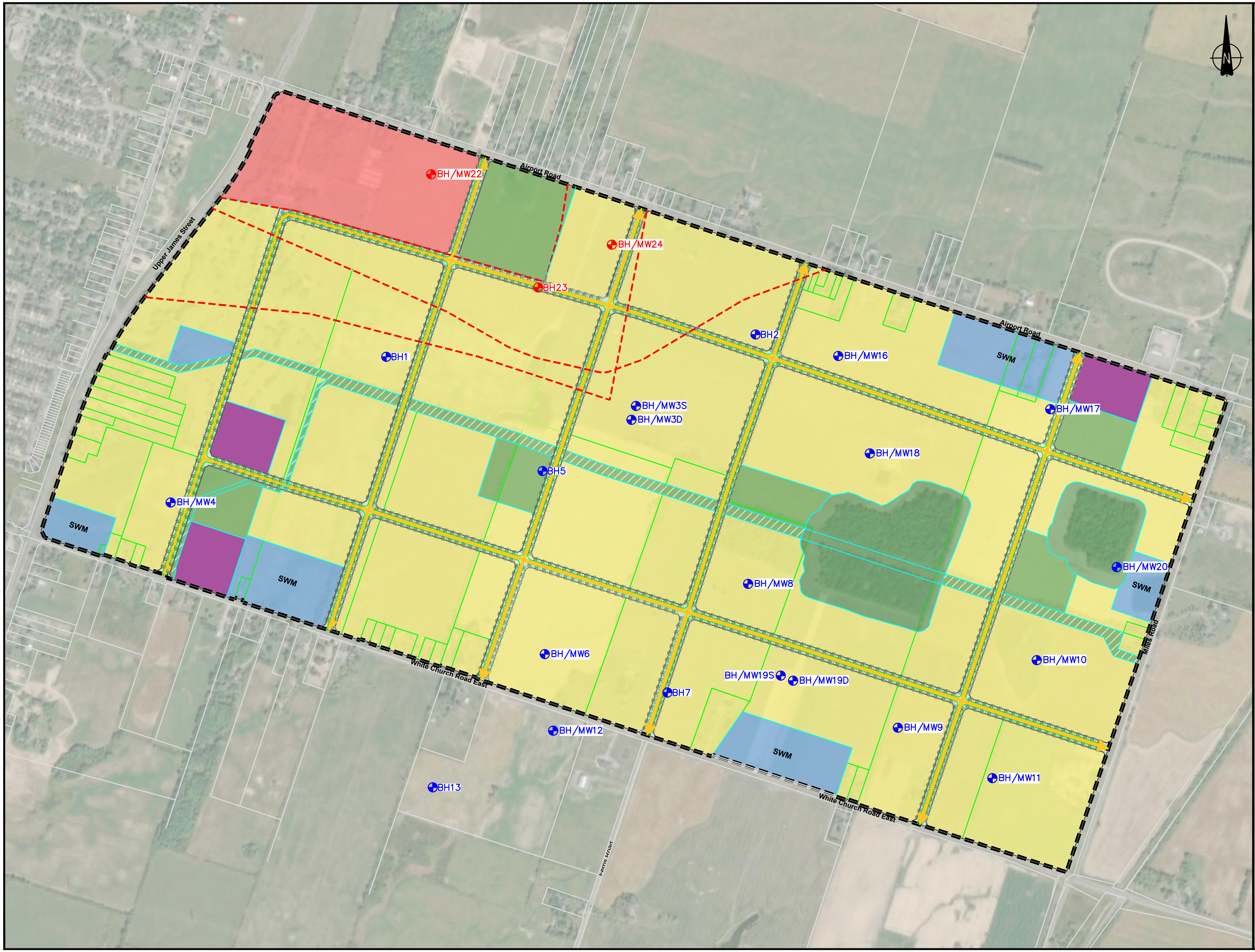
APPENDIX B CONTINUED
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES
ASTM Designation: D 2487 - 69 AND D 2488 – 69
(Unified Soil Classification System)

Major Divisions			Group Symbols	Typical Names	Classification Criteria		
Coarse-grained soils More than 50% retained on No. 200 sieve *	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean gravels	**GW**	Well-graded gravels and gravel-sand mixtures, little or no fines	Classification on basis of percentage of fines Less than 5% pass No. 200 sieve . . . . . . GW, GP, SW, SP More than 12% pass No. 200 sieve . . . . . GM, GC, SM, SC 5 to 12% pass No.200 sieve . . . . Borderline classifications requiring use of dual symbols	C_u=D60/D10 greater than 4; C_z = $(D30)^2$/(D10xD60) between 1 and 3	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines		Not meeting both criteria for GW	
		Gravels with fines	**GM**	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below “A” line or P.I. less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above “A” line with P.I. greater than 7	
	Sands More than 50% of coarse fraction passes No. 4 sieve	Clean Sands	**SW**	Well-graded sands and gravelly sands, little or no fines		C_u=D60/D10 greater than 6; C_z = $(D30)^2$/ (D10xD60) between 1 and 3	
			SP	Poorly graded sands and gravelly sands, little or no fines		Not meeting both criteria for SW	
		Sands with fines	**SM**	Silty sands, sand-silt mixtures		Atterberg limits below “A” line or P.I. less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above “A” line with P.I. greater than 7	
Fine-grained soils 50% or more passes No. 200 sieve *	Silts and clays Liquid limit 50% or less		**ML**	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands	Plasticity Chart For classification of fine-grained soils and fine fraction of coarse-grained soils. Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols. Equation of A-line: PI=0.73 (LL-20) 60 50 CH Plasticity 40 Index 30 OH and MH 20 CL 10 CL – ML ML and OL 0 10 20 30 40 50 60 70 80 90 100 Liquid Limit		
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silts			
			OL	Organic silts and organic silts of low plasticity			
	Silts and clays Liquid limit greater than 50%		**MH**	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity			
	Highly organic soils		**Pt**	Peat, much and other highly organic soils	* Based on the material passing the 3 in. (76mm) sieve.		



APPENDIX C

DRAWING 23354-01 – EXPLORATORY HOLE LOCATION PLAN
BOREHOLE LOGS



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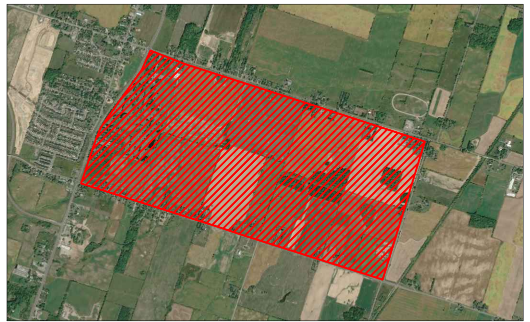
project location



Key plan an extract from Google Earth Pro©

Key:

●	Approximate location of borehole (BH) drilled or groundwater monitoring well (BH/MW) installed by Landtek Limited between 3 and 8 july 2024.
●	Approximate location of borehole (BH) drilled or groundwater monitoring well (BH/MW) installed by Landtek Limited on january 6 2025.
🟨	Future residential development
🟥	Future commercial development
🟪	Future institutional development
▨	Existing and future greenspace (woodland, parkland)

Notes:

Base plan taken from the drawing "*White Church Boundary Expansion Area*", as issued by Urban Solutions Planning & Land Development, with a background extract provided by A. T. McLaren and Aerial Imagery from Google Earth Pro©.

revisions/submissions

#	date	description
1	7 july 2024	issued for draft report
2	28 october 2024	updated property boundary
3	2 december 2024	updated property boundary
4	28 january 2025	additional investigation data

client
White Church Landowners Group Inc.

municipality
The Corporation of the City of Hamilton

project
Geotechnical Investigation
White Church Lands

sheet
Borehole and Monitoring Well Location Plan

date: 7 july 2024
drawn: mdc
checked: jd
project #: 23354
scale: 1:10,000

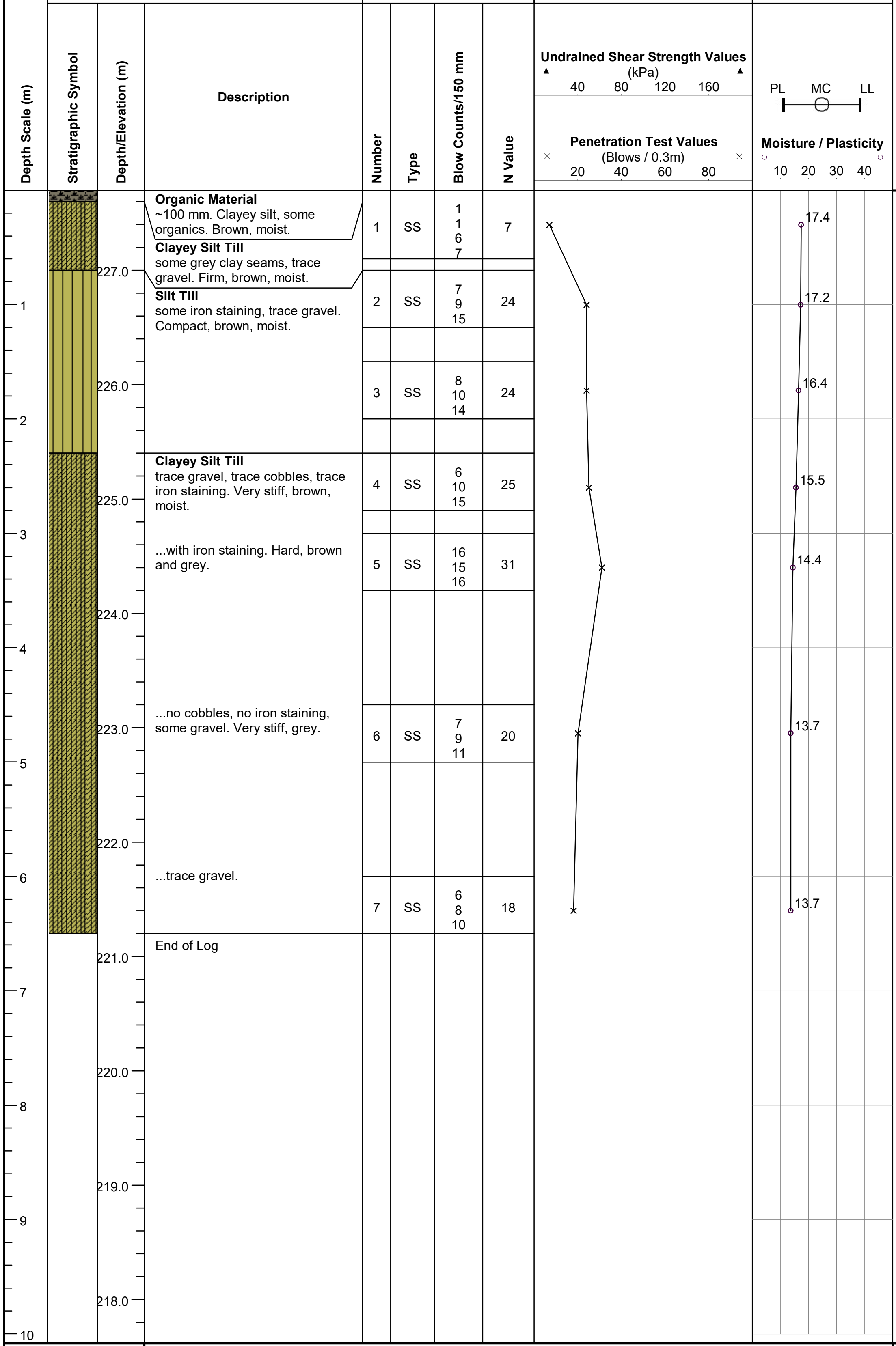
23354-01

LOG OF BOREHOLE BH1

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton
Drill Date: 2024-03-11
Drilling Method: Solid Stem
Datum: Geodetic
Northing: 43.149397
Easting: -79.908197
Ground Surface Elevation: 227.7

Subsurface Conditions	Samples				Well Details	Groundwater Conditions	Headspace / PID (ppm) [LEL(%)] / ppm	Comments
Description	Number	Type	Blow Counts/150 mm	N Value				
Organic Material ~100 mm. Clayey silt, some organics. Brown, moist.	1	SS	1 1 6 7	7				
Clayey Silt Till some grey clay seams, trace gravel. Firm, brown, moist.								
Silt Till some iron staining, trace gravel. Compact, brown, moist.	2	SS	7 9 15	24				
	3	SS	8 10 14	24				
Clayey Silt Till trace gravel, trace cobbles, trace iron staining. Very stiff, brown, moist.	4	SS	6 10 15	25				
...with iron staining. Hard, brown and grey.	5	SS	16 15 16	31				
...no cobbles, no iron staining, some gravel. Very stiff, grey.	6	SS	7 9 11	20				
...trace gravel.	7	SS	6 8 10	18				
End of Log								



Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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LOG OF BOREHOLE BH2

SHEET 1 of 1

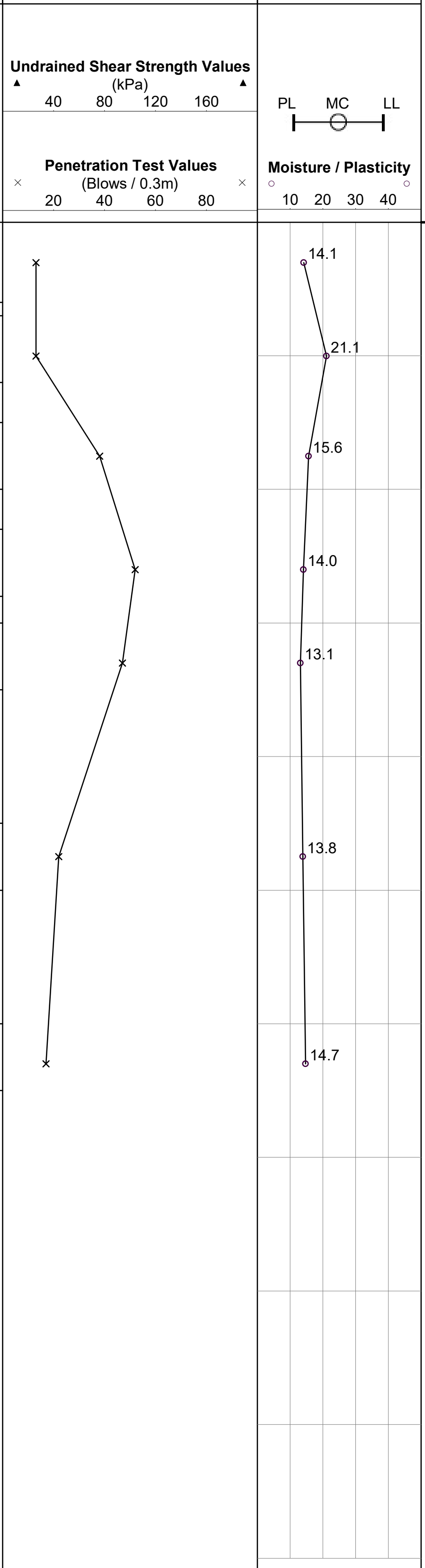
Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-04
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.149763
Easting: -79.896422
Ground Surface Elevation: 227.5

Subsurface Conditions		Samples			
Depth/Elevation (m)	Description	Number	Type	Blow Counts/150 mm	N Value
	Organic Material ~100 mm. Clayey silt, some organics. Brown, dry to moist. **Clayey Silt** some iron staining, trace grey clay seams. Stiff, brown, moist.	1	SS	3 6 7 6	13
227.0		2	SS	3 5 8	13
226.0	...trace iron staining. Hard.	3	SS	7 17 21	38
225.0	**Clayey Silt Till** trace gravel, trace iron staining. Hard, grey, moist.	4	SS	11 19 33	52
224.0		5	SS	9 21 26	47
223.0	...no iron staining. Very stiff.	6	SS	4 10 12	22
222.0		7	SS	6 5 12	17
221.0	End of Log				
220.0					
219.0					
218.0					

Depth Scale (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10



Well Details | Groundwater Conditions | Headspace / PID (ppm) [LEL(%)] / ppm | Comments

Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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LOG OF BOREHOLE BHMW3S

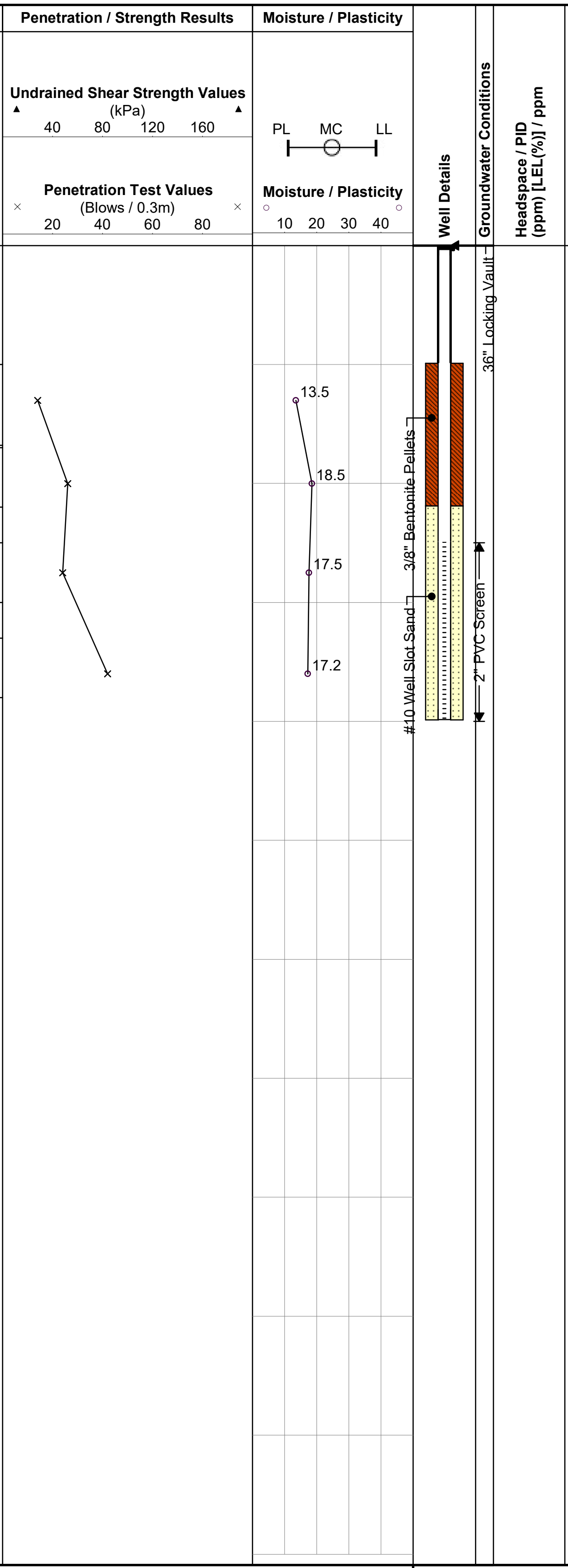
SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-04
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.148049
Easting: -79.900399
Ground Surface Elevation: 230

Depth Scale (m)	Depth/Elevation (m)	Description	Number	Type	Blow Counts/150 mm	N Value	Moisture (%)
0	230.0	**Organic Material** ~100 mm. Clayey silt, some organics. Brown, dry to moist.	1	SS	3 5 9 12	14	13.5
		Clayey Silt trace grey clay seams. Stiff, brown, moist.					
1	229.0	...very stiff.	2	SS	5 9 17	26	18.5
		Clayey Silt Till some grey clay seams, trace gravel. Very stiff, brown, moist.	3	SS	6 9 15	24	17.5
2	228.0	...hard.					
		Silt Till some clay, trace gravel. Dense, grey, wet.	4	SS	7 17 25	42	17.2
3	227.0	...compact.					
		End of Log					
4	226.0						
5	225.0						
6	224.0						
7	223.0						
8	222.0						
9	221.0						
10	220.0						



Comments

Additional Notes:
1. Borehole open to approximately 3.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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LOG OF BOREHOLE BH3D

SHEET 1 of 1

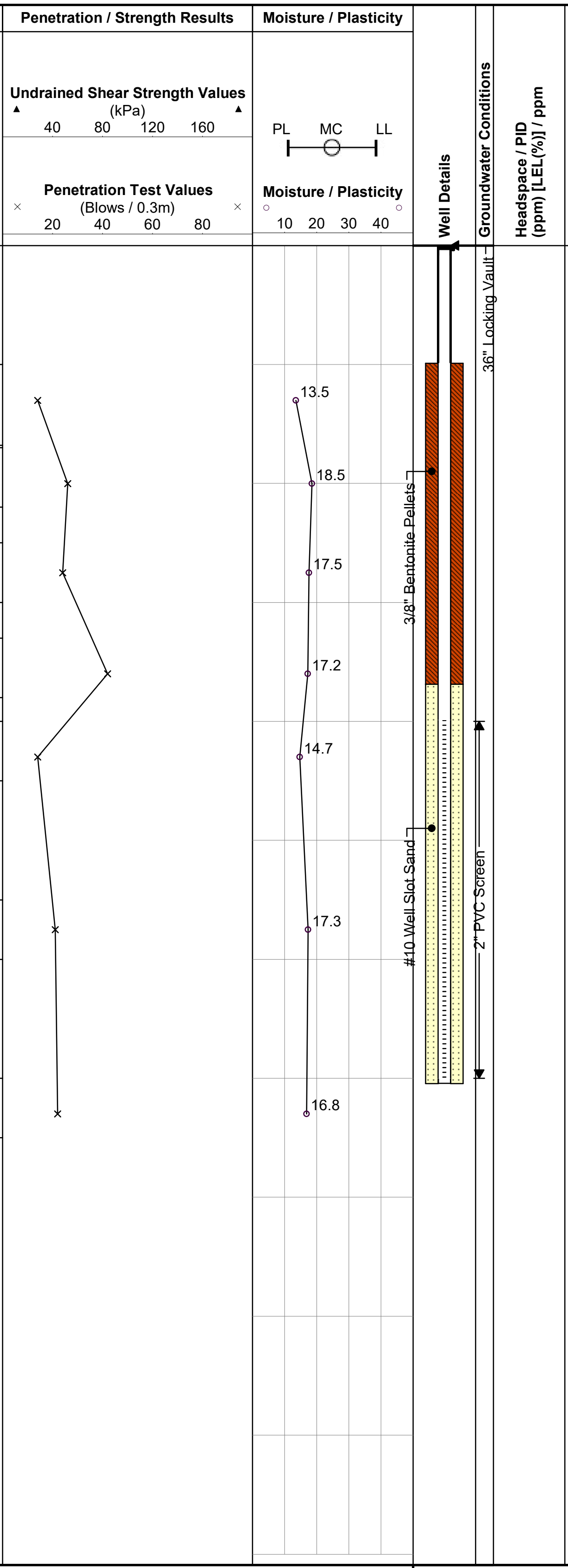
Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-04
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.148164
Easting: -79.900243
Ground Surface Elevation: 230

Depth Scale (m)	Depth/Elevation (m)	Description
0	230.0	**Organic Material** ~100 mm. Clayey silt, some organics. Brown, dry to moist.
		Clayey Silt trace grey clay seams. Stiff, brown, moist. ...very stiff.
1	229.0	
		Clayey Silt Till some grey clay seams, trace gravel. Very stiff, brown, moist. ...hard.
2	228.0	
		Silt Till some clay, trace gravel. Dense, grey, wet. ...compact.
3	227.0	
4	226.0	
5	225.0	
6	224.0	
		End of Log
7	223.0	
8	222.0	
9	221.0	
10	220.0	

Number	Type	Blow Counts/150 mm	N Value
1	SS	3 5 9 12	14
2	SS	5 9 17	26
3	SS	6 9 15	24
4	SS	7 17 25	42
5	SS	5 6 8	14
6	SS	6 10 11	21
7	SS	4 9 13	22



Comments

Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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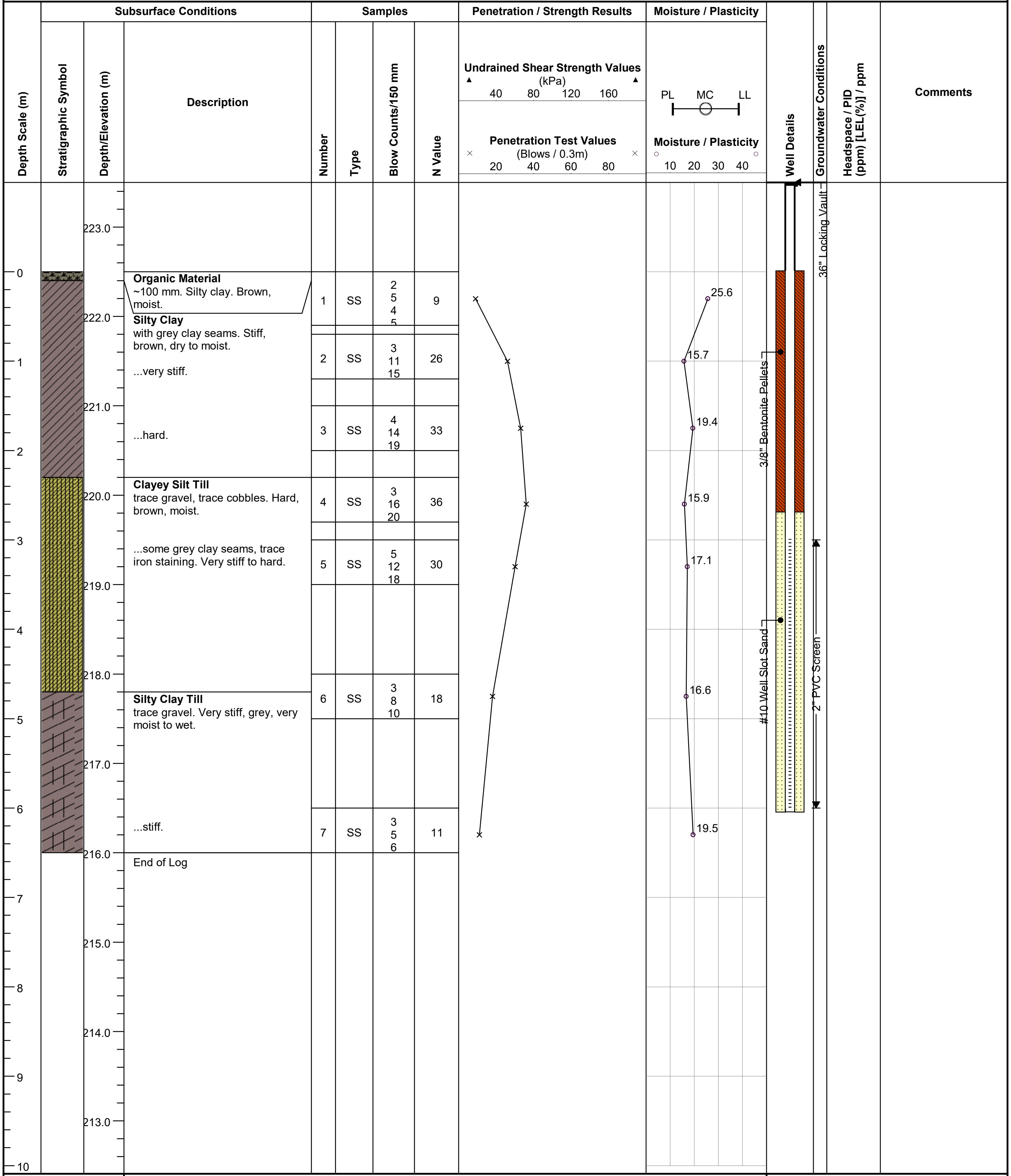
LOG OF BOREHOLE BHMW4

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-09
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.145765
Easting: -79.915462
Ground Surface Elevation: 222.5



Description	Number	Type	Blow Counts/150 mm	N Value
Organic Material ~100 mm. Silty clay. Brown, moist.	1	SS	2 5 4 5	9
Silty Clay with grey clay seams. Stiff, brown, dry to moist.				
...very stiff.	2	SS	3 11 15	26
...hard.	3	SS	4 14 19	33
Clayey Silt Till trace gravel, trace cobbles. Hard, brown, moist.	4	SS	3 16 20	36
...some grey clay seams, trace iron staining. Very stiff to hard.	5	SS	5 12 18	30
Silty Clay Till trace gravel. Very stiff, grey, very moist to wet.	6	SS	3 8 10	18
...stiff.	7	SS	3 5 6	11
End of Log				

Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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LOG OF BOREHOLE BH5

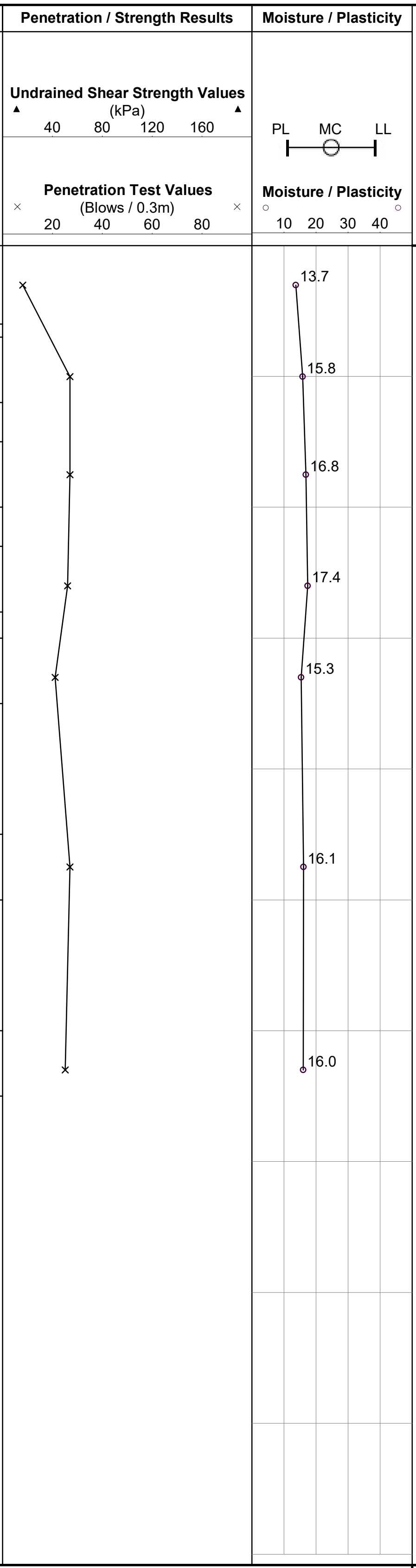
SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-04
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.146519
Easting: -79.903092
Ground Surface Elevation: 227

Depth Scale (m)	Depth/Elevation (m)	Subsurface Conditions – Description	Samples – Number	Type	Blow Counts/150 mm	N Value
		Organic Material ~50 mm. Clayey silt, trace organics. Brown, dry.	1	SS	3 4 4 5	8
		Clayey Silt trace iron staining. Firm to stiff, brown, dry.				
1	226.0	...very stiff.	2	SS	5 12 15	27
		...moist.	3	SS	6 11 16	27
2	225.0					
		Clayey Silt Till trace gravel, trace iron staining. Very stiff, brownish grey, moist.	4	SS	5 10 16	26
3	224.0		5	SS	6 8 13	21
4	223.0					
		...grey, wet.	6	SS	5 12 15	27
5	222.0					
6	221.0	...moist.	7	SS	4 8 17	25
		End of Log				
7	220.0					
8	219.0					
9	218.0					
10	217.0					



Well Details	Groundwater Conditions	Headspace / PID (ppm) [LEL(%)] / ppm	Comments

Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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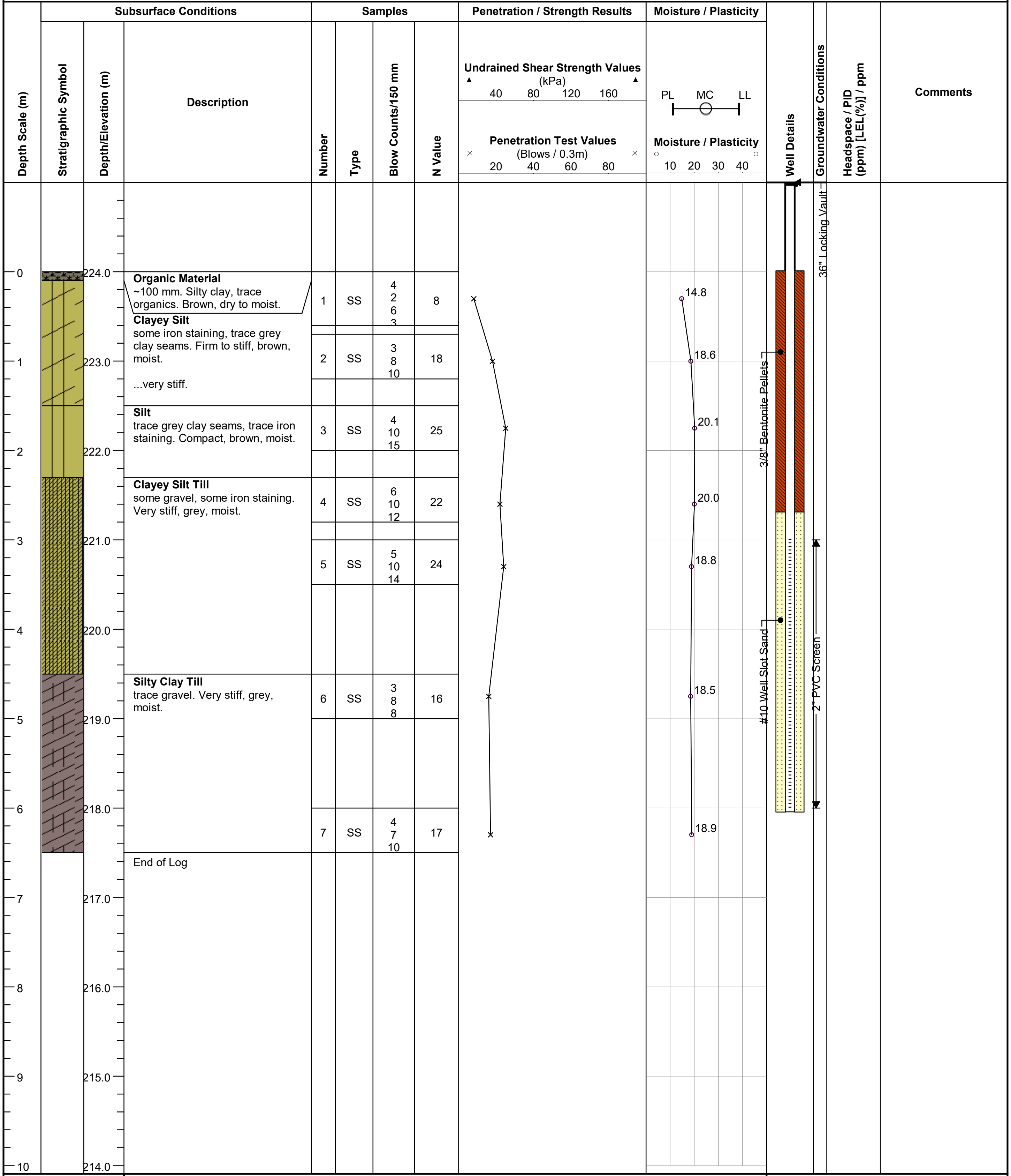
LOG OF BOREHOLE BHMW6

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-04
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.141969
Easting: -79.903206
Ground Surface Elevation: 224



Depth/Elevation (m)	Description	Number	Type	Blow Counts/150 mm	N Value	Moisture (%)
224.0	**Organic Material** ~100 mm. Silty clay, trace organics. Brown, dry to moist.	1	SS	4 2 6 3	8	14.8
	Clayey Silt some iron staining, trace grey clay seams. Firm to stiff, brown, moist. ...very stiff.	2	SS	3 8 10	18	18.6
	Silt trace grey clay seams, trace iron staining. Compact, brown, moist.	3	SS	4 10 15	25	20.1
	Clayey Silt Till some gravel, some iron staining. Very stiff, grey, moist.	4	SS	6 10 12	22	20.0
		5	SS	5 10 14	24	18.8
	Silty Clay Till trace gravel. Very stiff, grey, moist.	6	SS	3 8 8	16	18.5
		7	SS	4 7 10	17	18.9
	End of Log					

Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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LOG OF BOREHOLE BH7

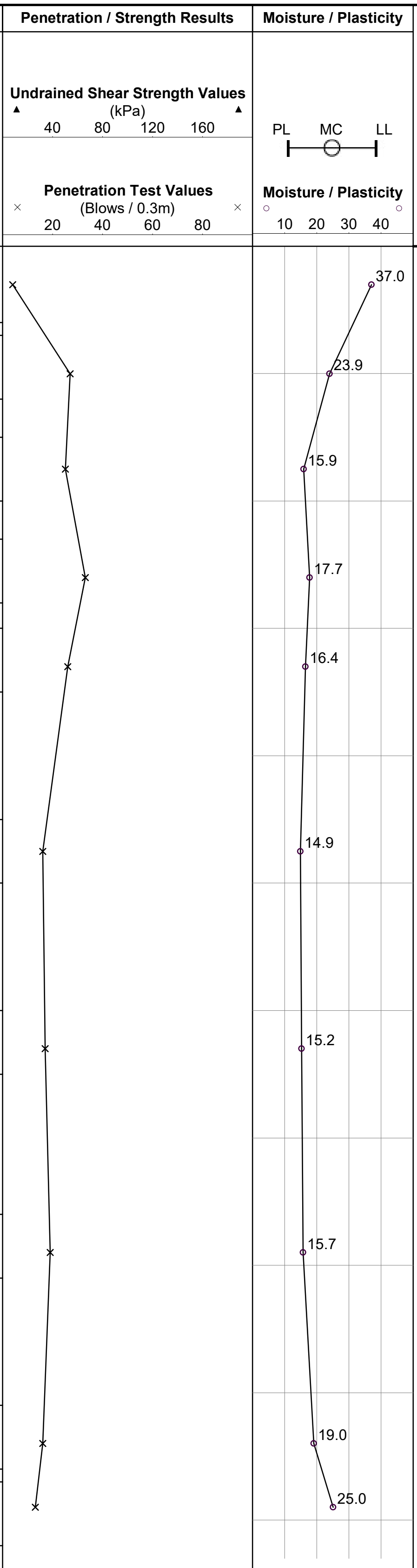
SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-05
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.141126
Easting: -79.899115
Ground Surface Elevation: 224.1

Depth/Elevation (m)	Description	Number	Type	Blow Counts/150 mm	N Value	Well Details	Groundwater Conditions	Headspace / PID (ppm) [LEL(%)] / ppm	Comments
224.0	**Organic Material** ~100 mm. Silty clay, some organics and wood debris. Brown, moist.	1	SS	1 2 2 3	4				
	Clayey Silt trace sand, trace gravel. Soft to firm, brown, dry to moist.	2	SS	4 13 14	27				
223.0	...very stiff.								
	...trace grey clay seams, trace red shale fragments.	3	SS	5 10 15	25				
222.0	**Clayey Silt Till** trace gravel. Hard, brown, moist.	4	SS	7 13 20	33				
221.0	...some iron staining. Very stiff.	5	SS	5 12 14	26				
	...grey.	6	SS	4 7 9	16				
218.0	...very moist.	7	SS	5 7 10	17				
		8	SS	6 9 10	19				
		9	SS	3 6 10	16				
	...stiff, very moist to wet.	10	SS	4 5 8	13				
214.0	End of Log								



Additional Notes:
1. Borehole open to approximately 9.3 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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LOG OF BOREHOLE BHMW8

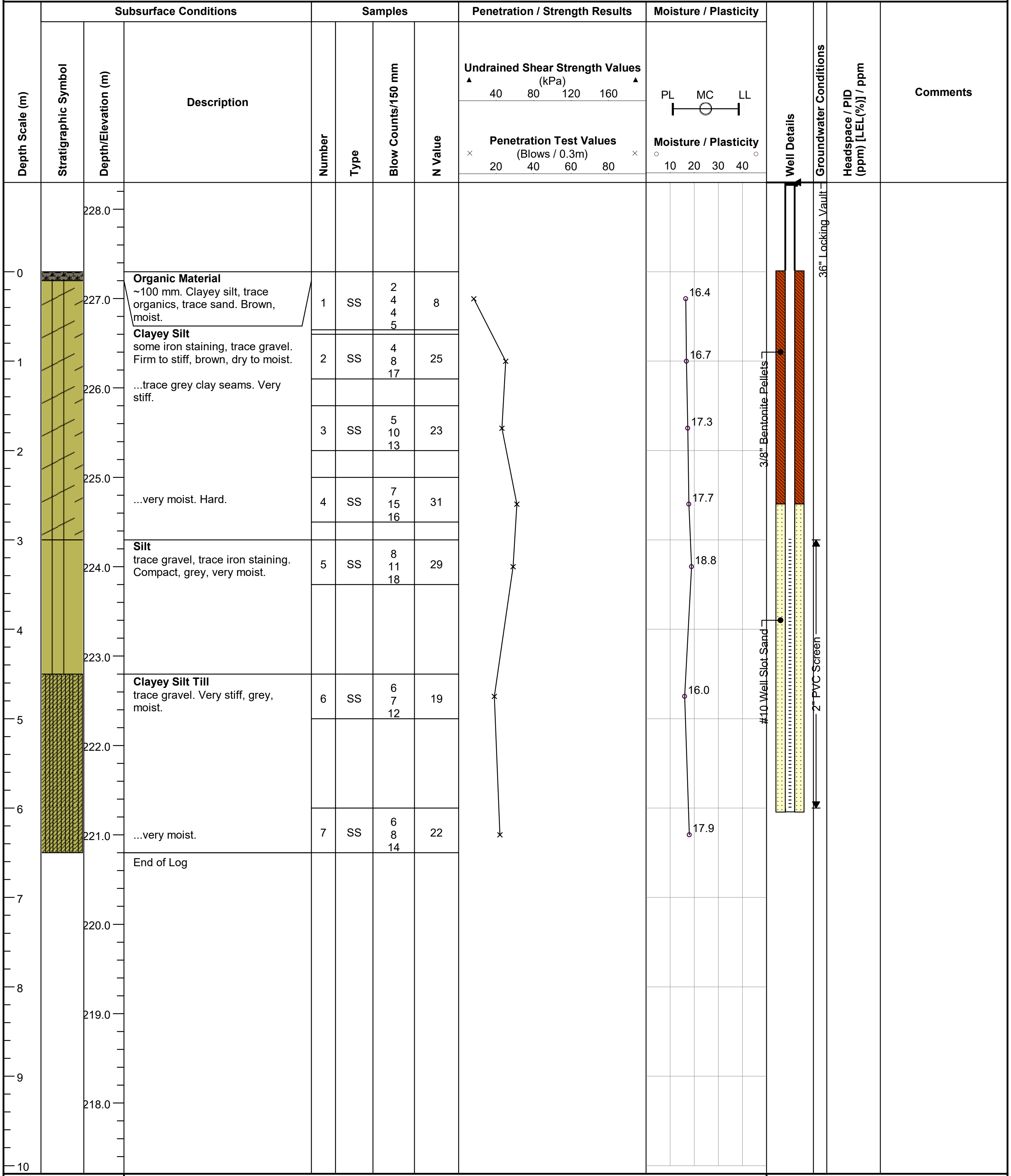
SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-05
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.143731
Easting: -79.896422
Ground Surface Elevation: 227.3

Subsurface Conditions		Samples			
Depth/Elevation (m)	Description	Number	Type	Blow Counts/150 mm	N Value
	Organic Material ~100 mm. Clayey silt, trace organics, trace sand. Brown, moist.	1	SS	2 4 4 5	8
	Clayey Silt some iron staining, trace gravel. Firm to stiff, brown, dry to moist.	2	SS	4 8 17	25
	...trace grey clay seams. Very stiff.	3	SS	5 10 13	23
	...very moist. Hard.	4	SS	7 15 16	31
	Silt trace gravel, trace iron staining. Compact, grey, very moist.	5	SS	8 11 18	29
	Clayey Silt Till trace gravel. Very stiff, grey, moist.	6	SS	6 7 12	19
	...very moist.	7	SS	6 8 14	22
	End of Log				



Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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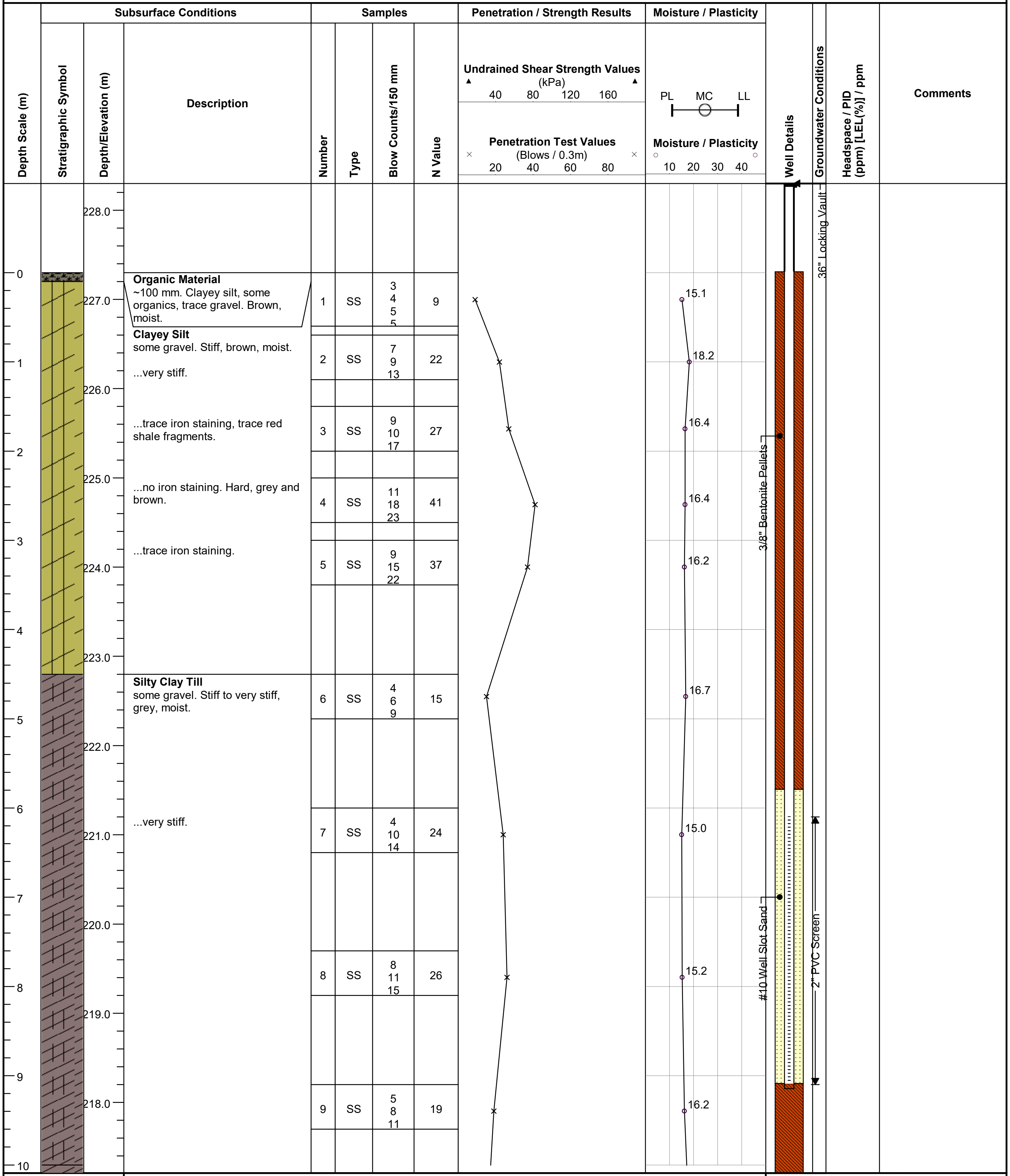
LOG OF BOREHOLE BHMW9

SHEET 1 of 2

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-08
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.139595
Easting: -79.892163
Ground Surface Elevation: 227.3



Description	Number	Type	Blow Counts/150 mm	N Value	Moisture
Organic Material ~100 mm. Clayey silt, some organics, trace gravel. Brown, moist.	1	SS	3 4 5 5	9	15.1
Clayey Silt some gravel. Stiff, brown, moist. ...very stiff.	2	SS	7 9 13	22	18.2
...trace iron staining, trace red shale fragments.	3	SS	9 10 17	27	16.4
...no iron staining. Hard, grey and brown.	4	SS	11 18 23	41	16.4
...trace iron staining.	5	SS	9 15 22	37	16.2
Silty Clay Till some gravel. Stiff to very stiff, grey, moist.	6	SS	4 6 9	15	16.7
...very stiff.	7	SS	4 10 14	24	15.0
	8	SS	8 11 15	26	15.2
	9	SS	5 8 11	19	16.2

Additional Notes:
1. Borehole open to approximately 12.1 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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LOG OF BOREHOLE BHMW9

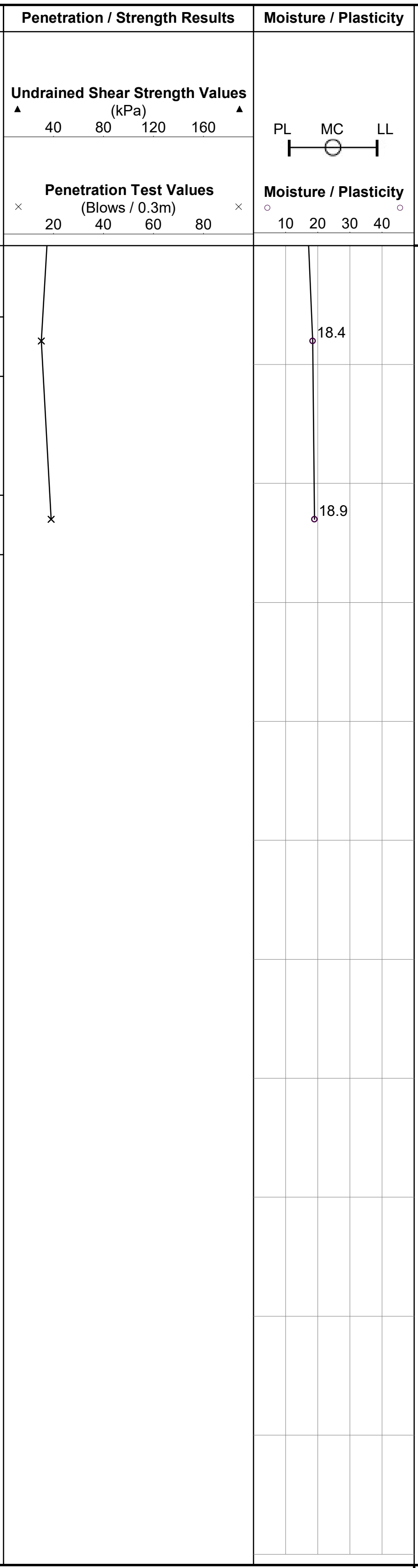
SHEET 2 of 2

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-08
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.139595
Easting: -79.892163
Ground Surface Elevation: 227.3

Depth Scale (m)	Depth/Elevation (m)	Description	Number	Type	Blow Counts/150 mm	N Value	Moisture (%)	Comments
	217.0	(continued)						
11		...stiff to very stiff, moist to very moist.	10	SS	5 7 8	15	18.4	
12	216.0							
	215.0	...very stiff.	11	SS	4 8 11	19	18.9	
13		End of Log						
14	214.0							
15	213.0							
16	212.0							
17	211.0							
18	210.0							
19	209.0							
20	208.0							
21	207.0							



Well Details | Groundwater Conditions | Headspace / PID (ppm) [LEL(%)] / ppm

Additional Notes:
1. Borehole open to approximately 12.1 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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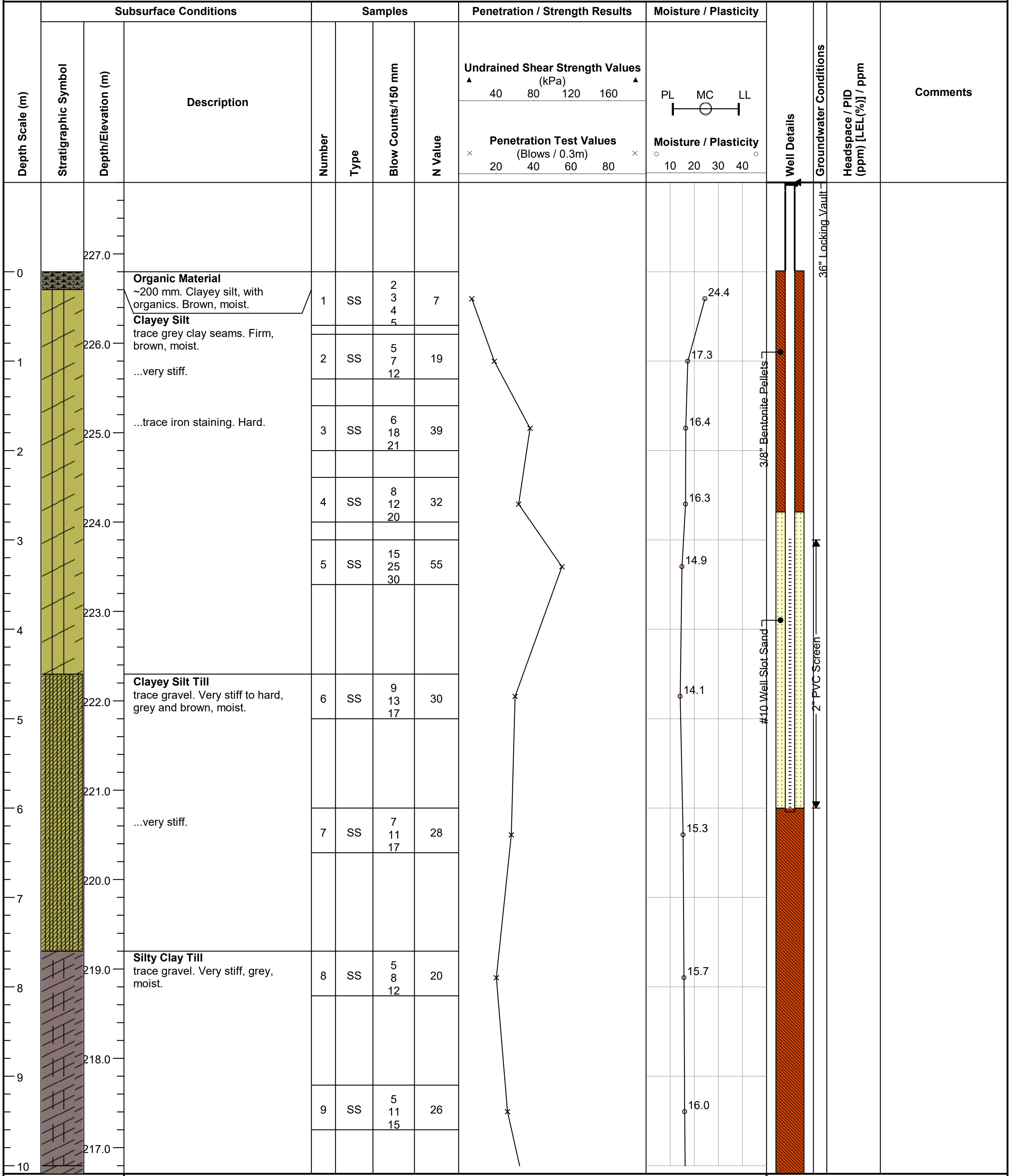
LOG OF BOREHOLE BHMW10

SHEET 1 of 2

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-08
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.142154
Easting: -79.886746
Ground Surface Elevation: 226.8



Description	Number	Type	Blow Counts/150 mm	N Value	Moisture (%)
Organic Material ~200 mm. Clayey silt, with organics. Brown, moist.	1	SS	2 3 4 5	7	24.4
Clayey Silt trace grey clay seams. Firm, brown, moist. ...very stiff.	2	SS	5 7 12	19	17.3
...trace iron staining. Hard.	3	SS	6 18 21	39	16.4
	4	SS	8 12 20	32	16.3
	5	SS	15 25 30	55	14.9
Clayey Silt Till trace gravel. Very stiff to hard, grey and brown, moist.	6	SS	9 13 17	30	14.1
...very stiff.	7	SS	7 11 17	28	15.3
Silty Clay Till trace gravel. Very stiff, grey, moist.	8	SS	5 8 12	20	15.7
	9	SS	5 11 15	26	16.0

Additional Notes:
1. Borehole open to approximately 12.1 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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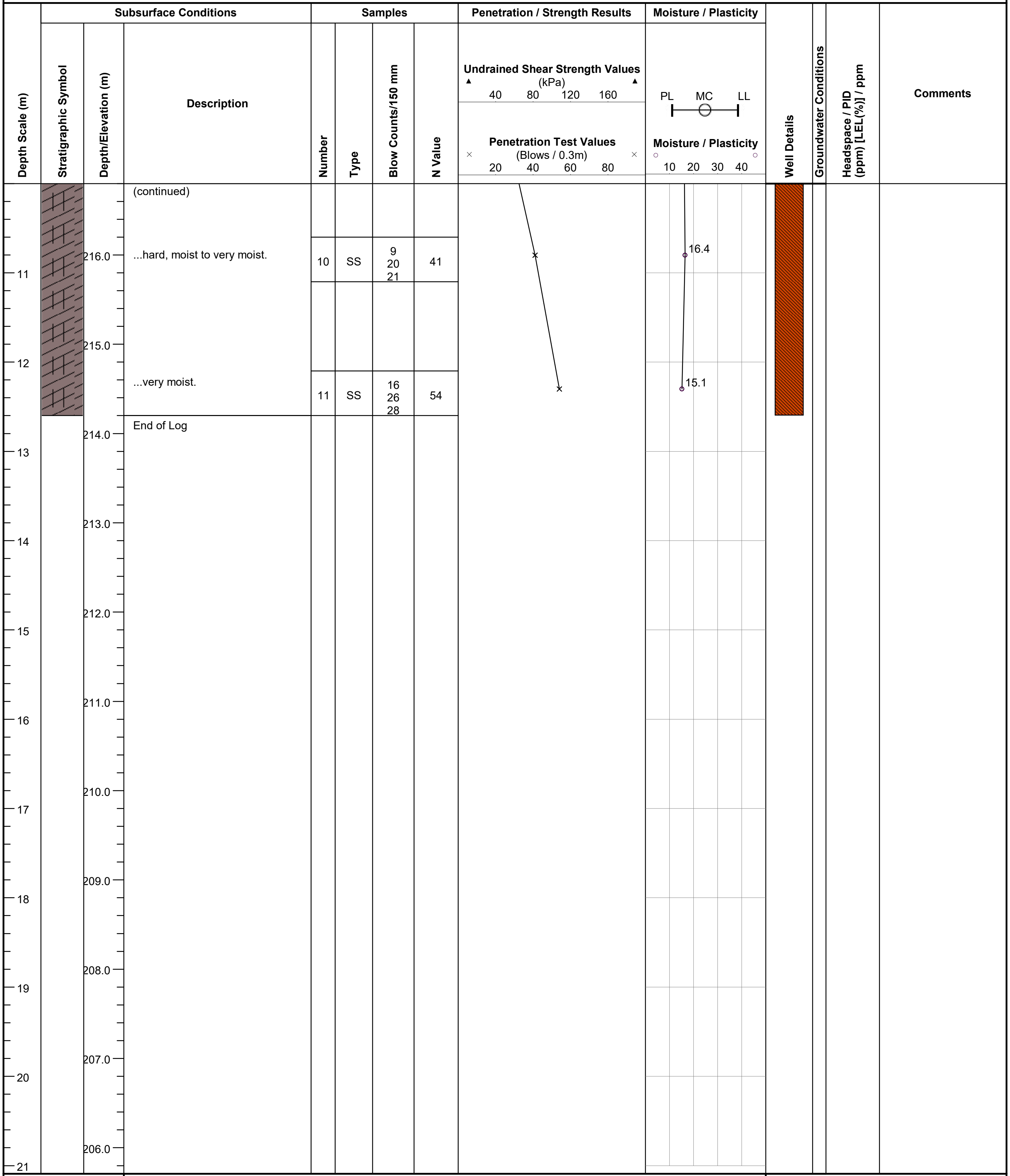
LOG OF BOREHOLE BHMW10

SHEET 2 of 2

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-08
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.142154
Easting: -79.886746
Ground Surface Elevation: 226.8



Description	Number	Type	Blow Counts/150 mm	N Value	Moisture / Plasticity
(continued)					
...hard, moist to very moist.	10	SS	9 20 21	41	16.4
...very moist.	11	SS	16 26 28	54	15.1
End of Log					

Additional Notes:
1. Borehole open to approximately 12.1 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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Ph: (905) 383-3733

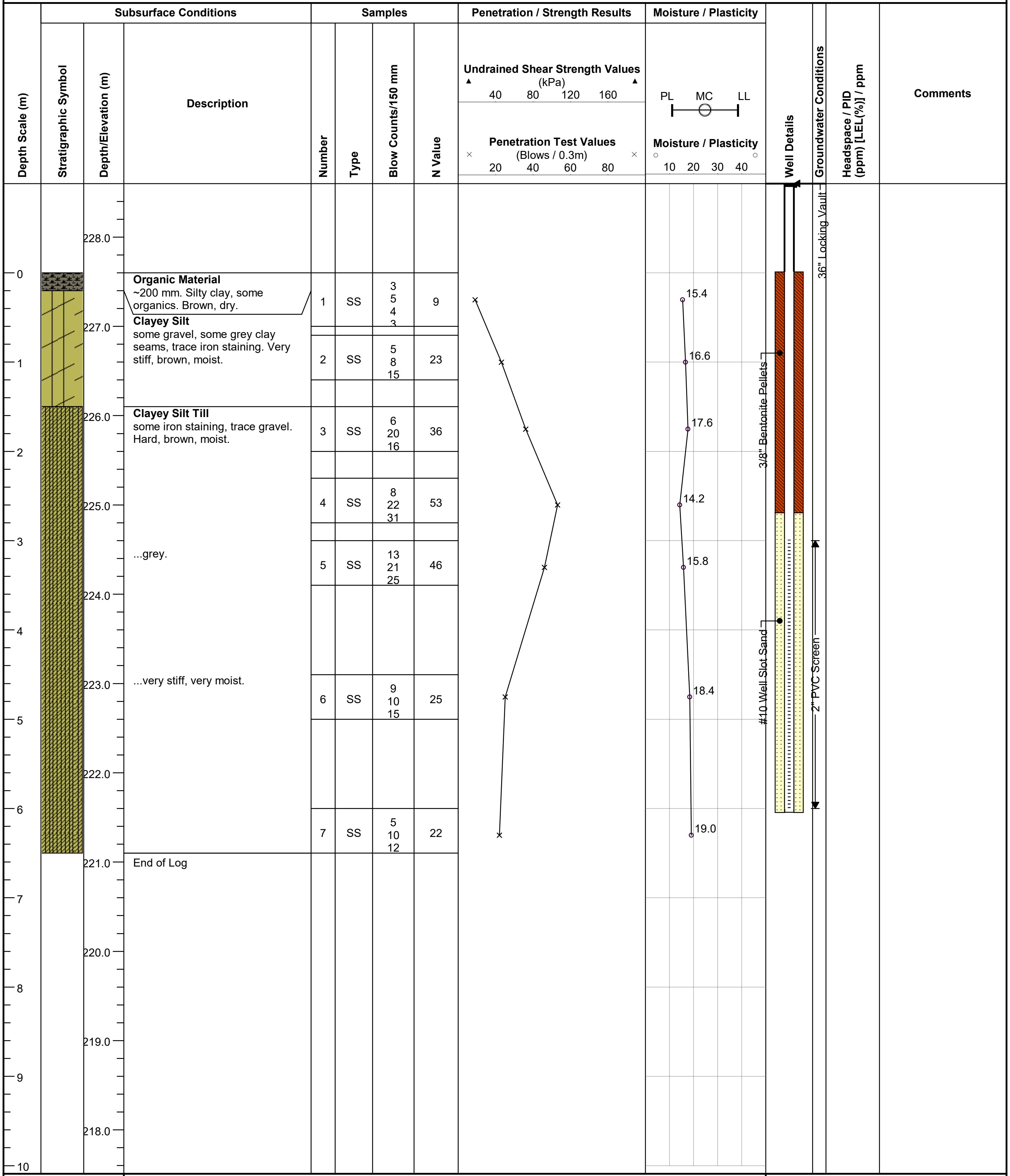
LOG OF BOREHOLE BHMW11

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-08
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.13907
Easting: -79.888437
Ground Surface Elevation: 227.6



Description	Number	Type	Blow Counts/150 mm	N Value
Organic Material ~200 mm. Silty clay, some organics. Brown, dry. **Clayey Silt** some gravel, some grey clay seams, trace iron staining. Very stiff, brown, moist.	1	SS	3 5 4 3	9
	2	SS	5 8 15	23
Clayey Silt Till some iron staining, trace gravel. Hard, brown, moist.	3	SS	6 20 16	36
	4	SS	8 22 31	53
...grey.	5	SS	13 21 25	46
...very stiff, very moist.	6	SS	9 10 15	25
	7	SS	5 10 12	22
End of Log				

Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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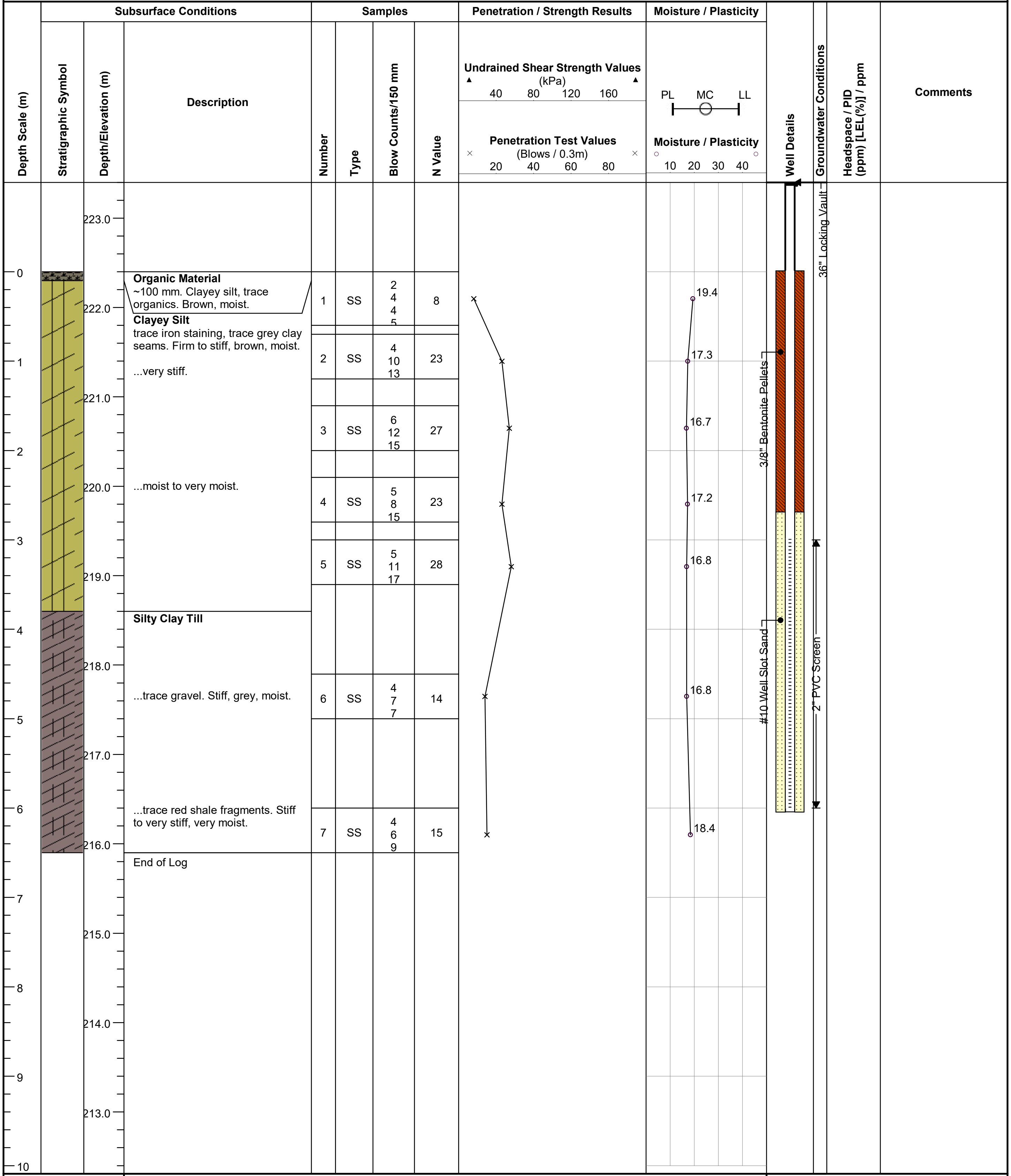
LOG OF BOREHOLE BHMW12

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-05
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.140212
Easting: -79.902967
Ground Surface Elevation: 222.4



Description
Organic Material ~100 mm. Clayey silt, trace organics. Brown, moist.
Clayey Silt trace iron staining, trace grey clay seams. Firm to stiff, brown, moist. ...very stiff. ...moist to very moist.
Silty Clay Till ...trace gravel. Stiff, grey, moist. ...trace red shale fragments. Stiff to very stiff, very moist.
End of Log

Number	Type	Blow Counts/150 mm	N Value	Moisture
1	SS	2 4 4 5	8	19.4
2	SS	4 10 13	23	17.3
3	SS	6 12 15	27	16.7
4	SS	5 8 15	23	17.2
5	SS	5 11 17	28	16.8
6	SS	4 7 7	14	16.8
7	SS	4 6 9	15	18.4

Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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LOG OF BOREHOLE BH13

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-07-04
Drilling Method: Solid Stem
Datum: Geodetic

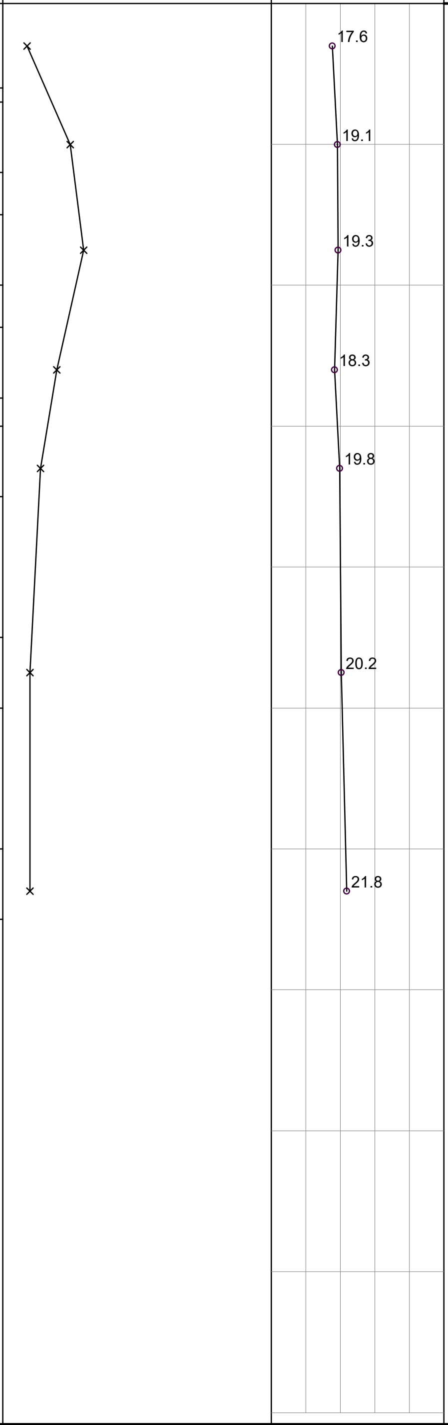
Northing: 43.138818
Easting: -79.90685
Ground Surface Elevation: 220.1

Subsurface Conditions

Depth/Elevation (m)	Description
220.0	**Organic Material** ~50 mm. Silt, trace clay, trace organics. Brown, moist.
	Clayey Silt trace grey clay seams. Stiff, brown, moist.
219.0	...very stiff.
	Clayey Silt Till trace gravel, trace iron staining. Very stiff to hard, grey, moist.
218.0	...no iron staining. Very stiff.
217.0	...stiff.
214.0	...very moist.
	End of Log

Samples

Number	Type	Blow Counts/150 mm	N Value
1	SS	4 5 4 4	9
2	SS	4 7 18	25
3	SS	5 10 20	30
4	SS	4 8 12	20
5	SS	3 6 8	14
6	SS	2 5 5	10
7	SS	2 4 6	10



Depth Scale (m): 1 2 3 4 5 6 7 8 9 10

Depth/Elevation (m): 220.0 219.0 218.0 217.0 216.0 215.0 214.0 213.0 212.0 211.0

Well Details | Groundwater Conditions | Headspace / PID (ppm) [LEL(%)] / ppm | Comments

Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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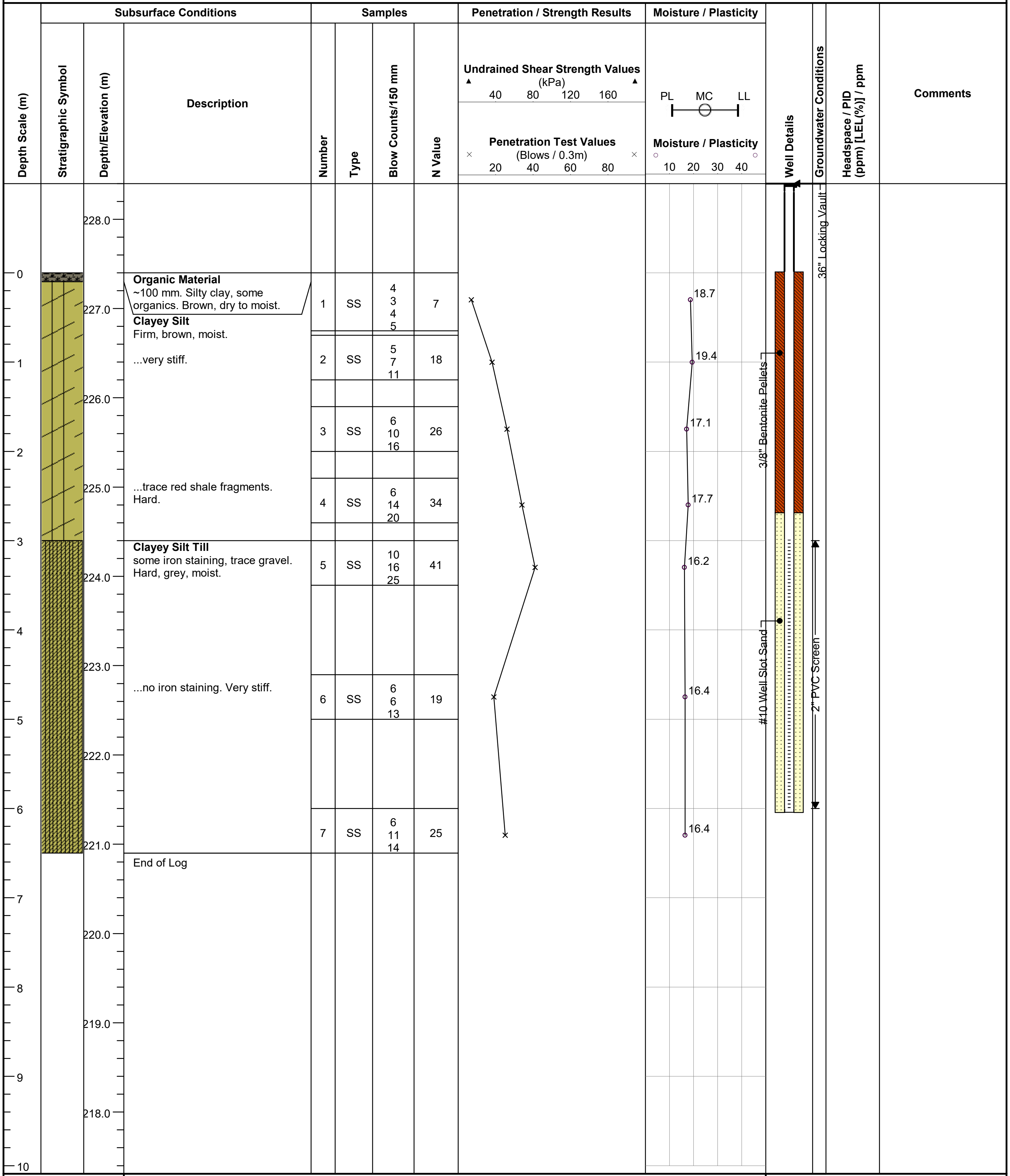
LOG OF BOREHOLE BHMW16

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-08-06
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.14914
Easting: -79.893228
Ground Surface Elevation: 227.4



Organic Material
~100 mm. Silty clay, some organics. Brown, dry to moist.

Clayey Silt
Firm, brown, moist.

...very stiff.

...trace red shale fragments. Hard.

Clayey Silt Till
some iron staining, trace gravel. Hard, grey, moist.

...no iron staining. Very stiff.

End of Log

Number	Type	Blow Counts/150 mm	N Value	Moisture (%)
1	SS	4 3 4 5	7	18.7
2	SS	5 7 11	18	19.4
3	SS	6 10 16	26	17.1
4	SS	6 14 20	34	17.7
5	SS	10 16 25	41	16.2
6	SS	6 6 13	19	16.4
7	SS	6 11 14	25	16.4

Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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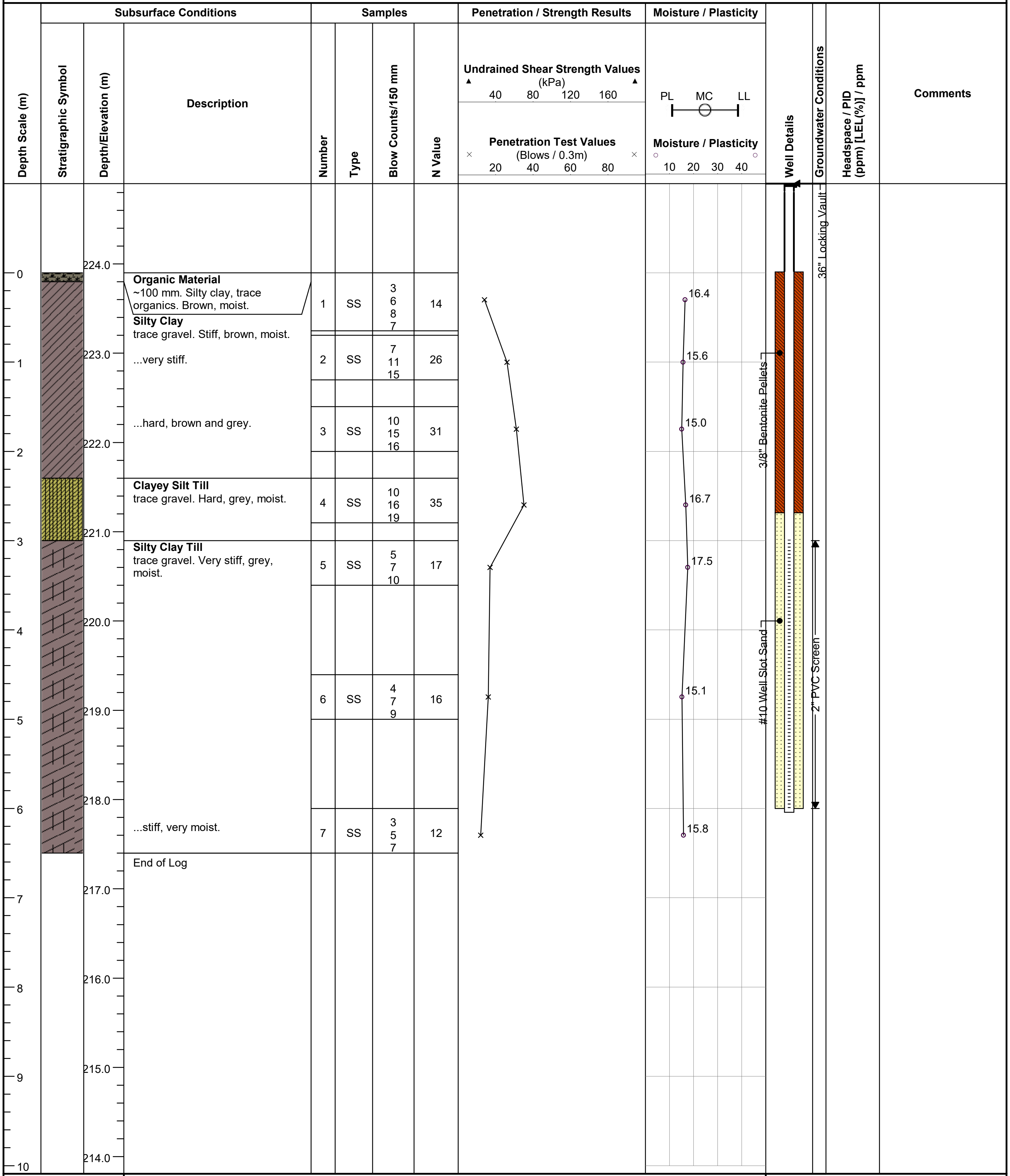
LOG OF BOREHOLE BHMW17

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-08-06
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.147912
Easting: -79.886182
Ground Surface Elevation: 223.9



Description	Number	Type	Blow Counts/150 mm	N Value	Moisture (%)
Organic Material ~100 mm. Silty clay, trace organics. Brown, moist. **Silty Clay** trace gravel. Stiff, brown, moist.	1	SS	3 6 8 7	14	16.4
...very stiff.	2	SS	7 11 15	26	15.6
...hard, brown and grey.	3	SS	10 15 16	31	15.0
Clayey Silt Till trace gravel. Hard, grey, moist.	4	SS	10 16 19	35	16.7
Silty Clay Till trace gravel. Very stiff, grey, moist.	5	SS	5 7 10	17	17.5
	6	SS	4 7 9	16	15.1
...stiff, very moist.	7	SS	3 5 7	12	15.8
End of Log					

Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

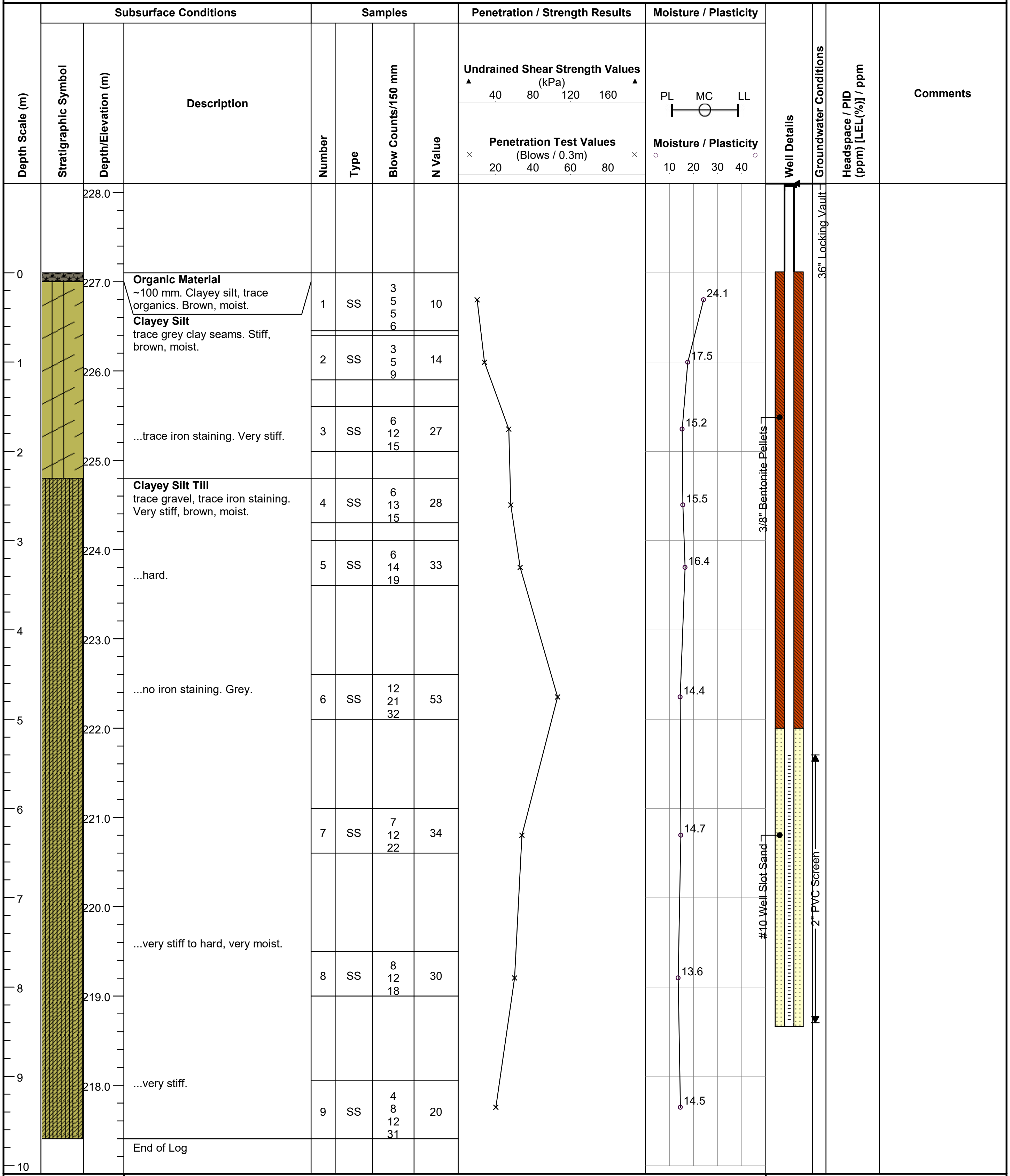
LOG OF BOREHOLE BHMW18

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-08-08
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.147067
Easting: -79.892351
Ground Surface Elevation: 227.1



Description	Number	Type	Blow Counts/150 mm	N Value	Moisture (%)
Organic Material ~100 mm. Clayey silt, trace organics. Brown, moist. **Clayey Silt** trace grey clay seams. Stiff, brown, moist.	1	SS	3 5 5 6	10	24.1
	2	SS	3 5 9	14	17.5
...trace iron staining. Very stiff.	3	SS	6 12 15	27	15.2
Clayey Silt Till trace gravel, trace iron staining. Very stiff, brown, moist.	4	SS	6 13 15	28	15.5
...hard.	5	SS	6 14 19	33	16.4
...no iron staining. Grey.	6	SS	12 21 32	53	14.4
	7	SS	7 12 22	34	14.7
...very stiff to hard, very moist.	8	SS	8 12 18	30	13.6
...very stiff.	9	SS	4 8 12 31	20	14.5
End of Log					

Additional Notes:
1. Borehole open, with cave, to approximately 8.4 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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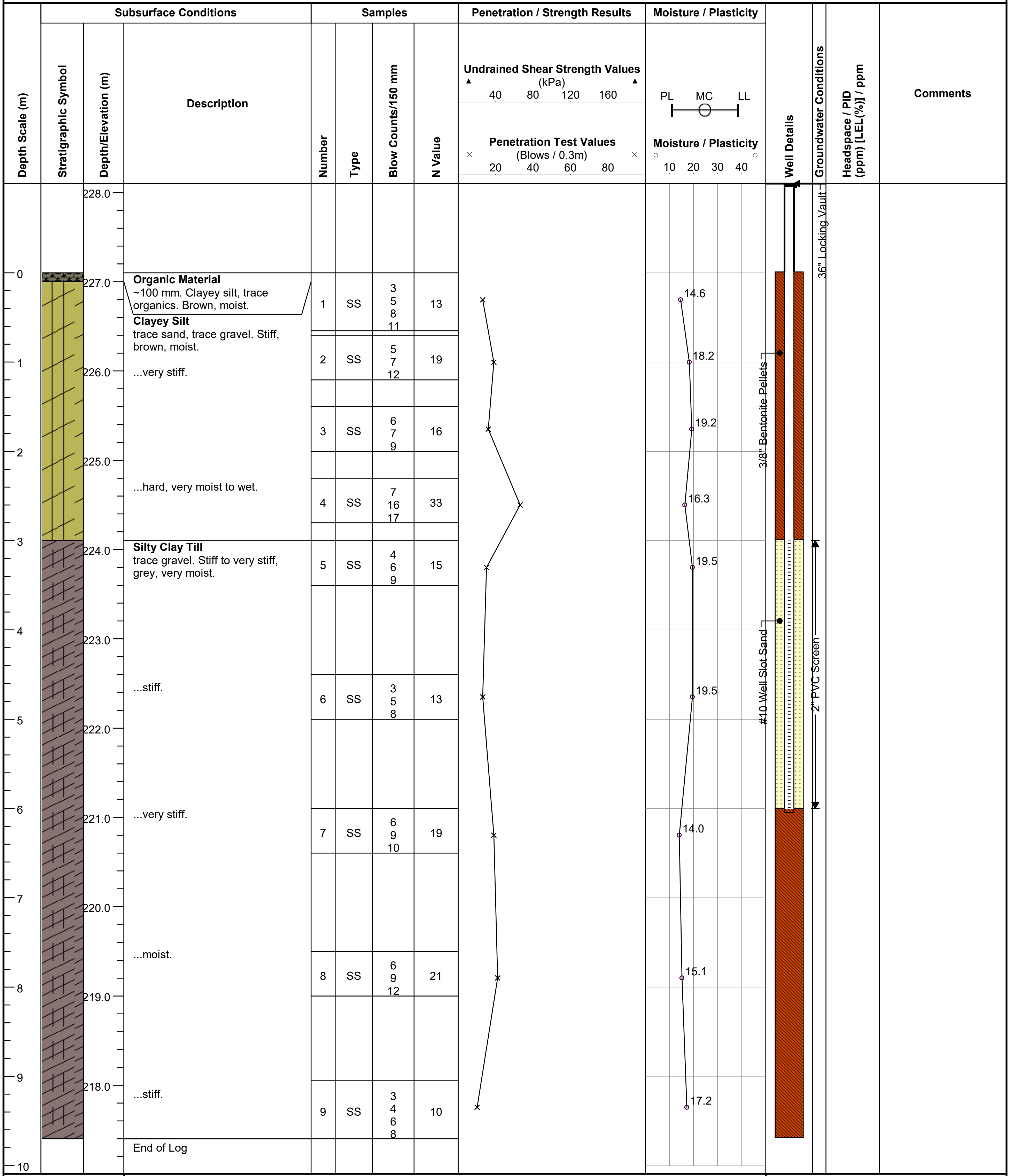
LOG OF BOREHOLE BHMW19D

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-08-07
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.141857
Easting: -79.894982
Ground Surface Elevation: 227.1



Depth/Elevation (m)	Description
227.0	**Organic Material** ~100 mm. Clayey silt, trace organics. Brown, moist.
	Clayey Silt trace sand, trace gravel. Stiff, brown, moist.
	...very stiff.
	...hard, very moist to wet.
224.0	**Silty Clay Till** trace gravel. Stiff to very stiff, grey, very moist.
	...stiff.
	...very stiff.
	...moist.
	...stiff.
	End of Log

Number	Type	Blow Counts/150 mm	N Value	MC
1	SS	3 5 8 11	13	14.6
2	SS	5 7 12	19	18.2
3	SS	6 7 9	16	19.2
4	SS	7 16 17	33	16.3
5	SS	4 6 9	15	19.5
6	SS	3 5 8	13	19.5
7	SS	6 9 10	19	14.0
8	SS	6 9 12	21	15.1
9	SS	3 4 6 8	10	17.2

Additional Notes:
1. Borehole open to approximately 9.1 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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LOG OF BOREHOLE BHMW19S

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-08-07
Drilling Method: Solid Stem
Datum: Geodetic

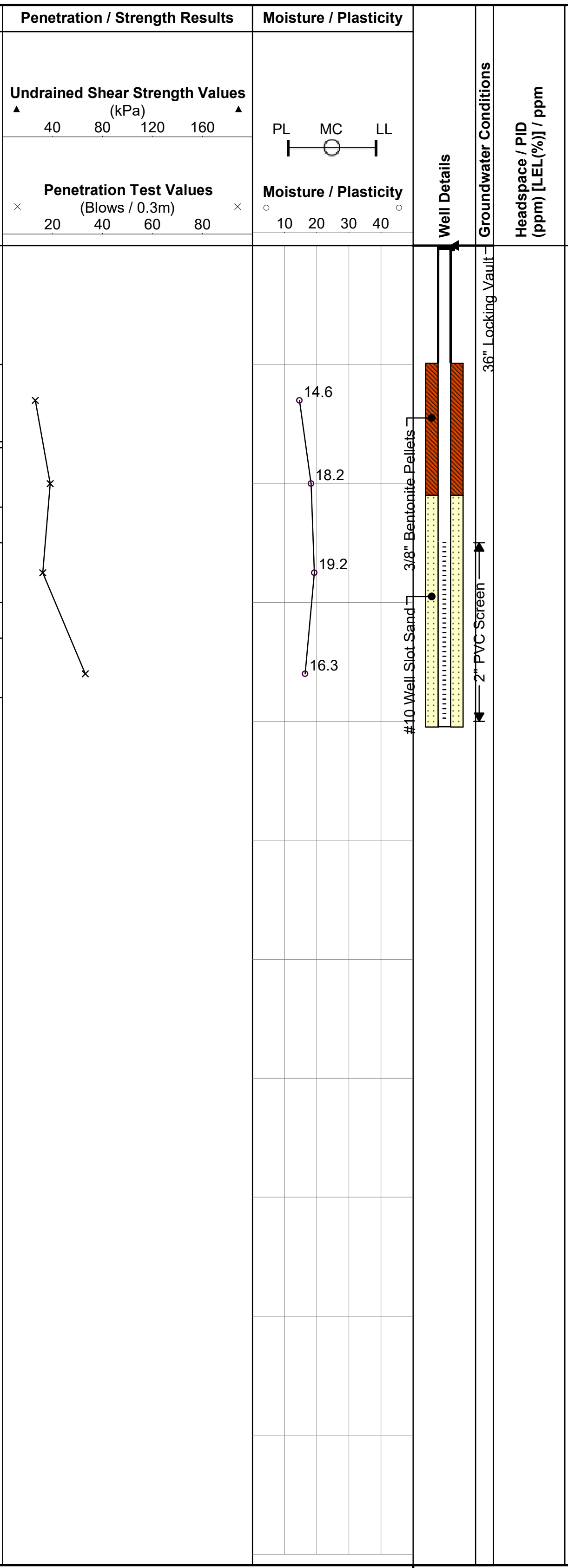
Northing: 43.141812
Easting: -79.894825
Ground Surface Elevation: 227.1

Subsurface Conditions

Depth Scale (m)	Depth/Elevation (m)	Description
0	227.0	**Organic Material** ~100 mm. Clayey silt, trace organics. Brown, moist.
		Clayey Silt trace sand, trace gravel. Stiff, brown, moist.
1	226.0	...very stiff.
2	225.0	...hard, very moist to wet.
3	224.0	End of Log

Samples

Number	Type	Blow Counts/150 mm	N Value
1	SS	3 5 8 11	13
2	SS	5 7 12	19
3	SS	6 7 9	16
4	SS	7 16 17	33



Comments

Additional Notes:
1. Borehole open to approximately 3.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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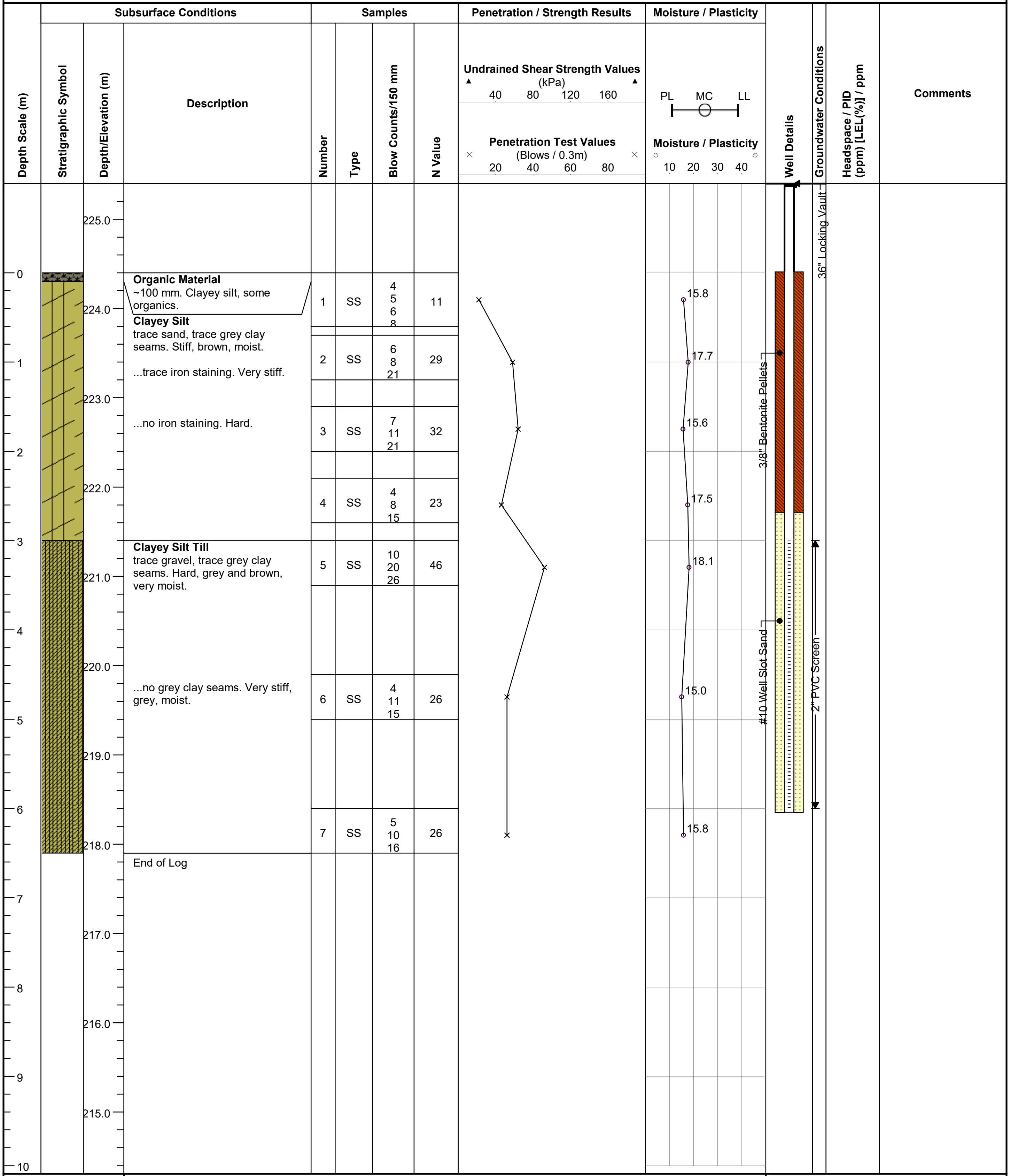
LOG OF BOREHOLE BHMW20

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2024-08-07
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.144462
Easting: -79.884115
Ground Surface Elevation: 224.4



Description	Number	Type	Blow Counts/150 mm	N Value	Moisture (%)
Organic Material ~100 mm. Clayey silt, some organics.	1	SS	4 5 6 8	11	15.8
Clayey Silt trace sand, trace grey clay seams. Stiff, brown, moist. ...trace iron staining. Very stiff.	2	SS	6 8 21	29	17.7
...no iron staining. Hard.	3	SS	7 11 21	32	15.6
	4	SS	4 8 15	23	17.5
Clayey Silt Till trace gravel, trace grey clay seams. Hard, grey and brown, very moist.	5	SS	10 20 26	46	18.1
...no grey clay seams. Very stiff, grey, moist.	6	SS	4 11 15	26	15.0
	7	SS	5 10 16	26	15.8
End of Log					

Additional Notes:
1. Borehole open to approximately 6.0 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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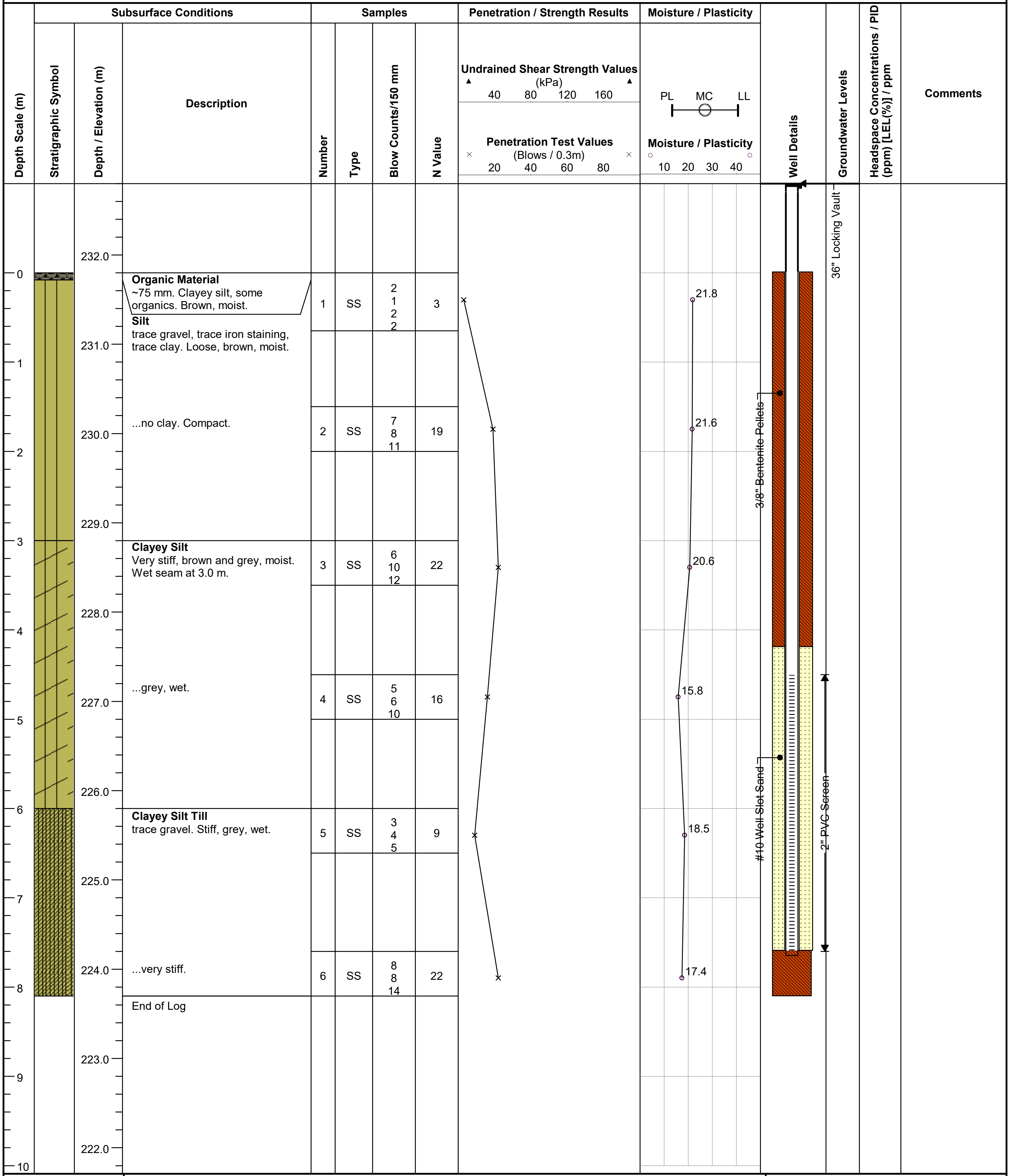
LOG OF BOREHOLE BH/MW22

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2025-01-06
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.153432
Easting: -79.906401
Ground Surface Elevation: 231.8



Subsurface Conditions	Samples			
Description	Number	Type	Blow Counts/150 mm	N Value
Organic Material ~75 mm. Clayey silt, some organics. Brown, moist. **Silt** trace gravel, trace iron staining, trace clay. Loose, brown, moist.	1	SS	2 1 2 2	3
...no clay. Compact.	2	SS	7 8 11	19
Clayey Silt Very stiff, brown and grey, moist. Wet seam at 3.0 m.	3	SS	6 10 12	22
...grey, wet.	4	SS	5 6 10	16
Clayey Silt Till trace gravel. Stiff, grey, wet.	5	SS	3 4 5	9
...very stiff.	6	SS	8 8 14	22
End of Log				

Additional Notes:
1. Borehole open to approximately 7.6 m depth on completion.
2. Groundwatere or water seepage encountered during drilling at approximately 3.0 m depth below the ground surface.
3.
4.

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LOG OF BOREHOLE BH23

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

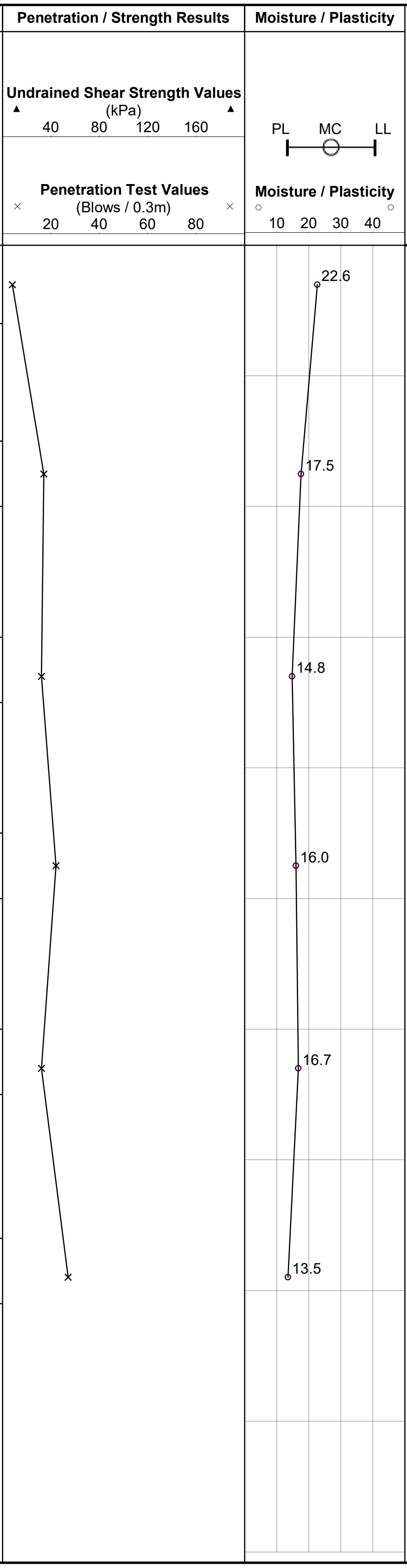
Drill Date: 2025-01-06
Drilling Method: Solid Stem
Datum: Geodetic

Northing: 43.150856
Easting: -79.903838
Ground Surface Elevation: 230.9

Description	Number	Type	Blow Counts/150 mm	N Value
Organic Material ~150 mm. Clayey silt, some organics. Brown, moist.	1	SS	2 1 3 4	4
Silt some clay, some iron staining, some gravel. Loose, brown, moist.				
...compact.	2	SS	2 8 9	17
...brownish grey.	3	SS	6 7 9	16
...grey.	4	SS	5 11 11	22
...trace clay, trace red shale fragments.	5	SS	3 5 11	16
Silt Till trace gravel. Compact, grey, moist.	6	SS	7 11 16	27
End of Log				

Depth Scale (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

Depth / Elevation (m): 230.0, 229.0, 228.0, 227.0, 226.0, 225.0, 224.0, 223.0, 222.0, 221.0



Well Details | Groundwater Levels | Headspace Concentrations / PID (ppm) [LEL(%)] / ppm | Comments

Additional Notes:
1. Borehole open to approximately 7.6 m depth on completion.
2. Groundwater or water seepage not encountered during drilling.
3.
4.

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LOG OF BOREHOLE BH/MW24

SHEET 1 of 1

Project No.: 23354
Project Name: White Church Lands
Location: White Church Rd. & Airport Rd., Hamilton

Drill Date: 2025-01-06
Drilling Method: Solid Stem
Datum: Geodetic

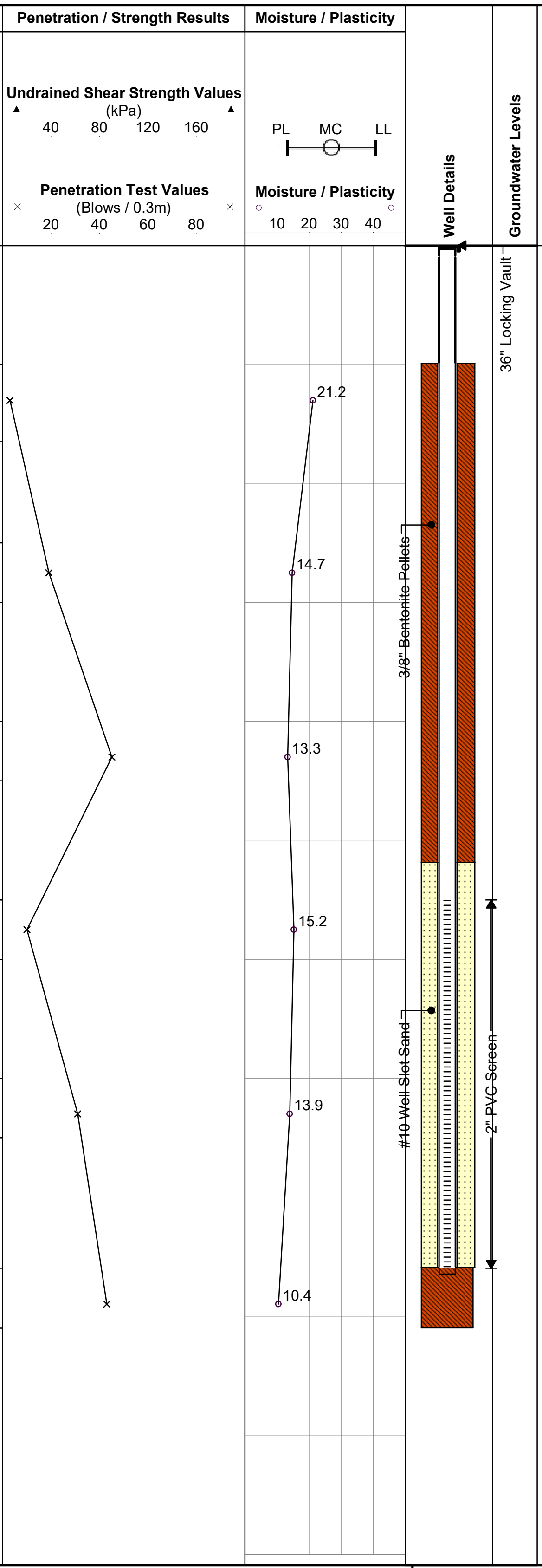
Northing: 43.151608
Easting: -79.900743
Ground Surface Elevation: 230.8

Subsurface Conditions

Depth Scale (m)	Depth / Elevation (m)	Description
0	231.0	**Organic Material** ~200 mm. Silty clay, some organics. Brown, moist.
		Silt with iron staining, some clay. Loose, brown, moist.
1	230.0	
		...compact.
2	229.0	
3	228.0	**Silt Till** with iron staining, trace gravel. Dense, brown, moist.
4	227.0	
5	226.0	...trace clay. Loose to compact, grey.
6	225.0	...no iron staining. Dense.
7	224.0	
8	223.0	...no clay. Dry to moist.
		End of Log
9	222.0	
10	221.0	

Samples

Number	Type	Blow Counts/150 mm	N Value
1	SS	1 1 2 2	3
2	SS	8 8 11	19
3	SS	11 21 24	45
4	SS	4 4 6	10
5	SS	9 14 17	31
6	SS	12 22 21	43



Headspace Concentrations / PID (ppm) [LEL(%)] / ppm

Comments

Additional Notes:
1. Borehole open to approximately 7.6 m depth on completion.
2. Groundwatere or water seepage not encountered during drilling.
3.
4.

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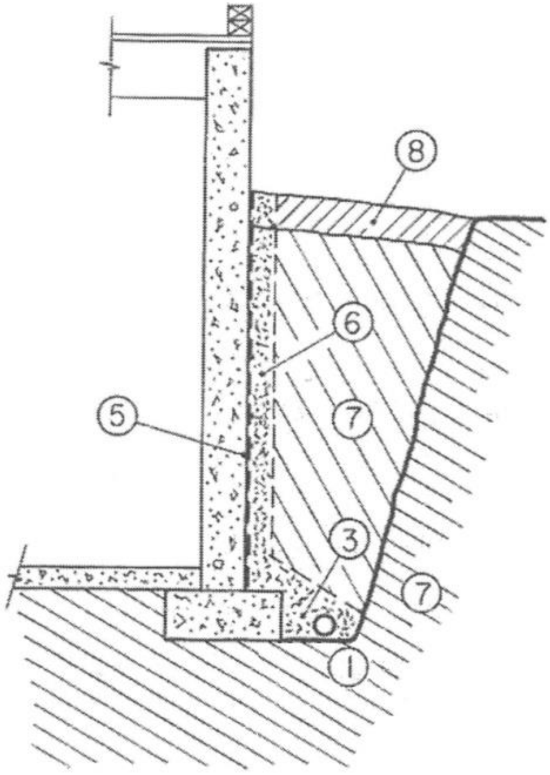
APPENDIX D

DRAWING 23354-02 - ENGINEERING COMMENTARIES – GENERAL REQUIREMENTS FOR DRAINAGE TO BASEMENT STRUCTURES

DRAWING 23354-03 - ENGINEERING COMMENTARIES – GENERAL REQUIREMENTS FOR UNDERFLOOR DRAINAGE SYSTEMS



① 100 mm, perforated or slotted pipe placed below the upper level of the floor slab.;
③ Filter material that is compatible with the grain size characteristics of the fine grained foundation and backfill soils, as well as with the perforations of the pipe;
④ Filter material continuously or intermittently placed next to the foundation wall to intercept water draining from window wells, down exterior walls and from low areas near the building;
⑤ Damp-proofing on wall – optional depending on the quality of the concrete wall;
⑥ Optional use of sheet drain, or synthetic fire blanket, next to the foundation wall to replace the soil filter according to ④;
⑦ Foundation and backfill soils, which may contain fine grained and erosion-susceptible materials;
⑧ "*Topping off*" material is to be graded such that it slopes outwards to lead surface water away from the building. It is usually desirable to use low permeability topsoil to reduce the risk of overloading the drainage pipe.

Based on Figure 12.1, Canadian Foundation Engineers Manual, Fourth Edition, 2006.

Additional Notes:

1. The perforated or slotted drainage pipe is to lead to a positive drainage sump or outlet. The invert of the pipe is to be a minimum of 150 mm below the underside of the proposed floor slab.
2. Backfill materials to the interior of the foundation walls may be clean, organic-free soils that can be compacted to the specified density within in a confined space.
3. Heavy, vibratory compaction equipment should not be used within 450 mm of the foundation wall. Fill is not to be placed or compacted within 1.8 m of the wall unless fill is being placed simultaneously on both sides of the wall.
4. The moisture barrier beneath the floor slab is to comprise at least 200 mm of compacted19mm clear stone or an equivalent free-draining material.
5. Should the 19 mm clear stone require surface blinding then 6mm stone chips are to be used.
6. The slab on grade should not be structurally connected to the foundation wall or footing.

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engineering@landteklimited.com
www.landteklimited.com

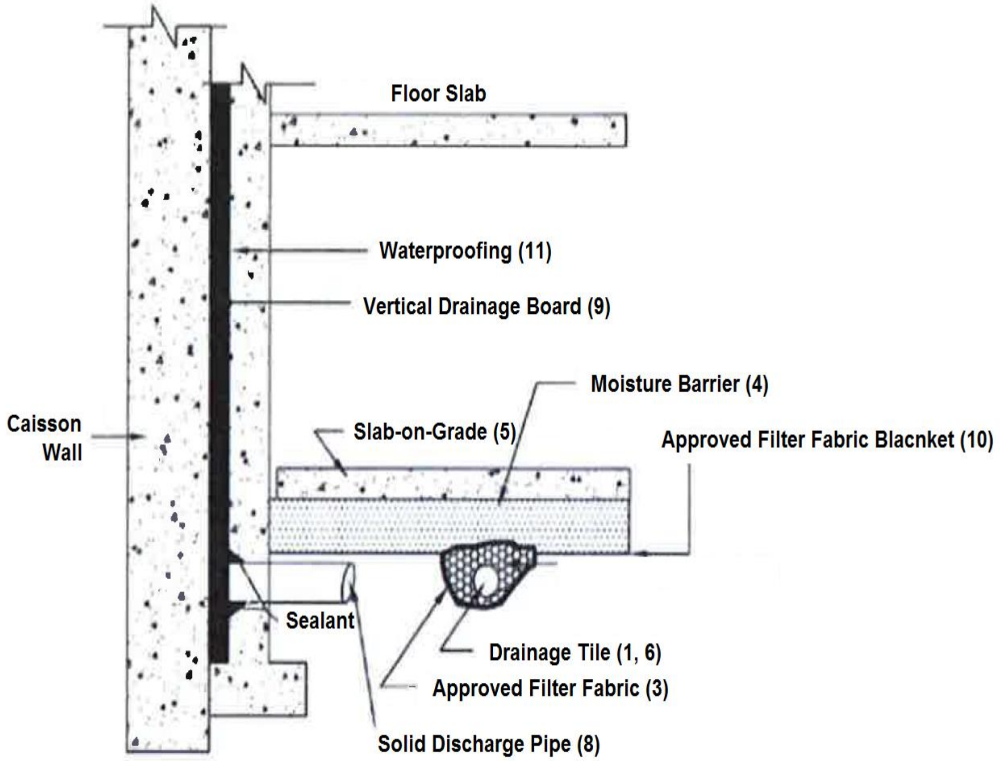
General Requirements for Drainage to Basement Structures

client **White Church Landowners Group Inc.**

project White Church Lands, Hamilton, Ontario

project # 23354

drawing # 23354-02



Notes:

1. Drainage tile, if required for permanent dewatering, to consist of 100 mm diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet, spaced between columns;
2. 19 mm clear stone – 150 mm top and side of drain. If the drain is not on the footing then place 100 mm of 19 mm clear stone below the drain;
3. Wrap the clear stone with an approved filter fabric (e.g., Terrafix 270R or equivalent);
4. Moisture barrier to be at least 200 mm of compacted, 19 mm clear stone or equivalent (and approved), free-draining material. A vapour barrier may be required for specialty floor coverings;
5. Typically, the slab-on-grade is not structurally connected to the wall or footing. However, if it is connected to the walls it should be designed accordingly;
6. Underfloor drain invert, where to be installed, to be at least 300 mm below underside of floor slab. Drainage tile should be placed in parallel rows 6 m to 8 m centres one way. Place drains on 100 mm of 19 mm clear stone and 150 mm of 19 mm clear stone on top and sides. Enclose clear stone with filter fabric as prescribed in Note (3);
7. Do not connect any underfloor drainage to perimeter drainage. The two systems are to remain separate.
8. Locate solid discharge at the middle of each bay between soldier piles;
9. Vertical drainage board (e.g., MiraDrain 6000 or equivalent) with filter cloth should be continuous from bottom to 1.2 m below exterior finished grade;
10. The entire subgrade is to be sealed with an approved filter fabric as in Note (3) where non-cohesive (silty/sandy/granular) soils are encountered below the groundwater table;
11. Where no permanent dewatering is proposed, the basement walls must be waterproofed below the seasonally highest groundwater level (plus 1.0 m to 1.5 m buffer) using bentonite or an equivalent waterproofing system;
12. The Geotechnical Report should be reviewed for site-specific details. Final detail must be approved before system is considered acceptable.

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engineering@landteklimited.com
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General Requirements for Underfloor Drainage Systems

client **White Church Landowners Group Inc.**

project White Church Lands, Hamilton, Ontario

project # 23354

drawing # 23354-03

APPENDIX E

CHEMICAL LABORATORY TESTING RESULTS

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Certificate of Analysis

Landtek Limited
205 Nebo Road, Unit 3
Hamilton, ON L8W 2E1
Attn: Marco Di Cienzo

Report Date: 30-Aug-2024
Order Date: 28-Aug-2024

Client PO: 23354
Project: 23354
Custody: 73194

Order #: 2435247

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

Paracel ID	Client ID
2435247-01	BH1-SS4 & SS5
2435247-02	BH3-SS4 & SS5
2435247-03	BH4-SS3 & SS5
2435247-04	BH6- SS4 & SS5
2435247-05	BH8- SS4 & SS5
2435247-06	BH9- SS3 & SS5
2435247-07	BH10- SS3 & SS5
2435247-08	BH11- SS3 & SS5
2435247-09	BH13- SS3 & SS5
2435247-10	BH16- SS3 & SS5
2435247-11	BH17- SS6 & SS7
2435247-12	BH20-SS6 & SS7

Approved By:

Alex Enfield, MSc
Lab Manager

Order #: 2435247

Certificate of Analysis

Client: **Landtek Limited**

Client PO: 23354

Report Date: 30-Aug-2024

Order Date: 28-Aug-2024

Project Description: 23354

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	29-Aug-24	29-Aug-24
Conductivity	MOE E3138 - probe @25 °C, water ext	29-Aug-24	29-Aug-24
Moisture, %	CWS Tier 1 - Gravimetric	28-Aug-24	29-Aug-24
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	28-Aug-24	29-Aug-24
Resistivity	EPA 120.1 - probe, water extraction	29-Aug-24	29-Aug-24
Solids, %	CWS Tier 1 - Gravimetric	28-Aug-24	29-Aug-24

Certificate of Analysis

Report Date: 30-Aug-2024

Client: **Landtek Limited**

Order Date: 28-Aug-2024

Client PO: 23354

Project Description: 23354

	Client ID:	BH1-SS4 & SS5	BH3-SS4 & SS5	BH4-SS3 & SS5	BH6- SS4 & SS5	-	-
	Sample Date:	27-Aug-24 11:00	27-Aug-24 11:00	27-Aug-24 11:00	27-Aug-24 11:00		
	Sample ID:	2435247-01	2435247-02	2435247-03	2435247-04		
	Matrix:	Soil	Soil	Soil	Soil		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	87.3	86.5	85.5	84.6	-	-
% Moisture	0.1 % by Wt.	12.7	13.5	14.5	15.4	-	-
General Inorganics							
Conductivity	5 uS/cm	507	143	217	129	-	-
pH	0.05 pH Units	7.71	7.81	7.81	7.77	-	-
Resistivity	0.10 Ohm.m	19.7	69.9	46.0	77.5	-	-
Anions							
Chloride	5 ug/g	<5	10	<5	11	-	-
Sulphate	5 ug/g	616	63	149	109	-	-

Certificate of Analysis

Report Date: 30-Aug-2024

Client: **Landtek Limited**

Order Date: 28-Aug-2024

Client PO: 23354

Project Description: 23354

	Client ID:	BH8- SS4 & SS5	BH9- SS3 & SS5	BH10- SS3 & SS5	BH11- SS3 & SS5	-	-
	Sample Date:	27-Aug-24 11:00	27-Aug-24 11:00	27-Aug-24 11:00	27-Aug-24 11:00		
	Sample ID:	2435247-05	2435247-06	2435247-07	2435247-08		
	Matrix:	Soil	Soil	Soil	Soil		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	85.9	86.7	86.6	87.0	-	-
% Moisture	0.1 % by Wt.	14.1	13.3	13.4	13.0	-	-
General Inorganics							
Conductivity	5 uS/cm	639	165	127	549	-	-
pH	0.05 pH Units	7.80	7.82	7.84	7.87	-	-
Resistivity	0.10 Ohm.m	15.7	60.5	78.6	18.2	-	-
Anions							
Chloride	5 ug/g	<5	<5	<5	<5	-	-
Sulphate	5 ug/g	934	42	29	770	-	-

Certificate of Analysis

Report Date: 30-Aug-2024

Client: **Landtek Limited**

Order Date: 28-Aug-2024

Client PO: 23354

Project Description: 23354

	Client ID:	BH13- SS3 & SS5	BH16- SS3 & SS5	BH17- SS6 & SS7	BH20-SS6 & SS7	-	-
	Sample Date:	27-Aug-24 11:00	27-Aug-24 11:00	27-Aug-24 11:00	27-Aug-24 11:00		
	Sample ID:	2435247-09	2435247-10	2435247-11	2435247-12		
	Matrix:	Soil	Soil	Soil	Soil		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	85.5	85.4	86.9	87.2	-	-
% Moisture	0.1 % by Wt.	14.5	14.6	13.1	12.8	-	-
General Inorganics							
Conductivity	5 uS/cm	387	151	483	340	-	-
pH	0.05 pH Units	7.87	7.84	7.88	7.89	-	-
Resistivity	0.10 Ohm.m	25.9	66.3	20.7	29.4	-	-
Anions							
Chloride	5 ug/g	<5	6	<5	<5	-	-
Sulphate	5 ug/g	479	116	672	428	-	-

Method Quality Control: Blank

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

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	5.01	5	ug/g	ND			NC	20	
Sulphate	627	5	ug/g	616			1.8	20	
General Inorganics									
Conductivity	526	5	uS/cm	507			3.7	5	
pH	7.44	0.05	pH Units	7.47			0.4	10	
Resistivity	19.0	0.10	Ohm.m	19.7			3.7	20	
Physical Characteristics									
% Moisture	11.2	0.1	% by Wt.	10.3			8.2	25	
% Solids	88.8	0.1	% by Wt.	89.7			1.0	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	11.4	5	ug/g	ND	109	80-120			
Sulphate	71.3	5	ug/g	61.6	97.7	80-120			