1544 & 1546 Four Mile Creek Road, Niagara-on-the-Lake, ON

Times Group Corp

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Geotechnical Investigation Report

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Proposed Mixed-Use Development
1544 & 1546 Four Mile Creek Road
Niagara-on-the-Lake, Ontario

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1. Introduction and Background

This report presents the results of a geotechnical investigation carried out at the site of the proposed mixed-use development at 1544 and 1546 Four Mile Creek Road in Niagara-on-the-Lake, Ontario. The investigation was authorized by Mr. Stephen Aghaei on behalf of Times Group Corp. The proposed development will include a 2-storey commercial building with no basement and a 4-storey residential building with one level of underground parking, which extends below the residential building as well as the surface parking area.

The purpose of this investigation was to determine the subsoil and groundwater conditions at the site by advancing eight (8) boreholes and based on an assessment of the factual subsurface data, provide an engineering report containing general geotechnical recommendations pertinent to the proposed construction. Another objective of the investigation was to evaluate the stability of the existing slopes along the north and west sides of the property. Additional fieldwork and reporting were completed for a Phase Two Environmental Site Assessment (ESA) as well as a hydrogeological study.

The comments and recommendations given in this report assume that the above-described design concept will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these modifications. The result of this review may be a modification of our recommendations or the requirement of additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

1.1 Site Description & Geological Setting

The subject site is located at 1544 and 1546 Four Mile Creek Road in Niagara-on-the-Lake, Ontario and is bound by Four Mile Creek Road to the east, residential dwellings to the south, a reservoir to the west and a valley that slopes down to the creek to the north. The subject site is currently occupied by two residential dwellings with associated driveways. Historical aerial photographs from 1965 and earlier appear to show that the creek had previously extended onto the subject property; the presence of deep fill at the north end of the site found during the investigation is consistent with the understanding that the creek was rerouted.

Based on the Ontario Geological Survey, Map 2496, *Quaternary Geology, Niagara-Welland*, the overburden at the site consists of modern alluvium: clay, silt, sand, and gravel with organic matter and Halton Till: silt and clay.

2. Field Investigation

2.1 General Fieldwork

EXP advanced a total of eight (8) boreholes at the approximate borehole locations shown on Drawing No. 1 in Appendix A. The boreholes were advanced to depths ranging from approximately 8.2 to 11.3 m below existing grade.

The fieldwork for this investigation was carried out on September 24 to 26, 2024. Drilling and sampling operations were completed by a combination of auger and split-spoon techniques using track mounted drilling equipment owned and operated by a specialist drilling subcontractor. Prior to the commencement of the drilling, the public and private-owned underground services were located to minimize the risk of contacting any such services during the investigation.

Soil samples were obtained using a 51 mm (2 inch) outside diameter split-spoon sampler driven in conjunction with Standard Penetration Test procedure (ASTM D1586) at the depths noted graphically on the borehole logs. Pocket penetrometer and field vane shear tests were carried out in cohesive soils for assessment of undrained shear strengths (ASTM D2573). The retained soil samples were logged in the field and then carefully packaged and transported to our Hamilton laboratory for detailed visual, textural, and olfactory classification. The Standard Penetration Test (SPT) N values and undrained shear strength measurements were recorded and used to provide an assessment of the compactness condition or consistency of the in-situ soils.

Groundwater levels within the boreholes were measured prior to backfilling. Three (3) 50 mm diameter monitoring wells were installed in Boreholes BH-3, BH-4, and BH-7 to allow for stabilized groundwater level measurements. The remaining boreholes were backfilled upon completion of drilling in accordance with O.Reg. 903.

The boreholes were located in accessible areas on site by EXP field personnel. Ground surface elevations at the borehole locations were surveyed by EXP and referenced to a temporary benchmark (TBM), described as follows:

TBM:	Top of catch basin located on southbound lane along the west curb line of Four Mile Creek Road, approximately 3 m south of north driveway of 1546 Four Mile Creek Road.
Elevation:	92.71 m as per the topographical survey titled, *Plan of Survey (with topographic detail) of Part of Township Lot 112 & Part of Road Allowance Between Township Lots 111 & 112 (Geographic Township of Niagara) in the Town of Niagara-on-the-Lake Regional Municipality of Niagara*, dated August 22, 2024 by Barich Grenkie Surveying Ltd.

3. Subsurface Conditions

Details of the subsurface conditions encountered during the drilling program are summarized in the borehole logs in Appendix A. The logs include textural descriptions of the subsoil and groundwater conditions and indicate the soil boundaries inferred from non-continuous sampling and observations during drilling. These boundaries reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The "Notes on Sample Description" preceding the borehole logs form an integral part of and should be read in conjunction with this report.

3.1 Soil Stratigraphy

Fill material was encountered at all borehole locations, except for Boreholes BH-2 and BH-3, extending from below the surficial topsoil/granular fill to depths ranging from approximately 0.8 to 9.1 m below grade. The underlying native soil consisted of predominantly silty clay with sandy silt till in Borehole BH-1. Details of the encountered soils are provided in the following subsections.

3.1.1 Granular Fill

Boreholes BH-1 and BH-6 were advanced in the gravel driveway and encountered approximately 250 and 450 mm of granular fill. Borehole BH-7 encountered approximately 200 mm of granular fill beneath the surficial topsoil layer. The granular fill typically consisted of crushed limestone.

3.1.2 Topsoil

Surficial topsoil was encountered at Boreholes BH-2, BH-3, BH-4, BH-5, and BH-8. The topsoil was noted to have a thickness of approximately 50 to 150 mm. It is noted that topsoil thicknesses may further vary across the site, especially in low-lying areas.

3.1.3 Fill

A layer of fill was encountered below the surficial topsoil/granular in all boreholes, except for Boreholes BH-2 and BH-3, extending to depths of 0.4 to 8.9 m below grade. The fill consisted of silty clay, gravelly sand, silty sand, or sandy silt; was brown, dark brown, or grey; and was in a moist to wet/saturated state, with moisture contents ranging from 5 to 105% (due to organics). The fill was noted to contain trace to some organics, trace wood, brick, and asphalt fragments, and deleterious materials.

Standard Penetration Test blow counts (SPT ‘N’) of 1 to 25 (mostly less than 10) blows per 0.3 m recorded in the fill indicate the fill is generally poorly compacted.

3.1.4 Silty Clay

Native silty clay was encountered below the fill or topsoil at each of the borehole locations, except for BH-1, extending to the borehole termination depths ranging from 8.2 to 9.8 m below grade. The brown silty clay contained trace sand and gravel, and becomes grey in a reduced environment below ground water level. The moist to wet silty clay has moisture contents ranged from 10 to 34% and SPT N values ranged from 3 to 37 blows per 305 mm of penetration. Based on the ‘N’ values, and on the estimated undrained shear strengths from pocket penetrometer measurements, ranging from 25 kPa to greater than 225 kPa, the silty clay exists in a soft to hard state with a trend to become weaker with depth.

3.1.5 Sandy Silt Till

Native sandy silt till was encountered below the fill in Borehole BH-1, extending to the borehole termination depth of 11.3 m below grade. The brown sandy silt till contained some clay, trace gravel, and existed in a moist state with moisture content of 10%. The sandy silt till has SPT N values ranged from 20 to 24 blows per 305 mm of penetration, indicating it is generally compact.

3.2 Groundwater Conditions

Groundwater conditions were monitored in the open boreholes during and upon completion of the investigation. Upon completion, water was encountered in Boreholes BH-1, BH-5 and BH-8 at depths ranging from approximately 3.0 to 7.6 m below grade. However, groundwater levels are not anticipated to have stabilized during the short term of the investigation. 50 mm diameter groundwater monitoring wells were installed in Boreholes BH-3, BH-4 and BH-7 with the groundwater depths and elevations summarized in the table below.

Table 3-1: Groundwater Level Measurements

Borehole No.	Groundwater Depth and Elevation (m)			
	Upon Completion		October 24, 2024	
BH-3	6.1	82.8	7.6	84.9

Borehole No.	Groundwater Depth and Elevation (m)			
	Upon Completion		October 24, 2024	
BH-4	6.9	82.8	3.5	89.1
BH-7	2.1	84.1	7.8	84.6

Seasonal variations in the water table should be anticipated, with higher levels occurring during wet weather conditions (spring thaw and late fall) and lower levels occurring during dry weather conditions.

4. Discussion and Recommendations

The new development is expected to consist of a 2-storey commercial building with no basement and a 4-storey residential building with one level of basement which extends beneath the above ground parking and access road areas. We offer the following comments and recommendations for the proposed construction.

4.1 Slope Stability Analysis

4.1.1 General

The subject site is located adjacent to existing slopes to the north and west of the property. As required by the Niagara Peninsula Conservation Authority (NPCA), a slope stability assessment of the existing slope needs to be carried out to establish the Long-Term Stable Top of Slope (LTSTOS) and relevant setback limits for the proposed development.

On August 7, 2024, NPCA came to the site to stake out the physical top of bank (PTOB) for the slopes on the north and the west sides of the site. This line is shown on Drawing 1 attached. Subsequently a survey of the slopes revealed that the slopes near the south end of the PTOB line are less than 3 m high. Such slopes are not a concern to NPCA. This stability study is focused on the slopes higher than 3 m.

4.1.2 Site Visit

Site reconnaissance was undertaken on July 8, 2024, to evaluate the existing slope conditions, and identify potential areas of concern from a geotechnical perspective. Based on site observations, the slope conditions are further described below.

4.1.2.1 Slope Configuration

Based on our observation and topographic information, the height differences between the crest and the toe of the existing slope are about 2.8 to 3.2 m. The overall gradient of the existing slope varies from about 2H:1V to shallower than 5H:1V, with locally short sections approaching 1.3H:1V. Figures B1 to B8 in shows the general topography of the existing slope.

4.1.2.2 Watercourse Features

Northwestern of the existing slope, the Four Mile Creek traverse from southwest to northeast direction, eventually reaches Lake Ontario. The horizontal distances between the bank of the Creek and the slope toe are generally greater than 15 m in the southern portion of the existing slope, while in the northern portion of the existing slope, the separation can be as narrow as 9 m. The width of the creek varies from approximately 6 to 8 m. At the time of our site visit there was about 0.3 m of water flowing in the creek.

4.1.2.3 Nearby Structures

A pedestrian wood bridge provides access across the Creek is located northwest of the slope toe.

4.1.2.4 Signs of Previous Landslide Activity and Erosion

No sign of surface erosion or previous slope failure was observed on the existing slope face during the site reconnaissance.

4.1.2.5 Soil/Bedrock Stratigraphy Exposure

No soil stratum exposure was observed at the crest, face, or toe of the existing slope. No rock outcrop was found.

4.1.2.6 Surface flow and seepage zone

No surface flow or seepage zone was identified on the slope crest, face and toe of the existing slope.

4.1.2.7 Vegetative cover

The southern portion of the existing slope was mostly vegetated with short grasses while the northern portion of the existing slope was overgrown with matured trees and shrubs. Majority of the tree trunks were straight and upright. Some fallen tree trunks were also found lying in random directions.

4.1.3 Slope Stability Analysis

According to the *Technical Guide – River and Stream Systems: Erosion Hazard Limit* published by Ontario Ministry of Natural Resources (MNR) as well as the Policies for Planning and Development in the Watersheds of the NPCA, the erosion hazards limit for confined systems comprises a) toe erosion allowance, b) the (geotechnical) stable slope allowance, and c) erosion access allowance. These allowances establish the long term stable top of slope (LTSTOS), and provide the setback distances from the physical top of the slope behind which all development must be located.

4.1.3.1 Toe Erosion Allowance

Recession rate (or erosion magnitude) of slope toe caused by the erosive action of water flow is closely related to the susceptibility of the slope toe materials to erosion, the proximity of the slope toe to the flowing water as well as the presence of signs of active erosion. According to the MNR Guidelines, the regression of the slope toe, due to creek erosion, over the course of 100 years design life of a typical structure can be compensated by the introduction of a toe erosion allowance, which is a requirement in areas where the watercourse position is within 15 m of the slope toe. For the two slope sections analyzed (see following section of report), the identified slope toes fall within 15 m from the Creek. Therefore, toe erosion allowance has to be considered, given the banks are unprotected. Based on site observation, the bank of the creek is composed of non-cohesive soils with no indication of active erosion. The MNR guideline recommends a toe erosion allowance of 5 m.

4.1.3.2 Geotechnical Stable Slope Allowance

For this assignment, the geotechnical stable slope allowance is established based on stability analyses using site-specific subsoil and groundwater conditions obtained from geotechnical investigations. The stable top of bank is the point beyond which all potential sliding surfaces have factors of safety (FS) of 1.5 or higher, and is used to establish the LTSTOS for the existing slope.

The slope stability analyses were carried out on the basis of the cross sections perpendicular to the physical top of slope and toe of slope by preparing various models under different defined conditions regarding the slope profiles, soils properties , field data and seismic loading conditions.

4.1.3.3 Cross Section Selection

Lidar data from the Ontario digital terrain model (published in Aug 2019) and spot elevations in the Survey Plan produced by Barich Grenkie Surveying Ltd. were used to generate the contours of the existing slope and prepared the cross sections for this study. Based on the contours of the existing slopes, and the physical top of bank staked out by NPCA (modified by the actual slope heights), two (2) cross sections (Sections 1-1 to 2-2) were selected for analysis. These sections represent the critical conditions and steepest gradients of the existing slope. The locations of the cross sections are shown in Drawing 1 in Appendix A. The general slope configurations of the selected cross sections are presented in Table 4-1.

Table 4-1: Summary of Slope Profiles Analyzed

Cross Section	Average Slope Gradient (xH:1V)	Steepest Local Slope Gradient (xH:1V)	Overall Slope Height (m)	Reference Borehole
1-1	2.0	1.3	3.2	BH-8
2-2	3.3	1.3	3.0	BH-5

4.1.3.4 Soil and Groundwater Parameters

Based on the information of the borehole logs as well as our understanding of the subsurface condition in the area, the soil strata and the corresponding soil parameters adopted in the analysis are summarized in Table 4-2.

The soil strength parameters were estimated from index properties of the soil strata, SPT “N” values measured in boreholes, accepted correlations in literature and EXP’s experience with the similar soils. They are considered to be appropriate for this site.

Table 4-2: Soil Parameters

Material / Stratum	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
Fill	18.0	0	28	50
Native Silty Clay	19.0	2	28	50
Creek Bank (Alluvial Deposit)	20.0	0	28	-

On the basis of water levels recorded in BH-5 and BH-8 upon completion of drilling operation, the groundwater levels were at elevations of 85.6m and 89.6 m, respectively. The groundwater level at 85.6 m in BH-5 was below the bottom of the creek channel but could be influenced by seasonal fluctuations. Hence, a water level of 89.6m is adopted for all the analyzed sections.

Groundwater near sloped ground typically tapers down to the stream level, following the natural hydraulic gradient. Beyond the slope toe, the phreatic surface remains confined below the ground surface, except where it intersects with the watercourse.

4.1.3.5 Seismic Loading

The selected cross sections were analyzed under static and seismic (pseudo-static) conditions. For seismic events, we have used the pseudo-static analysis approach. This method has inherent limitations; however, since the existing slope are of limited height, we feel that the use of pseudo-static analysis for the objective of this study is appropriate. The design ground acceleration used in the analysis corresponds to a seismic event that has a 2% probability of exceedance in a 50-year period (0.000404 per annum probability or 1:2,475 year return period loading conditions). According to the 2020 Building Code of Canada Seismic Hazard suggested values for a site class “D” as determined in our geotechnical investigation Report, the peak ground acceleration (PGA) for the site is 0.284 g, where “g” denotes acceleration due to gravity. For sustained earthquake loading, PGA is adopted as the horizontal seismic coefficient for the analyses. Vertical seismic force is not included in the analysis since that could give less conservative results.

4.1.3.6 Results of Slope Stability Analysis

The stability analyses were undertaken for the selected cross sections using the commercial two-dimensional slope stability computer program Slope/W (GeoStudio 2018). The factor of safety (FS) against slope failure was evaluated based on the limit equilibrium analysis method proposed by Morgenstern and Price for circular sliding surfaces. This method was applied under drained and undrained conditions, in static as well as pseudo-static (seismic) analyses to simulate earthquake loading effect.

The analysis results for the slope are shown in Figures B1 through B8 and are summarized in Table 4-3 below.

Table 4-3: Results of Slope Stability Analysis

Cross Section	Static / Seismic	Drained / Undrained	Calculated FS	Min. Required FS
1-1	static	Drained	1.0 (Figure B1)	1.5 (static) 1.0 (seismic)
1-1	static	Undrained	2.5 (Figure B2)	
1-1	seismic	Undrained	1.2 (Figure B3)	
1-1 (FS 1.5)	static	Drained	1.5 (Figure B4)	
2-2	static	Drained	1.1 (Figure B5)	
2-2	static	Undrained	1.9 (Figure B6)	
2-2	seismic	Undrained	1.0 (Figure B7)	
2-2 (FS 1.5)	static	Drained	1.5 (Figure B8)	

As per the Policies for Planning and Development in the Watersheds of the Niagara Peninsula Conservation Authority as well as the Geotechnical Design and Factors of Safety Technical Bulletin published by MNR, the minimum FS for LTSTOS under static and seismic conditions are 1.5 and 1.0 respectively.

For Section 1-1, the calculated minimum FS under static drained condition is 1.0 (Figure B1), which is less than the requirement minimum FS for long-term stability. Additional analysis was carried out to determine the location of failure slip surface where FS of 1.5 is achieved. This point, shown in Figure B4, is located at about 7.2 m behind the physical top of bank.

Similar analysis was carried out for Section 2. The point of FS 1.5 is found to be at 6.4 m setback from the physical top of bank.

4.1.3.7 Erosion Access Allowance

In addition to the two allowances above, NPCA hazard policies suggest a 7.5 m set back from the stable top of slope, to ensure a sustainable and adequate protection zone for people, ecological features and property associated with valley and stream corridors.

4.1.3.8 Long Term Stable Top of Slope

The following table provides a summary of the recommended allowances at each selected section:

Table 4-4: Distance from Physical Top of Slope

Section	Toe Erosion Allowance (m)	Stable Slope Allowance (m)	NPCA Erosion Access Allowance (m)	LTSTOS (m)
1-1	5	7.2	7.5	14.7
2-2	5	6.4	7.5	13.9

The LTSTOS setback distances shown in Table 4-4 are the sum of Stable Slope Allowance plus Erosion Access Allowance from the physical top of slope. The toe erosion allowance is not added to the set back distance since the toes of the slopes are more than 5 m from the creek bank. The Stable Slope Allowance, along with the LTSTOS lines (i.e. Stable Slope Allowance plus Erosion Access Allowance) are shown in Drawing 1 in Appendix A.

4.1.3.9 Comment on LTSTOS Line

As can be seen in Drawing No. 1, the proposed internal access road is located beyond the stable slope allowance but encroaches into the erosion access setback at the north and northwest end of the site; however, since the erosion access setback is intended for permanent structures rather than roads, the proposed road is considered acceptable given it satisfies with the stable slope allowance.

Drawing No. 1 also shows that the proposed residential building is located south of the PTOB line (as modified by the actual slope height). In this location, the requirements of LTSTOS and development setback do not apply.

4.1.3.10 Slope Protection

For the northern section of the existing slope, it is understood that the slope geometry will not be altered by the construction of the proposed buildings or road. To ensure that the existing slope will remain stable in the long term, the following are recommended:

- The site should be graded so that all surface water run-off is directed away from the slope surfaces
- The existing vegetation cover on the existing slope should be preserved
- No additional fill should be placed on the existing slope or near the slope crest during and after construction
- The configuration of the existing slope should not be altered

4.2 Building Foundation Recommendations

4.2.1 Shallow Foundations

As described in Section 3, a portion of the site is underlain by variable of amount of fill, with depths ranging from 0.7 to 6.1 m below existing grade in the southwest and up to 9.2 m below existing grade in the northeast. The fill is not well compacted and contained organic and deleterious materials, rendering it unsuitable for foundation support purposes.

Where the thickness of the existing fill is relatively thin, the buildings can be supported by conventional spread and continuous footings founded on the native soils below the fill. Footings founded on very stiff to hard native silty clay can be designed for geotechnical reaction of 200 kPa at Serviceability Limit State (SLS), and factored geotechnical resistance of 300 kPa at Ultimate Limit State (ULS). The table below shows the highest elevation at the borehole locations where the recommended footing pressures are available, subject to visual inspection of the founding soils during construction.

Table 4-5: Highest Founding Elevation
where the Recommended Geotechnical Reaction of 200 kPa (SLS) is Available

Borehole No.	Approximate Ground Surface Elevation (m)	Approximate Fill Depth (m)	Founding Soils	Recommended Highest Founding Depth / Elevation (m)
BH-2	92.6	0.0	Native, Hard Silty Clay	1.0/91.6
BH-3	92.5	0.0	Native, Hard Silty Clay	1.0/91.5
BH-4	92.6	3.0	Native, Very Stiff to Hard Silty Clay	3.3/89.3
BH-7	92.4	0.8	Native, Stiff to Hard Silty Clay	1.0/91.4

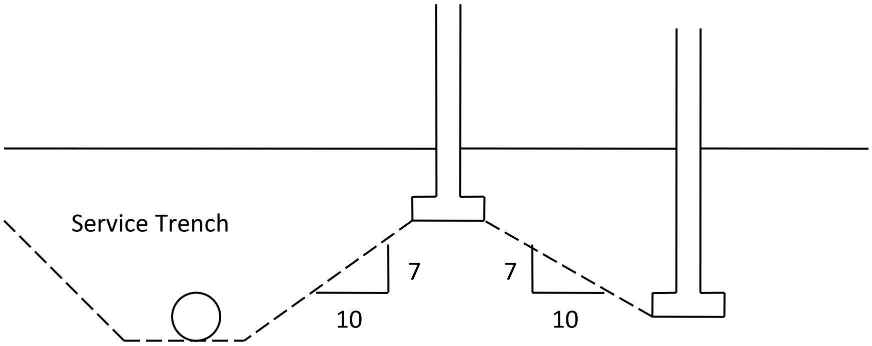
EXP should be contacted to review the proposed foundation details once available to confirm the recommendations provided. Prior to placement of concrete, the founding level subgrade should be inspected by geotechnical personnel from EXP Services Inc. to verify the competency of the founding material.

4.2.2 Deep Foundations

Where the thickness of the existing fill is too deep for conventional footings, the building columns and walls may be supported using deep foundations such as helical piles. A single helical pile founded in stiff silty clay can support 30 to 40 kN compression load at SLS, and 40 to 50 kN at ULS. Helical pile is a proprietary product which is designed by the specialist contractor. Additional boreholes may be completed to further evaluate this option. It is recommended that the actual pile capacity be verified with full scale load tests.

4.2.3 General Foundation Recommendations

Foundations which are to be at different elevations should be located such that higher footings/helical piers are set below a line drawn up at 10 horizontal to 7 vertical, from the near edge of the lower footing/helical piers, as indicated in the following sketch:



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

All foundations and grade beams exposed to freezing conditions must be provided with a minimum of 1.2 m of earth cover or equivalent insulation for frost protection, depending on the final grade requirements. Provided that the ground is not disturbed due to groundwater, precipitation, traffic, etc., and the aforementioned geotechnical resistance values are not exceeded, then total and differential settlements should be within the normally tolerated limits of 25 mm and 19 mm, respectively.

The recommended geotechnical resistances have been calculated by EXP from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, it should be appreciated that modifications to bearing levels may be required if unforeseen subsoil conditions are revealed after the excavation is exposed to full view or if final design decisions differ from those assumed in this report. For this reason, this office should be retained to review final foundation drawings and to provide field inspections during the construction stage.

4.3 Excavations

All existing building structures and associated underground services, if present in the area, are assumed to be removed as part of the demolition plan. It is anticipated that excavations will be carried out for the new footings as well as trenches for the new utilities. Given the proposed utility plan is not available at the time of preparing this report, trenches for new sewer and watermain installation are assumed to typically extend up to 4 m below existing grade. Based on the results of the investigation, excavation will generally be carried out within the existing fills, or native soils of silty clay and sandy silt till.

All unsupported excavations must be completed in accordance with the most recent regulations of the Ontario Occupation Health and Safety Act (OHSA). For the purpose of OHSA, the fills are classified as type 3 soils above groundwater, and Type 4 soils below groundwater. The native silty clay is classified as Type 3 soils.

4.4 Lateral Earth Pressure

The lateral earth pressure acting on the foundation walls of the underground garage may be calculated using the following equation:

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

where p = lateral earth pressure intensity at depth h (kPa)
K = earth pressure coefficient (assume 0.40)
γ = unit weight of retained soil, assume 21.0 kN/m^3 for granular backfill
h = depth to point of interest (m)
q = surcharge load acting adjacent to the wall at the ground surface (kPa)

The above expression assumes an effective perimeter drainage system will be incorporated to prevent the build-up of hydrostatic pressure behind the subsurface wall.

4.5 Groundwater Control During Construction

No major groundwater dewatering requirements are expected for one level basement excavation, footing excavations, and utilities installation. However, minor groundwater seepage into the excavation from perched water within the fill and pervious seams/layers within the native soils should be anticipated during construction. It should be possible to control and remove the minor seepage using conventional construction dewatering techniques, i.e. pumping from filtered sumps. Note that it is the responsibility of the contractor to ensure dry conditions are maintained within the excavation at all times.

Reference should be made to the EXP hydrogeological report for additional groundwater comments. Seasonal variations in the water table should be anticipated, with higher levels occurring during wet weather conditions (spring thaw and late fall) and lower levels occurring during dry weather conditions.

4.6 Pipe Bedding

It is anticipated that sewer and watermain pipes with inverts located at normal depths of up to 4 m below the existing grade should be founded on native silty clay in most areas. The clay should provide adequate support for the utilities. Pipe bedding and cover should comply with OPSD 802.

In some areas fill could be found at bedding level. The fill materials should be visually inspected. Highly organic fill, soft/wet fill, and fill containing a significant amount of construction debris should be sub-excavated and replaced with bedding materials. A layer of geogrid should be placed between the bedding material and the fill. If the site grade in the area of the utility pipes will be raised by more than 1 m, a settlement assessment should be carried out to verify that the pipes will not settle excessively.

If necessary, the pipe bedding may comprise minimum of 300 mm thick of 19 mm clear crushed limestone, wrapped with geotextile filter fabric (such as Terrafix 600R, Texel F300 or equivalent) to prevent migration of soil material from the underlying subgrade into the voids of the bedding material, which may cause loss of subgrade support and/or pavement settlement.

4.7 Building Floor Slab and Permanent Drainage

Where the slab subgrade is native silty clay, normal slab-on-grade construction may be used. The slabs should be founded on a 200 mm thick layer of Granular A, compacted to 100% standard Proctor maximum dry density (SPMDD). Prior to placing the granular materials, the exposed subgrade should be inspected and proof rolled with a heavy truck. Any soft areas detected during proof rolling should be sub-excavated and replaced with Granular A.

Normally we do not recommend placing slabs-on-grade on non-engineered fill. However, if the concrete slabs can accept some settlement and minor cracking, slabs-on-grade may be used provided that the upper 0.6 m of the existing fills are removed and replaced with imported Granular B or sand, compacted to 100% SPMDD. A 200 mm thick layer of Granular A should be placed between the Granular B and the slabs.

Permanent perimeter tile drains should also be placed around the exterior of the basement at foundation level to prevent the build-up of water. The perimeter drains should consist of 100 mm diameter perforated pipe surrounded by 300 mm of 19 mm clear stone and wrapped with a filter fabric with a filtration size of 60 microns or smaller. The drainage system should be connected to a frost free outlet from which the water can be removed.

The perimeter drainage system should be independent of any stormwater piping, such as rainwater leaders. Backflow prevention should be provided between the sumps and the drain headers.

Around the perimeter of the buildings, the finished ground surface should slope on a positive grade away from the structure to promote surface water run-off and to reduce groundwater infiltration adjacent to the basement.

4.8 Backfill Considerations

Backfill used to satisfy under floor slab requirements, footings and service trenches, etc., should be compactible fill, i.e. inorganic soil with its moisture content close to its optimum moisture content as determined in the standard Proctor test. The majority of excavated material will likely consist of fill or native silty clay. In general, the excavated material may be reused for backfill subject to the removal of any organics or other obviously unsuitable material. However, silty materials will require significant mechanical effort and strict moisture content control to achieve the specified compaction levels and should not be used in confined areas that aren't accessible to large compaction equipment.

Any organic, excessively wet, or otherwise deleterious material should not be used for backfilling purposes and should be sorted or left to dry as required. Any shortfall of suitable on-site excavated material can be made up with imported granular material such as OPSS Granular B. The backfill should be placed in lifts not more than 200 mm thick in the loose state, with each lift being compacted to at least 100% SPMDD in building areas and within 600 mm of the pavement structure, or at least 95% below this depth. The degree of compaction achieved in the field should be checked by in-place density tests and the compaction equipment used should be specific to the soil type, e.g. sheepsfoot roller for cohesive soils.

To minimize potential problem, any trench backfilling operations should follow closely after excavation so that only minimal length of trench slope is exposed. This will minimize wetting of the subgrade material. Should construction extend to the winter season, particular attention should be given to ensure that frozen material is not used as backfill.

In general, the on-site soils are not free draining and therefore should not be used where this characteristic is required, or in confined areas. Imported granular material conforming to OPSS Granular B Type I or II would be suitable for these purposes.

All backfilling and compaction operations must be closely examined by a qualified geotechnical consultant to ensure uniform compaction to specification requirements, especially in the vicinity of manholes and catch basins, and in all areas that are not readily accessible to compaction equipment.

4.9 Earthquake Considerations

The recommendations for the geotechnical aspects to determine the earthquake loading for design in accordance with Section 4.1.8 Earthquake Load and Effects in the Ontario Building Code (OBC) 2024, are presented below.

4.9.1 Subsoil Conditions

The subsoil consisted of topsoil or granular fill overlying fill and/or native silty clay or sandy silt till. Undrained shear strengths of the native firm to hard silty clay ranged from 25 kPa to greater than 225 kPa, mostly above or equal to 50 kPa. The SPT N values of the fill ranged from 1 to 25 blow while the sandy silt till has N values ranged from 20 to 24 blows. The foundation will be founded in stiff to hard silty clay or compact sandy silt till.

There have been no shear wave velocity measurements carried out at this site and therefore, N values and EXP's knowledge of the soil conditions in the area have been used to determine the site classification.

4.9.2 Depth of Boreholes

Table 4.1.8.4.-A Exceptions for Site Designation Using Vs30 Calculated from In Situ Measurements and Table 4.1.8.4.-B Site Classes, S, for Site Designation Xs in OBC (2024) indicated that to determine the site classification, the average properties in the top 30 m (below the lowest basement level) are to be used. Site Classification can be determined using the average shear wave velocity (Vs30) as per the classifications stated in Table 4.1.8.4.-A and Table 4.1.8.4.-B. If in-situ shear wave velocity measurements are not available, the site designation Xs shall be determined based on the energy-corrected average standard penetration resistance (SPT) N60 or the average undrained shear strength Su in accordance with Table 4.1.8.4.-B.

There are no shear wave measurements carried out at this site and therefore, the Site Designation will be determined based on the energy-corrected average SPT. The boreholes were advanced to depths of 8.2 to 11.3 m below existing grade. Therefore, the recommended site classification would be based on the available information as well as our interpretation of conditions below the boreholes based on our knowledge of the soil conditions in the subject area.

4.9.3 Site Classification

Based on the above assumptions and interpretations and the known soil conditions, the Site Class for this site is "D" as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2024, for foundations constructed in accordance with this report. It should be noted that an improved site classification may be achievable if shear wave velocity testing is carried out. EXP can be contacted to provide shear wave velocity testing, if required.

4.10 Internal Parking Areas and Access Road Pavement

The pavement subgrade could be either native silty clay, or the existing fill. The native soils should provide adequate pavement support. Where fill is found at pavement subgrade level, the fill materials should be inspected and proof-rolled as described in Section 4.7 of this report.

Pavement subdrains should be provided on both sides of the access road, and in the parking areas.

The recommended pavement structure provided in the table below is based upon an estimate of the subgrade soil properties determined from visual examination and textural classification of the soil samples. Other thickness combinations can be used provided the Granular Base Equivalency (GBE) is maintained and any minimum component thickness specified by the Town of Niagara-on-the-Lake.

Table 4-6: Recommended Pavement Structures

Pavement Layer	Compaction Requirements	Light Duty Area (Car Parking)	Heavy Duty Area (Access Road)
Asphalt (OPSS 1150)	Minimum 92%* Maximum Relative Density (MRD)	40 mm HL3 50 mm HL8	40 mm HL3 80 mm HL8
Granular A (OPSS 1010)	100% SPMDD**	150 mm	150 mm
Granular B Type II (OPSS 1010)	100% SPMDD**	300 mm	450 mm

*Denotes maximum relative density, MTO LS-264
**Denotes standard Proctor maximum dry density, MTO LS-706

The granular base and sub-base must be placed in maximum 200 mm lifts and compacted to 100% SPMDD at moisture content within 2% of the optimum moisture content. The recommended pavement structures outlined assumes adequate provision for drainage.

The foregoing design assumes construction is carried out during dry periods and the subgrade is proof-rolled and reviewed by a geotechnical representative, with any soft areas dug out and replaced. If construction is carried out during wet weather, and heaving or rolling of the subgrade is experienced, additional thickness of sub-base course material may be required.

The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure uniform subgrade moisture and density conditions are achieved. In addition, the need for adequate drainage cannot be over emphasized. The finished pavement surface and underlying subgrade should be free of depressions and should be sloped to provide effective surface drainage toward catch basins. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas. Subdrains should be installed to intercept excess subsurface moisture and prevent subgrade softening.

Additional comments on the construction of the paved areas are as follows:

- As part of the subgrade preparation, the proposed roadways should be stripped of vegetation, topsoil/organics and weak/loose subgrade material. The exposed subgrade surface should be compacted and proof-rolled in the presence of a representative of our office. Soft subgrade areas should be further sub-excavated and replaced with suitable approved backfill compacted to 98% SPMDD for pavement areas. Fill required to raise the grades to design elevations should be organic-free and at a moisture content which will permit compaction to 98% SPMDD. Any shortfall of suitable on-site excavated material can be made up with imported clean approved fill or granular material, OPSS Granular 'B' or equivalent. The imported fill or granular material should be placed in maximum 300 mm lifts and uniformly compacted to at least 98% SPMDD. The final subgrade surface should be properly shaped and crowned.

- Longitudinal subdrains should be installed along the curbs on both sides of the internal access road and above ground parking area, and radially to catch basins in parking areas at least 300 mm below the granular subbase. This will ensure no water collects in the granular courses which could lead to pre-mature pavement failure during freeze/thaw cycles.

- To minimize problems of differential movement between the pavement and catch basins/manholes due to frost action, the backfill around the structures should consist of free draining granular fill. The granular material should be compacted to 98% SPMDD with a small tamper to avoid damaging the structures. In addition, catch basins should be perforated just above the drainpipe and the holes screened with filter cloth.

- The most severe loading conditions on pavement areas and the subgrade may occur during construction. Consequently, special provisions such as half loads during paving, etc. may be required, especially if construction is carried out during unfavourable weather.

5. General Comments

The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions of the subject property. The conclusions presented in this report reflect site conditions existing at the time of the investigation.

EXP Services Inc. should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, EXP Services Inc. will assume no responsibility for interpretation of the recommendations in the report.

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

More specific information, with respect to the conditions between samples, or the lateral and vertical extent of materials, may become apparent during excavation operations. Consequently, during the future development of the property, conditions not observed during this investigation may become apparent; should this occur, EXP Services Inc. should be contacted to assess the situation and additional testing and reporting may be required. EXP Services Inc. has qualified personnel to provide assistance in regard to future geotechnical and environmental issues related to this property.

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

LICENSED PROFESSIONAL ENGINEER
JN
J. Sai-Ming NG
2025 04 02
PROVINCE OF ONTARIO

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Geoscientist

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Appendix A

Drawings & Borehole Logs

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LEGEND:

◐	APPROXIMATE BOREHOLE LOCATION
—	APPROXIMATE CROSS-SECTION LOCATION
—	MODIFIED PHYSICAL TOP OF BANK (considering slope height greater or equal to 3 m only)
—	STABLE SLOPE ALLOWANCE (FS 1.5)
- -	LONG-TERM STABLE TOP OF SLOPE (includes Erosion Access Allowance)

TITLE AND LOCATION:

BOREHOLE AND CROSS-SECTION LOCATION PLAN
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
1544 & 1566 FOUR MILE CREEK ROAD, NIAGARA-ON-THE-LAKE, ON

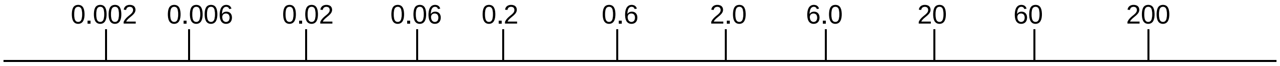
JOB NO.: HAM-24000672-A0	DRAWN BY: RY
SCALE: 1:750	CHECKED BY: JN
DATE: MARCH 2025	DWG NO.: 1

Notes On Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Society for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by exp also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		



EQUIVALENT GRAIN DIAMETER IN MILLIMETERS

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC	SAND			GRAVEL	

UNIFIED SOIL CLASSIFICATION

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Log of Borehole BH-1

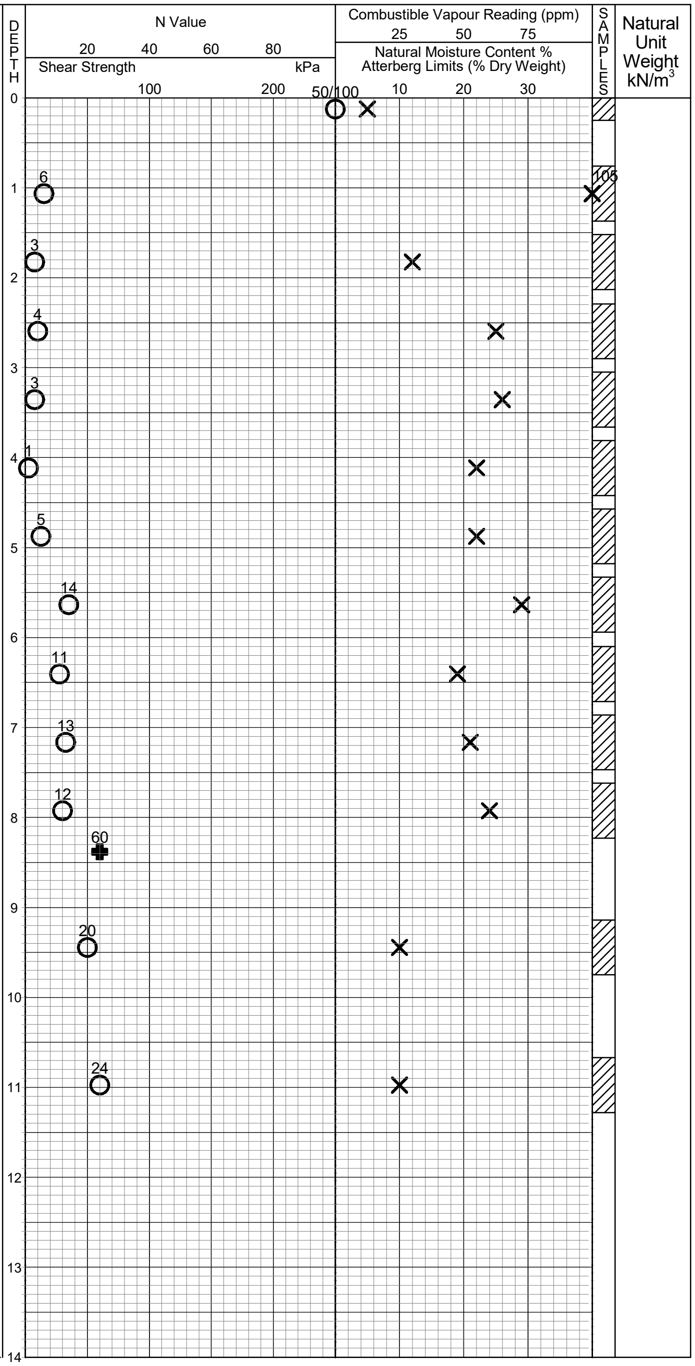
Project No. HAM-24000672-A0
Drawing No. 3
Project: Proposed Mixed-Use Development
Sheet No. 1 of 1
Location: 1544 & 1546 Four Mile Creek Road, Niagara-on-the-Lake, Ontario

Date Drilled: September 25, 2024
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

Soil Description	ELEV. m
	92.07
GRANULAR FILL: (~250 mm thick)	~91.8
FILL: gravelly sand, trace organics, trace asphalt fragments, trace wood pieces, dark brown, moist saturated below 0.8 m	~90.6
FILL: sandy silt, trace gravel, trace organics, dark brown, moist	~89.8
FILL: silty clay, trace gravel, trace organics, trace deleterious materials, trace wood pieces, grey, wet	~86.7
FILL: silty sand, some gravel, trace clay, trace organics, trace deleterious materials, dark brown, wet	~86.0
FILL: silty clay, trace organics, trace asphalt fragments, trace wood pieces, trace deleterious materials, grey, wet	~82.9
SANDY SILT TILL: some clay, trace gravel, brown, moist, compact	~80.8
Borehole terminated at 11.3 m depth below grade.	

NOTES:
1. This drawing is to be read with the subject report and project number as presented above.
2. Interpretation assistance by EXP is required before use by others.



Time	Water Level (m)	Depth to Cave (m)
on completion	7.6	7.3

EXP Services Inc.
Hamilton, Ontario
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Facsimile: 905.573.9693

LAGWGLJFHAM-EXP BHLOGS 1541 FMC CR.GPJ NEW.GDT 11/21/24

Log of Borehole BH-2

Project No. HAM-24000672-A0

Drawing No. 4

Project: Proposed Mixed-Use Development

Sheet No. 1 of 1

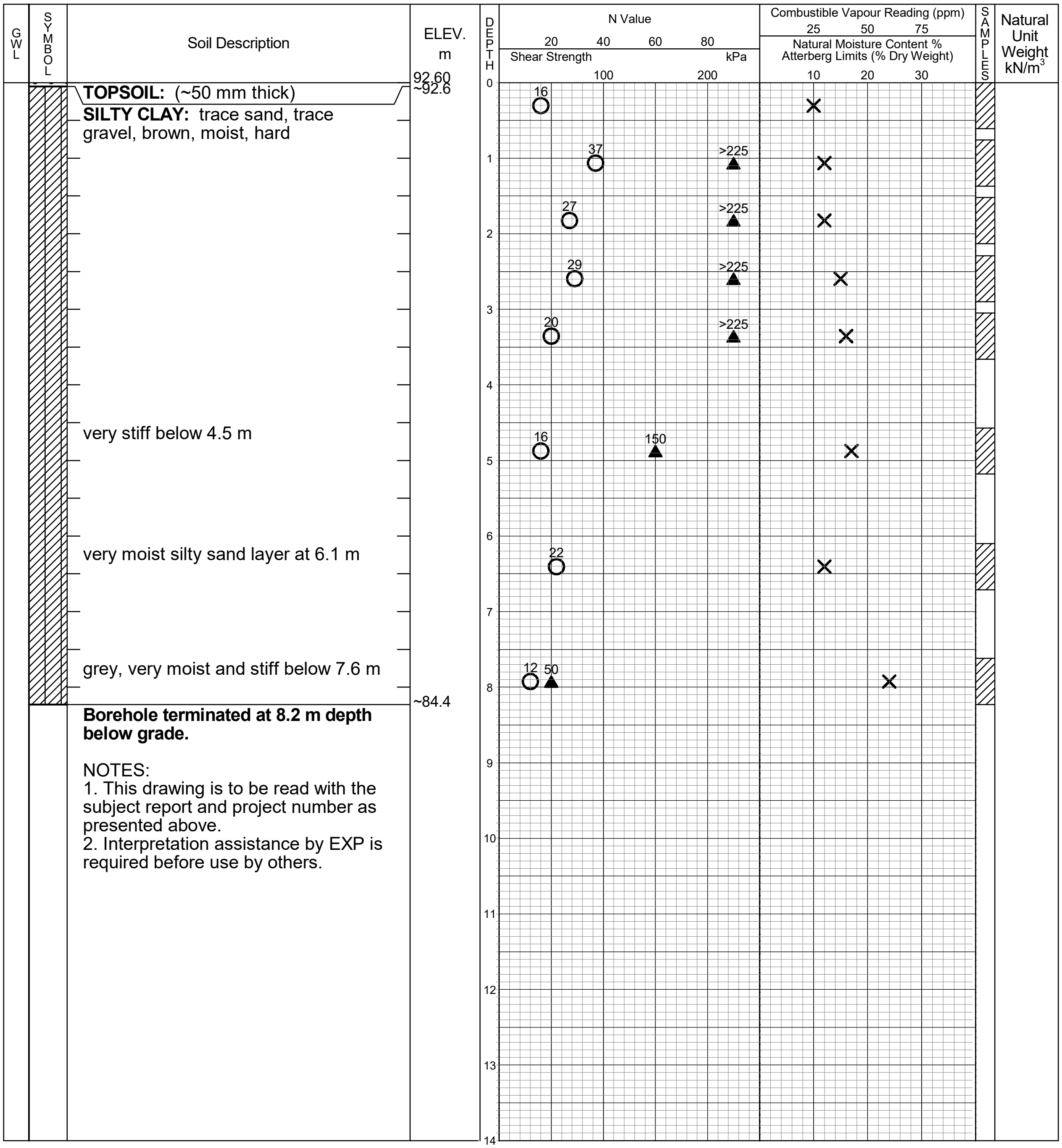
Location: 1544 & 1546 Four Mile Creek Road, Niagara-on-the-Lake, Ontario

Date Drilled: September 24, 2024

Drill Type: D-50 Track Mount. Solid Stem.

Datum: Geodetic

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	×
Plastic and Liquid Limit	├──○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲



TOPSOIL: (~50 mm thick)

SILTY CLAY: trace sand, trace gravel, brown, moist, hard

very stiff below 4.5 m

very moist silty sand layer at 6.1 m

grey, very moist and stiff below 7.6 m

Borehole terminated at 8.2 m depth below grade.

NOTES:
1. This drawing is to be read with the subject report and project number as presented above.
2. Interpretation assistance by EXP is required before use by others.

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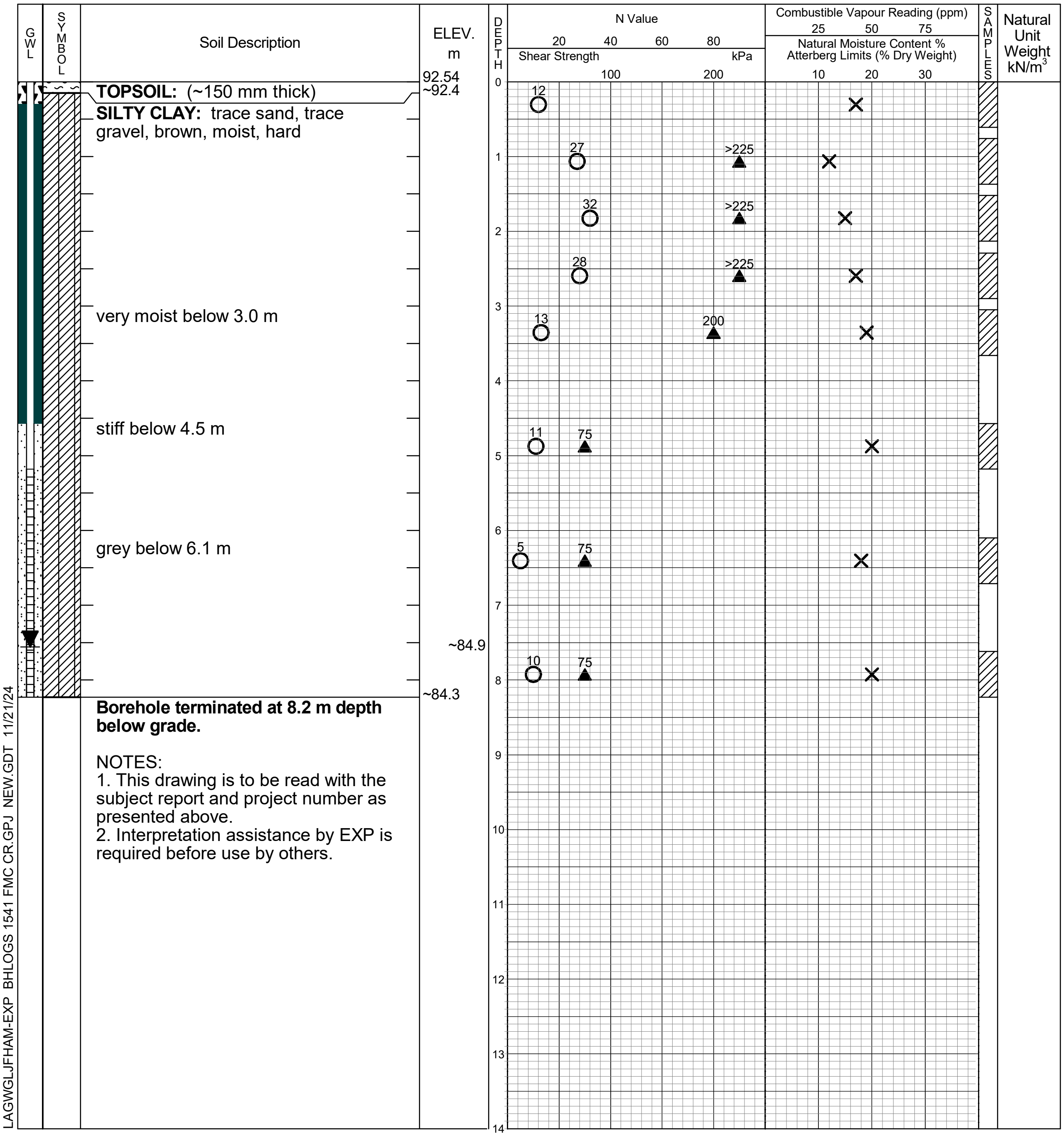
Time	Water Level (m)	Depth to Cave (m)
on completion	dry	open

Log of Borehole BH-3

Project No. HAM-24000672-A0
Drawing No. 5
Project: Proposed Mixed-Use Development
Sheet No. 1 of 1
Location: 1544 & 1546 Four Mile Creek Road, Niagara-on-the-Lake, Ontario

Date Drilled: September 26, 2024
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	✕
Plastic and Liquid Limit	├──○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲



TOPSOIL: (~150 mm thick)

SILTY CLAY: trace sand, trace gravel, brown, moist, hard

very moist below 3.0 m

stiff below 4.5 m

grey below 6.1 m

Borehole terminated at 8.2 m depth below grade.

NOTES:
1. This drawing is to be read with the subject report and project number as presented above.
2. Interpretation assistance by EXP is required before use by others.

LAGWGLJFHAM-EXP BHLOGS 1541 FMC CR.GPJ NEW.GDT 11/21/24

EXP Services Inc.
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Telephone: 905.573.4000
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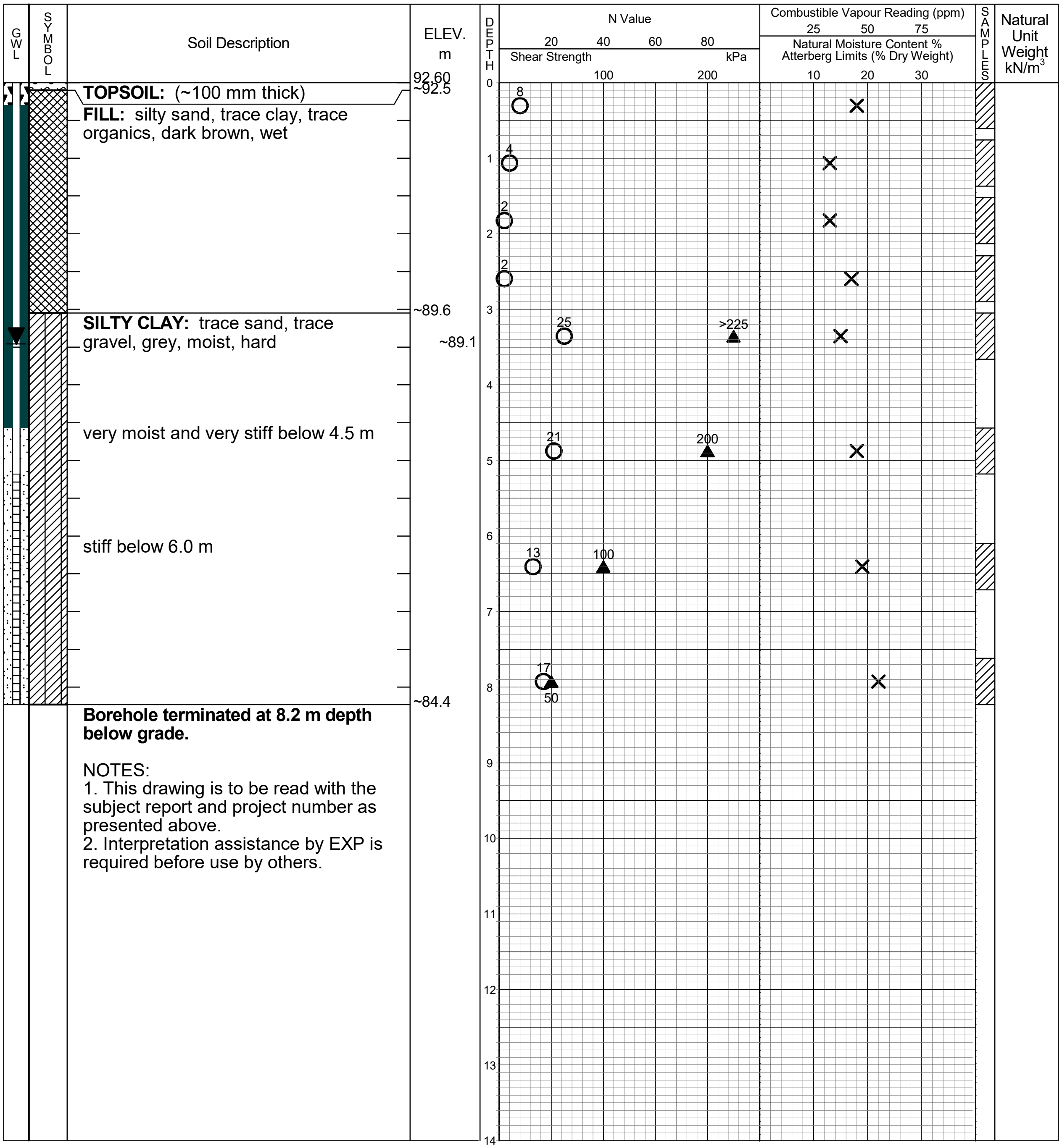
Time	Water Level (m)	Depth to Cave (m)
on completion	dry	open
October 24, 2024	7.6	-

Log of Borehole BH-4

Project No. HAM-24000672-A0
Drawing No. 6
Project: Proposed Mixed-Use Development
Sheet No. 1 of 1
Location: 1544 & 1546 Four Mile Creek Road, Niagara-on-the-Lake, Ontario

Date Drilled: September 24, 2024
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├─○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



TOPSOIL: (~100 mm thick)

FILL: silty sand, trace clay, trace organics, dark brown, wet

SILTY CLAY: trace sand, trace gravel, grey, moist, hard

very moist and very stiff below 4.5 m

stiff below 6.0 m

Borehole terminated at 8.2 m depth below grade.

NOTES:
1. This drawing is to be read with the subject report and project number as presented above.
2. Interpretation assistance by EXP is required before use by others.

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Time	Water Level (m)	Depth to Cave (m)
on completion	dry	1.2
October 24, 2024	3.5	-

Log of Borehole BH-5

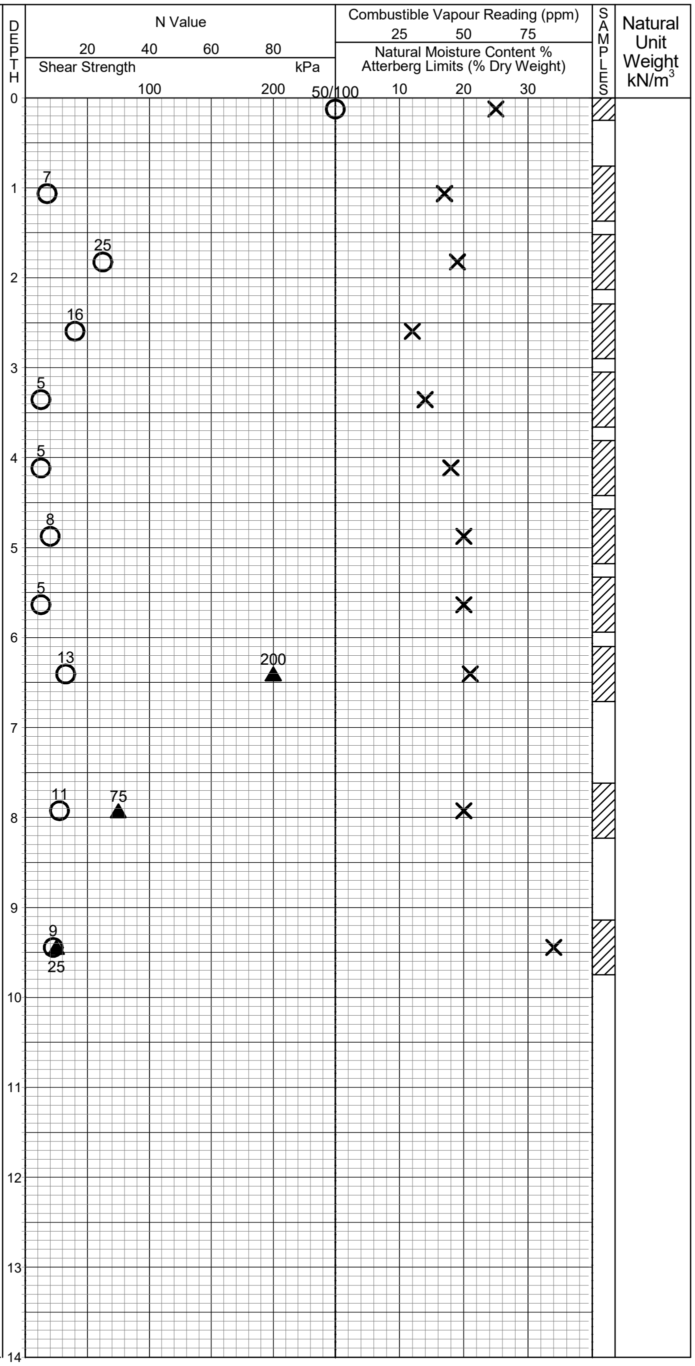
Project No. HAM-24000672-A0
Drawing No. 7
Project: Proposed Mixed-Use Development
Sheet No. 1 of 1
Location: 1544 & 1546 Four Mile Creek Road, Niagara-on-the-Lake, Ontario

Date Drilled: September 25, 2024
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

ELEV. m	Soil Description
91.68 ~91.6	**TOPSOIL:** (~50 mm thick)
	FILL: silty sand, trace gravel, trace asphalt fragments, black, wet
~90.9	**FILL:** sandy silt, some clay, trace gravel, trace deleterious materials, dark brown, wet
~90.2	**FILL:** silty clay, trace sand, trace gravel, trace organics, trace asphalt fragments, brown, moist
~86.4	**FILL:** silty sand, some clay, trace organics, brown and grey, wet
~85.6	**SILTY CLAY:** trace sand, trace gravel, brown, very moist, hard grey and stiff below 7.6 m wet and firm below 9.1 m
~81.9	**Borehole terminated at 9.8 m depth below grade.**

NOTES:
1. This drawing is to be read with the subject report and project number as presented above.
2. Interpretation assistance by EXP is required before use by others.



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Time	Water Level (m)	Depth to Cave (m)
on completion	6.1	6.7

Log of Borehole BH-6

Project No. HAM-24000672-A0
Drawing No. 8
Project: Proposed Mixed-Use Development
Sheet No. 1 of 1
Location: 1544 & 1546 Four Mile Creek Road, Niagara-on-the-Lake, Ontario

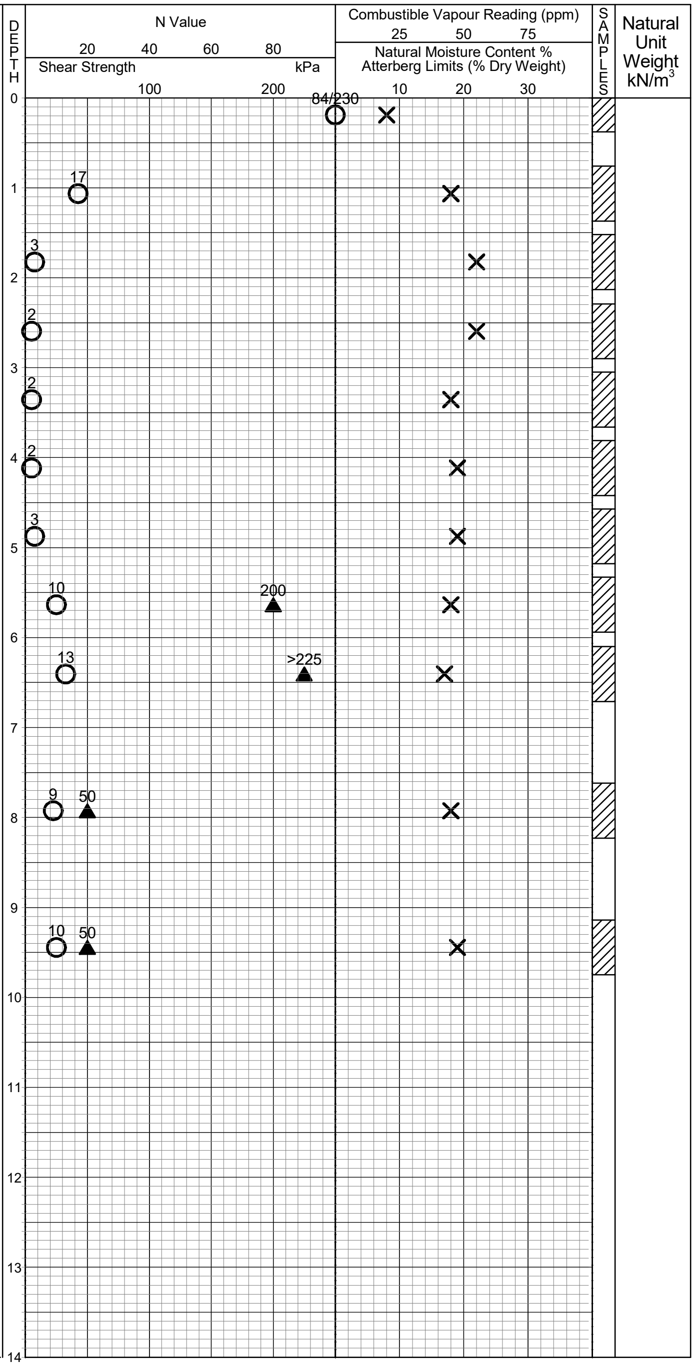
Date Drilled: September 25, 2024
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	⊠
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	×
Plastic and Liquid Limit	├—○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m
			92.12
		GRANULAR FILL: (~450 mm thick)	~91.7
		FILL: gravelly sand, trace asphalt fragments, brown, moist	~91.4
		FILL: silty clay, trace sand, trace gravel, trace plastic fragments, brown, moist	~90.6
		FILL: sandy silt, some clay, brown, wet	~89.8
		FILL: silty clay, some sand, trace gravel, trace organics, dark brown, very moist	~86.8
		SILTY CLAY: trace sand, trace gravel, brown, very moist, hard grey and stiff below 7.6 m	~82.4

Borehole terminated at 9.8 m depth below grade.

NOTES:
1. This drawing is to be read with the subject report and project number as presented above.
2. Interpretation assistance by EXP is required before use by others.



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Facsimile: 905.573.9693

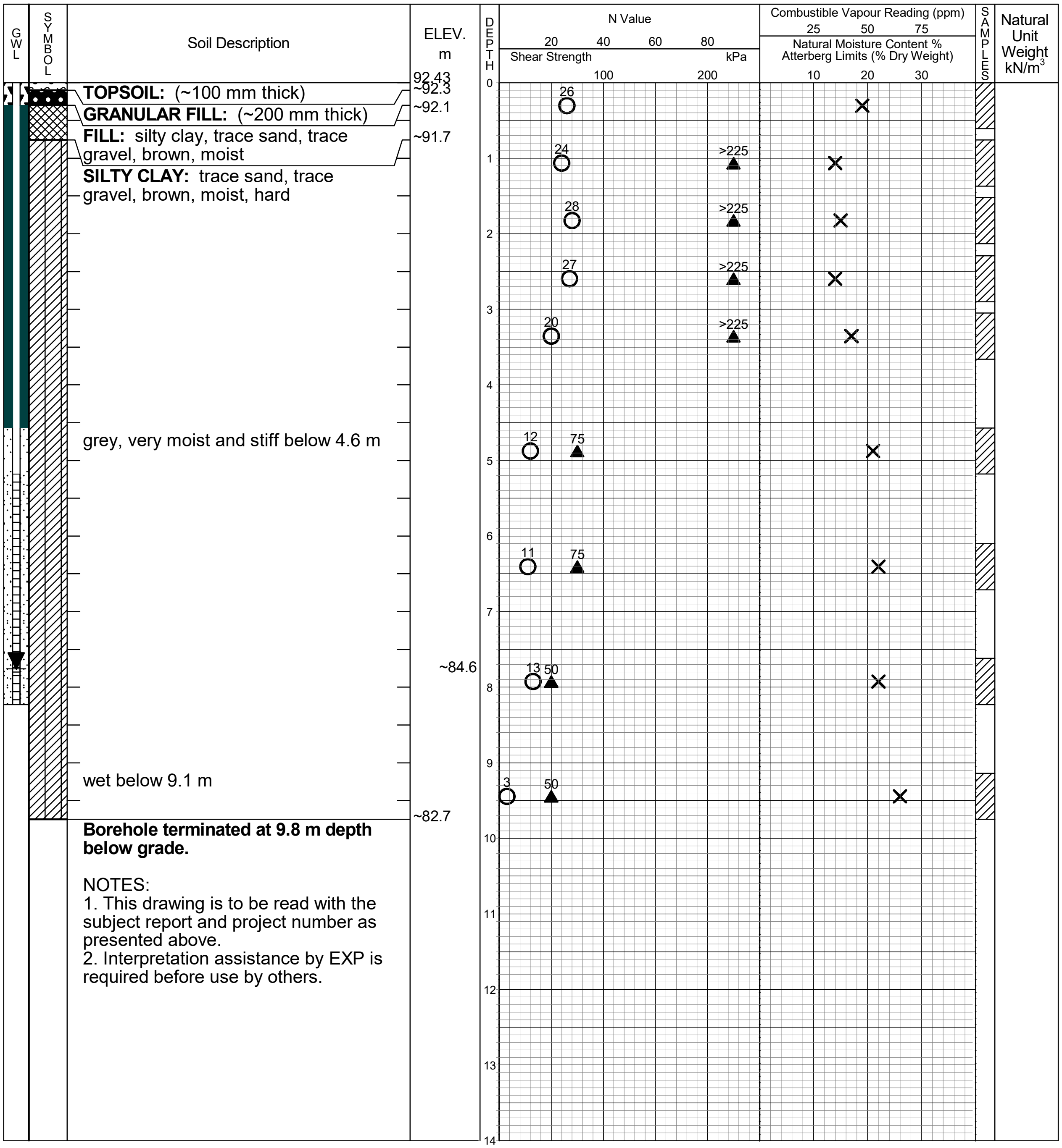
Time	Water Level (m)	Depth to Cave (m)
on completion	dry	open

Log of Borehole BH-7

Project No. HAM-24000672-A0 | Drawing No. 9
Project: Proposed Mixed-Use Development | Sheet No. 1 of 1
Location: 1544 & 1546 Four Mile Creek Road, Niagara-on-the-Lake, Ontario

Date Drilled: September 24, 2024
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



ELEV. m	Soil Description
92.43	
~92.3	**TOPSOIL:** (~100 mm thick)
~92.1	**GRANULAR FILL:** (~200 mm thick)
~91.7	**FILL:** silty clay, trace sand, trace gravel, brown, moist
	SILTY CLAY: trace sand, trace gravel, brown, moist, hard
	grey, very moist and stiff below 4.6 m
	wet below 9.1 m
~82.7	**Borehole terminated at 9.8 m depth below grade.**

NOTES:
1. This drawing is to be read with the subject report and project number as presented above.
2. Interpretation assistance by EXP is required before use by others.

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Facsimile: 905.573.9693

Time	Water Level (m)	Depth to Cave (m)
on completion	dry	open
October 24, 2024	7.8	-

Log of Borehole BH-8

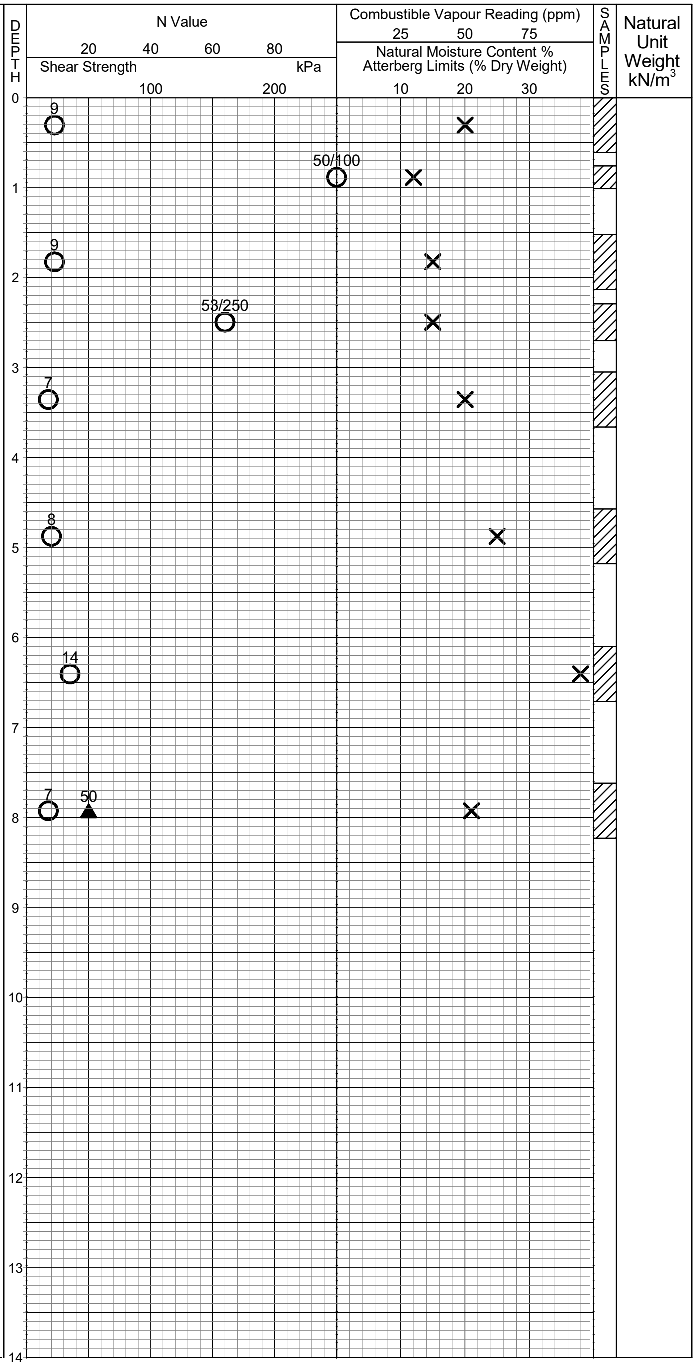
Project No. HAM-24000672-A0
Drawing No. 10
Project: Proposed Mixed-Use Development
Sheet No. 1 of 1
Location: 1544 & 1546 Four Mile Creek Road, Niagara-on-the-Lake, Ontario

Date Drilled: September 26, 2024
Drill Type: D-50 Track Mount. Solid Stem.
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

ELEV. m	Soil Description
92.64	**TOPSOIL:** (~130 mm thick)
~92.5	**FILL:** silty clay, trace sand, trace gravel, brown, moist
~91.9	**FILL:** gravelly sand, trace silt, trace brick fragments, brown, moist
~91.1	**FILL:** silty clay, some gravel, trace sand, trace organics, dark brown, moist trace ashphalt fragments at 2.3 m
~86.5	**POSSIBLE FILL:** silty clay, some organics, trace sand, black, wet (possible alluvial deposit)
~85.0	**SILTY CLAY:** trace sand, trace gravel, grey, very moist, stiff
~84.4	**Borehole terminated at 8.2 m depth below grade.**

NOTES:
1. This drawing is to be read with the subject report and project number as presented above.
2. Interpretation assistance by EXP is required before use by others.



EXP Services Inc.
Hamilton, Ontario
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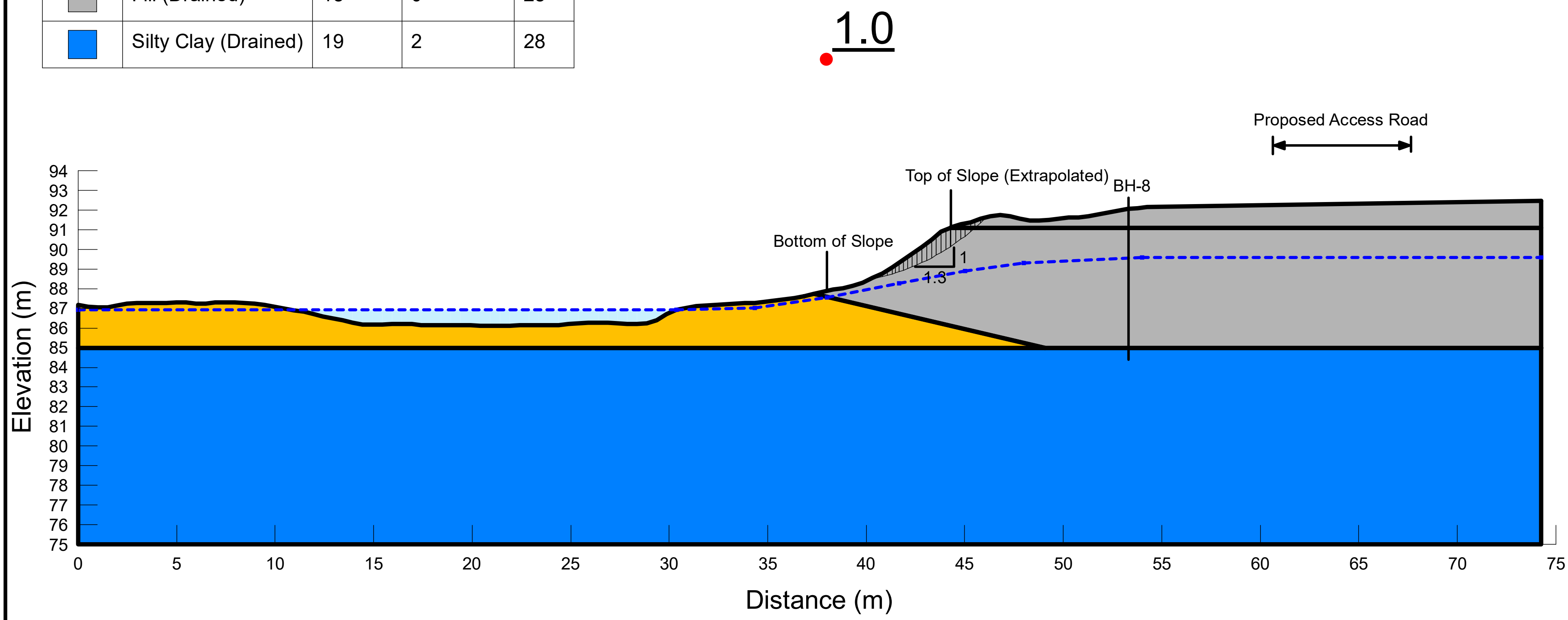
Facsimile: 905.573.9693

Time	Water Level (m)	Depth to Cave (m)
on completion	3.0	open

Appendix B

Slope Stability Analysis

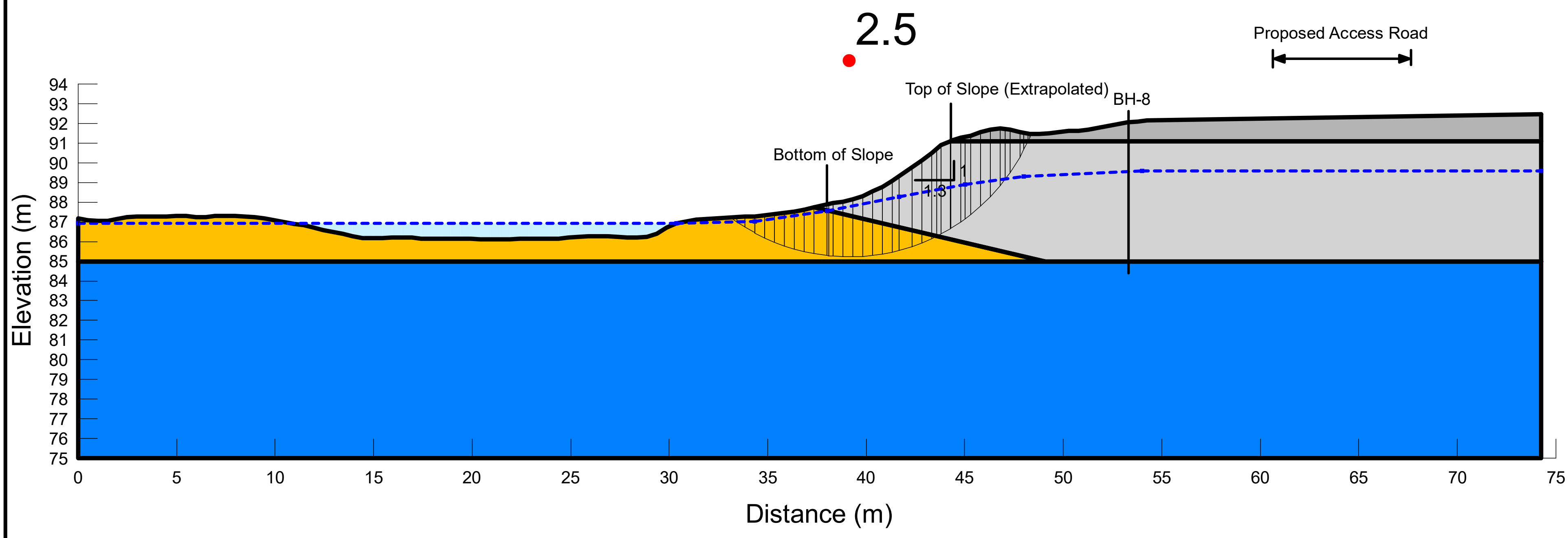
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟨	Creek Bank	20	0	28
⬜	Fill (Drained)	18	0	28
🟦	Silty Clay (Drained)	19	2	28



Section : 1-1
Slope Height : 3.2
Slope Gradient : 2.0 : 1
Drained / Undrained Condition : Drained
Static / Seismic Scenario : Static
Analysis Method : Morgenstern - Price
Surcharge : Nil

Figure B1

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)
🟧	Creek Bank	20		0	28
■	Fill (Drained)	18		0	28
□	Fill (Undrained)	18	50		
🟦	Silty Clay (Undrained)	19	50		

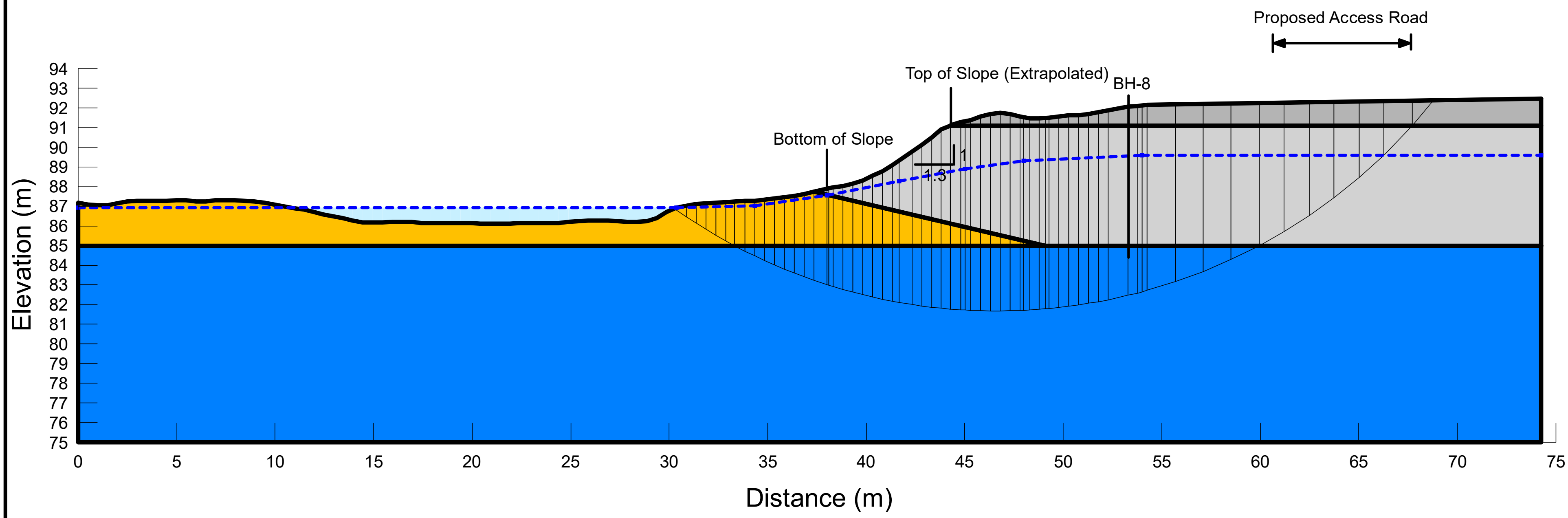


Section : 1-1
Slope Height : 3.2
Slope Gradient : 2.0 : 1
Drained / Undrained Condition : Undrained
Static / Seismic Scenario : Static
Analysis Method : Morgenstern - Price
Surcharge : Nil

Figure B2

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)
🟨	Creek Bank	20		0	28
▨	Fill (Drained)	18		0	28
⬜	Fill (Undrained)	18	50		
🟦	Silty Clay (Undrained)	19	50		

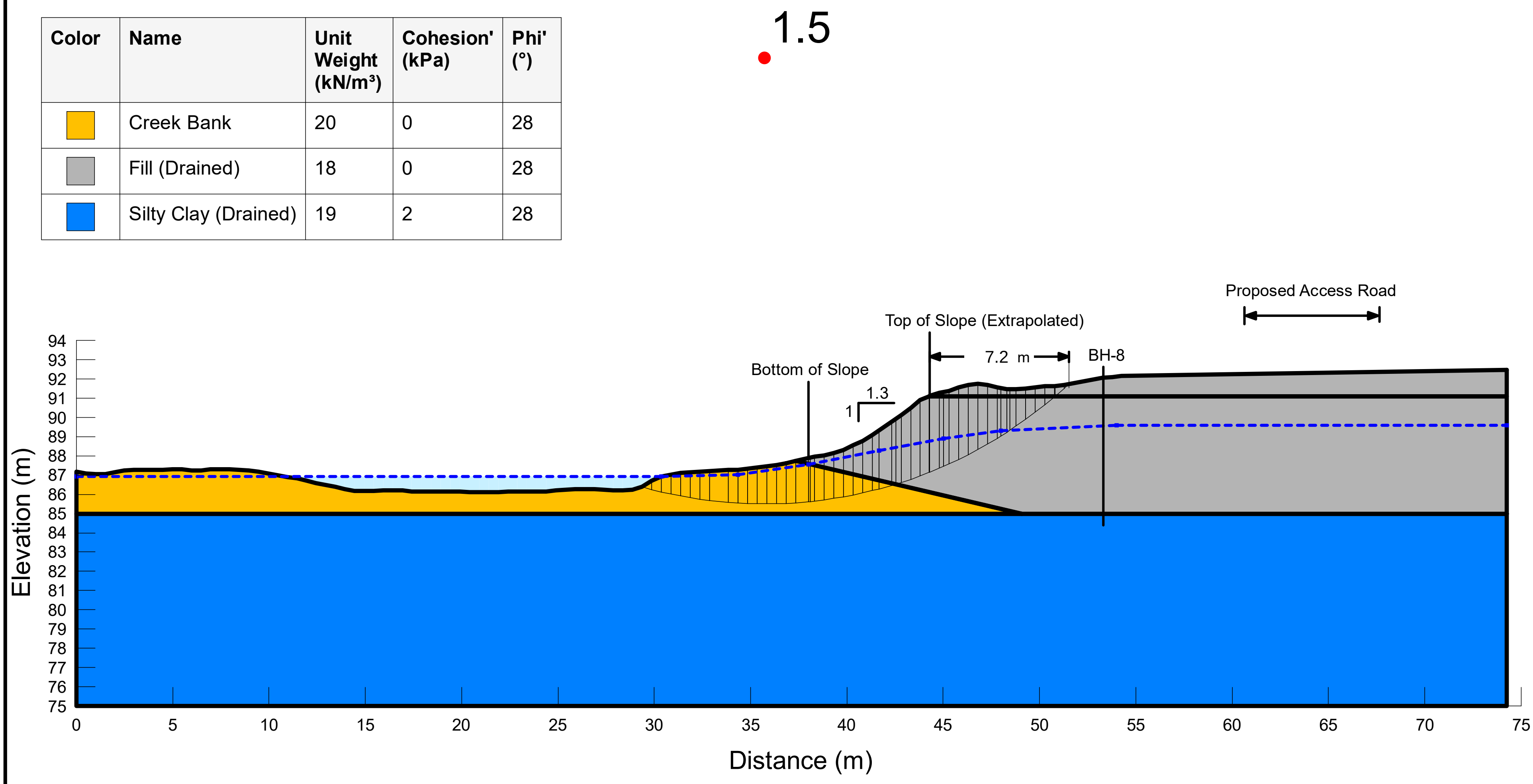
1.2



Section : 1-1
Slope Height : 3.2
Slope Gradient : 2.0 : 1
Drained / Undrained Condition : Undrained
Static / Seismic Scenario : Seismic
Analysis Method : Morgenstern - Price
Surcharge : Nil

Figure B3

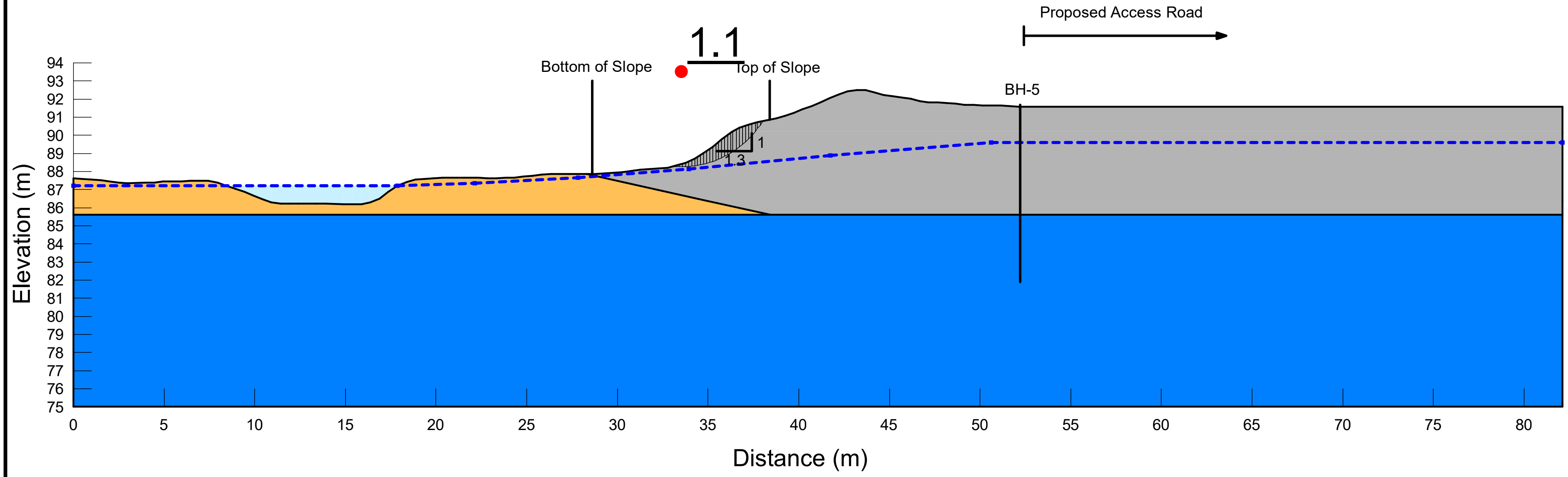
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟨	Creek Bank	20	0	28
⬜	Fill (Drained)	18	0	28
🟦	Silty Clay (Drained)	19	2	28



Section : 1-1 (FS 1.5)
Slope Height : 3.2
Slope Gradient : 2.0 : 1
Drained / Undrained Condition : Drained
Static / Seismic Scenario : Static
Analysis Method : Morgenstern - Price
Surcharge : Nil

Figure B4

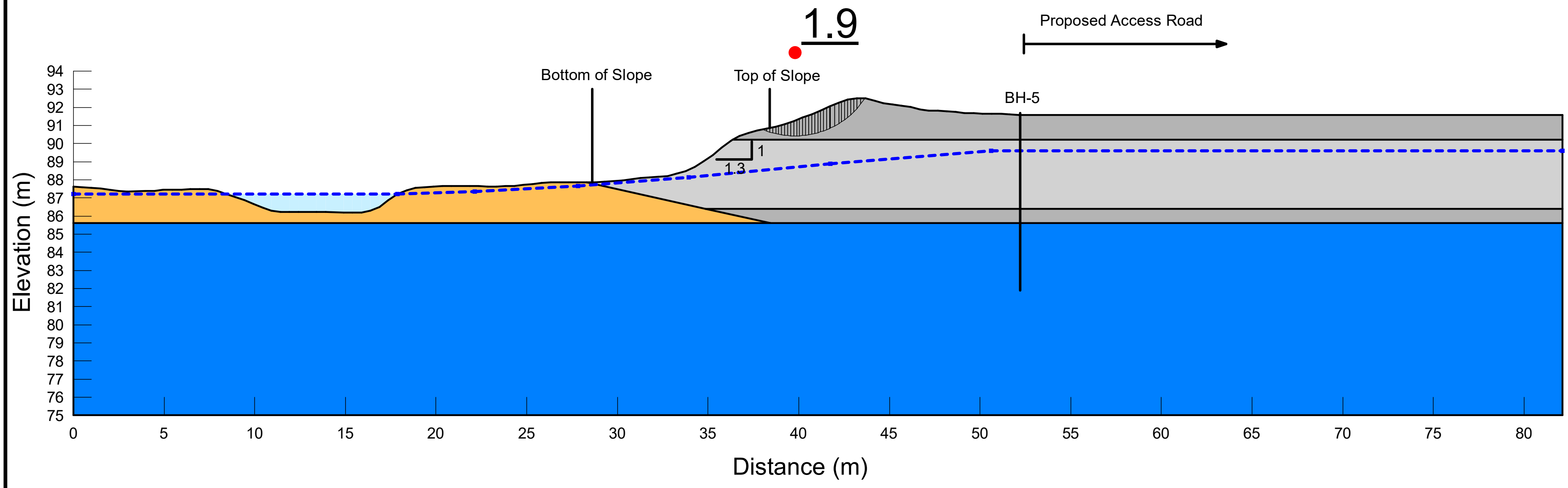
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟧	Creek Bank	20	0	28
⬜	Fill (Drained)	18	0	28
🟦	Silty Clay (Drained)	19	2	28



Section : 2-2
Slope Height : 3.0
Slope Gradient : 3.3 : 1
Drained / Undrained Condition : Drained
Static / Seismic Scenario : Static
Analysis Method : Morgenstern - Price
Surcharge : Nil

Figure B5

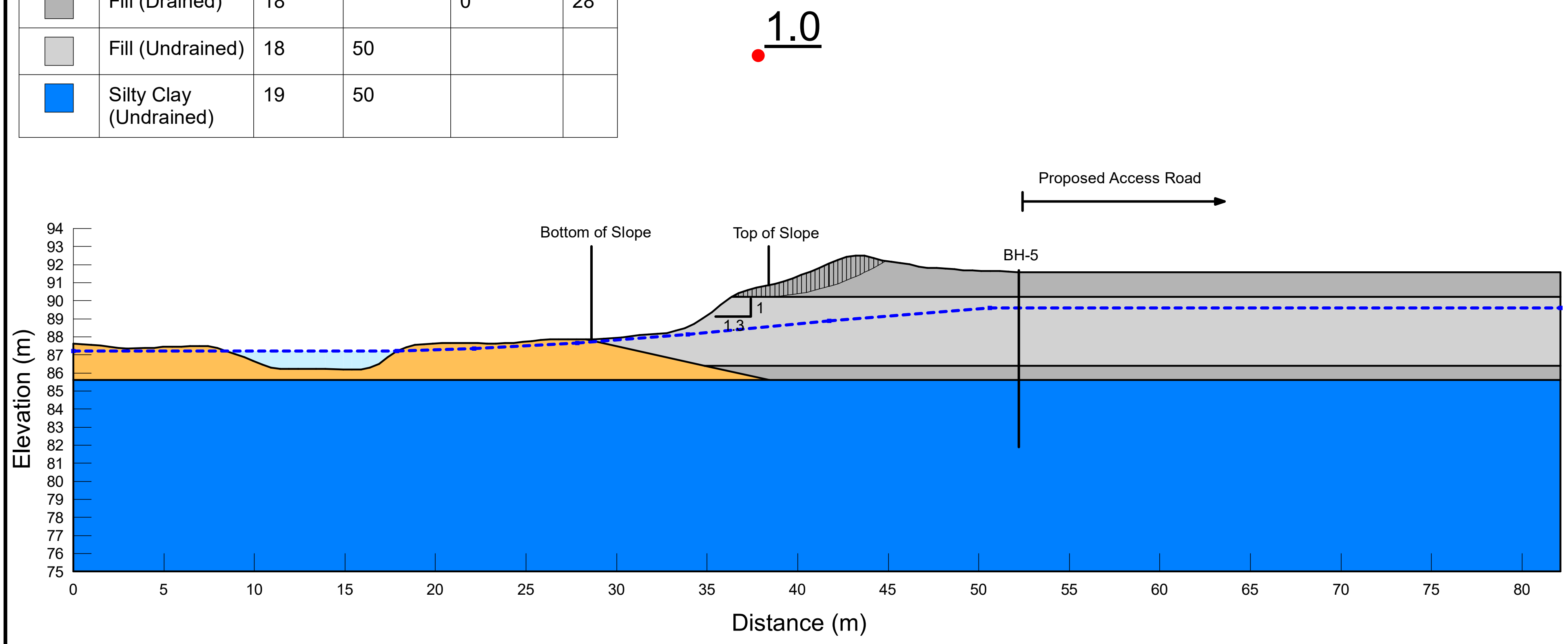
Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)
🟧	Creek Bank	20		0	28
◼	Fill (Drained)	18		0	28
◻	Fill (Undrained)	18	50		
🟦	Silty Clay (Undrained)	19	50		



Section : 2.2
Slope Height : 3.0
Slope Gradient : 3.3 : 1
Drained / Undrained Condition : Undrained
Static / Seismic Scenario : Static
Analysis Method : Morgenstern - Price
Surcharge : Nil

Figure B6

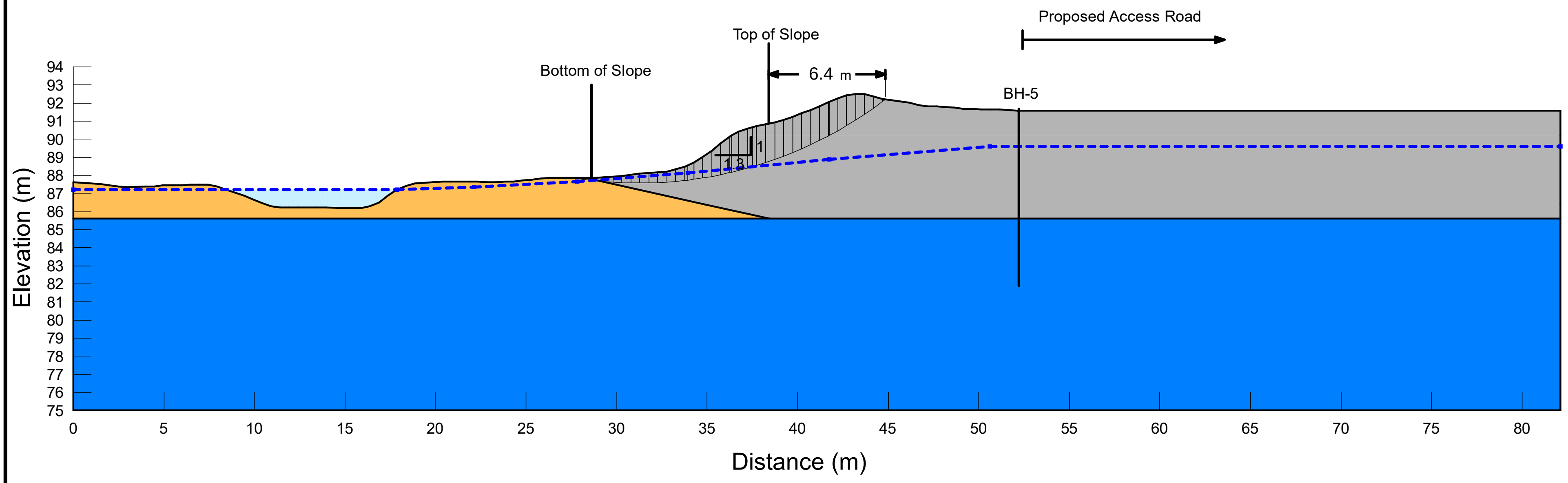
Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)
🟧	Creek Bank	20		0	28
■	Fill (Drained)	18		0	28
□	Fill (Undrained)	18	50		
🟦	Silty Clay (Undrained)	19	50		



Section : 2-2
Slope Height : 3.0
Slope Gradient : 3.3 : 1
Drained / Undrained Condition : Undrained
Static / Seismic Scenario : Seismic
Analysis Method : Morgenstern - Price
Surcharge : Nil

Figure B7

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟧	Creek Bank	20	0	28
⬜	Fill (Drained)	18	0	28
🟦	Silty Clay (Drained)	19	2	28



Section : 2-2 (FS 1.5)
Slope Height : 3.0
Slope Gradient : 3.3 : 1
Drained / Undrained Condition : Drained
Static / Seismic Scenario : Static
Analysis Method : Morgenstern - Price
Surcharge : Nil

Figure B8