- # DCR Phoenix Group of Companies

Geotechnical Investigation

Type of Document
Final

Project Name
Hillside Vista Walk-up Condos
St-Joseph Boulevard and Tenth Line Road, Ottawa, ON

Project Number
OTT-00241432-A0

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Date Submitted
November 27, 2017

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Executive Summary

A geotechnical investigation was undertaken at the site of the proposed Hillside Vista Walk-up Condos to be located on De La Recolte Private in Orleans, City of Ottawa, Ontario. This work was authorized by Mr. Michael Boucher on behalf of DCR Phoenix Group of Companies on July 14, 2017.

It is proposed to construct five 18-unit blocks of walk-up condos. Each block will be three-storeys high with one basement level. The basements are to be used for underground parking. A series of retaining walls are also proposed on the southside of the proposed development. Global stability of these retaining walls was outside the scope of work of EXP Services Inc. (EXP) and will be completed as part of the retaining wall design.

The geotechnical investigation has revealed that the surficial soil at the site is fill, which extends to 0.7 m to 10.4 m depth. The fill is underlain by silty clay, which extends to 4.6 m to 18.5 m depth. The silty clay is hard to firm. It is likely that the silty clay is underlain by bedrock at a depth of 1.0 m to 18.5 m. The groundwater table was established at 4.8 m to 16.5 m depth, i.e. Elev. 55.6 to 60.6 m. The groundwater was established at depths of 4.8 m to 16.5 m below the ground surface, i.e. Elev. 55.6 m to 60.6 m. It should be noted that some areas of the site were not accessible to the drill rig or the excavator, and therefore, assumptions were made on the subsurface conditions in these areas of the site.

The proposed structures may be founded on spread and strip footings set on bedrock in areas where the bedrock is shallow and on piles driven to practical refusal on bedrock in areas where the bedrock is overlain by considerable depth of fill and silty clay. The footings and pile foundations may be designed as per the recommendations contained in the report. The lowest level floors of the garages should be paved for easy maintenance since the underlying fill may be prone to some settlement. Perimeter drains should be provided for the proposed structures. Whether underfloor drains are required should be determined during construction.

The site of Block 1 has been classified as Class C for seismic site response in accordance with the requirements of Table 4.1.8.4A of the Ontario Building Code, 2012. The sites of all the other blocks have been specified as Class D.

The on-site soils are not susceptible to liquefaction during a seismic event.

As the proposed structures will be located partly uphill of the toe of the slope, slope stability analyses were undertaken. It revealed that the existing slope is currently stable and is also expected to be stable post-construction in the long-term and during any seismic event. However, cut slopes at Section C-C would have to be flattened to 2.75H:1V and cut at Section E-E to 2.5H:1V. Slope stability analyses of the proposed retaining walls must be undertaken once the type, depth, and height of the proposed retaining walls have been finalized.

Excavations at the site will extend to a maximum depth of 6 m to 7 m below the existing ground surface through the fill into the underlying silty clay. They will be predominantly above the groundwater table. These excavations may be cut back at 45 degrees in the fill except where they terminate in the natural silty clay in which case the lowermost 1.2 m of the excavations may be cut vertical.

The above and other related considerations have been discussed in greater detail in the report.

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1 Introduction

A geotechnical investigation was undertaken at the site of the proposed Hillside Vista Walk-up Condos to be located on De La Recolte Private in the Orleans suburb of the City of Ottawa, Ontario (Figure 1). This work was authorized by Mr. Michael Boucher on behalf of DCR Phoenix Group of Companies on July 14, 2017.

It is proposed to construct five blocks of walk up condos on the site. Each block would be three storeys high with one basement level. The basements are to be used for underground parking. Each block will contain 18 units. A number of retaining walls are also proposed on the southside of the proposed development. Recommendations pertaining to the design of these retaining walls was outside the scope of work of EXP Services Inc. (EXP) and will be completed by others.

The investigation was undertaken to establish:

a.) Subsurface and groundwater conditions at the location of the boreholes and test pits;

b.) Foundation options for the various structures proposed, i.e. conventional spread/strip footings, deep foundation, depth of suitable founding stratum and Serviceability Limit State (SLS) and Ultimate Limit State (ULS) bearing pressure of the founding stratum;

c.) Grade-raise restrictions;

d.) Anticipated total and differential settlements;

e.) Classification of the site for seismic site classification in accordance with requirements of Ontario Building Code and liquefaction potential of on-site soils;

f.) Feasibility of construction of slab-on-grade floors and drainage requirements;

g.) Lateral earth pressure on sub-surface walls;

h.) Excavation conditions anticipated;

i.) Backfilling requirements and suitability of on-site soils for backfilling purposes;

j.) Subsurface concrete requirements; and

k.) Slope stability assessment.

The comments and recommendations given in this report are based on the assumption that the above-described design concepts 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 it may require additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

2 Background Information

Previous geotechnical investigations undertaken at the site by others were made available by Phoenix Home. In addition, previous geotechnical investigations undertaken on the site by Trow (now EXP) were reviewed. Relevant information from these documents was used to plan the geotechnical investigation and to prepare this report. The following is a list of documents consulted.

1) *Supplemental Geotechnical Investigation, Proposed Residential Development, Orleans Town Centre East Townhouses, Tenth Line Road, Ottawa, Ontario*; prepared for Novatech Engineering Consultants by Paterson Group under Report PG 1743-3; dated October 26, 2011.
2) *Preliminary Geotechnical Investigation, Orleans Town Centre, Centrum Boulevard, Orleans, Ontario*; prepared for OTCP Arts Centre Limited Partnership et al by Fondex Ontario Limited; reference T020062-A1; dated November 7, 2006.
3) *Geotechnical Investigation, Proposed Condo Apartments, 3277 St. Joseph Boulevard, Ottawa, Ontario*; prepared for DCR Phoenix Homes by Trow Consulting Engineers Ltd (now EXP); project MA15643A; dated December 20, 2002.
4) *Preliminary Geotechnical Investigation, Proposed Residential Development, CH Lands, St. Joseph Boulevard, Ottawa, Ontario*; prepared for City of Ottawa by Trow Associates Inc. (now EXP), Project No. OTGE00016754A; dated October 6, 2003.
5) *Additional Slope Stability Analysis, Proposed Condo Apartments; 3277 St. Joseph Boulevard, Ottawa, Ontario*; prepared for Phoenix Homes by Trow Associates Inc.; project OTGE00015643C; dated December 15, 2003.
6) *Geotechnical Investigation, Hillside Terrace Flats, Hwy 174 and 10th Line Road, Orleans, Ontario,* prepared for Phoenix Homes by exp under Project No. OTT-00218474-A0 dated September 9, 2015.

Based on the previous work undertaken at the site, it is known that the ground surface elevations at the site slope down to the north from Elevation 73.5 m on top of the escarpment to Elevation 63.0 m in the area south of Highway 174. The overall slope inclination varies from approximately 2.5H:1V to 4.5H:1V with localized steeper sections. It is noted that a clay filled gulley up to 19 m deep is present in the vicinity of Blocks 3, 4 and 5.

3 Procedure

The fieldwork for the geotechnical investigation was undertaken on August 17 and 23, 2017. It comprised of excavating seven test pits to refusal at 1.5 m to 5.9 m depth (Test Pits A to E, I and J). In addition, nine boreholes were drilled at the site to 5.5 m to 18.5 m depth. (Boreholes F to H, and K to P). A dynamic cone penetration test was also performed in Borehole M from 17.3 m depth to refusal at 18.5 m depth.

The fieldwork was undertaken with a track-mounted CME-55 drill rig equipped with continuous flight follow stem auger and with a backhoe. The fieldwork was supervised on a full-time basis by a representative of EXP. The locations of the boreholes and test pits are shown on Site Plan, Figure 2. It should be noted that some areas of the site were not accessible to the drill rig or the excavator and therefore assumptions were made on the subsurface conditions in these areas of the site.

Standard penetration tests were performed in all the boreholes at 0.75 m to 1.5 m depth intervals and soil samples retrieved by split-barrel sampler in accordance with ASTM 1586. The undrained shear strength of the clay was established by field vane shear tests as per ASTM 2573. Relatively undisturbed thin wall tube samples of the clay were also obtained from some of the boreholes at selected depths in accordance with ASTM D1587.

All the soil samples were visually examined in the field for textural classification, logged, preserved in plastic bags and identified. The thin wall tube samples were logged, capped and identified. All the test pits were logged based on the examination of the excavated soil and representative soil samples were obtained. The soil samples were logged, preserved in plastic bags and identified. All the test pits were backfilled on completion.

On completion of the fieldwork, all the soil samples were transported to the EXP laboratory in the city of Ottawa, Ontario. All the soil samples were visually examined in the laboratory by a geotechnical engineer and borehole logs prepared. The engineer also assigned the laboratory testing which consisted of performing natural moisture content, unit weight, grain-size analyses, Atterberg Limit, one-dimensional oedometer, pH, sulphate, chloride content and electrical resistivity tests on selected soil samples.

Water levels were measured in the open boreholes on completion of drilling. In addition, long-term groundwater monitoring installations consisting of 19 mm diameter PVC (polyvinyl chloride) pipe were placed in selected boreholes in accordance with EXP standard practice. The installation configuration is documented on the respective borehole logs. All the boreholes were backfilled upon completion of the fieldwork. The locations and elevations of the boreholes were surveyed by Novatech. The elevations of the boreholes refer to the geodetic datum.

4 Soil Description

A detailed description of the subsurface soil and groundwater conditions determined from the boreholes and test pits are given on the attached Borehole Logs, Figures 3 to 18 inclusive. The borehole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted. Boreholes were drilled to provide representation of subsurface conditions as part of a geotechnical exploration program and are not intended to provide evidence of potential environmental conditions.

It should be noted that the soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The “Note on Sample Descriptions” preceding the borehole logs form an integral part of this report and should be read in conjunction with this report.

The soil stratigraphy encountered at the site is discussed below in descending order from the existing ground surface.

4.1 Topsoil

A surficial layer of topsoil 250 mm thick was encountered in Test Pit A.

4.2 Fill

Underlying the topsoil in Test Pit A and from the existing ground surface elsewhere on the site, fill was encountered, which extends to a depth of 0.7 m to 10.4 m (Elev. 59.7 m to 66.8 m). The fill mainly consists of silty clay although it is mixed with sand and gravel in localized areas. It is generally very loose to loose although some compact zones were also encountered as indicated by the N values, which vary from 2 to 14. The natural moisture content of the fill and its unit weight are 5 to 64 percent and 17.6 to 20.7 kN/m^3 respectively.

4.3 Topsoil

A layer of topsoil 300 mm thick was encountered underlying the fill in Test Pit B, which extends to 4 m depth. Refusal to excavation in this test pit was encountered at 4 m depth.

4.4 Silty Clay Crust

The fill in all the boreholes and test pits except Test Pits A, B and D is underlain by silty clay crust, which varies in thickness from 0.3 m to 5.9 m. The undrained shear strength of the silty clay crust varies from 120 kPa to 216 kPa. It has a natural moisture content of 23 to 59 percent.

4.5 Firm to Stiff Silty Clay

The silty clay crust is underlain by firm to stiff silty clay, which extends to 4.6 m to 18.5 m depth (Elev. 55.3 m to 64.9 m). The shear strength of this stratum varies from 62 kPa to 120 kPa. Its moisture content varies from 17.0 to 66.0 percent. It has a unit weight of 15.7 to 16.3 kN/m^3.

The results of the three grain-size analyses performed on the silty clay samples are given on Figures 19 to 21 inclusive. A review of these figures indicated that the silty clay comprises of 56 to 75 percent clay, 23 to 43 percent silt and 1 to 2 percent sand.

Three Atterberg Limit Tests were performed on the silty clay samples. The test results have been tabulated on Table 1 and plotted on the Casagrande Chart (Figure 22). A review of this drawing indicates that the clay is inorganic and of high plasticity.

Table 1: Results of Atterberg Limit Tests on Clay Samples

Borehole ID	Sample Depth (m)	Natural Moisture Content (%)	Unit Weight	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index
H	6.1 – 6.7	60	-	81.4	30.8	50.6
N	7.6 – 8.2	52	-	63.9	25.0	38.9
O	14.5 – 15.1	57	-	76.1	29.9	46.2

Three one-dimensional oedometer tests were performed on the stiff to firm silty clay. The test results have been summarized on Table 2 and have been plotted on Figures 23 to 25 inclusive. A review of Table 2 indicates that the silty clay at the site is over-consolidated by 64 to 139 kPa. Its compression and re-compression indices are 1.15 to 1.29 and 0.026 to 0.055 respectively.

Table 2: Results of Consolidation Tests

Borehole ID	Depth (m)	e_o	p'_c	p'_o	c_r	c_c	$p'_c - p'_o$
M	13.6 – 13.7	1.833	280	216	0.026	1.15	64
M	16.6 – 16.7	1.639	340	235	0.026	1.24	105
N	7.2 – 7.3	1.738	240	101	0.055	1.29	139

4.6 Glacial Till

In Test Pit D, the fill is underlain by glacial till, which extended to refusal at a depth of 3.3 m (Elev. 63.7 m). The refusal was met on the bedrock surface.

4.7 Weathered Bedrock

The fill in Test Pit A is underlain by weathered bedrock, which extends to 1.0 m depth (Elev. 65.5 m). Refusal to excavation was met at this level.

4.8 Bedrock

Refusal to excavation or auguring was met in all the test pits and boreholes at a depth of 1.0 m to 18.5 m. The refusal in the test pits was met on bedrock. The refusal to auguring in the boreholes was most likely met on bedrock although it is possible that in some cases this refusal may have been met on cobbles and boulders in the glacial till.

Available information indicates that the bedrock at the site is likely to be limestone with some shaley partings of the Ottawa Formation.

4.9 Groundwater

Water level observations were made in all the boreholes on completion of drilling. In addition, water level observations were made in wells installed in Boreholes F, L, N and P.

A review of water level observations made in all the boreholes previously drilled at the site by EXP in the area under consideration was undertaken. All water level observations have been tabulated on Table 3. A review of this table indicates that the groundwater table at the site is present at a depth of 4.8 m to 16.5 m below the ground surface, i.e. Elev. 55.6 m to 60.6 m. Generally, the groundwater table at the site slopes down towards the northeast.

Table 3: Water Level Observations

Block No.	Borehole ID	Elapsed Time (days)	Date	Water Level Depth (m)	Water Level Elev. (m)
1	13	On completion	9 May 2014	dry	
	14	On completion	9 May 2014	dry	
	17	On completion	13 May 2014	dry	
	18	On completion	9 May 2014	dry	
	3A – 11	On completion	18 May 2011	dry	
2	15	62	29 Aug 2014	6.8	60.6
	F	33	25 Sept 2017	7.9	58.9
	G	On completion	23 Aug 2017	dry	--
3	H	On completion	23 Aug 2017	9.4	56.6
	20	56 days	10 Jul 2014	5.3	63.1
4	02 – 16	13	13 Aug 2002	9.1	56.3
	21	56	15 Jul 2014	4.8	58.9
	02 – 1	61	26 Aug 2002	9.2	57.9
	L	33	25 Sep 2017	dry	--
	M	On completion	23 Aug 2017	14.6	56.4
5	02 – 15	13	13 Aug 2002	5.8	57.7
	N	33	25 Sep 2017	5.4	58.3
	02 – 14	On completion	01 Aug 2002	dry	--
	O	On completion	23 Aug 2017	16.5	55.6
	P	33	23 Aug 2017	6.0	58.2

5 Foundation Considerations

The investigation has revealed that the geotechnical conditions at the site present a few challenges as they are highly variable. The site contains fill, which extends to a depth of 1.0 m to 10.4 m. The depth to bedrock varies from 1 m to 18.4 m. The proposed structures will consist of 3-storeys plus a basement. The basements are to be used for underground parking. The proposed structures will cut into the toe of the escarpment resulting in steeper slopes. Therefore, stability of the escarpment slope will be a major consideration. Slope stability considerations have been discussed in Section 10 of this report. The geotechnical conditions encountered at the site have been summarized on Table 4 for the various blocks proposed. This table also lists the recommended type of foundations for each structure (i.e. 3-storeys and basement). Bearing capacity and settlement considerations of footings set on bedrock and piles driven to practical refusal on bedrock, frost protection and footing bed review requirements have been discussed below. It is recommended that the proposed structures should not be founded on fill. Also, the structures should not be founded partly in the overburden and partly on the bedrock as settlements beyond tolerable limits may result.

Table 4: Geotechnical Conditions in Blocks 1 to 5 and Foundation Recommendations

Block No. and Founding Level	Borehole No.	Ground Surface Elev. (m)	Fill & Topsoil Thickness (m)	Clay/Till Thickness (m)	Refusal Elev. (m)	Recommended Foundations
1 Elev. 65.66 m	13	65.5	1.0	--	64.5	Footings on bedrock
	14	65.7	2.2	1.6	61.9	
	17	65.7	1.2	--	64.5	
	TP B	67.5	4.0	--	63.5	
	18	66.2	1.5	0.4	64.3	
	3 – 11	66.1	1.2	--	64.9	
	3A - 11	66.1	1.8	--	64.3	
2 Elev. 65.36 m	TP D	67.0	2.9	0.4	63.7	Spread footings on bedrock where bedrock is shallow. Driven piles to refusal on bedrock where bedrock is deep.
	15	67.4	6.3	1.9	53.8	
	F	66.8	5.4	3.7	57.7	
	TP E	69.4	4.3	0.3	64.8	
	G	67.4	4.6	0.9	61.9	
3 Elev. 64.84 m	H	66.0	4.8	5.9	55.3	Spread footings on bedrock where rock is present at shallow depth. Driven piles to refusal on bedrock where rock is deep.
	TP I	68.5	4.7	1.2	62.6	
	20	68.4	6.3	0.8	61.3	
	TP J	70.0	5.5	--	64.5	
	4A – 11	66.5	3.7	0.9	61.9	

Table 4: Geotechnical Conditions in Blocks 1 to 5 and Foundation Recommendations

Block No. and Founding Level	Borehole No.	Ground Surface Elev. (m)	Fill & Topsoil Thickness (m)	Clay/Till Thickness (m)	Refusal Elev. (m)	Recommended Foundations
4 Elev. 64.34 m	16	65.4	7.6	10.8	47.0	Footings on bedrock where rock is shallow. Driven piles to refusal on bedrock where bedrock is deep.
	21	63.7	3.2	6.8	53.7	
	L	72.1	8.8	0.9	62.4	
	1	67.1	5.9	10.0	51.2	
	M	71.0	10.0	8.5	52.5	
5 Elev. 64.49	15	63.5	2.4	6.4	54.7	Pipe piles driven to practical refusal on bedrock.
	N	63.7	3.1	5.7	54.9	
	14	65.2	3.8	5.4	56.0	
	O	72.1	10.4	6.4	55.3	
	P	64.3	3.1	3.1	58.1	

5.1 Footings on Bedrock

Footings set on the bedrock may be designed for factored geotechnical resistance at Ultimate Limit State (ULS) of 1 MPa. The settlements of these footings are expected to be very small, i.e. less than 10 mm.

Footings at the site will be set at different levels. Adjacent footings located at different levels should be set such that the higher footings are located below a line drawn up at 10H:7V from the near edge of the lower footing. This is to prevent transference of stress from the higher footings to the lower footings. Also, the lower footings should be constructed prior to the construction of the higher footings to prevent the latter from being undermined during subsequent construction.

5.2 Driven Piles

In areas where the bedrock is deep and footings are not feasible, the proposed structures may be founded on driven piles.

Closed end pipe piles driven to practical refusal on bedrock are considered to be the most suitable type of piles for the site. Closed end pipe piles driven to practical refusal in the bedrock are expected to meet refusal within 0.5 m of encountering the bedrock. Since the piles would be driven to bedrock, the factored geotechnical resistance at Ultimate Limit State would govern the design. The factored geotechnical resistance at Ultimate State Limit of the various piles is given in Table 5. The recommended design load of the piles would vary from 1350 kN to 2000 kN depending on the size of pile used. The recommended design loads assume that the piles will meet refusal in the bedrock. Any piles that meet refusal at a higher level on cobbles and boulders in till will have a lower load carrying capacity. This capacity should be established in the field using pile driving analyzer.

Table 5: Factored ULS Loads on Driven Pipe Piles

Type of Pile	Size	(1) Factored ULS, Load per Pile (kN)	(2) Negative Skin Friction (kN)	(3) Allowable Load per Pile (kN0
Steel Pipe	245 mm O.D. by 12 mm wall thickness	1350	Compute from expression (i)	Col (1) – Col (2)
	324 mm O.D. by 10.9 mm wall thickness	1750	Compute from expression (i)	Col (1) – Col (2)
	324 mm O.D. by 12 mm wall thickness	2000	Compute from expression (i)	Col (1) – Col (2)

The recommended factored ULS loads are based on 350 MPa yield strength steel and 30 MPa compressive strength concrete. This office should be contacted to review the design if steel with lower yield strength or concrete with lower compressive strength is used in the design of the piles.

It is noted that the grades at the site will be raised in some areas. Consequently, the piles will be subjected to negative skin friction. The negative skin friction may be computed from the following expression.

$$q_n = \beta \sigma_v' \text{ ------------------- (i)}$$

Where q_n = The unit negative skin friction at any depth z along the pile

β = reduction coefficient = 0.3
σ_v' = vertical effective stress adjacent to pile at depth z.

The computed negative skin friction must be subtracted from the factored ULS load to arrive at the allowable load per pile.

To achieve the capacity given previously, the pile-driving hammer must seat the pile into bedrock without overstressing the pile material. For guidance purposes, it is estimated that a hammer with rated energy of 54 kJ to 70 kJ (40,000 to 52,000 ft. lbs.) per blow would be required to drive the piles to practical refusal in the bedrock. Practical refusal is considered to have been achieved at a set of five blows for 6 mm or less of pile penetration. However, the driving criteria for a particular hammer-pile system must be established at the beginning of the project. This may be done with the Pile Driving Analyzer.

The site contains sand and gravel and till with numerous boulders. It is therefore recommended that the pile tips should be reinforced with a 25-mm thick steel plate and equipped with a driving Type II shoe in accordance with Ontario Provincial Standards Specifications (OPSS) Standard Drawing 3001.100 (Figure 26),
The pipe piles may have to be spliced because of the length of pile required. The number of splices should be limited to a maximum of one splice per pile.

A number of test piles (five percent of the total number of piles) should be monitored with the Pile Driving Analyzer during the initial driving and re-striking at the beginning of the project and should be subjected to

CAPWAP analysis. This monitoring will allow for the evaluation of transferred energy into the pile from the hammer, determination of driving criteria and an evaluation of the ultimate bearing capacity of the piles. Depending on the results of the pile driving analysis, the pile capacity may have to be proven by at least one pile load test for each pile type before production piling begins. If necessary, the pile load test should be performed in accordance with ASTM D 1143.

Closed end pipe piles tend to displace a relatively large volume of soil. When driven in a cluster or group, they may tend to jack up the adjacent piles in the group. Consequently, the elevation of the top of each pile in a group should be monitored immediately after driving and after all the piles in the group have been driven. This is to ensure that the piles are not heaving. Any piles found to heave more than 3 mm should be re-tapped.

Piles driven at the site may be subject to relaxation i.e. loss of load carrying capacity with time. It is therefore recommended that the piles should be re-struck minimum 24 hours after initial driving to determine if the piles have relaxed. If relaxation is observed, this procedure should be reported every 24 hours until it can be proven that relaxation is no longer a problem.

The installation of the piles at the site should be monitored on a full-time basis by a geotechnician working under the direction and supervision of a qualified geotechnical engineer to verify that the piles are driven in accordance with the project specifications.

Settlements induced by the above recommended pile loads are expected to be less than normally tolerated limits of 25 mm total and 19 mm differential movements.

5.3 Frost Protection Requirements

Adequate soil cover should be provided to all the footings to protect them against damage due to frost penetration. Exterior footings of a heated structure founded on bedrock should be provided with a minimum of 0.9 m of earth cover. Footings of an unheated structure, if founded on bedrock, should be provided with a minimum of 1.2 m of earth cover if snow will not be removed from its vicinity and 1.5 m of earth cover if snow will be removed from its vicinity. Footings of an unheated structure, if founded in overburden, should be provided with a minimum of 2.1 m of earth cover if snow will not be removed from its vicinity and 2.4 m of earth cover if snow will be removed from its vicinity.

5.4 Footing Bed Review Requirements

All the footing beds should be examined by an experienced geotechnician or geotechnical engineer to ensure that the recommended bearing pressure is available at the founding level and that the footing beds have been prepared satisfactorily. In addition, installation of the piles should also be monitored on a full-time basis by a geotechnician working under the direction of a geotechnical engineer.

6 Floor Slab and Drainage Requirements

It is noted that the subgrade of the garage floor may be in-fill in some areas.

It is therefore recommended that the garage floors should be paved instead of providing a concrete slab as it would be easier to maintain the latter in case there are any settlements due to the consolidation of the underlying fill. However, the fill should be evaluated during construction. Based on the evaluation of the subgrade conditions, EXP will be in a position to provide the pavement structure for the garage floors. For preliminary design purposes, the following pavement structure may be considered for light automobile traffic.

65 mm Asphaltic Concrete

150 mm OPSS Granular A

450 mm OPSS Granular B

Perimeter drains should be provided for the structures with basements. Whether underfloor drains are required should be established during construction. The underfloor drainage system, if required, may consist of 150 mm diameter perforated pipe or equivalent placed in parallel rows at 5 m to 6 m centres with its invert at least 300 mm below the underside of the floor slab. The drains should be set on 100 mm of pea gravel and covered on top and sides with 150 mm pea gravel. The pea gravel should be surrounded with suitable filter cloth, such as Terrafix 270R or equivalent. The perimeter and underfloor drains should preferably lead to separate positive sumps from where the water can be removed.

All subsurface walls should be properly damp-proofed. The exterior grade should be sloped away from the structures at an inclination of at least 1 to 2 percent to prevent the ingress of surface runoff.

7 Lateral Earth Pressure Against Subsurface Walls

The subsurface walls should be backfilled with free draining material, such as OPSS Granular B, Type II and equipped with a perimeter drainage system to prevent the buildup of hydrostatic pressure behind the wall. The walls will be subjected to lateral static and dynamic (seismic) earth forces.

For design purposes, the lateral static earth thrust against the subsurface walls may be computed from the following equations: (ii) and (iii) given below. These equations assume that the finished grade adjacent to the subsurface walls will be level.

$P = K_0 H (q + ½ \gamma H)$ -------------------- (ii)

where P = lateral earth thrust acting on the subsurface wall; kN/m

K_0 = lateral earth pressure coefficient for ‘at rest’ condition for Granular B Type II backfill material = 0.5

γ = unit weight of free draining granular backfill; Granular B Type II = 22 kN/m^3

H = Height of backfill adjacent to foundation wall, m

q = surcharge load, kPa

The grade adjacent to the south subsurface walls of some of the proposed blocks will support the slope. In such cases, the subsurface walls would be subjected to passive earth pressure, which may be computed from the expression given above by replacing the coefficient of lateral earth pressure at rest (K_0), with coefficient of passive earth pressure (K_p).

Where K_p = 3.0 for level ground;
= 3.3 for 12-degree slope
= 3.85 for 30-degree slope

The lateral seismic thrust may be computed from the equation given below:

$\Delta P_E = 0.3 \gamma H^2$ -------------------- (iii)

where ΔP_E = resultant thrust due to seismic activity; kN/m

γ = unit weight of free draining granular backfill; Granular B Type II = 22 kN/m^3

H = height of backfill adjacent to foundation wall, (m)

The ΔP_E value does not take into account the surcharge load. The resultant load should be assumed to act at 0.6 H from the bottom of the wall.

8 Grade Raise Restrictions

A review of the grading plans, Drawing 106011-GR-WT1 and 106011-GR-WT2 indicates that the following grade raises will be undertaken in the vicinity of the proposed structures.

Table 6: Proposed Grade Raises Adjacent to Structures

Block No.	Proposed Grade Raises (m)			
	North Side	South Side	East Side	West Side
1	2.35	-0.5 to 1.35	1.35 to 2.35	-0.5 to 2.35
2	0.8 to 1.3	1.35	0.8 to 1.35	1.35 to 1.5
3	-0.5 to -1.5	0 to -3.0	-1.5 to -3.0	0 to -.05
4	-1.0	-3.6	-1.0 to -3.6	-1 to -3.5
5	-0.35 to 2.0	-2.0 to 3.3	2.0 to 3.3	-2.0 to -0.35

NOTE: negative numbers indicates cut areas.

It has been recommended that all the structures should be founded either on footings set on bedrock in areas where the bedrock is present at a shallow depth or on piles driven to practical refusal on bedrock. Grade raise in the vicinity of pile foundations would result in negative skin friction on the piles due to consolidation of the existing fill and silty clay because of the grade raise. The down drag forces that the piles will be subjected to should be subtracted from the factored ULS geotechnical resistance to arrive at the allowable load for pile. If this is undertaken, the proposed grades are considered feasible.

9 Site Classification for Seismic Site Response and Liquefaction Potential of On-Site Soils

9.1 Site Classification

The geotechnical conditions at the site consist of existing fill, which is predominantly cohesive (silty clay) underlain by silty clay deposit, which extends to the bedrock. The site classification was established by completing the average N60 value to 30 m depth. The N60 values established from the standard penetration tests in the fill and silty clay overburden were used in the computations. The N value of the bedrock was assumed to be 100. On this basis, the following site classifications were established for the different blocks.

Table 7: Site Classifications for Blocks 1 to 5

Block No.	Site Classification for Seismic Site Response
1	C
2	D
3	D
4	D
5	D

9.2 Liquefaction Potential of On-site Soils

Atterberg Limit Tests were performed on three samples of the silty clay obtained from the site. The results of the tests were plotted on the Bray et al (2004) criteria chart for liquefaction assessment of fine grained soils (Figure 28). A review of this figure indicates that the on-site soils are not susceptible to liquefaction during a seismic event.

10 Slope Stability Analyses

As indicated previously, construction of the proposed Blocks 1 to 5 would be undertaken by partially excavating the toe of the existing escarpment slope. Slope stability analyses were undertaken to determine:

1.) Existing stability of the slope assuming the slope is in a drained condition.

2.) Stability of the slope on completion of excavation (undrained condition).

3.) Stability of the slope on completion of construction assuming a building load of 100 kPa and undrained shear strength parameters.

4.) Long-term stability of the slope subsequent to construction assuming drained conditions.

5.) Stability of the slope under seismic conditions subsequent to construction using undrained shear strength parameters.

One cross-section of the slope for each block was analyzed as follows:

Block Number	Section Identification
1	A-A
2	B-B
3	C-C
4	D-D
5	E-E

The locations of the cross-sections have been plotted on the Borehole Location Plan, Figure 2. The cross-sections of the slope analyzed were surveyed by Novatech Engineers, Planners and Landscape Architects and supplemented with site contours.

The stability of the slope was analyzed using Morgenstern and Price Method, GeoStudio 2012, Version 8.13.1.9253 (May 2013) computerized system. The soil stratigraphy of the various cross-sections was obtained from the boreholes drilled and test pits excavated for the current project supplemented by boreholes and test pits undertaken at the site during previous geotechnical investigations.

The engineering properties of the various soils excavated in the slope used in slope stability analyses have been listed on Table 8. These properties are based on limited lab testing, previous experience in the area and literature review.

Table 8: Engineering Properties of Soils Used in Slope Stability Analyses

Soil Type	Unit Weight (kN/m^3)	Total Strength Parameters		Effective Stress Parameters	
		Undrained Shear Strength (kPa)	Angle of Internal Friction (Ø) Degrees	Effective Cohesion C' (kPa)	Effective Angle of Internal Friction (Ø′) Degrees
Existing Fill (low strength)	18.5	50	0	5	28
Existing Fill (high strength)	18.5	50	0	7	31
New Fill	19.0	--	0	0	33
Top Soil	15.0	--	0	0	10
Silty Clay (low strength)	18.8	75	0	7	33
Silty Clay (high strength)	18.8	75	0	9	33
Bedrock	Impenetrable Layer				
Retaining Wall	Impenetrable Layer				

The results of the analyses have been summarized in Table 9.

Table 9: Results of Slope Stability Analyses

Block Number	Section	Computed Factor of Safety for Condition				Figure Numbers
		Existing	End of Construction	Long Term	Seismic	
1	A-A	1.74	6.64	2.11	3.27	29 to 32
3	B-B	1.27	5.84	1.90	3.59	33 to 36
3	C-C	1.70	3.56	1.30 (2H:1V)	1.99	37 to 40
				1.44 (2.5H:1V)		49
				1.52 (2.75H:1V)		50
4	D-D	1.46	8.99	1.52	6.71	41 to 44
5	E-E	1.27	4.76	1.41 (2H:1V)	2.82	45 to 48
				1.84 (2.5H:1V)		51

A review of Table 9 indicates that the factor of safety of the existing slope varies from 1.27 to 1.74 indicating that the slope is currently stable.

Factors of safety at end of construction varied from 3.56 to 8.99.

Post-construction factors of safety in the long term varied from 1.30 to 2.11 indicating that the slopes will be stable post-construction. However, it is noted that the post construction factor of safety of the slope is 1.3 for Section C-C and 1.41 for Section E-E. These factors of safety are less than 1.5, which is the current industry standard. It is noted that in both cases, these lower factors of safety are due to the proposed cuts at a slope of 2H:1V. Section C-C was re-analyzed cutting the slope back at 2.5H:1V (Figure 49 and at 2.75H:1V (Figure 50). These figures indicate that a factor of safety of 1.52 would be available if the slope is cut back at 2.75H:1V. Section E-E was re-analyzed cutting the slope back at 2.5H:1V (Figure 51). A factor of safety of 1.83 was obtained. It is therefore recommended that the slope at Section C-C should be cut back at 2.7H:1V and that at Section E-E at 2.5H:1V so that the maximum requisite factors of safety will be available.

Factors of safety during a seismic event would vary from 1.9 to 6.71, which comply with the current industry practice.

The slope stability analyses are based on the assumption that the escarpment slope comprises of silty clay. It was not possible to drill boreholes at the crest of the slope or along the slope because of access difficulties. It is recommended that once the site has been cleared of vegetation, representative boreholes or test piles should be drilled/excavated at the crest and along the slope to verity the bedrock profile and the soil conditions assumed in the analyses.

It is noted that the slope stability analysis did not assess the stability of the retaining walls proposed. The global stability of the retaining walls should be checked at the design stage.

11 Retaining Walls

As part of site grading, a portion of the existing slope will be cut back. For this purpose, retaining walls are to be constructed south of Blocks 1 to 3 and Block 4. These retaining walls will support the upper portion of the slope to the escarpment. As such, they will be subjected to a higher lateral earth pressure than normal. The passive resistance to stability of each retaining wall will be provided by the friction between the retaining wall footing and the founding soil/bedrock and the passive resistance of the backfill in front of the retaining wall. The passive resistance of the soil in front of the wall to 2.1 m depth should be ignored due to freeze and thaw action. However, the weight of the soil in front of the retaining wall up to 2.1 m depth may be considered to provide passive support.

The proposed retaining walls may be founded on natural undisturbed soil or on bedrock. A SLS bearing pressure of 100 kPa may be used when founding in the overburden. The corresponding factored geotechnical resistance at ULS will be 150 kPa. The settlements of the retaining walls are expected to be within the normally tolerated limits of 25 mm total and 19 mm differential movements. A ULS bearing pressure of 1MPa may be used when founding on the bedrock. Settlement of the retaining walls founded on the bedrock are expected to be negigibile.

The retaining wall will be subjected to lateral static earth as well as lateral dynamic earth forces during a seismic event.

Seismic loading will result in an increase in active lateral earth pressure and a decrease in passive lateral earth pressure on the wall. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio 0.28 applicable for the Ottawa area.

The dynamic pressure distribution is an inverted triangle with maximum pressure at the top of the wall and a minimum at the bottom of the wall. Therefore, the resultant of earthquake pressure on the retaining wall is assumed to be applied at a height of 0.6 H above the base of the wall where H is the height the wall. The total active pressure distribution can be separated into static component and dynamic components and may be determined as follows (Mononobe and Matsuo, 1929):

$$\sigma_{AE}(z) = k_a \gamma z + (K_{AE} - k_a)\gamma(H - z)$$

Where $\sigma_{AE}(z)$: the total combined active earth pressure (dynamic and static), (kPa).

z : depth below the top of the retaining wall.

K_a : static active earth pressure coefficient

K_{AE} : combined (static and dynamic) active earth pressure coefficient.

γ : unit weight of the backfill soil (KN/m^3).

H : Total height of the wall (m).

The total passive pressure in front of the wall can be similarly separated into static and dynamic components as follows:

$$\sigma_{PE} = k_p \gamma z + (K_{PE} - k_p)\gamma(h - z)$$

Where σ_{PE}: the total combined passive earth pressure (dynamic and static), (kPa).

z : depth below the ground surface in front of the wall.

K_p : static passive earth pressure coefficient

K_{PE} : combined (static and dynamic) passive earth pressure coefficient.

γ : unit weight of the backfill soil (KN/m^3).

h : depth of embedment of the wall (m).

The above earth pressure expression does not take into account any surcharge applied on the wall or on the backfill soil. It also assumes that the backfill against the subsurface walls will be free-draining granular material and drains will be presented at the footing level to prevent building up of hydrostatic pressure against the subsurface walls. The backfill should be compacted to 95 percent SPMDD. The method of compaction of the engineered fill (Granular B Type II) is not known. However, it is recommended that a minimum compaction surcharge of 20 kPa should be taken into account when designing the retaining wall. The lateral earth pressure parameters of the backfill material are given in Table 10.

Table 10: Lateral Earth Pressure Parameters

Soil layer				**Granular B Type II**
Wet Unit Weight of Soil (γ) , kN/m^3				22
Angle of Internal Friction (ϕ') (°)				33°
Coefficient of Earth Pressure at Rest (k_0)				0.46
Retained Slope Angle	**Static (k_a)**	**Static (k_p) ****	**Dynamic (k_a)**	**Dynamic ** (k_p)**
30°	0.62	3.4	2.6	- 0.6
12°	0.34	3.4	0.40	- 0.6
0°	0.30	3.4	0.26	- 0.6

** : Peak Ground Acceleration in Ottawa, a=0.28.*

***: Ground surface in front of the wall is assumed horizontal.*

It is imperative that once the design of the retaining walls has been completed, it should be checked for global slope stability. A factor of safety of 1.5 should be incorporated in the design for static slope stability analysis and 1.1 for seismic slope stability analysis.

The final design of the retaining walls should be reviewed by this office.

12 Excavations

Excavations for construction of proposed structures and installation of any underground services at the site are expected to extend to a maximum depth of 6 m to 7 m below the existing ground surface. These excavations will extend through the fill and terminate in the clay. The fill and the clay above and below the groundwater table have been classified as Type 3 soil in accordance with the requirements of Occupation Health and Safety Act, Ontario Regulations 213/91. The excavations at the site will be predominantly above the groundwater table. These excavations may be undertaken as open cut provided they are cut back at 45 degrees. The exception to this is that the lowermost 1.2 m of excavations which terminated in the natural clay which may be cut vertically. Excavations that terminate in the clay or in the granular soils above the groundwater table are not expected to experience a 'base-heave' type of failure of the excavation.

It is noted that the excavations on the south side of the proposed blocks will cut into the existing slope. A slope stability analysis of the south excavated slopes of the blocks, except Block 2, was undertaken to determine if these slopes will be stable when cut back. Excavation for Block 2 will not encroach on the existing slope and therefore, a stability analysis was not necessary. The results of the stability analysis are given on Table 11.

Table 11: Slope Stability Analysis of South Excavation Slope During Construction

Block Number	Section	Condition	Factor of Safety	Figure Number
1	A-A	On completion of excavation	3.59	52
3	C-C	On completion of excavation	2.36	53
4	D-D	On completion of excavation	2.19	54
5	E-E	On completion of excavation	1.95	55

A review of Table 11 indicates that the factors of safety against slope failure vary from 1.95 to 3.59. Therefore, these slopes are expected to be stable during construction. However, it is recommended that these slopes should be monitored during construction so that remedial measures can be implemented in case of evidence of any movement.

Seepage of the surface and subsurface water into the excavations is anticipated. However, it should be possible to collect any water entering the excavations in perimeter ditches and to remove it by pumping from sumps. Although this investigation has estimated the groundwater levels at the time of the field work, and commented on dewatering and general construction problems, conditions may be present which are difficult to establish from standard boring techniques and which may affect the type and nature of dewatering procedures used by the contractor in practice. These conditions include local and seasonal fluctuations in the groundwater table, erratic changes in the soil profile, thin layers of soil with large or small permeabilities compared with the soil mass, etc. Only carefully controlled tests using pumped wells and observation wells will yield the quantitative data on groundwater volumes and pressures that are necessary to adequately engineer construction dewatering systems.

Many geologic materials deteriorate rapidly upon exposure to meteorological elements. Unless otherwise specifically indicated in this report, walls and floors of excavations must be protected from moisture, desiccation, and frost action throughout the course of construction.

The clay at the site is susceptible to disturbance due to the movement of construction equipment, and personnel on its surface. It is therefore recommended that the excavation at the site should be undertaken by equipment which does not travel on the excavated surface e.g. a gradall or mechanical shovel. It is anticipated that temporary granular roads would be required to gain access to the site by construction equipment.

13 Backfilling Requirements and Suitability of On-site Soils for Backfilling Purposes

The backfill against subsurface walls and in footing and service trenches inside the structures should consist of free draining material preferably conforming to OPSS for Granular B, Type II. It should be compacted to 95 percent of SPMDD.

The backfill in service trenches outside the buildings should be compactable, i.e. free of organics and debris and with natural moisture content which is within 2 percent of the optimum moisture content. It also should be compacted to 95 percent SPMDD.

The material to be excavated during construction of the footings and installation of services is expected to be existing fill. The existing fill primarily comprises of clay with occasional cobbles and boulders. The exception to this may be close to the existing escarpment where it may comprise of the scree material. The scree material as well as the heterogenous fill containing boulders and cobbles are not considered suitable for use as backfill against subsurface walls and selected portion may be used as sub grade fill for the access roads pending further evaluation during construction and excavation. This material however may be used foe general grading purposes in the landscaped areas.

It is anticipated that the majority of the fill required to backfill the blocks would have to be imported and should comply to the requirement listed below;

- Engineered fill under floor slab and behind subsurface walls, OPSS 1010 Granular B, Type II placed and compacted to 98 percent of the Standard Proctor Maximum Dry Density (SPMDD) in the interior of the buildings and to 95 % SPMDD in the exterior of the buildings;
- Trench backfill and subgrade fill in parking area and access roadways – OPSS 1010 Select Subgrade Material (SSM), placed in 300 mm thick lifts and each lift compacted to 95 percent of the SPMDD. To minimize settlement of the pavement structure over services trenches, the trench backfill material within the frost zone should match the existing material along the trench walls to minimize differential frost heaving of the subgrade soil, provided this material is compactible. Otherwise, frost tapers may be required.

If the backfill for the service trenches will consist of granular fill, clay seals should be installed in the service trenches at select intervals as per City of Ottawa Drawing No. S8. The seals should be 1m wide, extend over the entire trench width and from the bottom of the trench to the underside of the pavement structure. The clay should be compacted to 95% SPMDD. The purpose of the clay seals is to minimize the permanent lowering of the groundwater level.

14 Access Roads and Driveways

Pavement structure thicknesses required for the access roads and driveways to be used by heavy traffic and light automobile traffic were computed. The pavement structures are shown on Table 12. It was designed using average annual daily traffic of between 2000 and 3000 vehicles per day. The thicknesses are based upon an estimate of the subgrade soil properties determined from visual examination and textural classification of the soil samples and functional design life of 15 to 18 years. The proposed functional design life represents the number of years to the first rehabilitation, assuming regular maintenance is carried out.

Table 12: Recommended Pavement Structure Thicknesses

Pavement Layer	**Compaction Requirements**	**Driveways**	**Subdivision Roads**	
			Sand Subgrade	**Clay Subgrade**
Asphaltic Concrete – PG 58-34	92-97% Maximum Relative Density	65 mm HL3	40 mm – SP12.5-CATB 50 mm - SP19.0-CATB	40 mm – SP12.5 50 mm – SP19
OPSS Granular A Base (crushed limestone)	100% SPMDD*	150 mm	150 mm	150 mm
OPSS Granular B Sub-Base, Type II	100% SPMDD*	300 mm	300 mm	450 mm

* *Denotes standard Proctor maximum dry density (SPMDD), ASTM-D698.*
Any subgrade fill must be compacted to 98% SPMDD for at least the upper 300 mm.

Construction procedures for the pavement structure are discussed below.

After all the underground services have been installed, backfilled and satisfactorily compacted, the entire road should be excavated to the subgrade level. The subgrade should be crowned with a centre edge to edge slope of at least 2 percent. It should then be proof rolled with a heavy roller. Any soft areas which become evident should be sub-excavated and replaced with approved native fill or free draining granular material. All subgrade fill should be placed in maximum 300 mm lifts and compacted to 98 percent of SPMDD. In-place density tests should be performed at regular intervals to ensure that the specified degree of compaction is being achieved.

1. It is stressed that the overall satisfactory performance of the recommended pavement structures is contingent upon the provisions of good drainage. Subsurface drains should be provided on both sides of the access roads. In parking areas, the drains should be located at low points and should be continuous between catch basins. The drains should be located with their invert approximately 300 mm below the subgrade level and may consist of 150 mm diameter perforated pipe set on 100 mm bed of 19 mm clear stone and covered top and sides with 150 mm of 19 mm stone. The stone should be surrounded with a suitable filter cloth, such as Terrafix 270 R or equivalent. The remainder of the trench should be backfilled with well compacted, free draining granular material.
2. To minimize the problems of differential movement between the pavement and catchbasins/manholes due to frost action, the backfill around the structures should consist of free-draining granular material preferably conforming to OPSS Granular B Type II. Weep holes should

be provided in the catchbasins and screened with filter cloth to facilitate drainage of any water which may accumulate in the granular fill around the catchbasins/manholes.

3. Relatively weaker subgrade may develop over service trenches at subgrade level due to soil disturbance. If this is the case, it is recommended that additional 150 mm of granular sub-base Granular B should be provided in these areas.
4. The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum cross fall of 2 percent) to provide effective surface drainage towards catch basins. Surface water should not be allowed to pond adjacent to the outside edges of paved areas.
5. The granular materials used for pavement construction should conform to OPSS for Granular A and Granular B, Type II and should be compacted to 100 percent SPMDD. The asphaltic concrete used and its placement should meet OPSS 1151 requirements. It should be placed and compacted to OPSS 310 and 313 requirements.

It is recommended that EXP be retained to review the final pavement structure design and drainage plans prior to construction to ensure that they are consistent with the recommendations of this report.

15 Subsurface Concrete Requirements

Chemical tests limited to pH and sulphate tests were performed on five selected soil samples. The results are given on Table 13. The testing was performed by AGAT Laboratories, Mississauga, Ontario.

Table 13: Results of Chemical Tests on Soil Samples

Parameter	**Borehole ID and Depth (m)**						**Threshold Values**
	Test Pit B **2.5 – 2.8 m**	**BH-G** **4.6 – 5.2 m**	**BH-H** **3.8 – 4.4 m**	**BH-H** **9.2 – 9.8 m**	**BH-L** **7.6 – 8.2 m**	**BH-N** **3.8 – 4.4 m**	
pH	7.59	7.18	7.64	7.82	7.32	7.33	<5
Sulphates (%)	0.0108	0.0121	0.0058	0.0184	0.0024	0.0193	<0.1
Chlorides (%)	0.0012	0.0077	0.0011	0.0079	0.0033	0.0073	>0.04
Electrical Resistivity Ohm/cm	3448	3095	3584	2304	4651	2512	<700 ohm.cm High corrosion potential

The test results indicate the soil contains a sulphate content of less than 0.1 percent, and a chloride content of less than 0.04 percent. This concentration of sulphates and chlorides in the soil would have a negligible potential of attack on subsurface concrete. Therefore, General Use (GU) Portland cement may be used in the subsurface concrete at this site. The concrete for the site should be designed in accordance with the requirements of CSA A23.1-14.

The resistivity results indicate that the subsurface soil is mildly corrosive to buried steel. It is noted that Zonecode 3-5 guidelines indicate that the thickness loss from corrosion (mm) of piles in soil above and below the groundwater table installed in undisturbed natural soil should be taken as 0.6 mm in 50 years. This value should be increased to 1.2 mm in non-compacted, non-compressive fill for 50 years. It is therefore recommended that a sacrificial steel thickness of 1.2 mm should be used in the design of piles for a life expectancy of 50 years of the structures.

16 Tree Planting

The clay in the Ottawa area is prone to shrinkage on drying. This process is largely not reversible. Therefore, settlement and cracking of the structures can result if trees are planted too close to the residences. During dry seasons, the tree roots draw moisture from the clay thereby resulting in the clay drying and shrinking.

City of Ottawa guidelines indicate that fast growing, high-water demand trees must not be plant closer to a building than a distance equal to their height at maturity. Only one of the small-sized trees listed below can be placed a minimum distance of 7.5 m away from any buildings, including when planting along road allowances. In addition, new planted trees must be a minimum of 2.5 m from the curb and have a small-sized canopy at maturity to allow sufficient space for snow and ice control purposes.

Table 14: List of Trees that can be Planted Close to Structures

Species	**Water Demand**
Amur Maple (Acer ginnala)	Moderate
Serviceberry (Amelanchier canadensis)	Low
Crabapple (Malus spp.)	Moderate
Japanese Lilac (Syringa reticulate)	Moderate
Green Colorado Spruce or any conifer species (Picea pungens)	Low

For further information, an arborist should be consulted.

17 General Comments

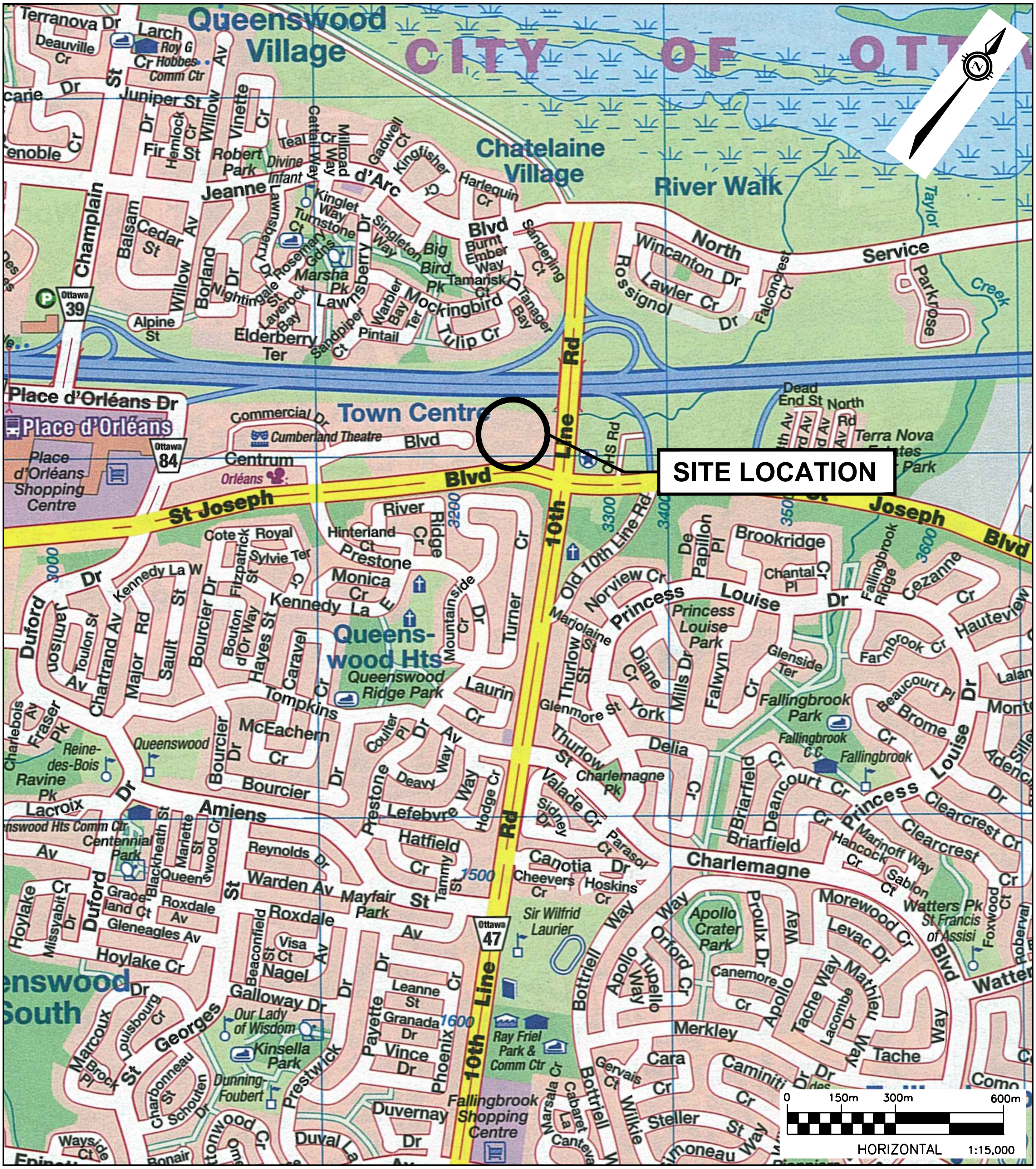
The investigation has indicated that the site topography and geotechnical conditions vary considerably across the site. It is therefore feasible that the depth of fill encountered between boreholes may vary considerably from the depths established at borehole locations.

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 the 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.

The information contained in this report is not intended to reflect on environmental aspects of the soils. Should specific information be required, including for example, the presence of pollutants, contaminants or other hazards in the soil, additional testing may be required.

We trust that the information contained in this report will be satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

Figures



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scale 1:15,000	CLIENT: DCR Phoenix Group of Companies	project no. OTT-00241432-A0
date AUGUST 2017	TITLE: Site Location Plan. Hill Side Walk-Up Condos St-Joseph Boulevard & Tenth Line Road. Ottawa, ON	FIG 1
drawn by J.R.		

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LEGEND

BH–A	PROPOSED BOREHOLE LOCATION
TP–D	PROPOSED TEST PIT LOCATION
BH17–11	BOREHOLE NUMBER AND LOCATION FROM PREVIOUS INVESTIGATIONS
TP02–1	TEST PIT NUMBER AND LOCATION FROM PREVIOUS INVESTIGATIONS
T @ ELEV. 65.5 m	TERMINATION ELEVATION
R @ ELEV. 65.5 m	REFUSAL TO AUGER OR CONE ELEVATION

NOTES :

1. THE BOUNDARIES AND SOIL TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL SAMPLES AND ROCK WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. TOPSOIL QUANTITIES SHOULD NOT BE ESTABLISHED FROM THE INFORMATION PROVIDED AT THE BOREHOLE LOCATIONS.
4. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING□S□OR FLOOR SLABS OR PARKING LOT□S□GRADES.
5. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
6. BASE PLAN OBTAINED FROM MORRIS MELAMED ARCHITECT, PRELIMINARY OVERALL SITE PLAN, DRAWING A00, REV. 15, DATED DEC. 10, 2012

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Last Plotted:11/3/2017 2:40:26 PM Plotted by: GhadbanM
Pen Table: ----
References: 218474-A0 FIG 1 :moved to WORLD:BH rev 4 - 2016-05.dwg

NOTES

THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, DETERMINE THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES AND ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

NO.	REVISION DESCRIPTION	DATE	BY	APPD
1	ISSUED FOR REVIEW	0/11/17	MZG	IT

SCALE

HORZ 1:300

0 3m 6m 12m

HORIZONTAL 1:300

DESIGNED BY

REVIEWED BY

CLIENT

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• BUILDINGS • EARTH & ENVIRONMENT • ENERGY •
• INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

BASEPLAN

DESIGN
S. AGGARWAL

CHECKED
S. AGGARWAL

CAD
M. NUGENT

PROJ. MAN
I. TAKI

APPROVED
I. TAKI

PROJECT

HILLSIDE VISTA
WALK–UP CONDOS
HIGHWAY 174 & 10TH LINE ROAD, ORLEANS, ON

TITLE

BOREHOLE
LOCATION PLAN

PROJ NO.
OTT–00241432–A0

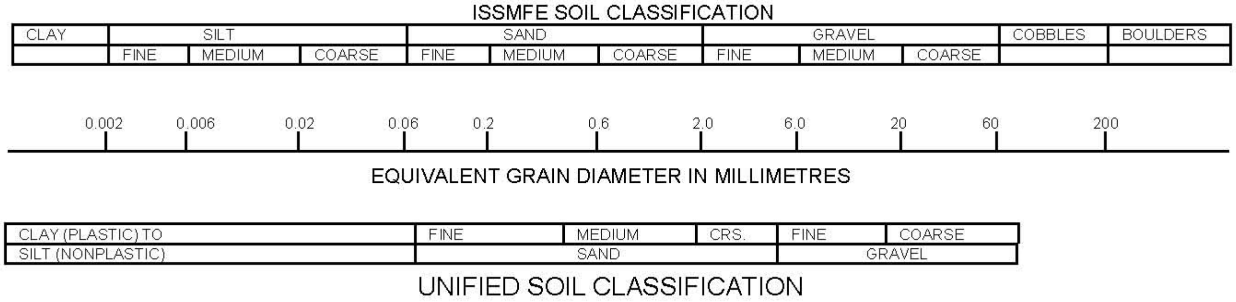
SURVEY

DATE
AUGUST 31, 2017

DRAWING NO.
FIG 2

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** Services Inc. 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.



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 Test Pit TP-A

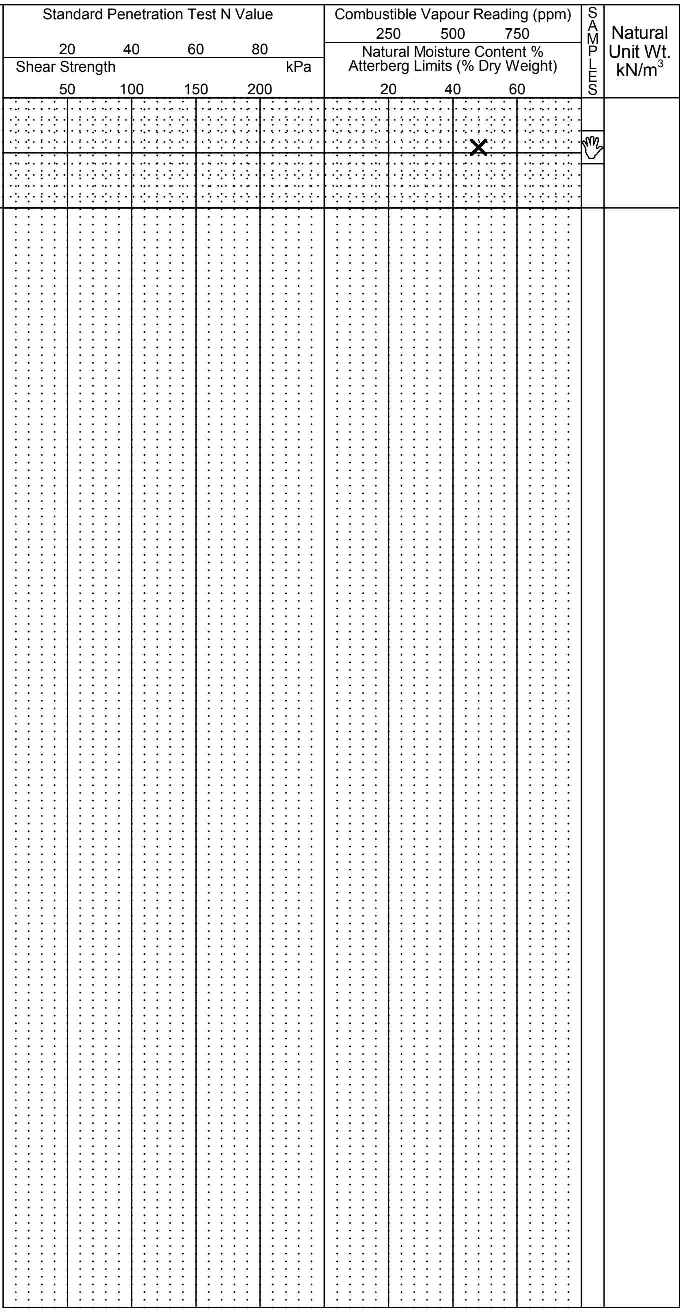
Project No: OTT-00241432-A0
Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos
Location: St-Joseph Blvd and Tenth Line Road, City of Ottawa, Ontario

Figure No. 3
Page. 1 of 1

Date Drilled: 'August 17, 2017
Drill Type: Excavator (30 ton)
Datum: Geodetic
Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	——	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth
			67.53	0
		TOPSOIL ~250 mm	67.3	
		FILL Silty clay, some cobbles and boulders	66.8	
		WEATHERED BEDROCK	66.5	1
		Test Pit Terminated at 1.0 m Depth Upon Refusal Bedrock Surface		



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Test Pit backfilled and compacted with excavator bucket upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	1.0

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Test Pit TP-B

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Road, City of Ottawa, Ontario

Figure No. 4

Page. 1 of 1

Date Drilled: 'August 17, 2017

Drill Type: Excavator (30 ton)

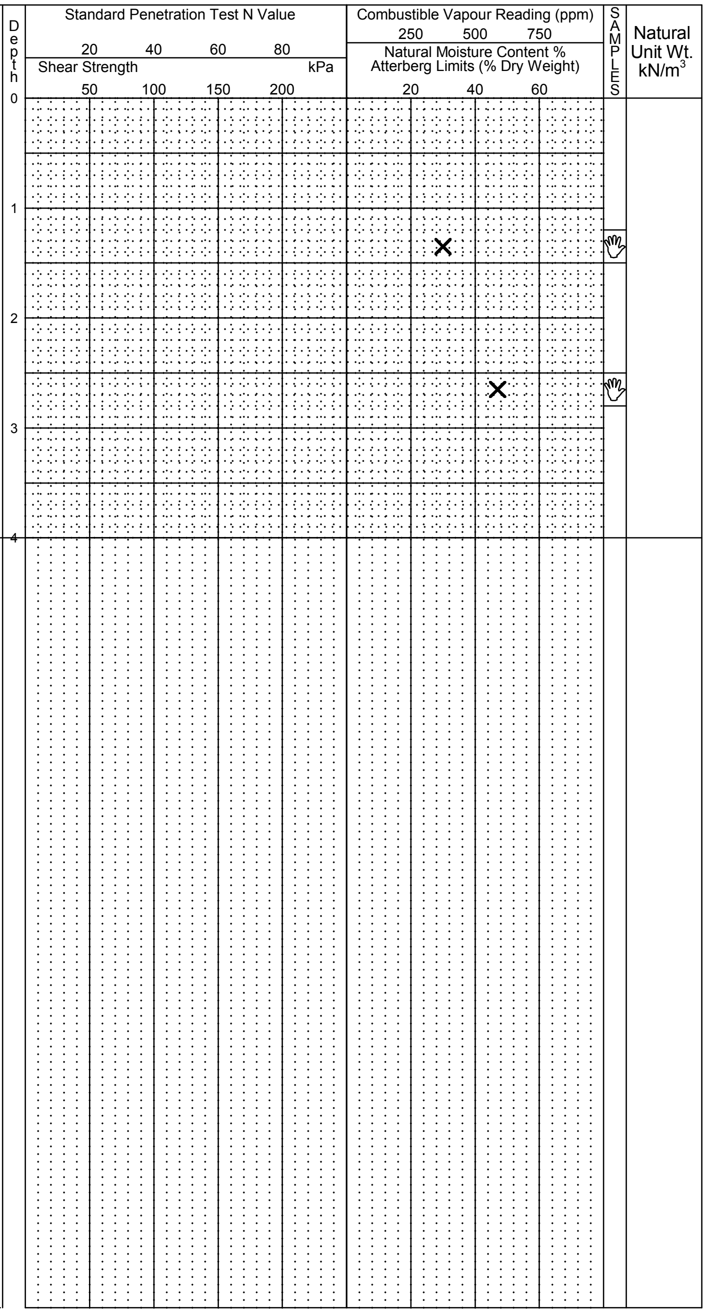
Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m
	▨	**FILL** Silty clay mixed with sand and gravel, frequent cobbles and boulders, grey-brown, moist	67.45
	▨	**FILL** Silty clay, trace sand and gravel, grey, moist to wet	65.3
		TOPSOIL Compressed	63.8 63.5

Test Pit Terminated at 4.0 m Depth On Bedrock Surface



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Test Pit backfilled and compacted with excavator bucket upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	4.0

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Test Pit TP-C

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Road, City of Ottawa, Ontario

Figure No. 5

Page. 1 of 1

Date Drilled: 'August 17, 2017

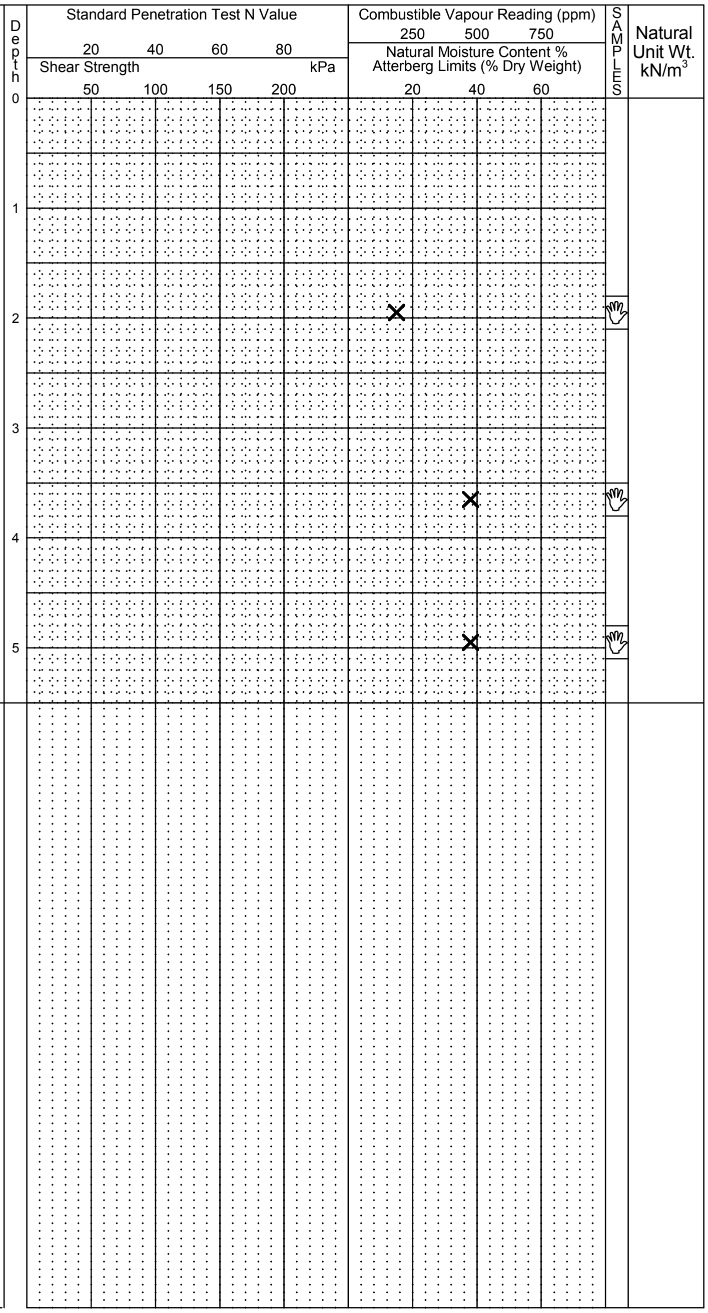
Drill Type: Excavator (30 ton)

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth
	▩	**FILL** Silty clay mixed with sand and gravel, frequent cobbles and boulders, grey-brown, moist	70.41	0
	▨	**SILTY CLAY (DESICCATED CRUST)** Silty clay, trace sand, weathered crust, brown, moist, (firm)	66.0	
		Test Pit Terminated at 5.5 m Depth Upon Refusal On Bedrock Surface	64.9	



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Test Pit backfilled and compacted with excavator bucket upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	5.5

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Test Pit TP-D

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Road, City of Ottawa, Ontario

Figure No. 6

Page. 1 of 1

Date Drilled: 'August 17, 2017

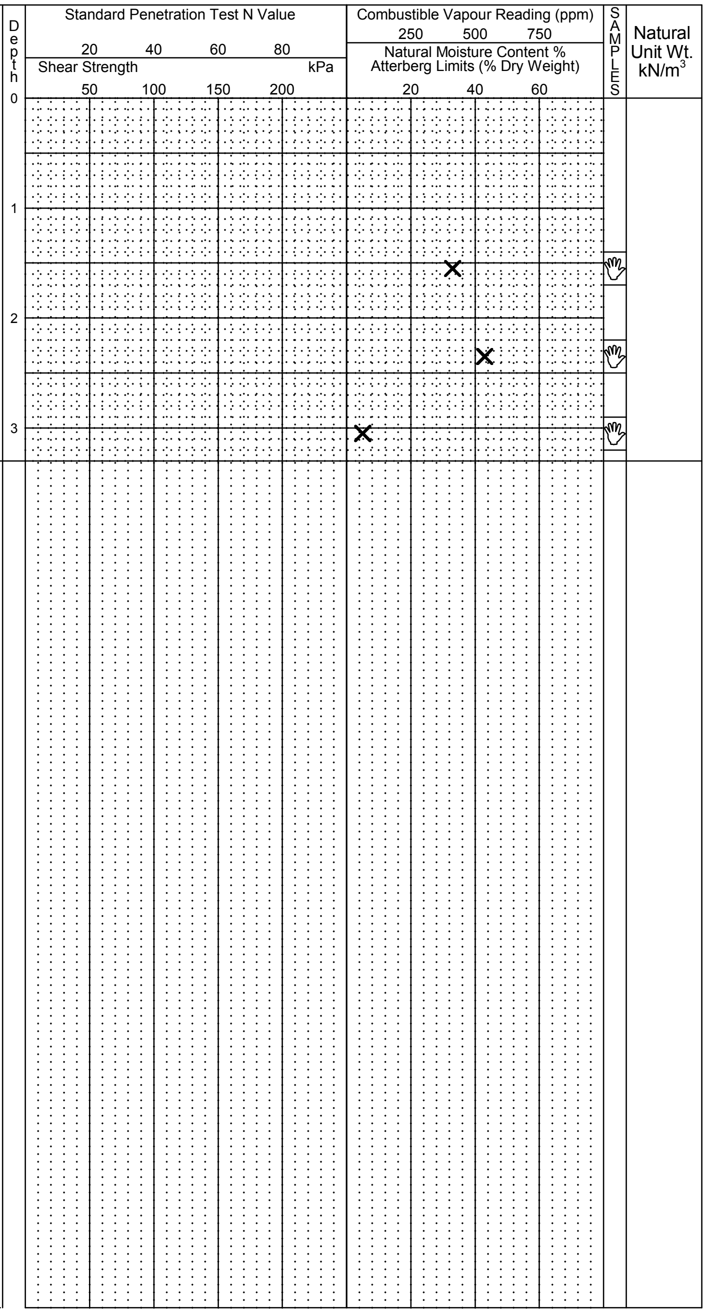
Drill Type: Excavator (30 ton)

Datum: Geodetic

Logged by: M.L.   Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	⊢—⊖
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth
	▨	**FILL** Silty clay mixed with sand and gravel, occasional cobbles and boulders, grey-brown, moist	67.04	0
	▨	**FILL** Silty clay, trace sand, grey, moist to wet, (loose)	66.0	1
				2
		GLACIAL TILL Gravelly clay, some sand and silt, brown, moist	64.1	3
		Test Pit Terminated at 3.3 m Depth Upon Refusal On Bedrock Surface	63.7	



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Test Pit backfilled and compacted with excavator bucket upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	3.3

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Test Pit TP-E

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Road, City of Ottawa, Ontario

Figure No. 7

Page. 1 of 1

Date Drilled: 'August 17, 2017

Drill Type: Excavator (30 ton)

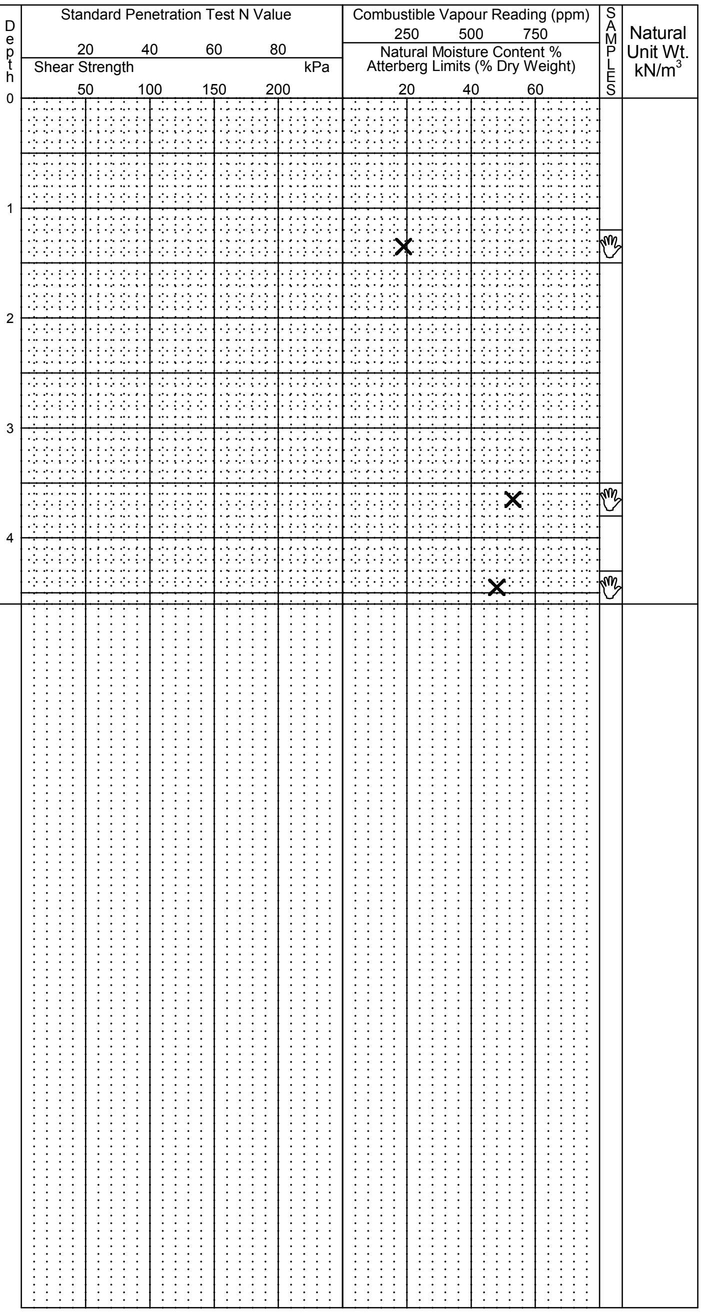
Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth
	▩	**FILL** Silty clay mixed with sand and gravel, occasional cobbles and boulders, grey-brown, moist	69.4	0
	▩	**FILL** Silty clay, trace sand, grey, moist to wet, (loose)	66.6	
	▨	**SILTY CLAY** Silty clay, trace sand, weathered crust, brown, moist	65.1	
			64.8	

Test Pit Terminated at 4.6 m Depth Upon Refusal on Bedrock Surface



NOTES:

1. Borehole data requires interpretation by exp. before use by others
2. Test Pit backfilled and compacted with excavator bucket upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	4.6

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-F

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario

Figure No. 8

Page. 1 of 1

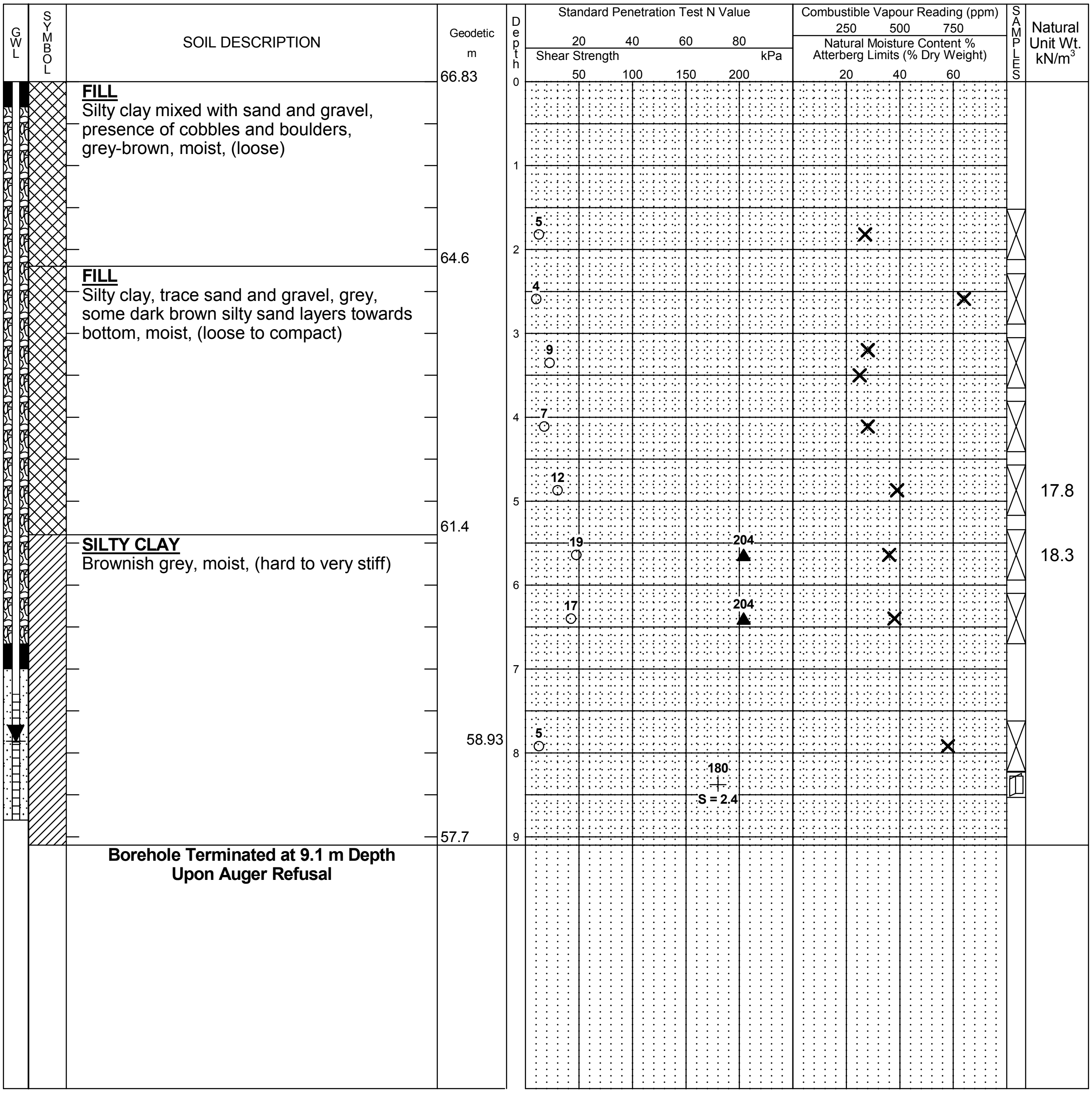
Date Drilled: 'August 23, 2017

Drill Type: CME-75 Trackmount

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



Geodetic m	Soil Description	Natural Unit Wt. kN/m³
66.83	**FILL** Silty clay mixed with sand and gravel, presence of cobbles and boulders, grey-brown, moist, (loose)	
64.6	**FILL** Silty clay, trace sand and gravel, grey, some dark brown silty sand layers towards bottom, moist, (loose to compact)	17.8
61.4	**SILTY CLAY** Brownish grey, moist, (hard to very stiff)	18.3
58.93		
57.7	**Borehole Terminated at 9.1 m Depth Upon Auger Refusal**	

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. 19 mm standpipe was installed in teh borehole upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	9.0
33 days	7.9	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-G

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario

Figure No. 9

Page. 1 of 1

Date Drilled: 'August 24, 2017

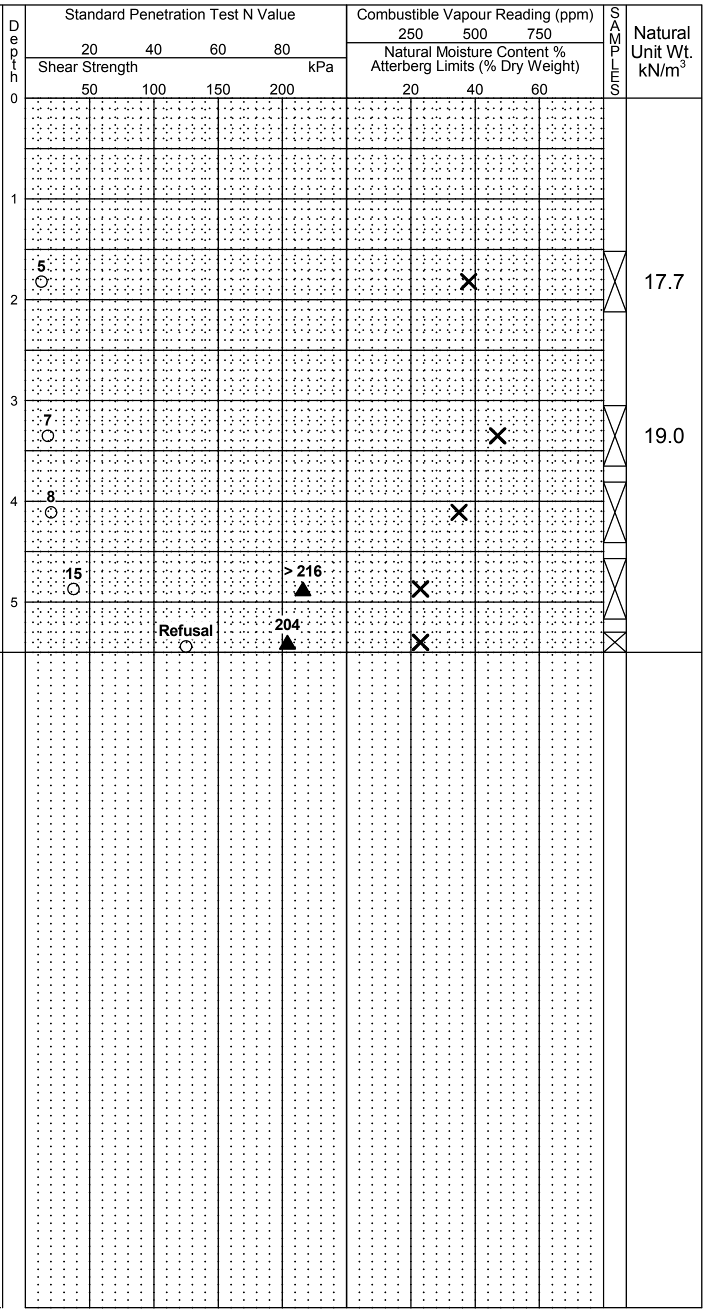
Drill Type: CME-75 Trackmount

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth
	▩	**FILL** Silty clay, trace sand and gravel, grey, moist, (loose)	67.43	0
	▩	**FILL** Silty sand, trace to some clay, some organics (grass), wood pieces, dark brown, moist, (loose)	64.0	
	▨	**SILTY CLAY** Brownish grey, moist, (hard)	62.8	
			61.9	
		Borehole Terminated at 5.5 m Depth Upon Auger Refusal		



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole backfilled with cuttings upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	5.5

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-H

Project No: OTT-00241432-A0

Figure No. 10

Page. 1 of 1

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario

Date Drilled: 'August 23, 2017

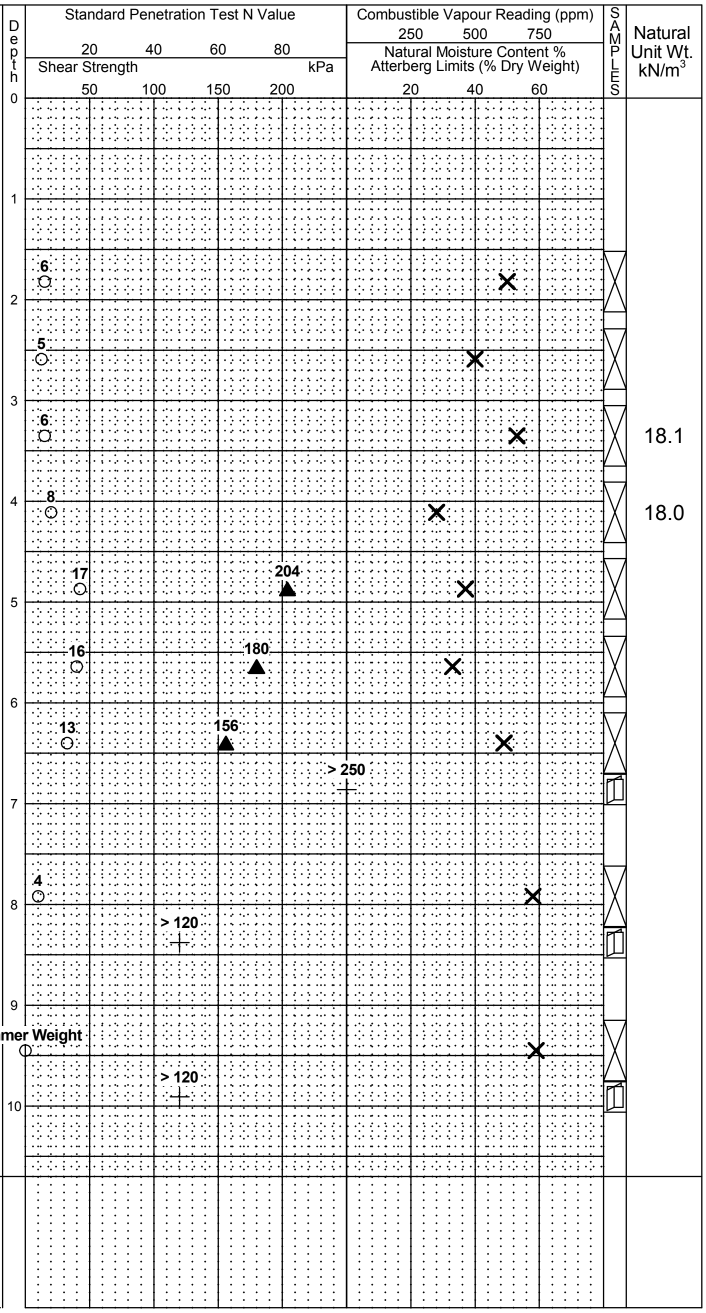
Drill Type: CME-75 Trackmount

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	▥	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	——	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m
	▨	**FILL** Silty clay mixed with sand and gravel, presence of cobbles, grey-brown, moist, (loose)	66.02
	▨	**FILL** Silty clay, trace sand, grey, moist, (loose)	64.5
	▨	**SILTY CLAY** Brownish grey to grey, moist, (hard to very stiff)	61.2
		Borehole Terminated at 10.7 m Depth Upon Auger Refusal	55.3



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole backfilled with cuttings upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	9.4	10.7

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Test Pit TP-I

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Road, City of Ottawa, Ontario

Figure No. 11

Page. 1 of 1

Date Drilled: 'August 17, 2017

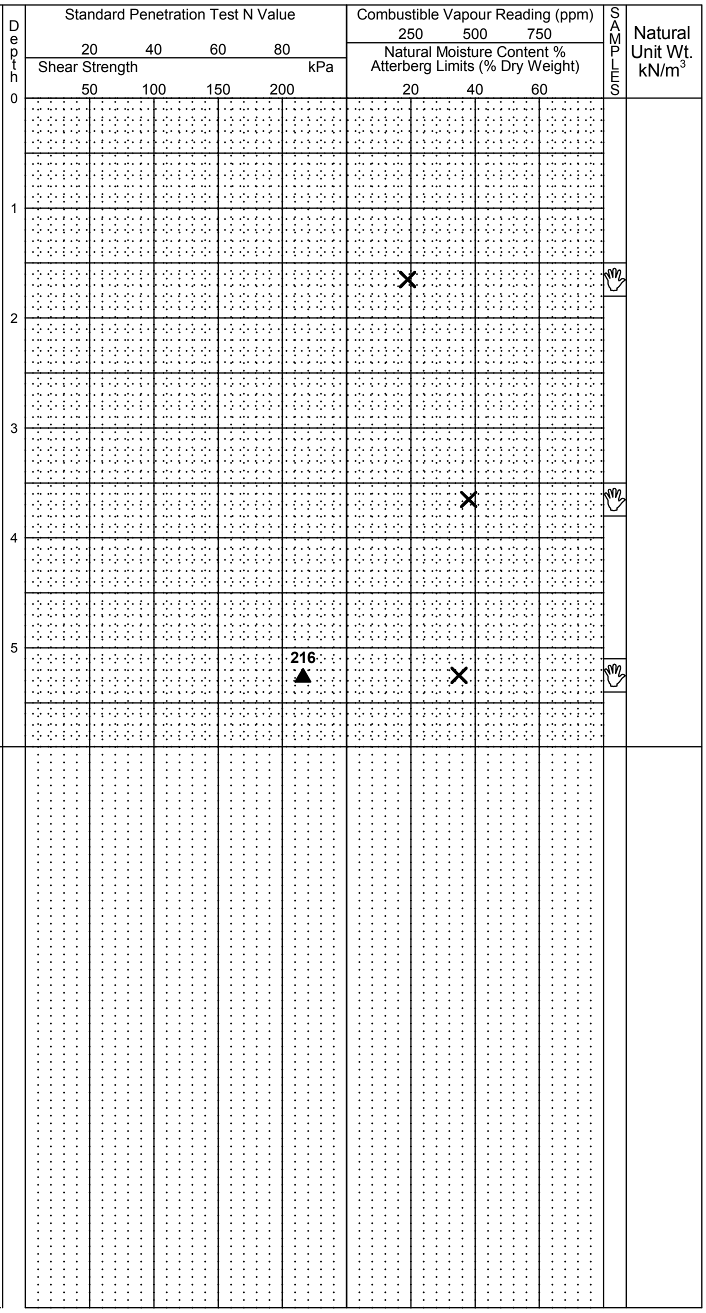
Drill Type: Excavator (30 ton)

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	⊢○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth
	▨	**FILL** Silty clay mixed with gravel and sand, frequent cobbles and boulders, brown, moist	68.53	0
	▨	**FILL** Silty clay, trace sand and gravel, grey, moist, (loose)	66.3	
	▨	**SILTY CLAY** Silty clay, brownish grey, moist, (hard)	63.8	
		Test Pit Terminated at 5.9 m Depth Upon Refusal On Bedrock Surface	62.6	



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Test Pit backfilled and compacted with excavator bucket upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	5.9

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Test Pit TP-J

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Road, City of Ottawa, Ontario

Figure No. 12

Page. 1 of 1

Date Drilled: 'August 17, 2017

Drill Type: Excavator (30 ton)

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m	Depth	Standard Penetration Test N Value 20 40 60 80 / Shear Strength kPa 50 100 150 200	Combustible Vapour Reading (ppm) 250 500 750 / Natural Moisture Content % / Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m^3
		FILL Silty to gravelly clay, some sand, frequent cobbles and boulders, brown, moist	69.95	0 1 2				
		FILL Silty clay, trace sand and gravel, grey, moist, (loose)	67.6	3 4 5				
		TOPSOIL Compressed	65.0					
		Test Pit Terminated at 5.5 m Depth Upon Refusal On Bedrock Surface Note Refusal confirmed on August 23, 2017 by augering through the test pit backfill.	64.5					

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Test Pit backfilled and compacted with excavator bucket upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	5.5

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-K

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario

Figure No. 13

Page. 1 of 1

Date Drilled: 'August 23, 2017

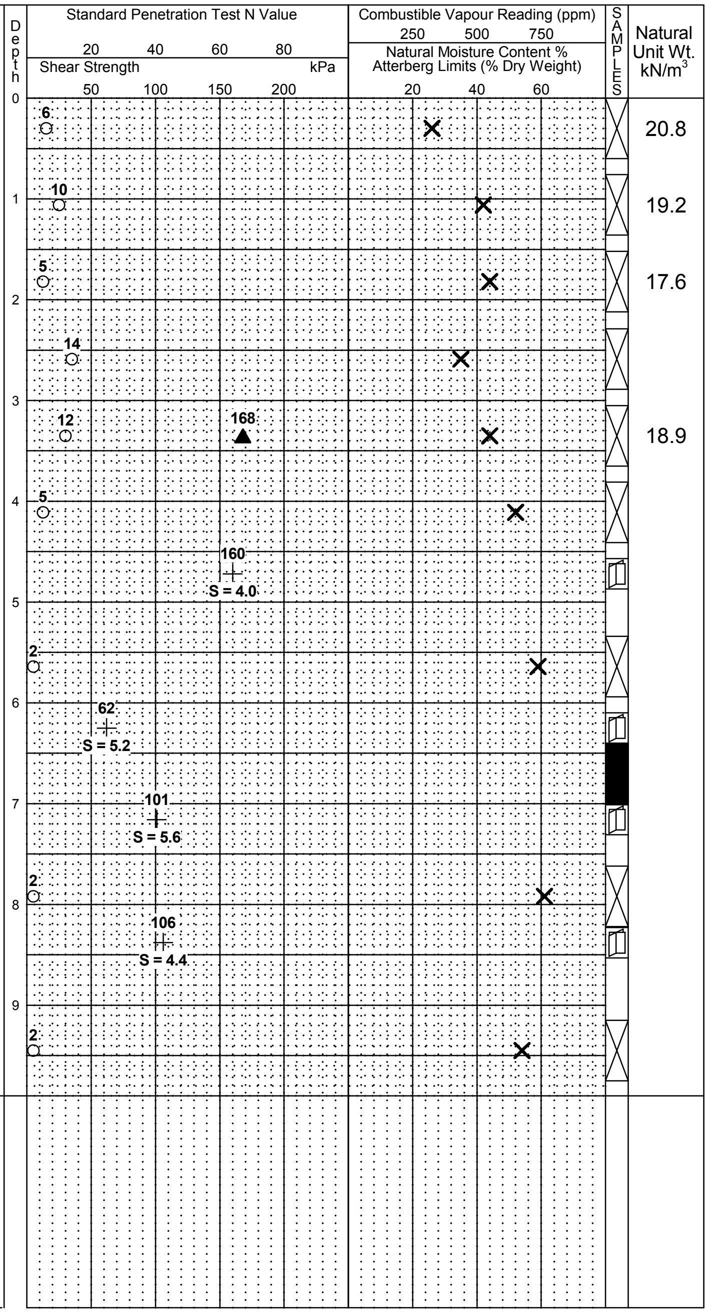
Drill Type: CME-75 Trackmount

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m
			62.7
	▩	**FILL** Silty clay, some sand, trace gravel, grey-brown, moist, (loose)	61.9
	▩	**FILL** Silty clay, trace sand, some organics (grass), grey, moist, (loose to compact)	59.7
	▨	**SILTY CLAY** Brownish grey to grey, moist, (very stiff to stiff)	52.8
		Borehole Terminated at 9.9 m Depth Upon Auger Refusal	



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole backfilled with cuttings upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	7.6	9.8

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-L

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario

Figure No. 14

Page. 1 of 1

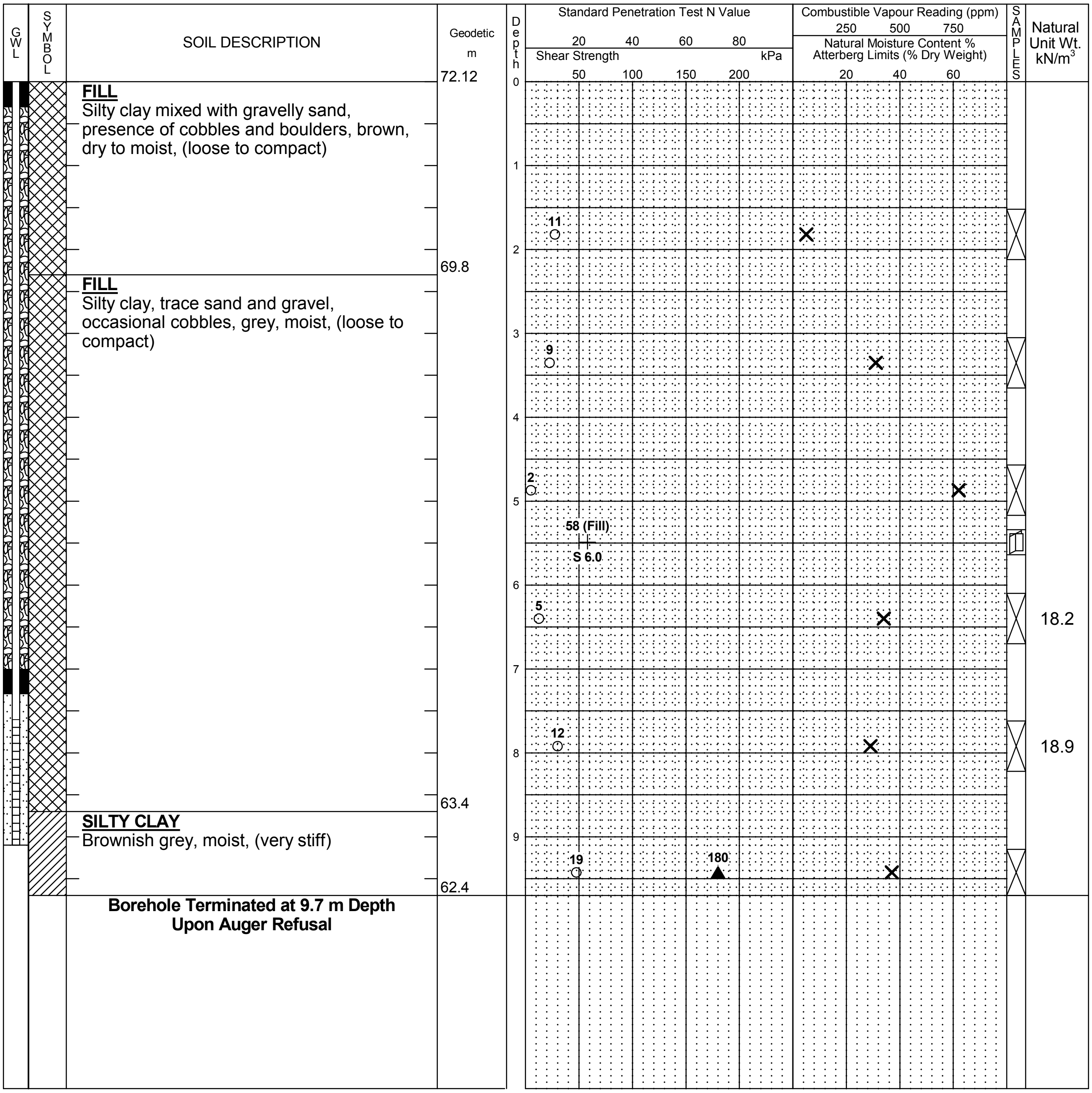
Date Drilled: 'August 25, 2017

Drill Type: CME-75 Trackmount

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample	◨	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



FILL
Silty clay mixed with gravelly sand, presence of cobbles and boulders, brown, dry to moist, (loose to compact)

FILL
Silty clay, trace sand and gravel, occasional cobbles, grey, moist, (loose to compact)

SILTY CLAY
Brownish grey, moist, (very stiff)

Borehole Terminated at 9.7 m Depth
Upon Auger Refusal

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. 19 mm standpipe was installed in teh borehole upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	9.2
31 days	dry	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-M

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario

Figure No. 15

Page. 1 of 2

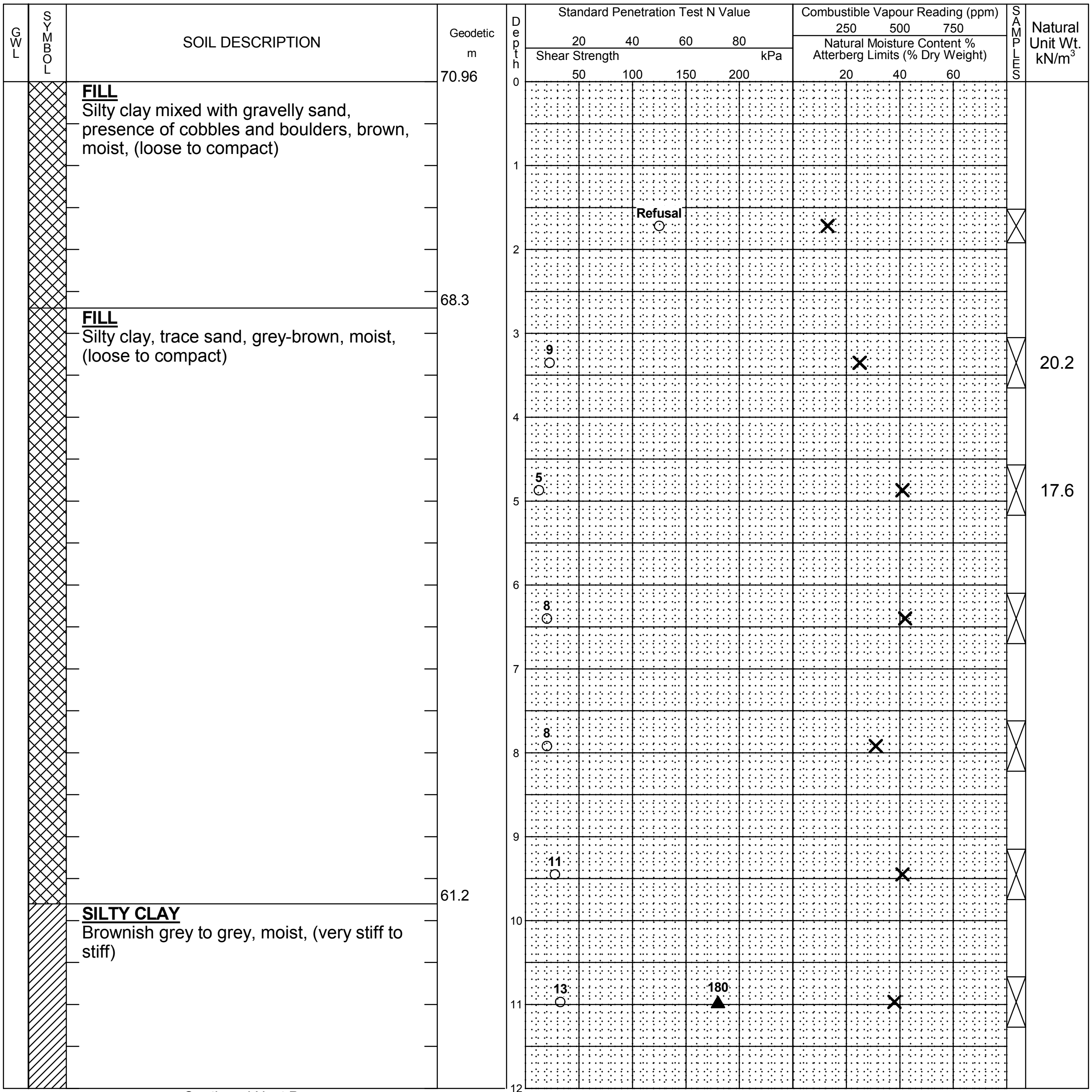
Date Drilled: 'August 25, 2017

Drill Type: CME-75 Trackmount

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	─	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



Geodetic m	Depth (m)	SOIL DESCRIPTION
70.96	0	**FILL** Silty clay mixed with gravelly sand, presence of cobbles and boulders, brown, moist, (loose to compact)
68.3		**FILL** Silty clay, trace sand, grey-brown, moist, (loose to compact)
61.2		**SILTY CLAY** Brownish grey to grey, moist, (very stiff to stiff)

Continued Next Page

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole backfilled with cuttings upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	14.6	15.2

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-M

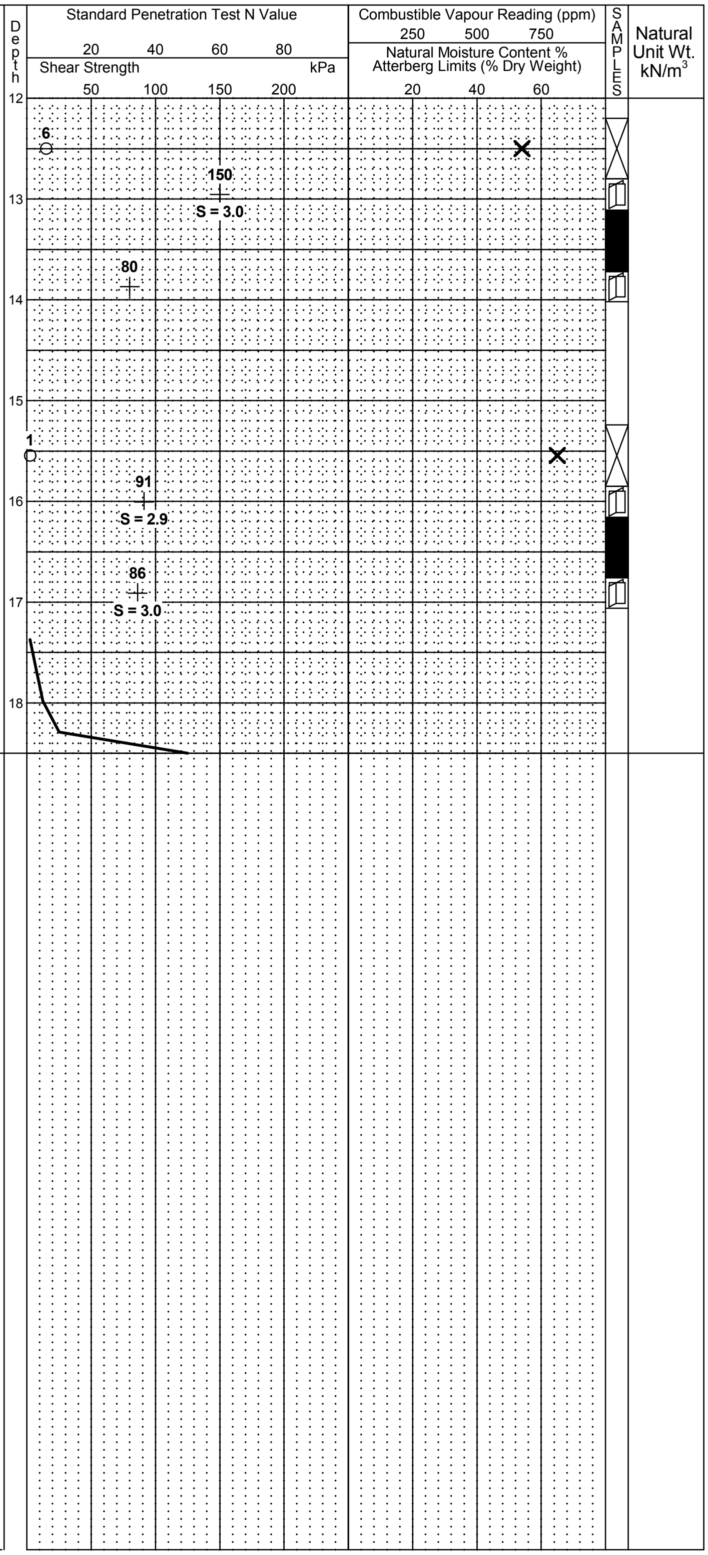
Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Figure No. 15

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m
	▨	**SILTY CLAY** Brownish grey to grey, moist, (very stiff to stiff) *(continued)*	58.96
	▨	**INFERRED SILTY CLAY** Cone Penetration Test advanced from 17.1 m depth to refusal at 18.5 m depth	53.9
		Borehole Terminated at 18.5 m Depth Upon Refusal of Cone Penetration Test	52.5



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole backfilled with cuttings upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	14.6	15.2

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-N

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario

Figure No. 16

Page. 1 of 1

Date Drilled: 'August 22, 2017

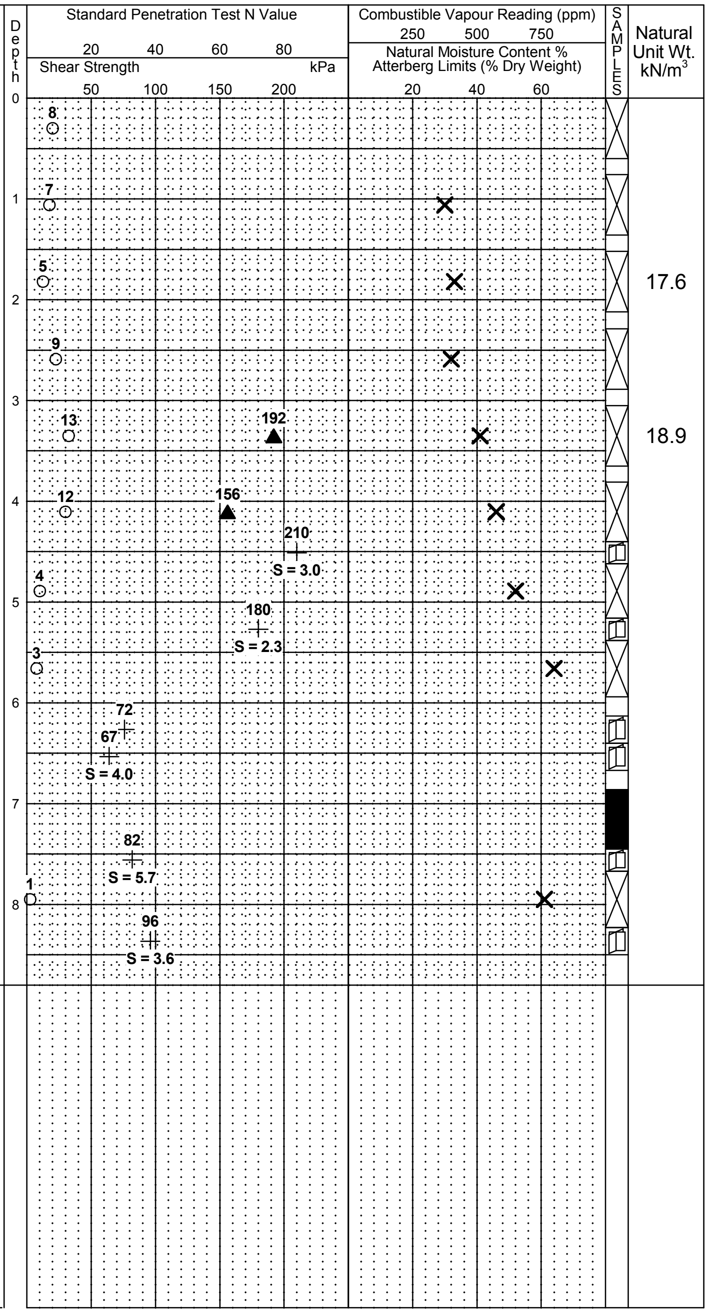
Drill Type: CME-75 Trackmount

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	─	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m
			63.66
		FILL Silty clay, trace to some sand and gravel, some organics (grass), brown to grey, moist, (loose)	
			60.6
		SILTY CLAY Brownish grey to grey, moist, (very stiff to stiff)	
▼			58.26
			54.9
		Borehole Terminated at 8.8 m Depth Upon Auger Refusal	



NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. 19 mm standpipe was installed in teh borehole upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	7.6	8.8
34 days	5.4	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-O

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario

Figure No. 17

Page. 1 of 2

Date Drilled: 'August 23, 2017

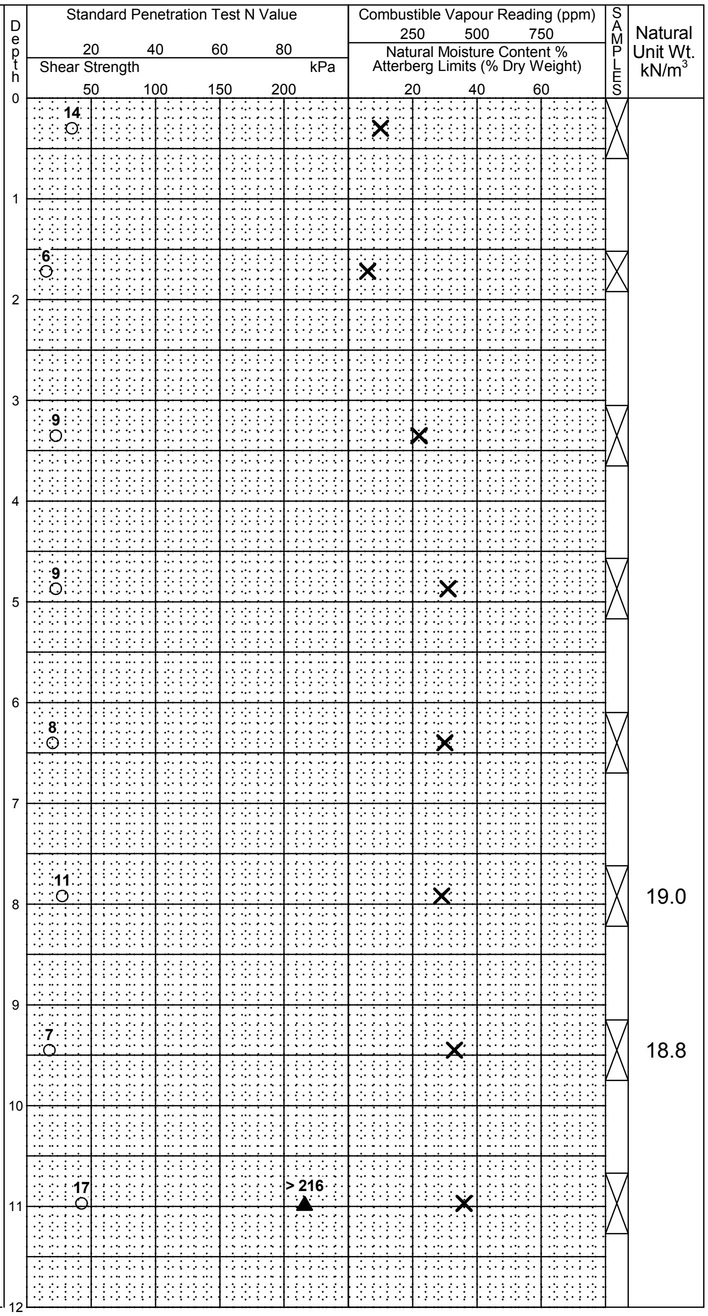
Drill Type: CME-75 Trackmount

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	⊢○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic m
			72.08
	▨	**FILL** Silty clay mixed with sand and gravel (crushed limestone), presence of cobbles and boulders, grey, moist, (loose)	
			69.4
	▨	**FILL** Silty clay, trace sand, grey-brown, moist, (loose)	
			61.7
	▨	**SILTY CLAY** Brownish grey to grey, moist, (hard to very stiff)	



Continued Next Page

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. Borehole backfilled with cuttings upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	16.5	16.8

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

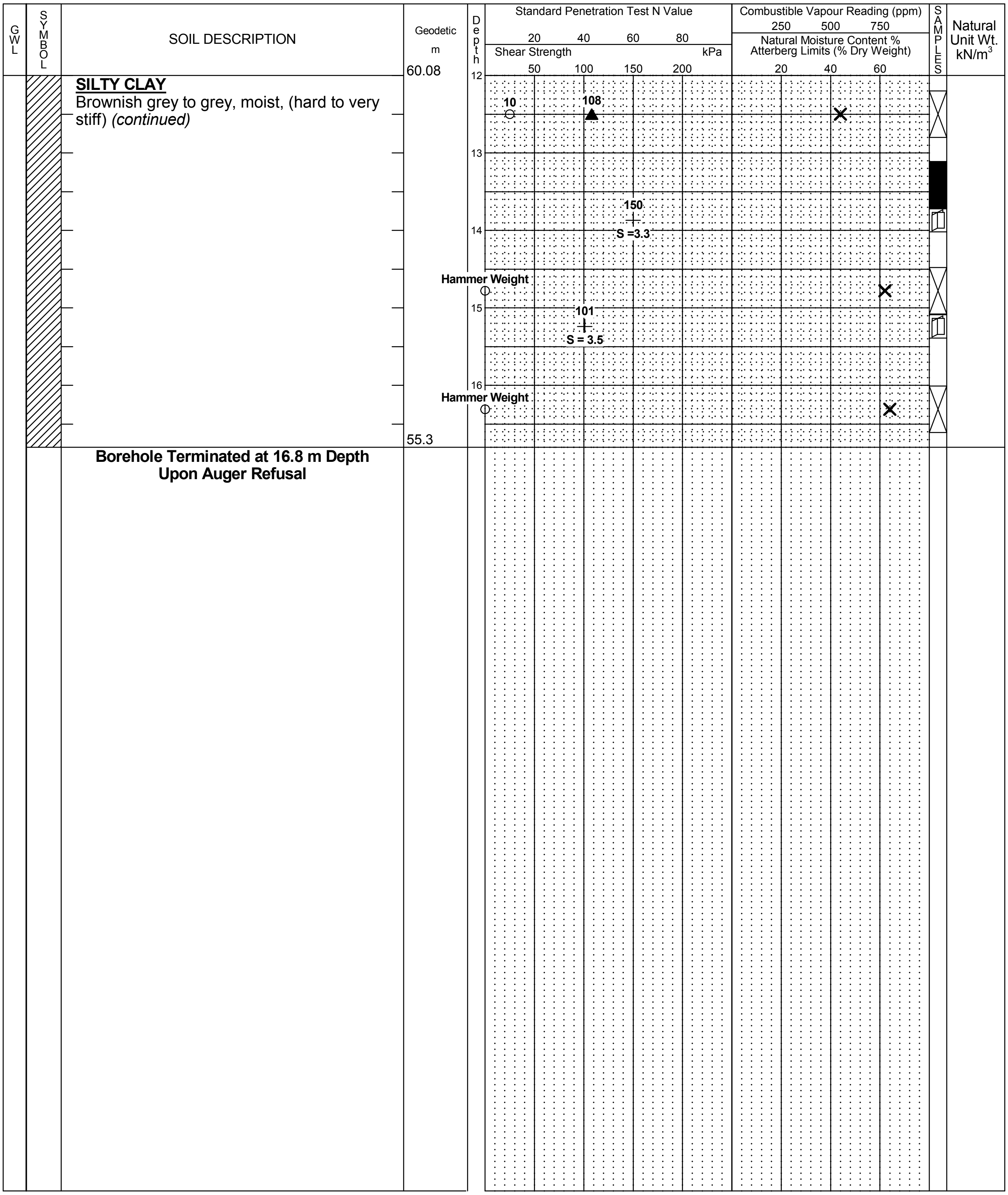
Log of Borehole BH-O

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Figure No. 17

Page. 2 of 2



SILTY CLAY
Brownish grey to grey, moist, (hard to very stiff) *(continued)*

Borehole Terminated at 16.8 m Depth
Upon Auger Refusal

NOTES:

1. Borehole data requires interpretation by exp. before use by others
2. Borehole backfilled with cuttings upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	16.5	16.8

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

LOG OF BOREHOLE BH LOGS - 241432 - HILLSIDE VISTA.GPJ TROW OTTAWA.GDT 11/2/17

Log of Borehole BH-P

Project No: OTT-00241432-A0

Project: Geotechnical Investigation. Hillside Vista Walk-Up Condos

Location: St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario

Figure No. 18

Page. 1 of 1

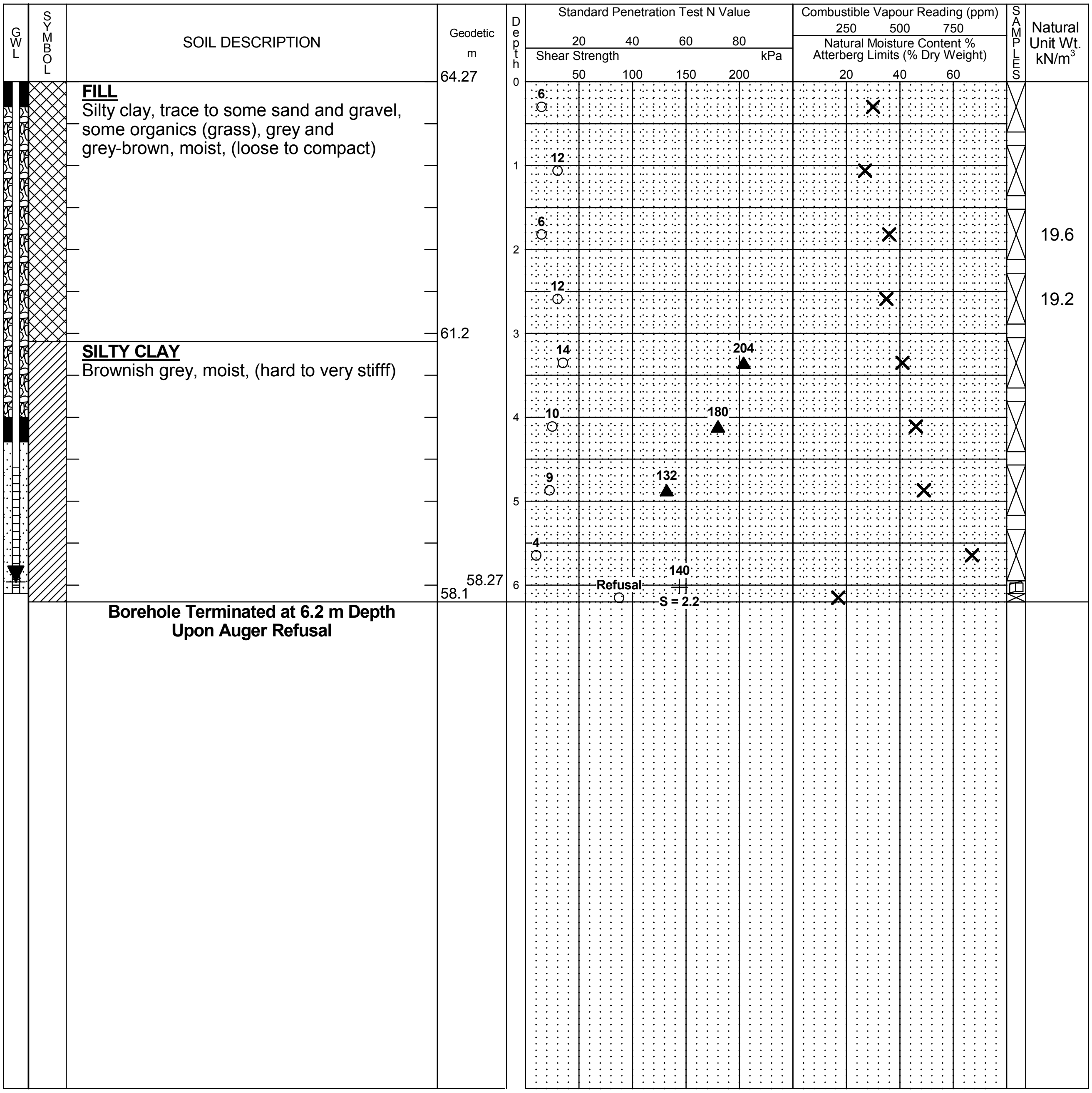
Date Drilled: 'August 22, 2017

Drill Type: CME-75 Trackmount

Datum: Geodetic

Logged by: M.L. Checked by: SKA

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



FILL
Silty clay, trace to some sand and gravel, some organics (grass), grey and grey-brown, moist, (loose to compact)

SILTY CLAY
Brownish grey, moist, (hard to very stifff)

Borehole Terminated at 6.2 m Depth Upon Auger Refusal

LOG OF BOREHOLE BH LOGS - 241432 - HILLSIDE VISTA.GPJ TROW OTTAWA.GDT 11/2/17

NOTES:
1. Borehole data requires interpretation by exp. before use by others
2. 19 mm standpipe was installed in teh borehole upon completion
3. Field work supervised by an **exp** representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-00241432-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
Completion	Dry	6.1
34 days	6.0	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

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Ottawa, ON K2B 8H6

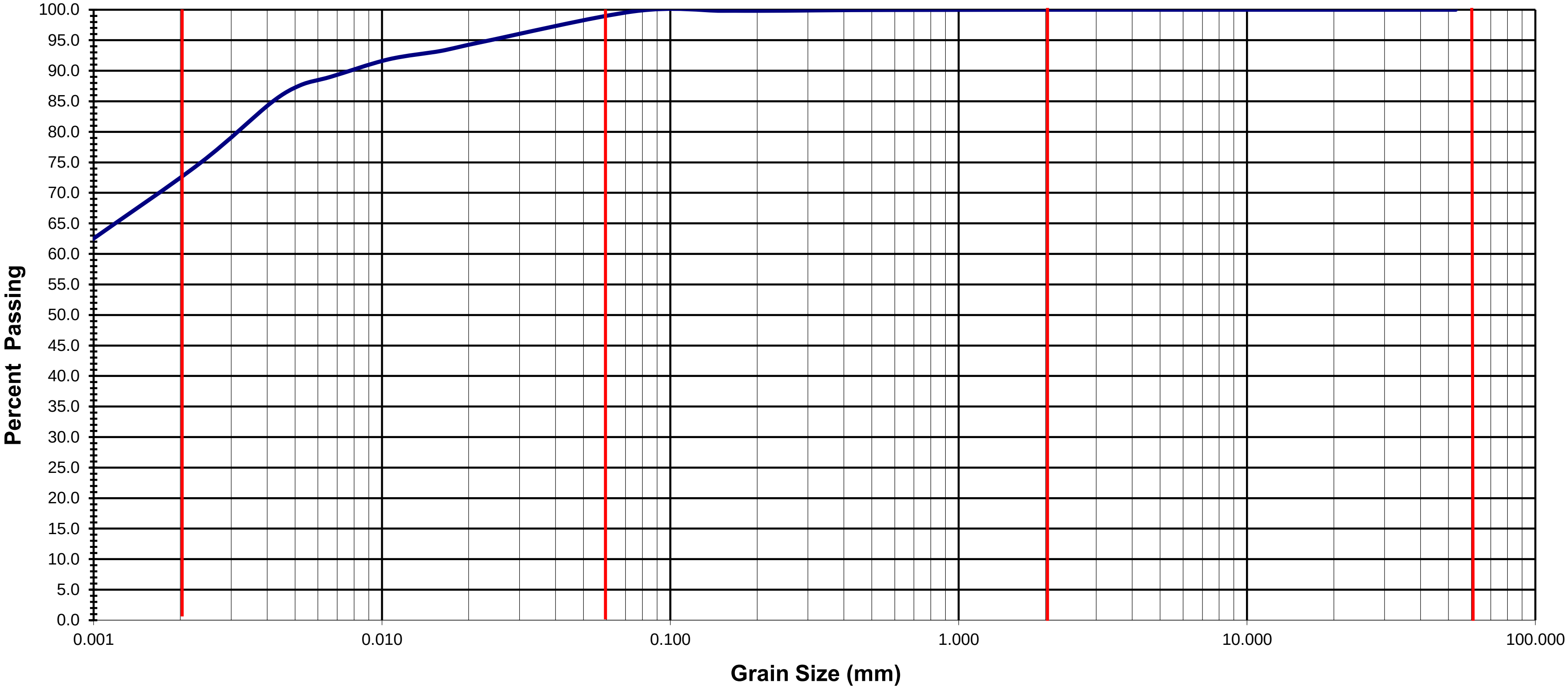
Grain-Size Distribution Curve

Method of Test for Particle Size Analysis of Soil

MTO Test Method LS - 702, Rev. No. 19 (ASTM D-422)

Modified M.I.T. Classification

CLAY	SILT			SAND			GRAVEL			
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	



Exp Project No.: OTT-00241432-AO	**Project Name :** Geotechnical Investigation. Hillside Vista Walk-Up Condos				
Client : DCR Pheonix Group of Companies	**Project Location :** St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario				
Date Sampled : August 23, 2017	**Bore Hole/Test Pit No.:** H	**Sample No.:** SS7	**Depth (m) :** 6.1-6.7		
Sample Description : Silty Clay, Trace Sand			**Figure :** 19		

exp Services Inc.
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Ottawa, ON K2B 8H6

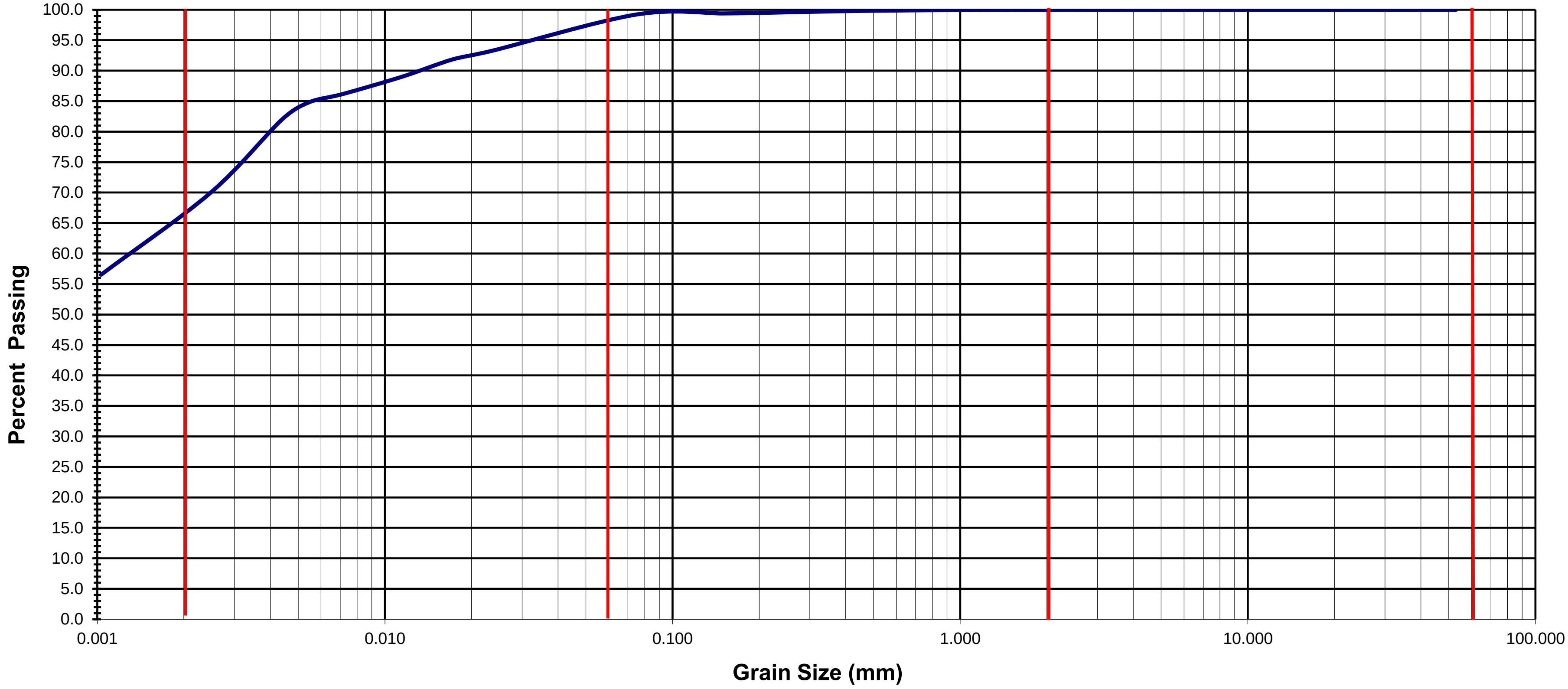
Grain-Size Distribution Curve

Method of Test for Particle Size Analysis of Soil

MTO Test Method LS - 702, Rev. No. 19 (ASTM D-422)

Modified M.I.T. Classification

CLAY	SILT			SAND			GRAVEL			
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	



Exp Project No.: OTT-00241432-AO		**Project Name :** Geotechnical Investigation. Hillside Vista Walk-Up Condos			
Client : DCR Pheonix Group of Companies		**Project Location :** St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario			
Date Sampled : August 22, 2017		**Bore Hole/Test Pit No.:** N	**Sample No.:** SS10	**Depth (m) :** 7.6-8.2	
Sample Description : Silty Clay, Trace Sand				**Figure :** 20	

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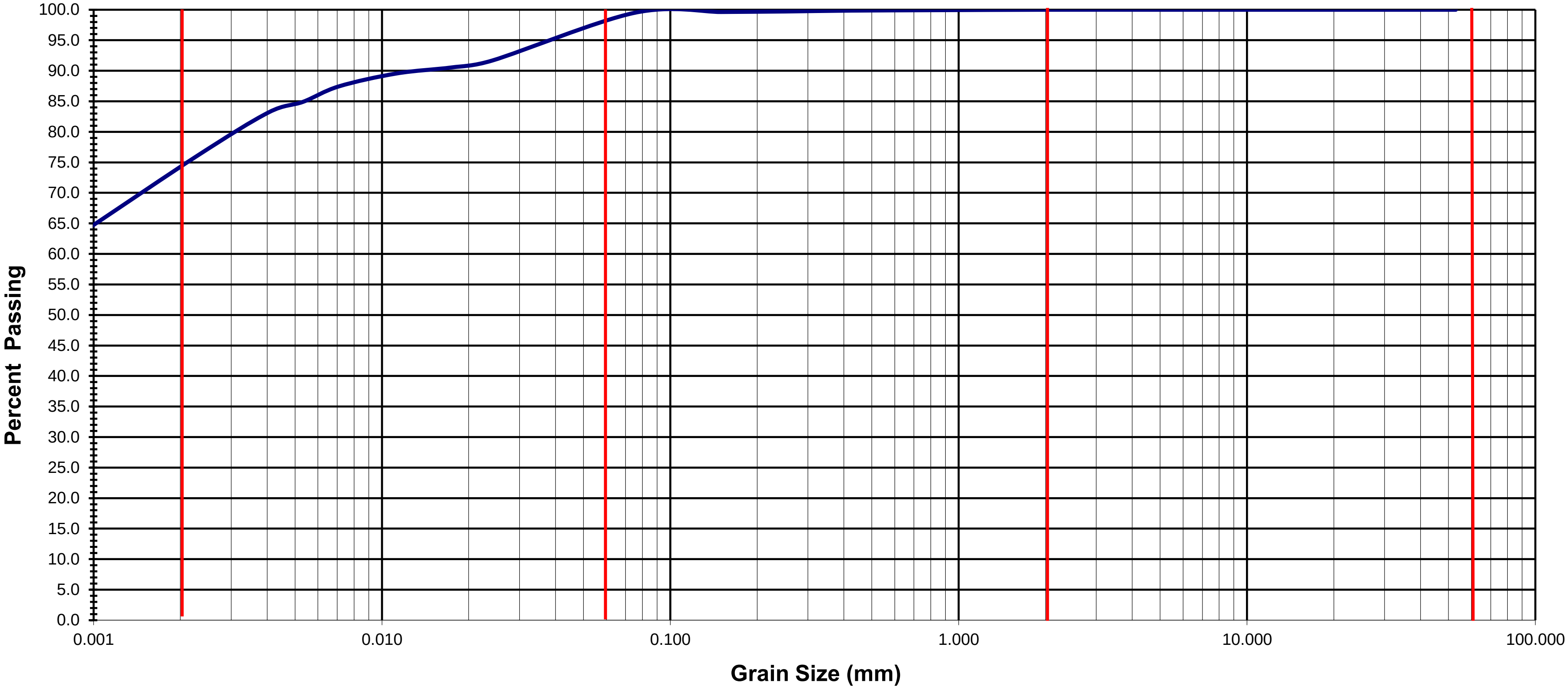
Grain-Size Distribution Curve

Method of Test for Particle Size Analysis of Soil

MTO Test Method LS - 702, Rev. No. 19 (ASTM D-422)

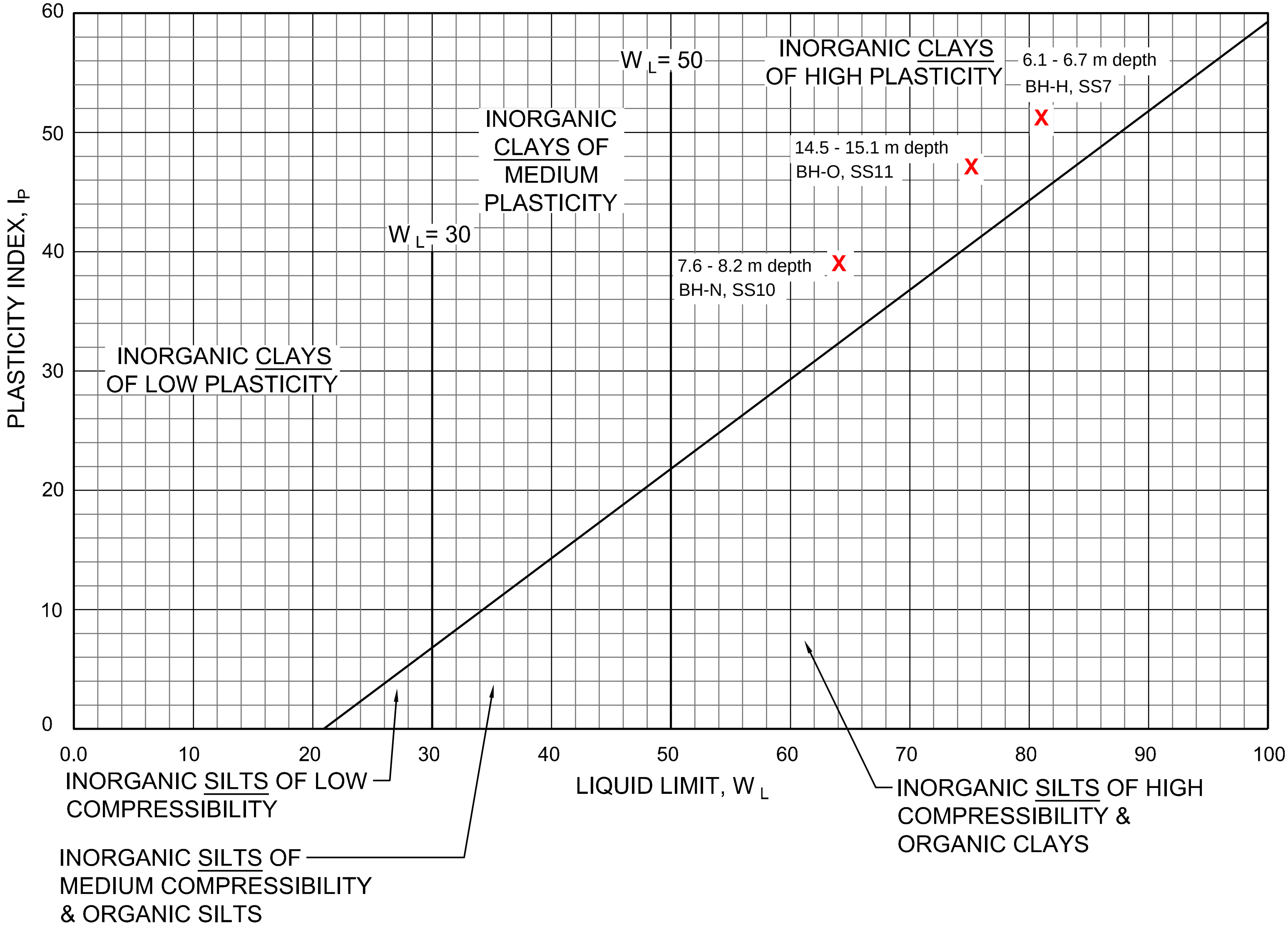
Modified M.I.T. Classification

CLAY	SILT			SAND			GRAVEL			
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	



Exp Project No.: OTT-00241432-AO	**Project Name :** Geotechnical Investigation. Hillside Vista Walk-Up Condos				
Client : DCR Pheonix Group of Companies	**Project Location :** St-Joseph Blvd and Tenth Line Rd., City of Ottawa, Ontario				
Date Sampled : August 23, 2017	**Bore Hole/Test Pit No.:** O		**Sample No.:** SS11	**Depth (m) :** 14.5-15.1	
Sample Description :	Silty Clay, Trace Sand			**Figure :** 21	

PLASTICITY CHART (AFTER CASAGRANDE, 1948)



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• INDUSTRIAL • INFRASTRUCTURE • SUSTAINABILITY •

scale: N.T.S.
date:
drawn by: J.R.

CLIENT: DCR Phoenix Group of Companies

TITLE: Hillside Vista Walk-Up Condos

project no. OTT-00241432-A0

FIG 22

Filename: s:\drafting\geotech\soil plasticity chart.dwg
Last Saved: 12/2/2016 9:15:45 AM
Last Plotted: 12/2/2016 9:16:15 AM
Plotted by: nugentm
Pen Table:: trow standard, july 01, 2004.ctb

Englobe

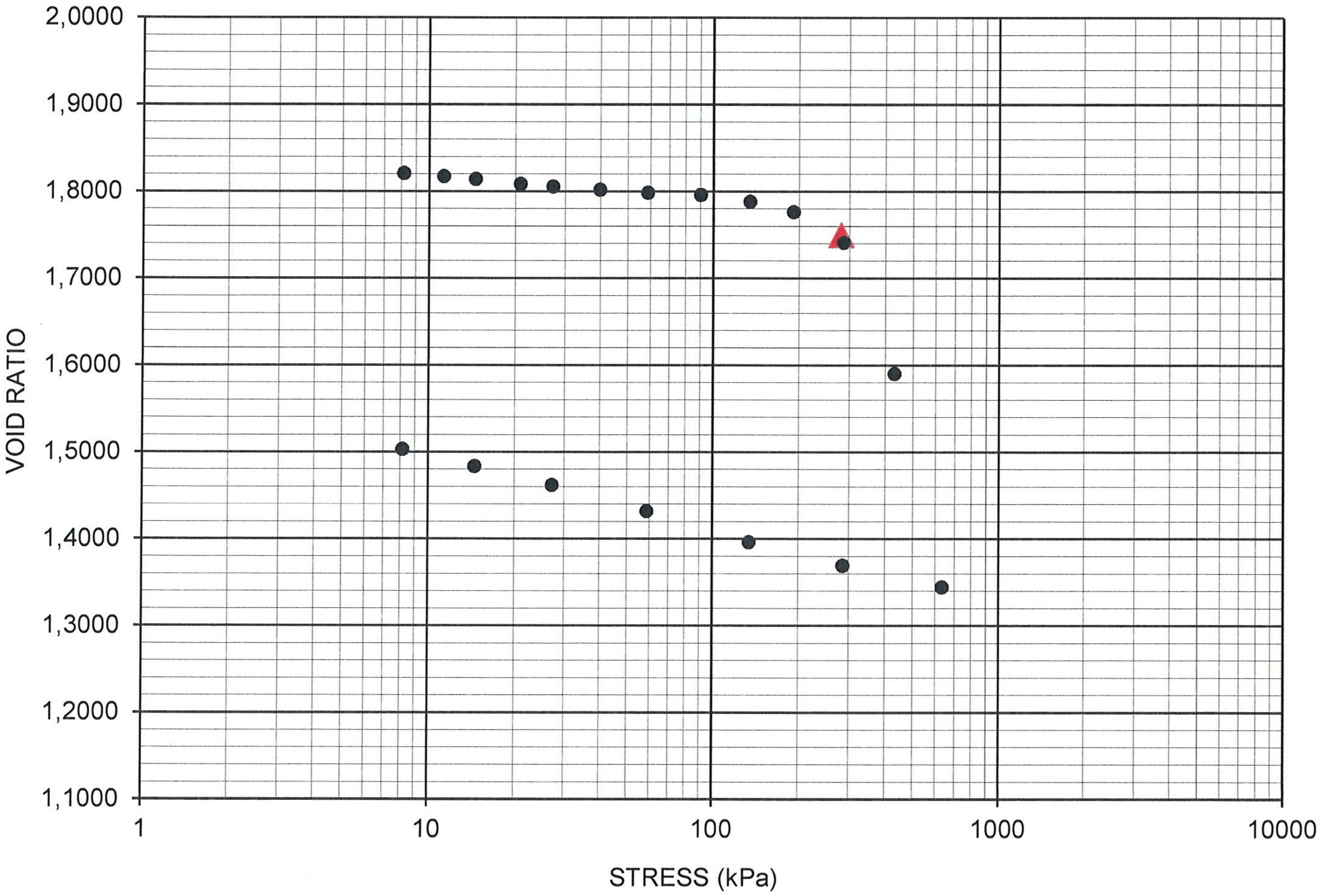
One-Dimensional Consolidation Properties of Soils Using Incremental Loading

ASTM D 2435 - Taylor Method

Client: EXP Ontario
Date : 2017-11-02
Project : OTT-00241432-A0
Our file No. : P-0011703-6-01-Rev01
Boring No. : BH-M, ST-9
Sample No. : 5
Depth (m) : 13,60 to 13,70m
Hydrostatic stress at the test (date) :
Provided by ☐ the client ☐ Englobe

Figure 23

STRESS vs VOID RATIO CURVE



Geotechnical Characteristics of Soils :

Initial void ratio (e_o) :	1,833	Recompression index (C_r) :	0,026
Initial water content (w) :	65,1%	Virgin compression index (C_c) :	1,15
Initial humid unit weight (γ_h) :	15,7 kN/m³	Initial effective stress (σ'_v) :	216 kPa
Initial saturation degree (S_r) :	97,6%	Preconsolidation pressure (σ'_p):	280 kPa
		Overconsolidation deviation ($\Delta\sigma$) :	64 kPa

Remarks : The sampling and transportation of the sample were carried out by a client's representative.
The initial effective stress has been provided by the client.

Prepared by :

A. Bouadma

Adlane Bouadma, eng. jr

Verified by :

Famakhan Fainke, eng.

One-Dimensional Consolidation Properties of Soils Using Incremental Loading

ASTM D 2435 - Taylor Method

Client: EXP Ontario

Date : 2017-11-02

Project : OTT-00241432-A0

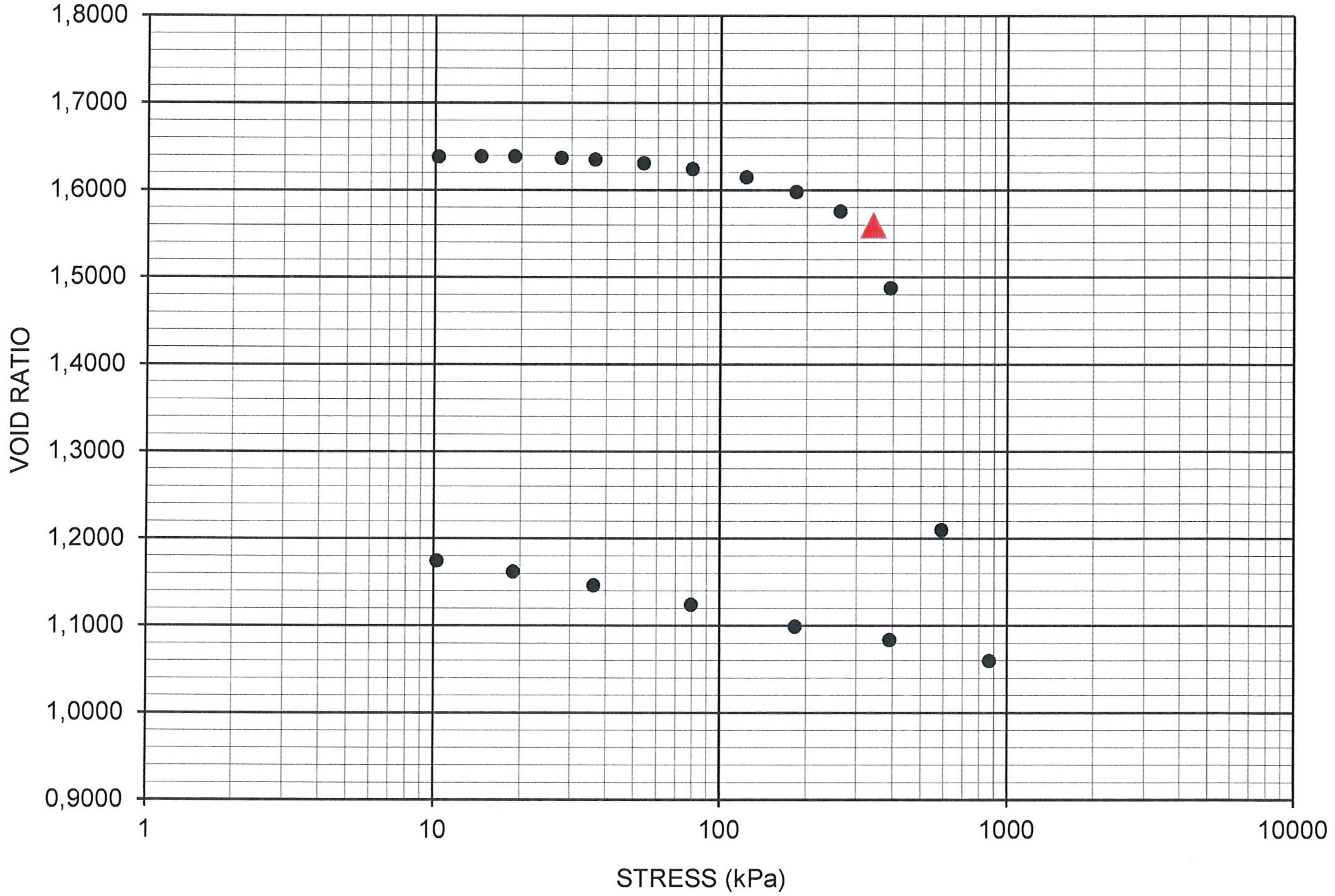
Our file No. : P-0011703-6-01-REv01

Boring No. : BH-M, ST-11 **Sample No. :** 6 **Depth (m) :** 16,60 to 16,70m

Hydrostatic stress at the test (date) : Provided by ☐ the client ☐ Englobe

Figure 24

STRESS vs VOID RATIO CURVE



Geotechnical Characteristics of Soils :

Initial void ratio (e_o) :	1,639	Recompression index (C_r) :	0,026
Initial water content (w) :	59,4%	Virgin compression index (C_c) :	1,24
Initial humid unit weight (γ_h) :	16,3 kN/m³	Initial effective stress (σ'_v) :	235 kPa
Initial saturation degree (S_r) :	99,6%	Preconsolidation pressure (σ'_p):	340 kPa
		Overconsolidation deviation ($\Delta\sigma$) :	105 kPa

Remarks : The sampling and transportation of the sample were carried out by a client's representative.
The initial effective stress has been provided by the client.

Prepared by :

A. Bouadma

Adlane Bouadma, eng. jr

Verified by :

Famakhan Fainke, eng

Englobe

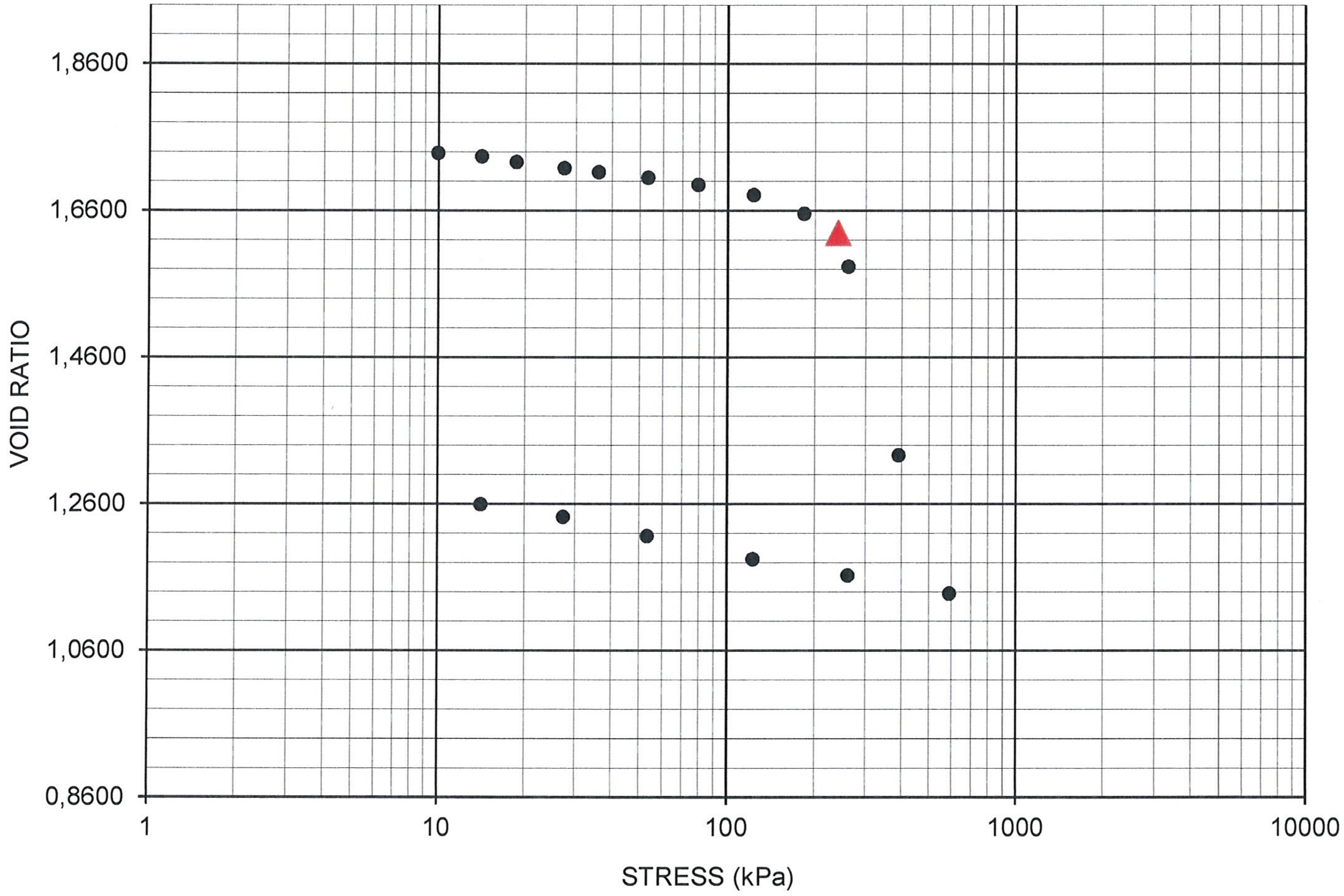
One-Dimensional Consolidation Properties of Soils Using Incremental Loading

ASTM D 2435 - Taylor Method

Client: EXP Ontario **Date :** 2017-11-02
Project : OTT-00241432-A0 **Our file No. :** P-0011703-6-01-Rev01
Boring No. : BH-N, ST-9 **Sample No. :** 7 **Depth (m) :** 7,20 to 7,30m
Hydrostatic stress at the test (date) : Provided by ☐ the client ☐ Englobe

Figure 25

STRESS vs VOID RATIO CURVE



Geotechnical Characteristics of Soils :

Initial void ratio (e_o) :	1,738	Recompression index (C_r) :	0,055
Initial water content (w) :	63,0%	Virgin compression index (C_c) :	1,29
Initial humid unit weight (γ_h) :	16,1 kN/m³	Initial effective stress (σ'_v) :	101 kPa
Initial saturation degree (S_r) :	99,7%	Preconsolidation pressure (σ'_p):	240 kPa
		Overconsolidation deviation ($\Delta\sigma$) :	139 kPa

Remarks : The sampling and transportation of the sample were carried out by a client's representative.
The initial effective stress has been provided by the client.

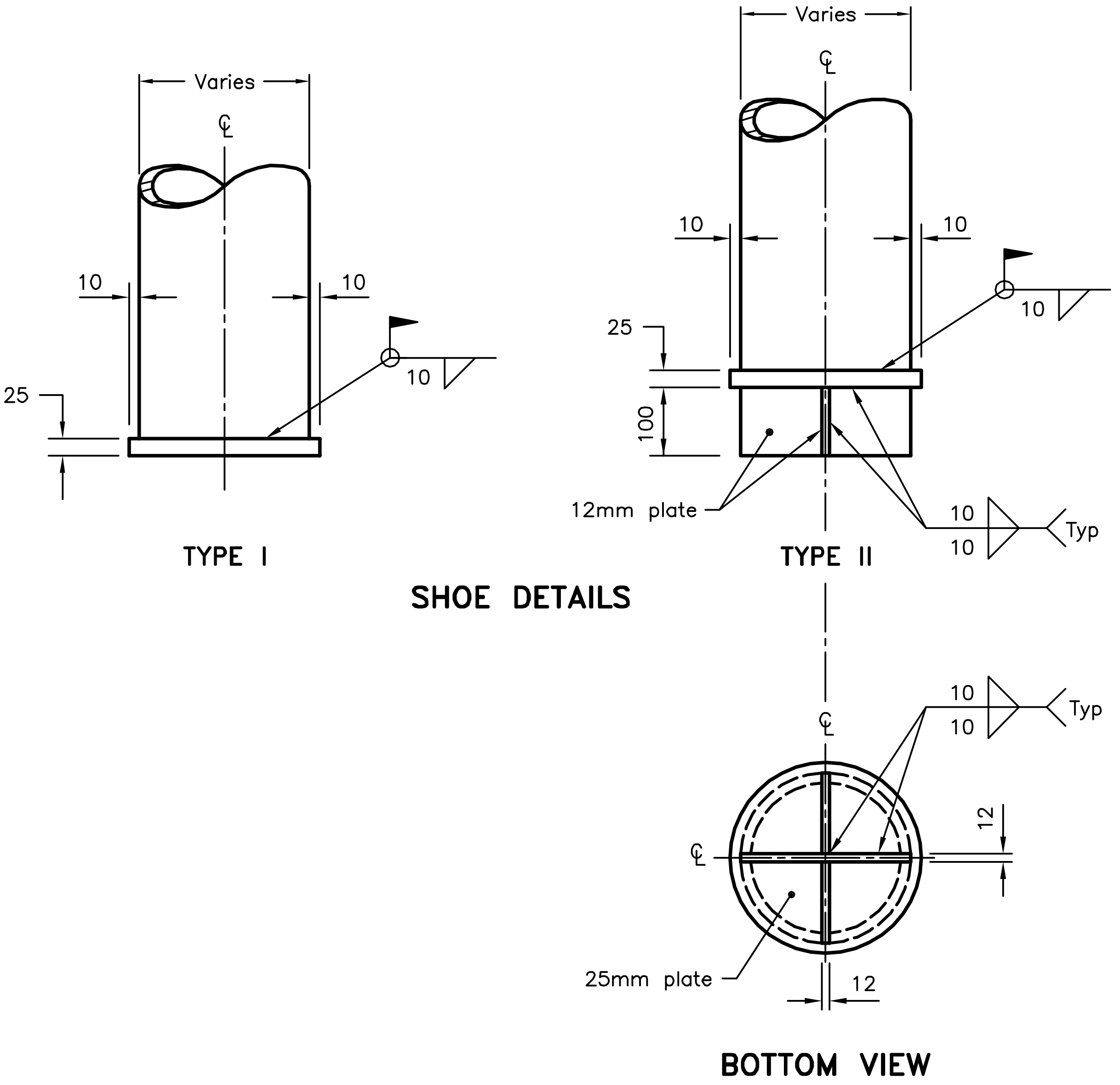
Prepared by :

A. Bouadma

Adlane Bouadma, eng. jr

Verified by :

Famakhan Fainke, eng.



NOTES:

- A Driving shoe Type I or II as specified.
- B Welding shall be according to CSA W59.
- C Steel plates shall be according to CSA G40.20/G40.21, Grade 300W.
- D All dimensions are in millimetres unless otherwise shown.

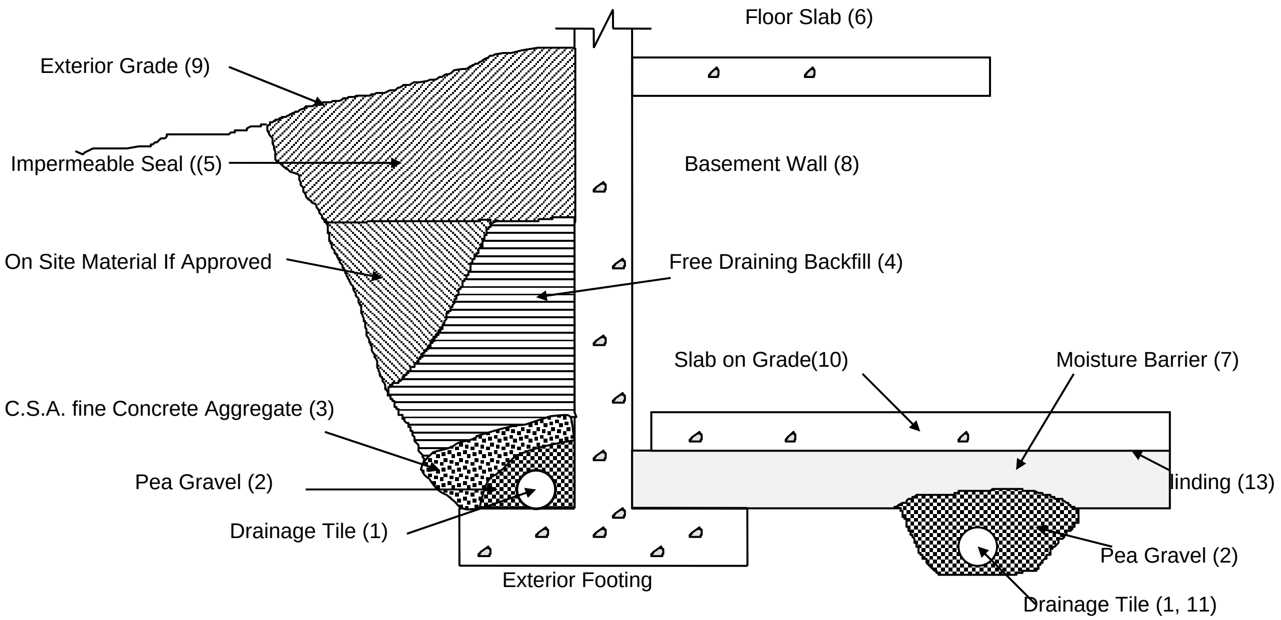
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 1

FOUNDATION
PILES
STEEL TUBE PILE DRIVING SHOE

Figure 26

OPSD 3001.100

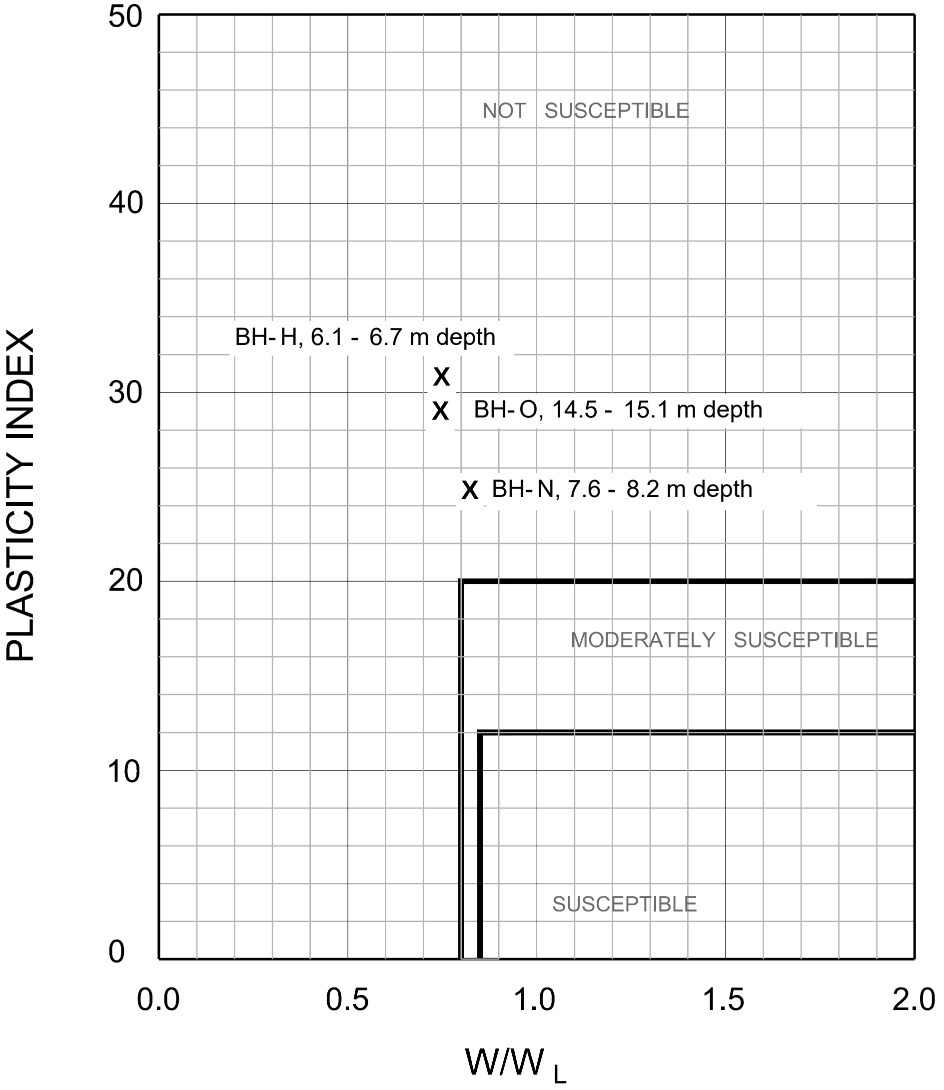


Notes

1. Drainage tile to consist of 150 mm (6") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet. Invert to be a minimum of 150 mm (6") below underside of floor slab.
2. Pea gravel - 150 mm (6") top and side of drain. If drain is not on footing, place 100 mm (4 inches) of pea gravel below drain. 20 mm (3/4") clear stone is an alternative provided it is surrounded by an approved porous plastic membrane (Terrafix 270R or equivalent).
3. C.S.A. fine concrete aggregate to act as filter material. Minimum 300 mm (12") top and side of tile drain. This may be replaced by an approved porous plastic membrane as indicated in (2).
4. Free Draining backfill - OPSS Granular B or equivalent compacted to the specified density. Do not use heavy compaction equipment within 450 mm (18") of the wall. Use hand controlled light compaction equipment within 1.8 m (6') of wall.
5. Impermeable backfill seal - compacted clay, clayey silt or equivalent. If original soil is free-draining, seal may be omitted.
6. Do not backfill until wall is supported by basement and floor slabs or adequate bracing.
7. Moisture barrier to be at least 200 mm (8") of compacted clear 20 mm (3/4") stone or equivalent free draining material.
8. Basement wall to be damp-proofed.
9. Exterior grade to slope away from building.
10. Slab on grade should not be structurally connected to the wall or footing.
11. Underfloor drain invert to be at least 300 mm (12") below underside of floor slab. Drainage tile placed in parallel rows 3 to 4 m (10 to 12') centres one way. Place drain on 100 mm (4") pea gravel with 150 mm (6") of pea gravel on top and sides. Provide filter material as noted in (3) if moisture barrier is not clear crushed stone.
12. Do not connect the underfloor drains to perimeter drains.
13. If the 20 mm (3/4") stone requires surface blinding, use 6 mm (1/4") clear stone chips.

DRAINAGE AND BACKFILL RECOMMENDATIONS

(not to scale)



BRAY ET AL. (2004) CRITERIA FOR LIQUEFACTION ASSESSMENT OF FINE-GRAINED SOILS

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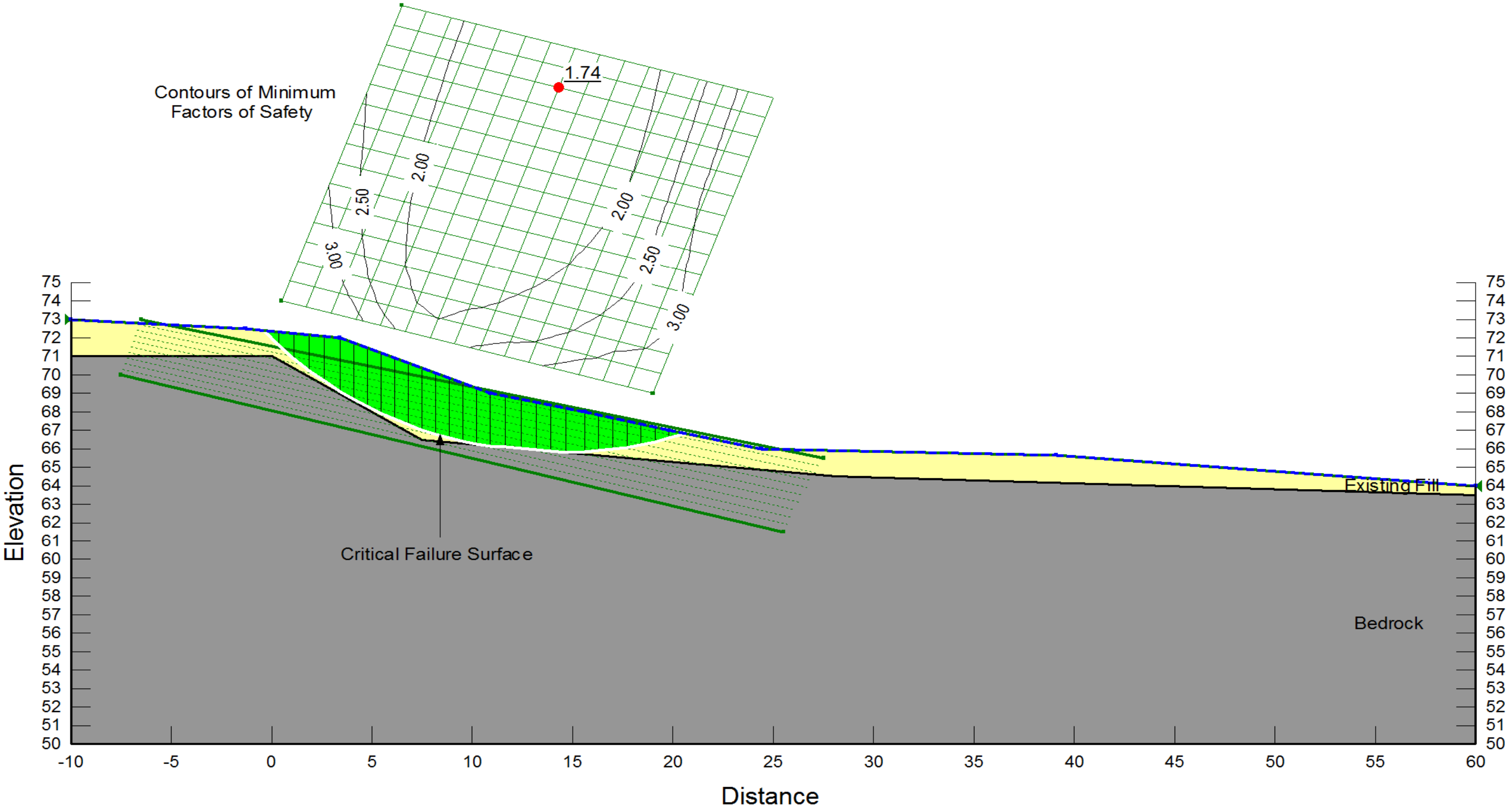
scale N.T.S.	CLIENT: DCR Phoenix Group of Companies	project no. OTT-00241432-A0
date		
drawn by M.N.	TITLE: Hillside Vista Walk-Up Condos	FIG 28

Filename: s:\drafting\geotech\liquefaction assessment.dwg
Last Saved: 9/23/2014 10:43:27 AM
Last Plotted: 9/23/2014 10:51:28 AM Plotted by: nugentm Pen Table:: trow standard, july 01, 2004.ctb

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 29



Section / Location : A-A
Original Slope : 4.0H:1V
Original Height : 6.5 m
Condition : Existing, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

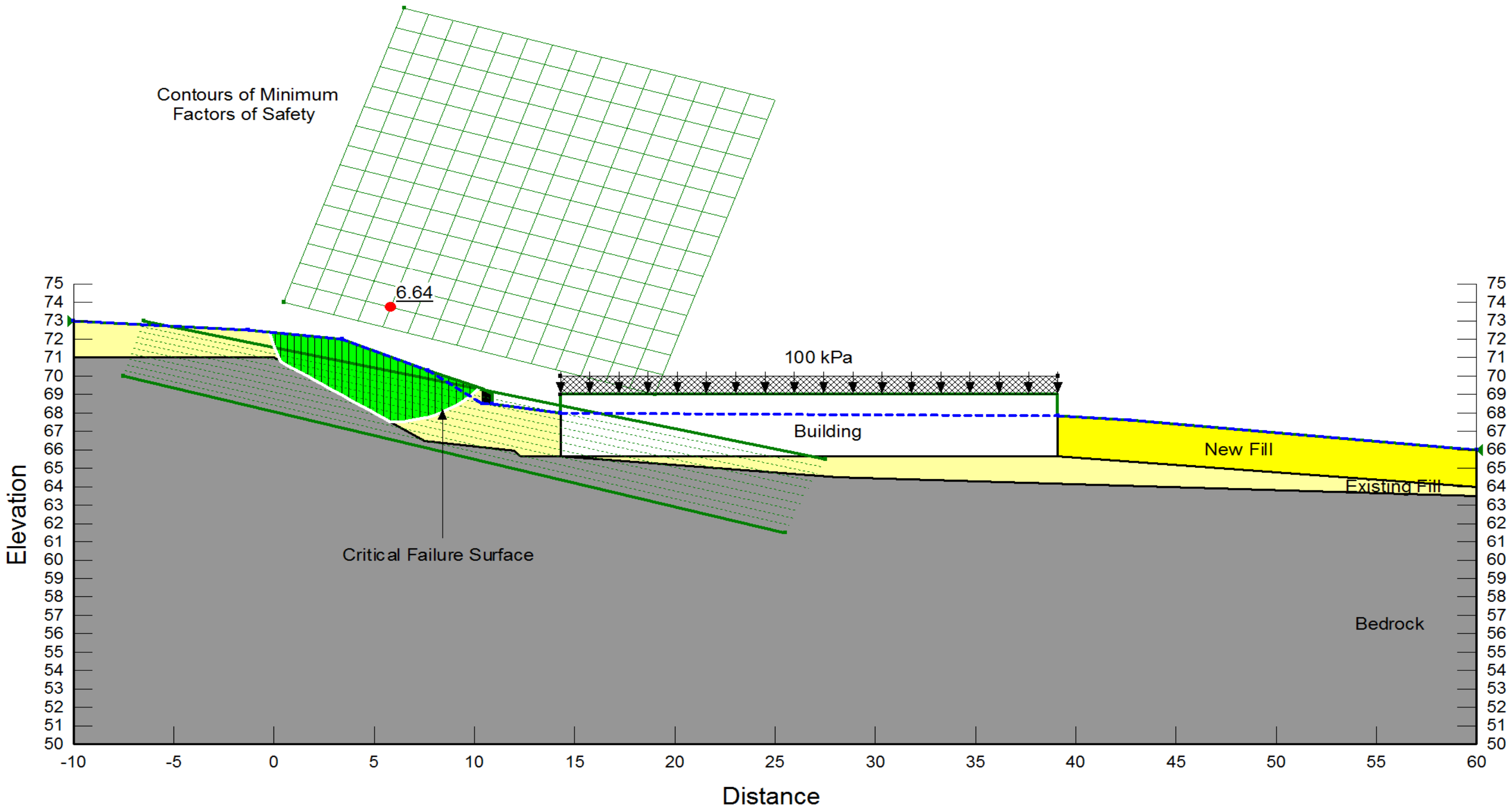
LEGEND

🟨 Existing Fill
⬛ Bedrock

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 30



Section / Location : A-A
Original Slope : 4.0H:1V
Original Height : 6.5 m
Condition : End of Construction, Short Term
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

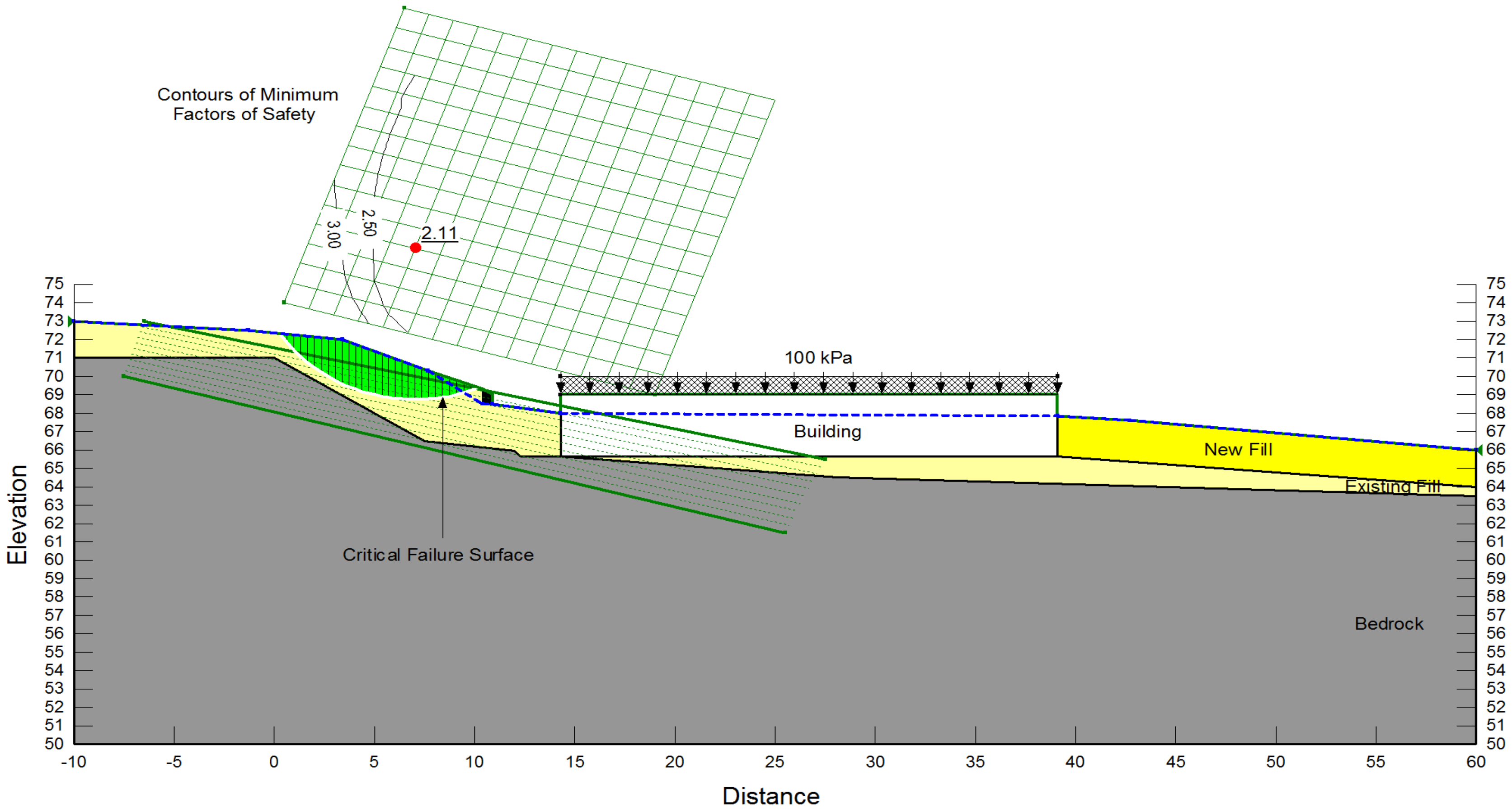
<u>LEGEND</u>

🟨 New Fill
🟨 Existing Fill
▩ Bedrock
⬛ Retaining Wall

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 31

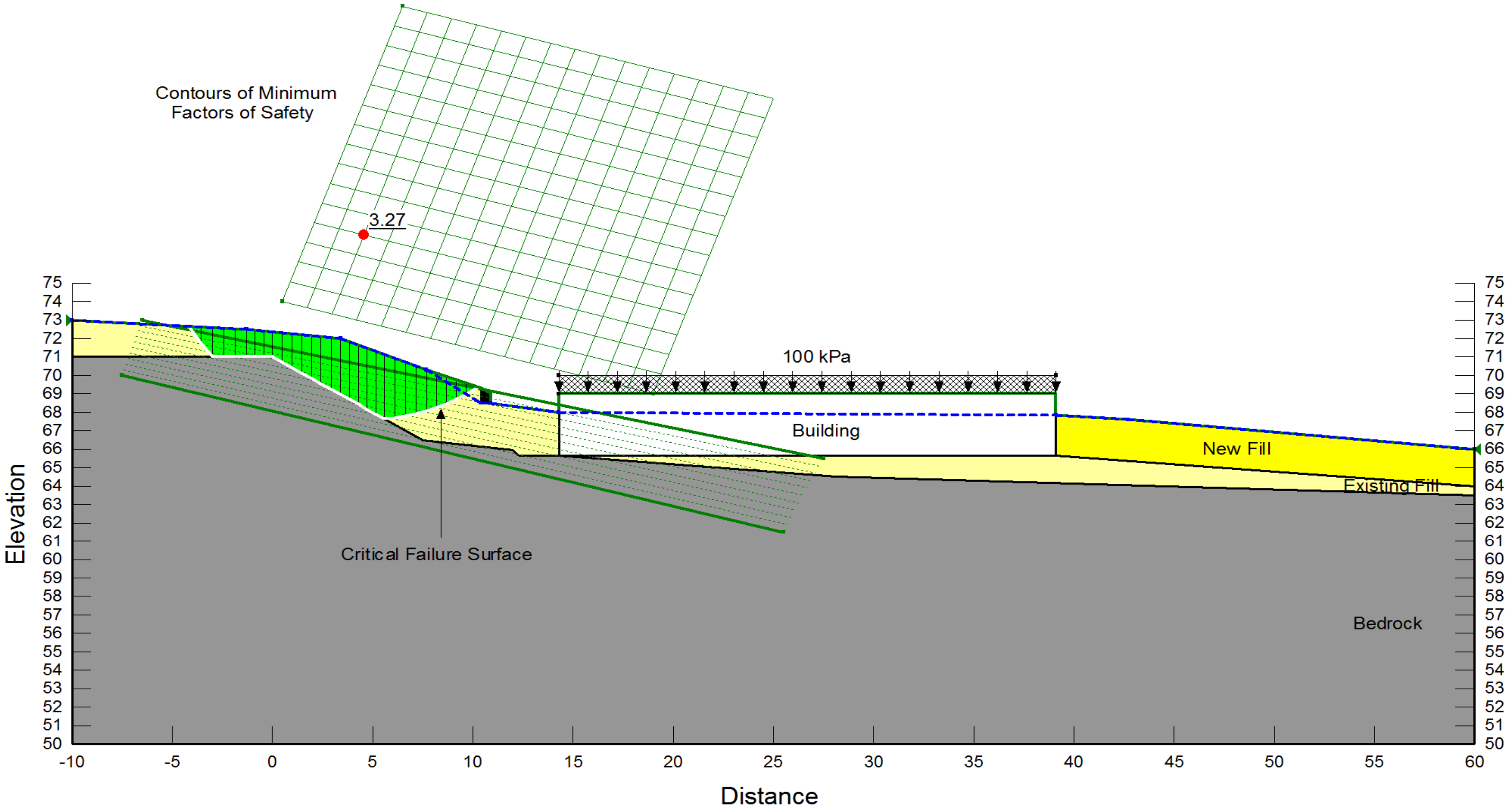


Section / Location : A-A
Original Slope : 4.0H:1V
Original Height : 6.5 m
Condition : End of Construction, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

LEGEND

🟨	New Fill
▭	Existing Fill
▩	Bedrock
⬛	Retaining Wall

PSEUDO-STATIC SLOPE STABILITY ANALYSIS **OTT-00241432-A0**
Hillside Terrace Condos
Ottawa, Ontario **Figure No. 32**



Section / Location : A-A
Original Slope : 4.0H:1V
Original Height : 6.5 m
Condition : End of Construction, Short Term, Seismic
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

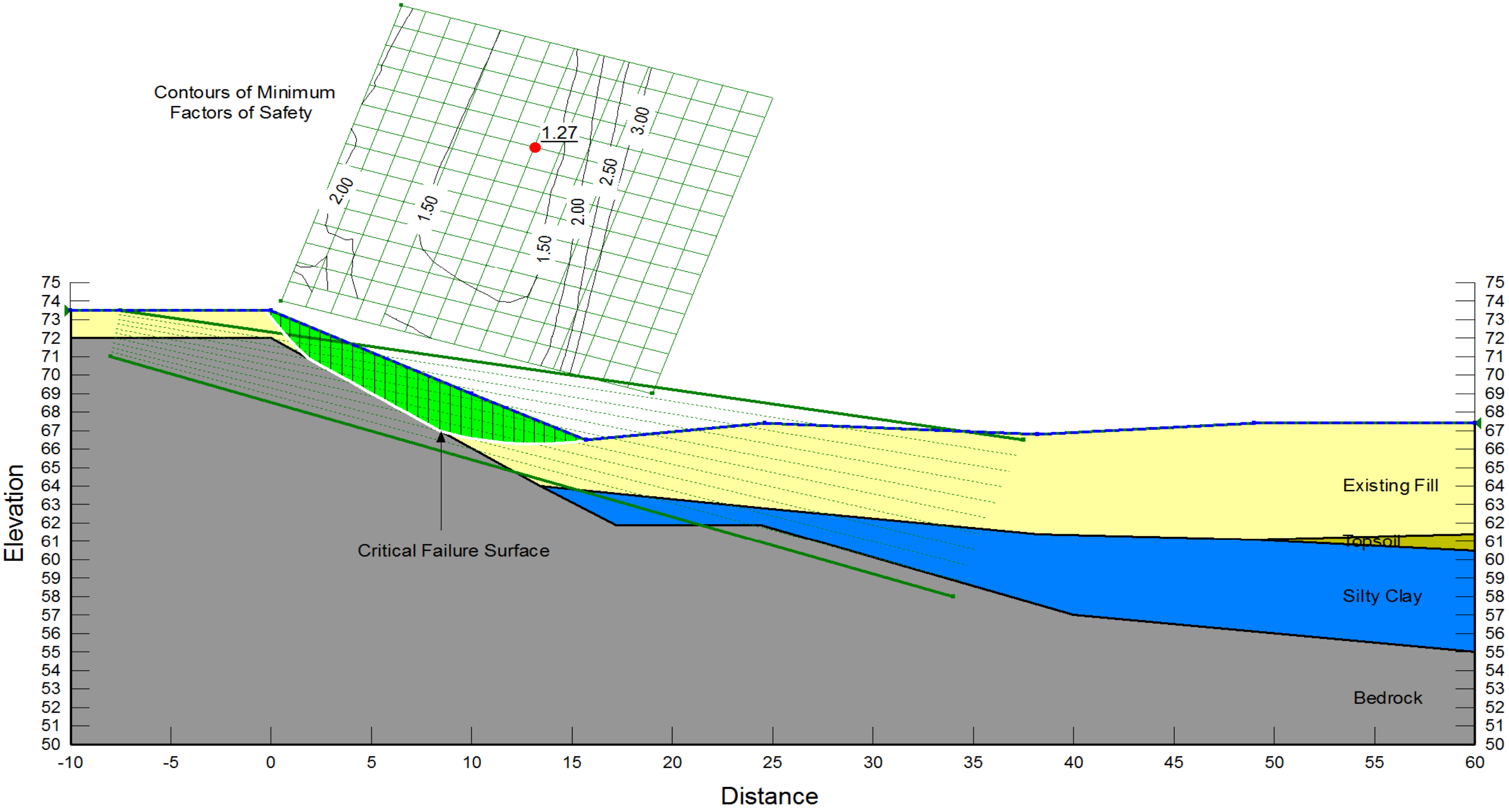
LEGEND

🟨	New Fill
🟨	Existing Fill
⬛	Bedrock
⬛	Retaining Wall

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 33



Section / Location : B-B
Original Slope : 2.2H:1V
Original Height : 7.0 m
Condition : Existing, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

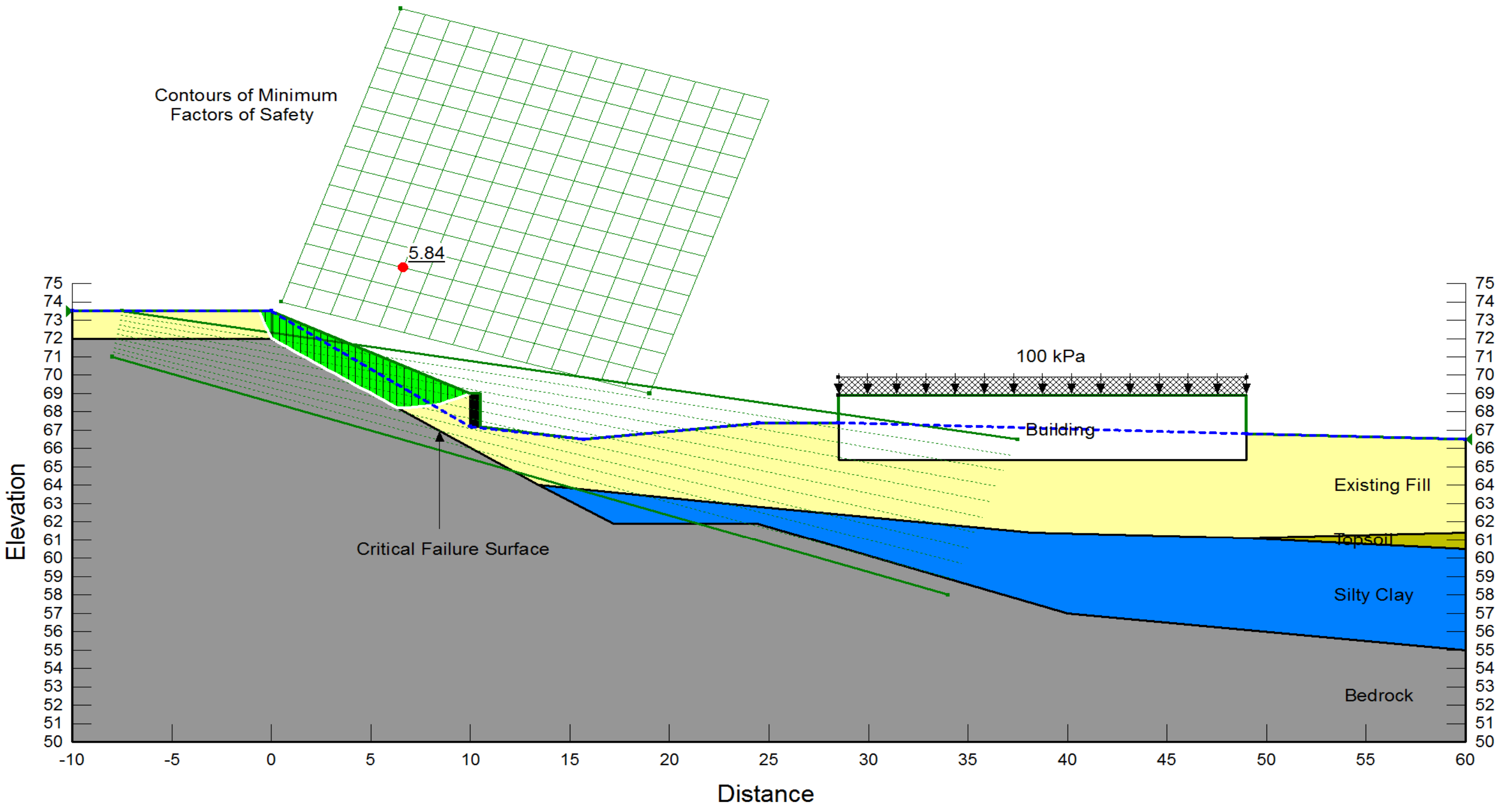
LEGEND

🟨	Existing Fill
🟫	Topsoil
🟦	Silty Clay
⬛	Bedrock

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 34

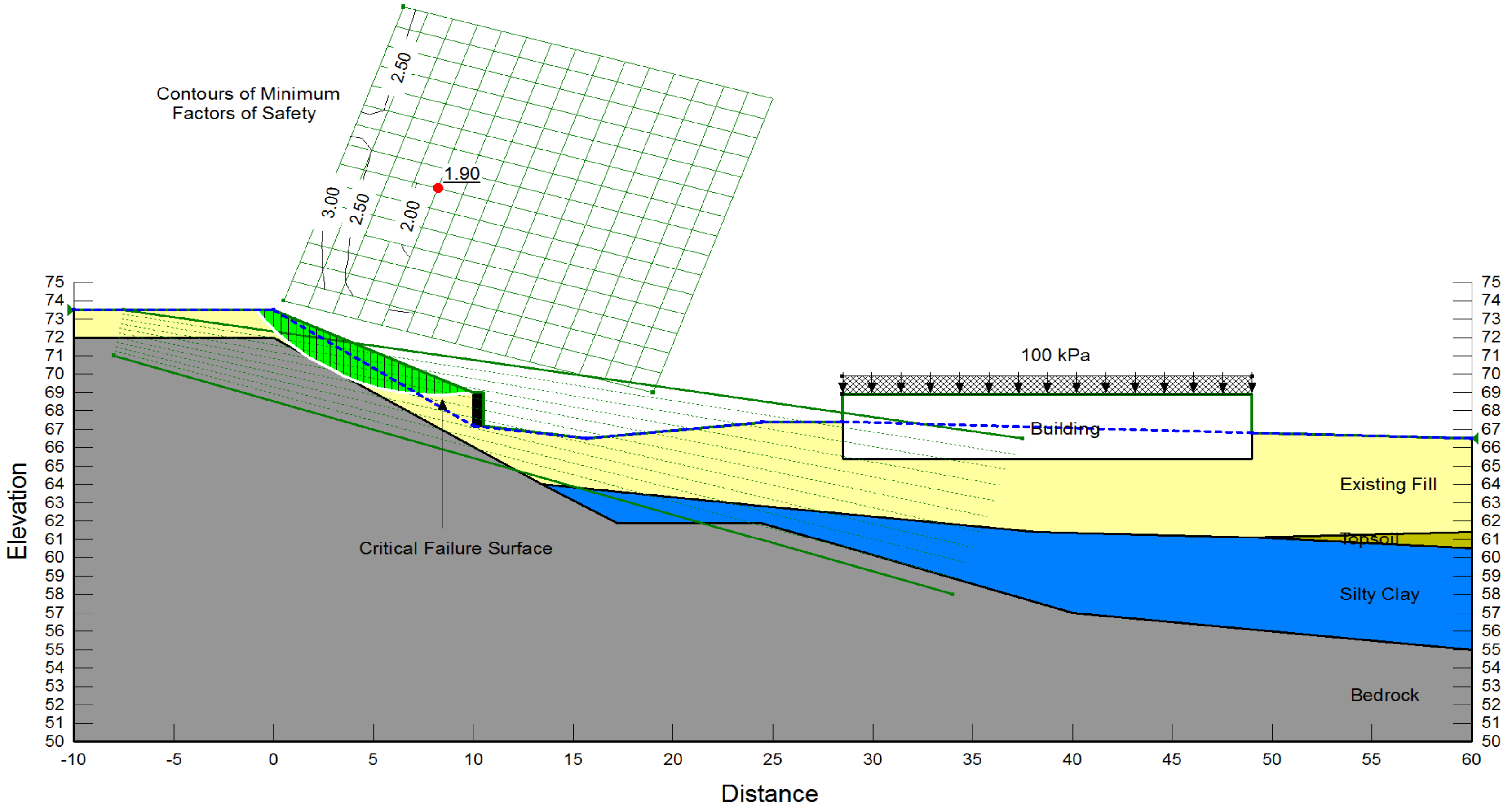


Section / Location : B-B
Original Slope : 2.2H:1V
Original Height : 7.0 m
Condition : End of Construction, Short Term
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

LEGEND

🟨 Existing Fill
🟫 Topsoil
🟦 Silty Clay
▒ Bedrock
■ Retaining Wall

STATIC SLOPE STABILITY ANALYSIS **OTT-00241432-A0**
Hillside Terrace Condos
Ottawa, Ontario **Figure No. 35**



Section / Location : B-B
Original Slope : 2.2H:1V
Original Height : 7.0 m
Condition : End of Construction, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

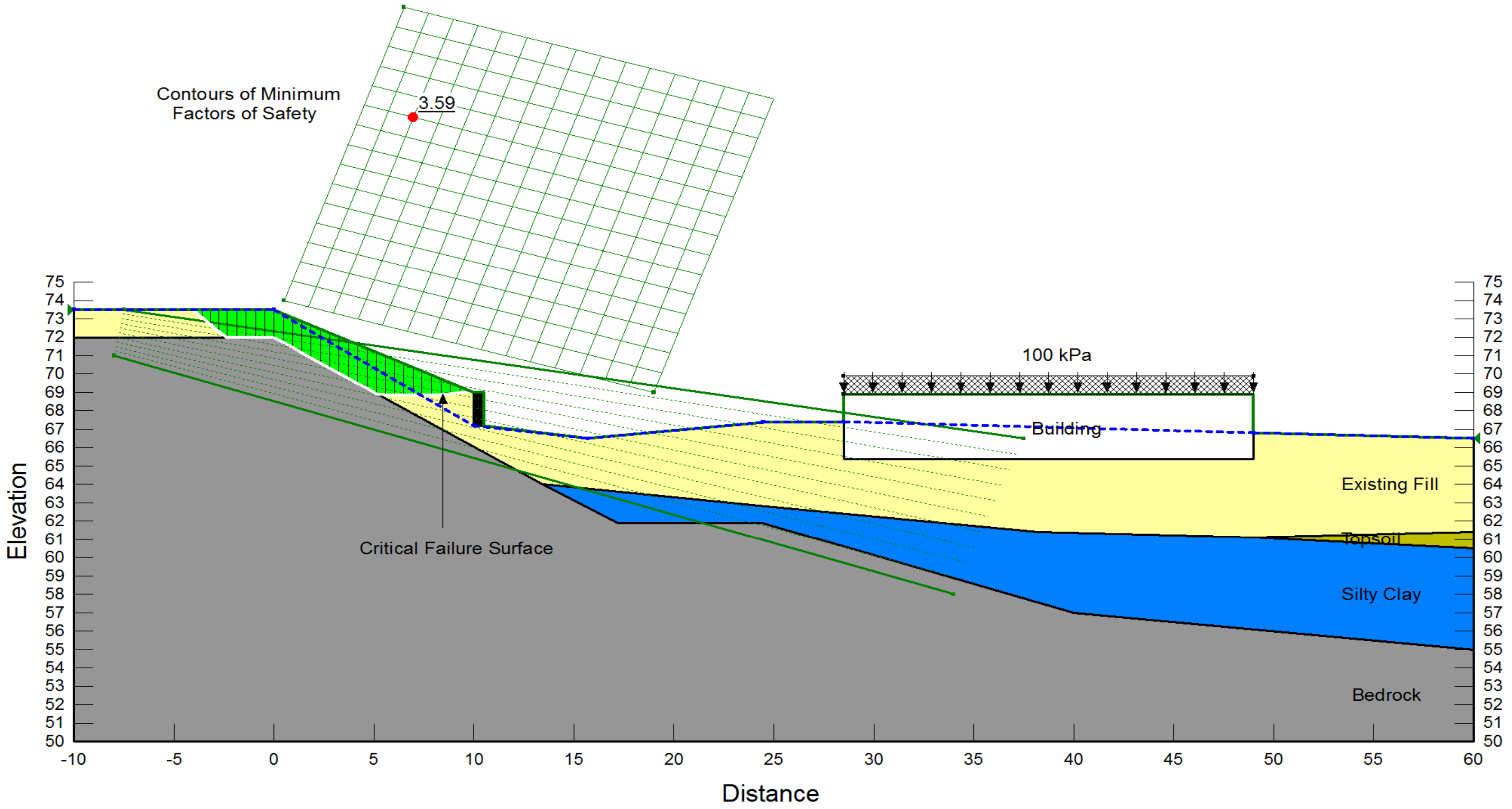
LEGEND

🟨	Existing Fill
🟫	Topsoil
🟦	Silty Clay
▩	Bedrock
⬛	Retaining Wall

PSEUDO-STATIC SLOPE STABILITY ANALYSIS

Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0
Figure No. 36



Section / Location : B-B
Original Slope : 2.2H:1V
Original Height : 7.0 m
Condition : End of Construction, Short Term, Seismic
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

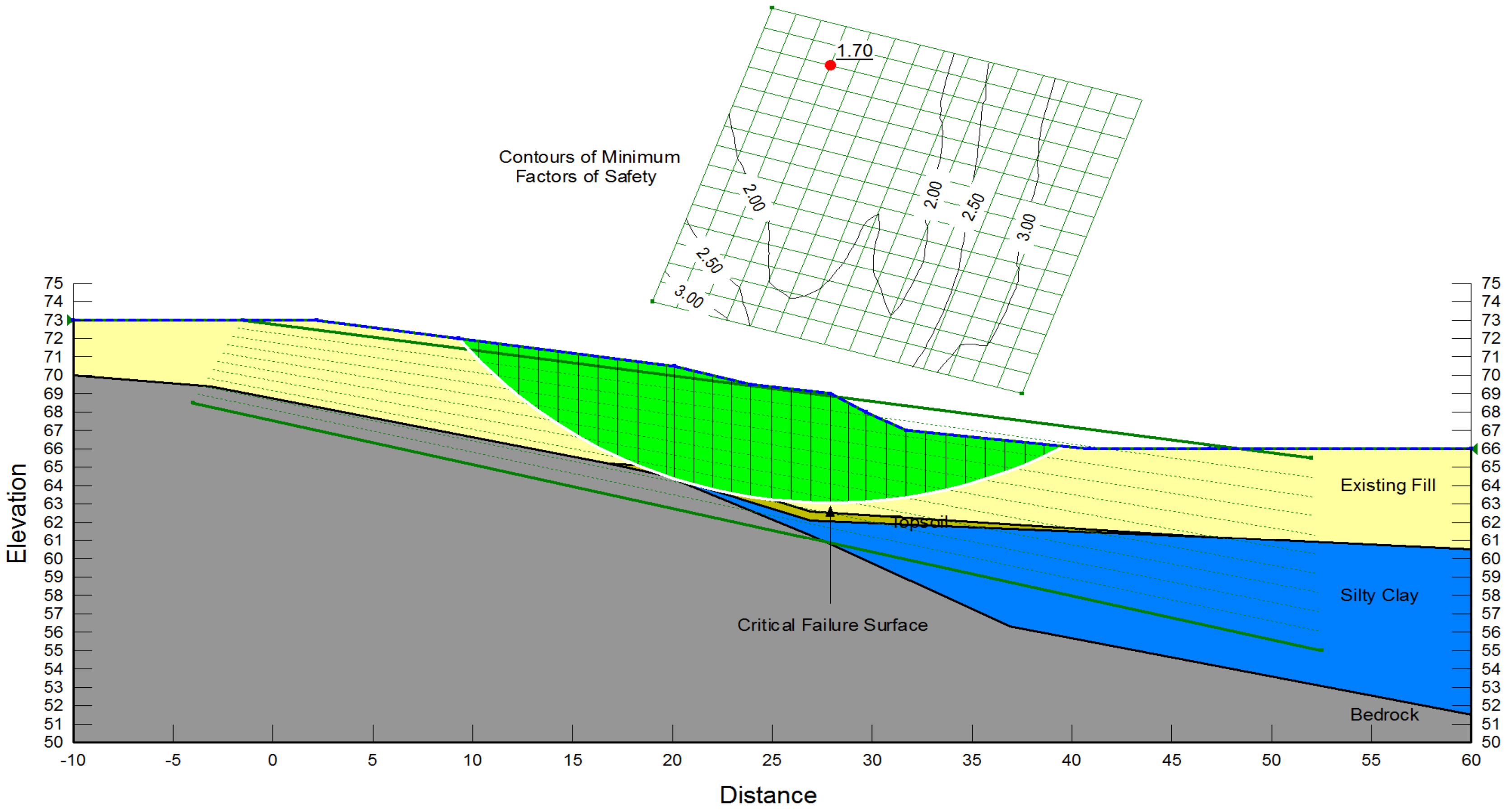
LEGEND

🟨 Existing Fill
🟫 Topsoil
🟦 Silty Clay
⬜ Bedrock
⬛ Retaining Wall

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 37



Section / Location : C-C
Original Slope : 5.5H:1V
Original Height : 7.0 m
Condition : Existing, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

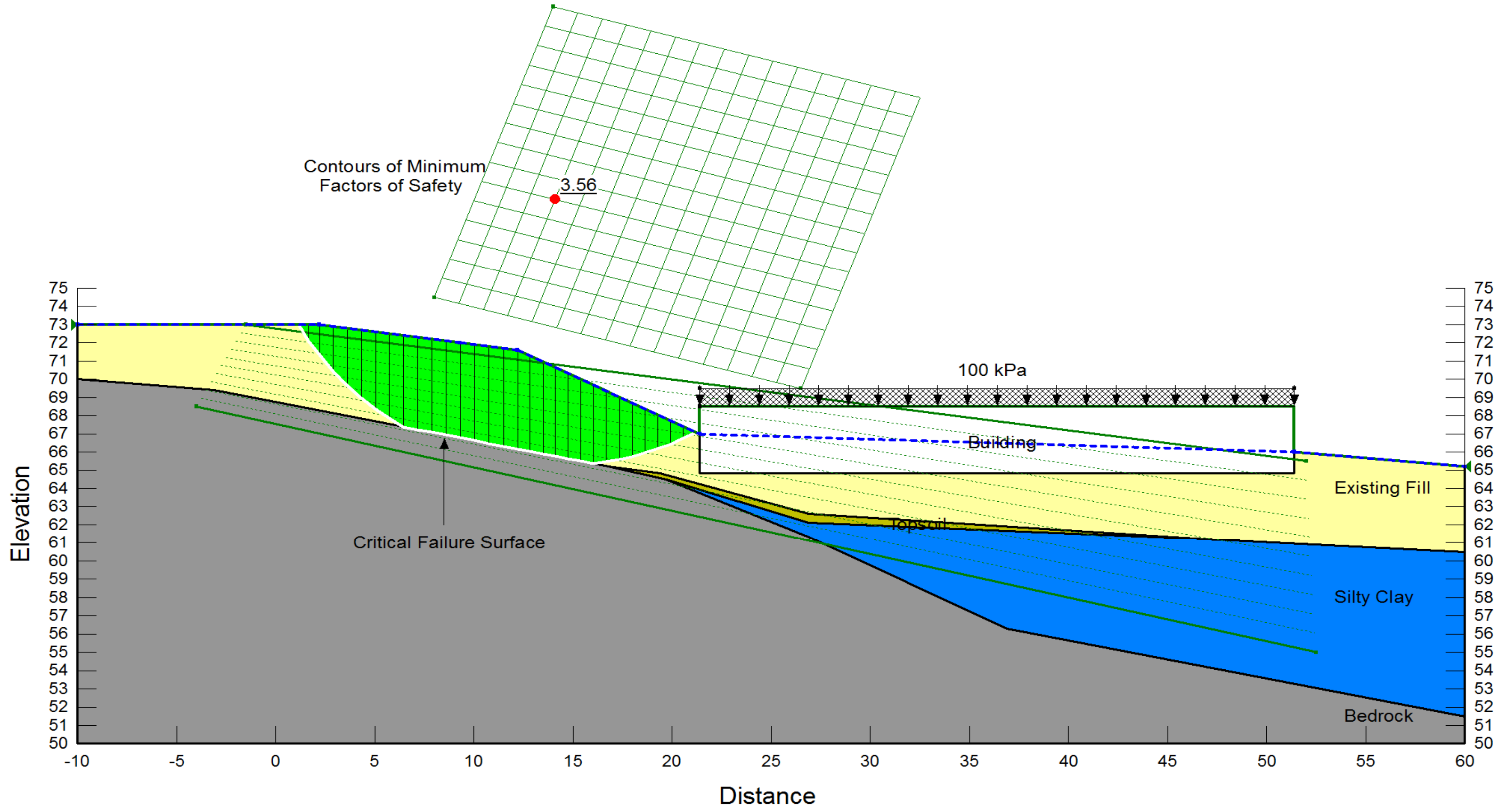
LEGEND

🟨 Existing Fill
🟫 Topsoil
🟦 Silty Clay
⬛ Bedrock

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 38



Section / Location : C-C
Original Slope : 5.5H:1V
Original Height : 7.0 m
Condition : End of Construction, Short Term
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

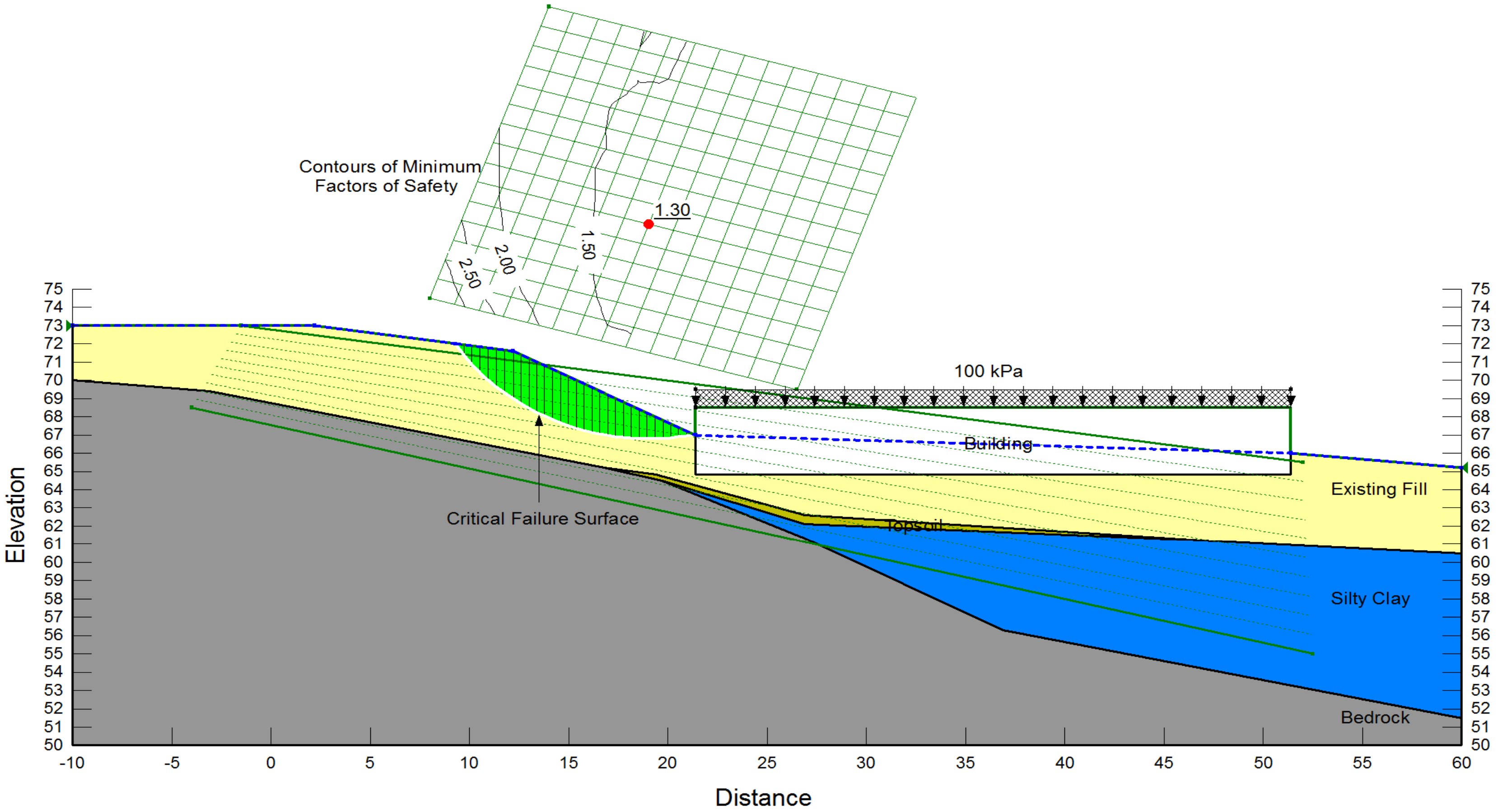
LEGEND

🟨	Existing Fill
🟫	Topsoil
🟦	Silty Clay
⬜	Bedrock

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 39



Section / Location : C-C
Original Slope : 5.5H:1V
Original Height : 7.0 m
Condition : End of Construction, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

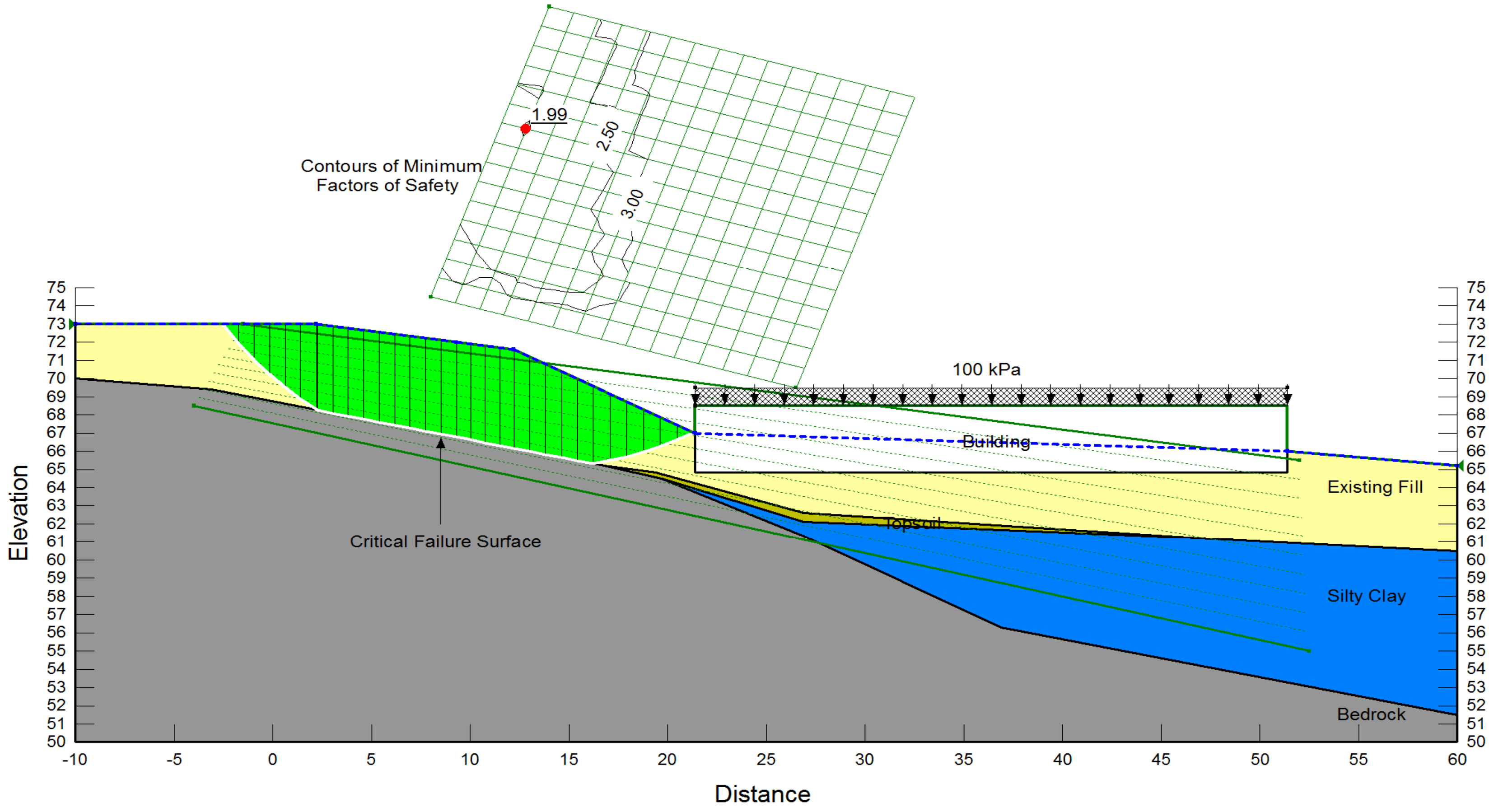
LEGEND

🟨 Existing Fill
🟫 Topsoil
🟦 Silty Clay
⬛ Bedrock

PSEUDO-STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 40



Section / Location : C-C
Original Slope : 5.5H:1V
Original Height : 7.0 m
Condition : End of Construction, Short Term, Seismic
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

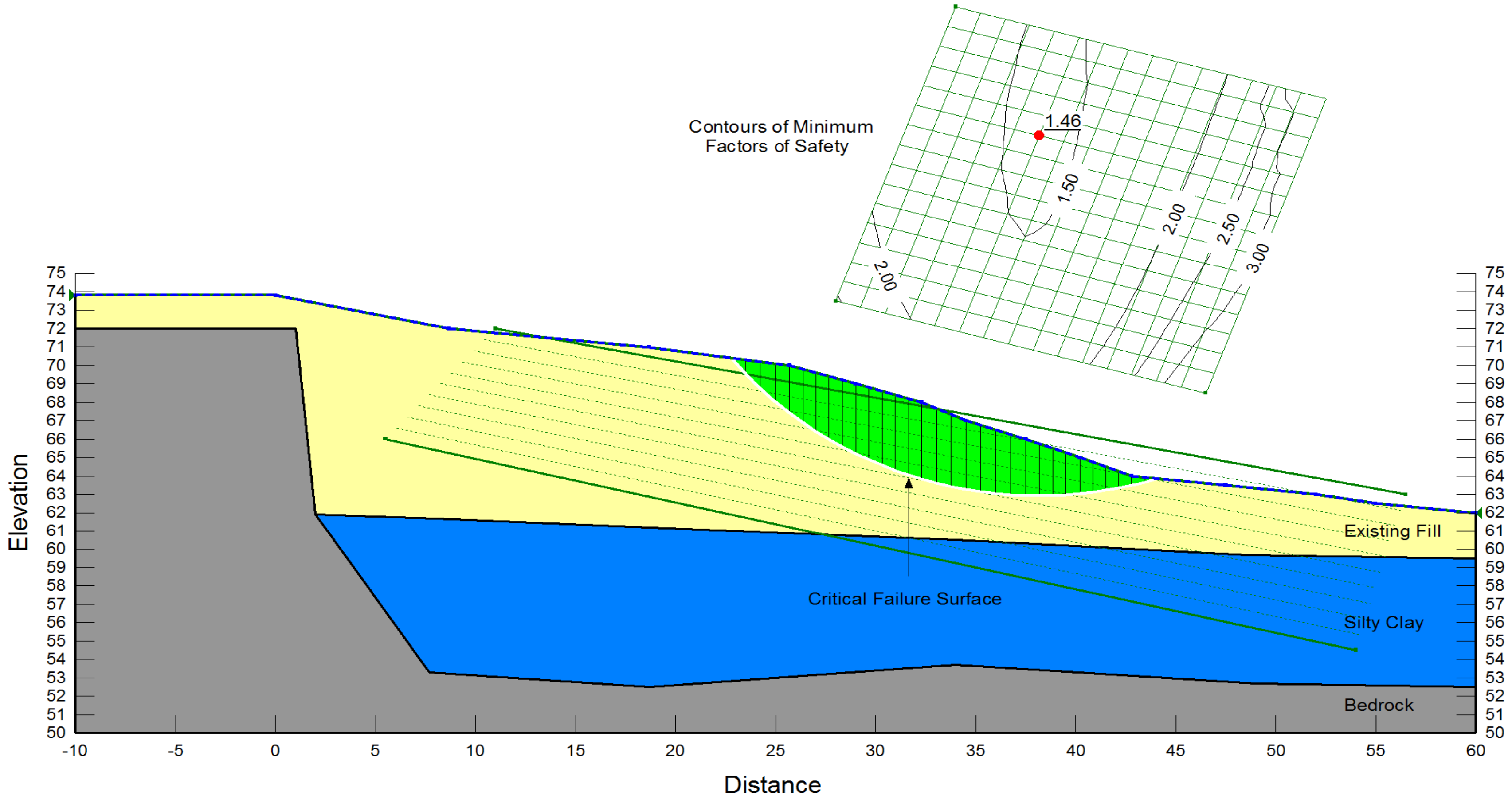
LEGEND

🟨	Existing Fill
🟫	Topsoil
🟦	Silty Clay
⬜	Bedrock

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 41



Section / Location : D-D
Original Slope : 4.4H:1V
Original Height : 9.8 m
Condition : Existing, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

LEGEND

🟨 Existing Fill
🟦 Silty Clay
⬛ Bedrock

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

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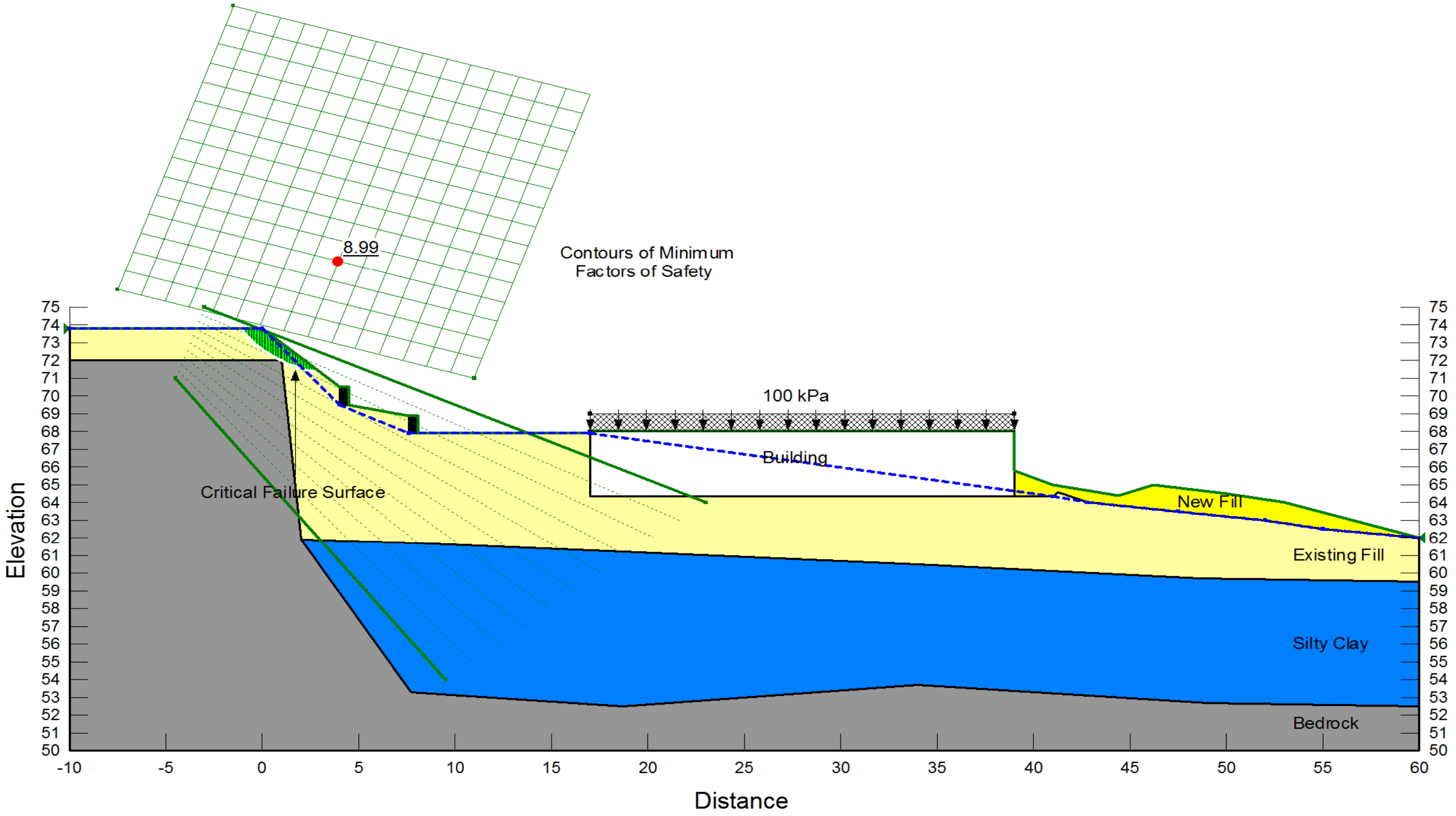
Figure No. 42



Section / Location : D-D
Original Slope : 4.4H:1V
Original Height : 9.8 m
Condition : End of Construction, Short Term
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

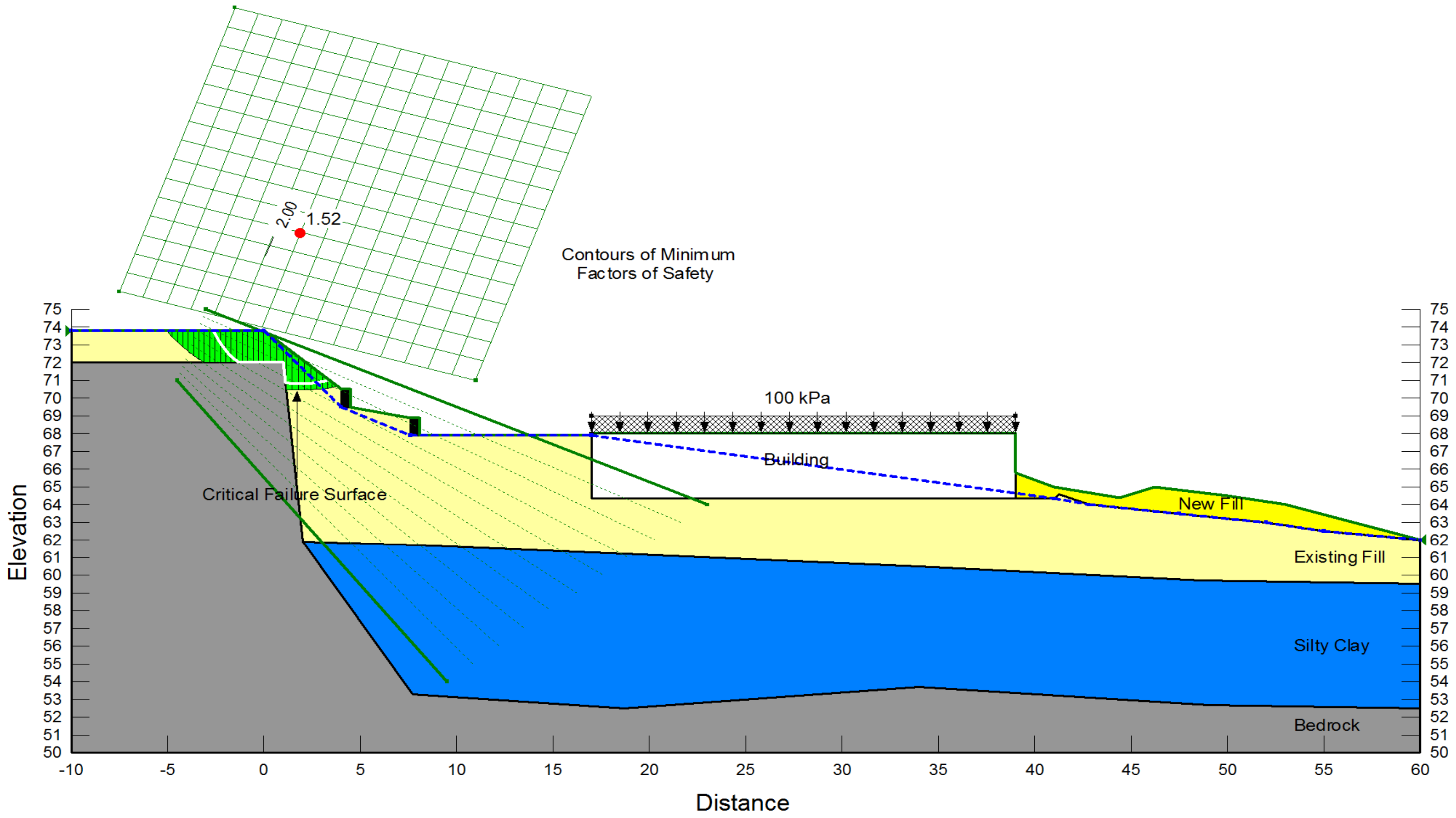
LEGEND

🟨 New Fill
🟨 Existing Fill
🟦 Silty Clay
⬛ Bedrock
⬛ Retaining Wall

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 43



Section / Location : D-D
Original Slope : 4.4H:1V
Original Height : 9.8 m
Condition : End of Construction, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

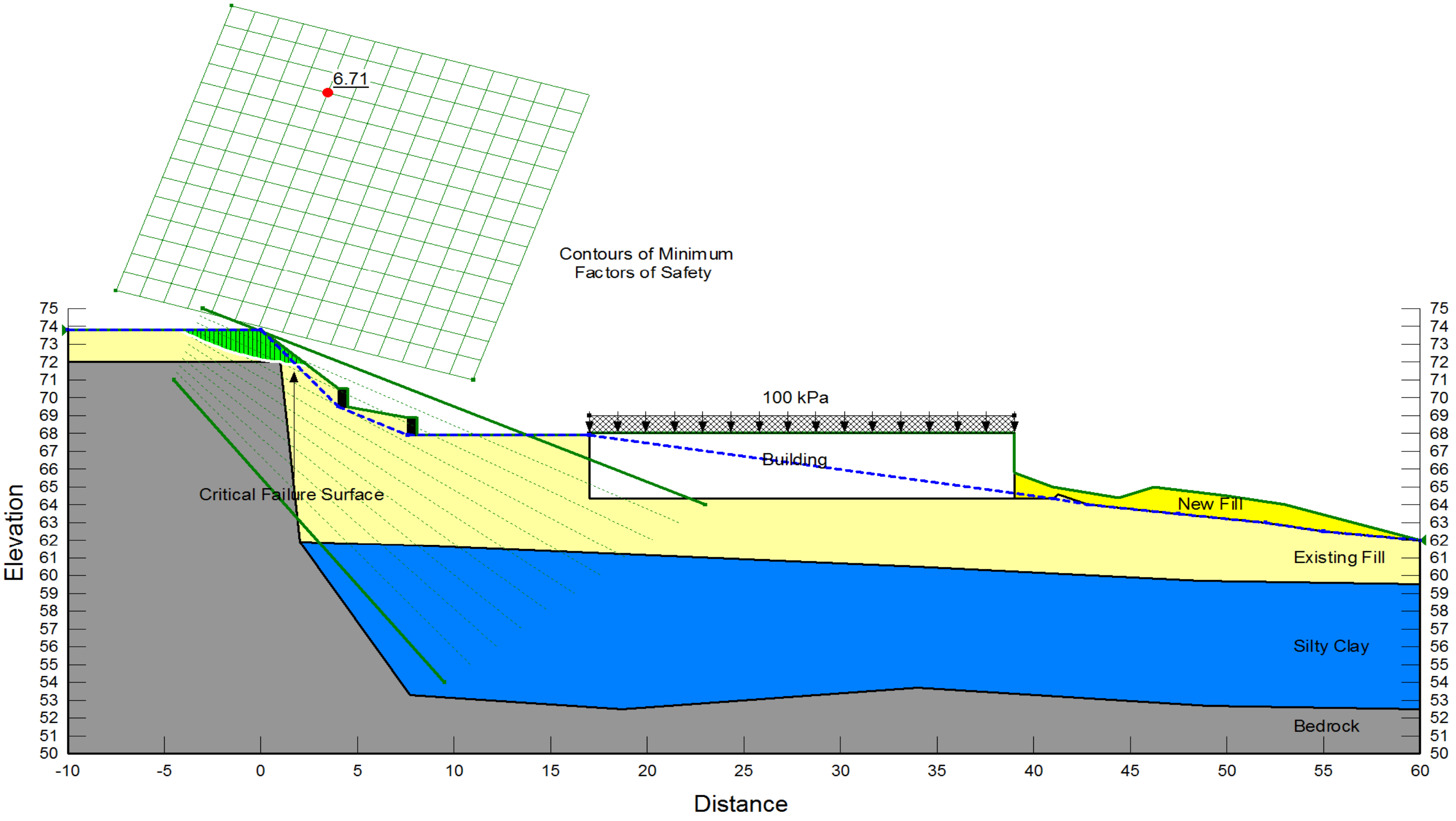
LEGEND

🟨 New Fill
🟨 Existing Fill
🟦 Silty Clay
⬛ Bedrock
⬛ Retaining Wall

PSEUDO-STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 44



Section / Location : D-D
Original Slope : 4.4H:1V
Original Height : 9.8 m
Condition : End of Construction, Short Term, Seismic
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

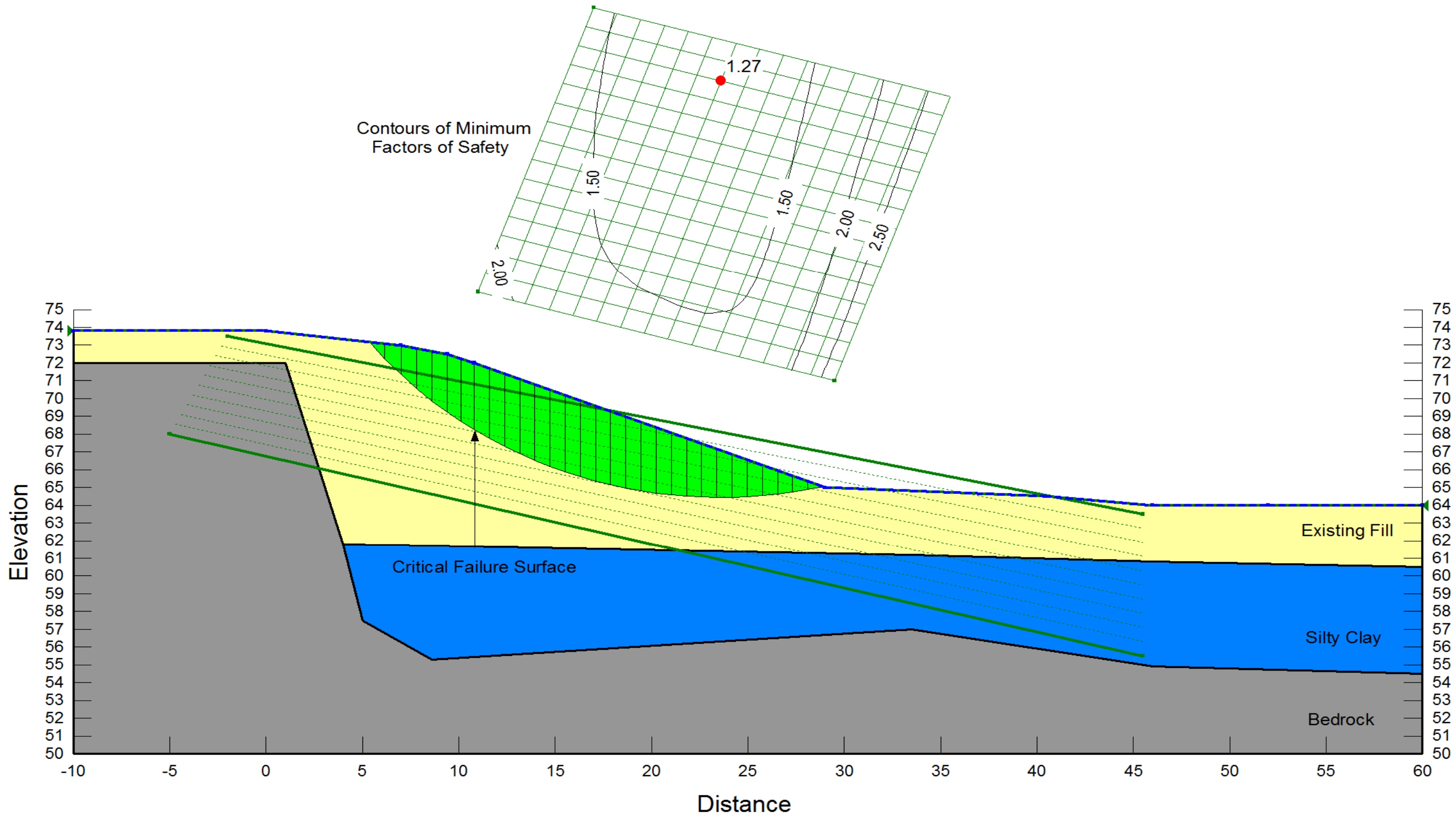
LEGEND

🟨 New Fill
🟨 Existing Fill
🟦 Silty Clay
◼ Bedrock
■ Retaining Wall

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 45



Section / Location : E-E
Original Slope : 3.3H:1V
Original Height : 8.8 m
Condition : Existing, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

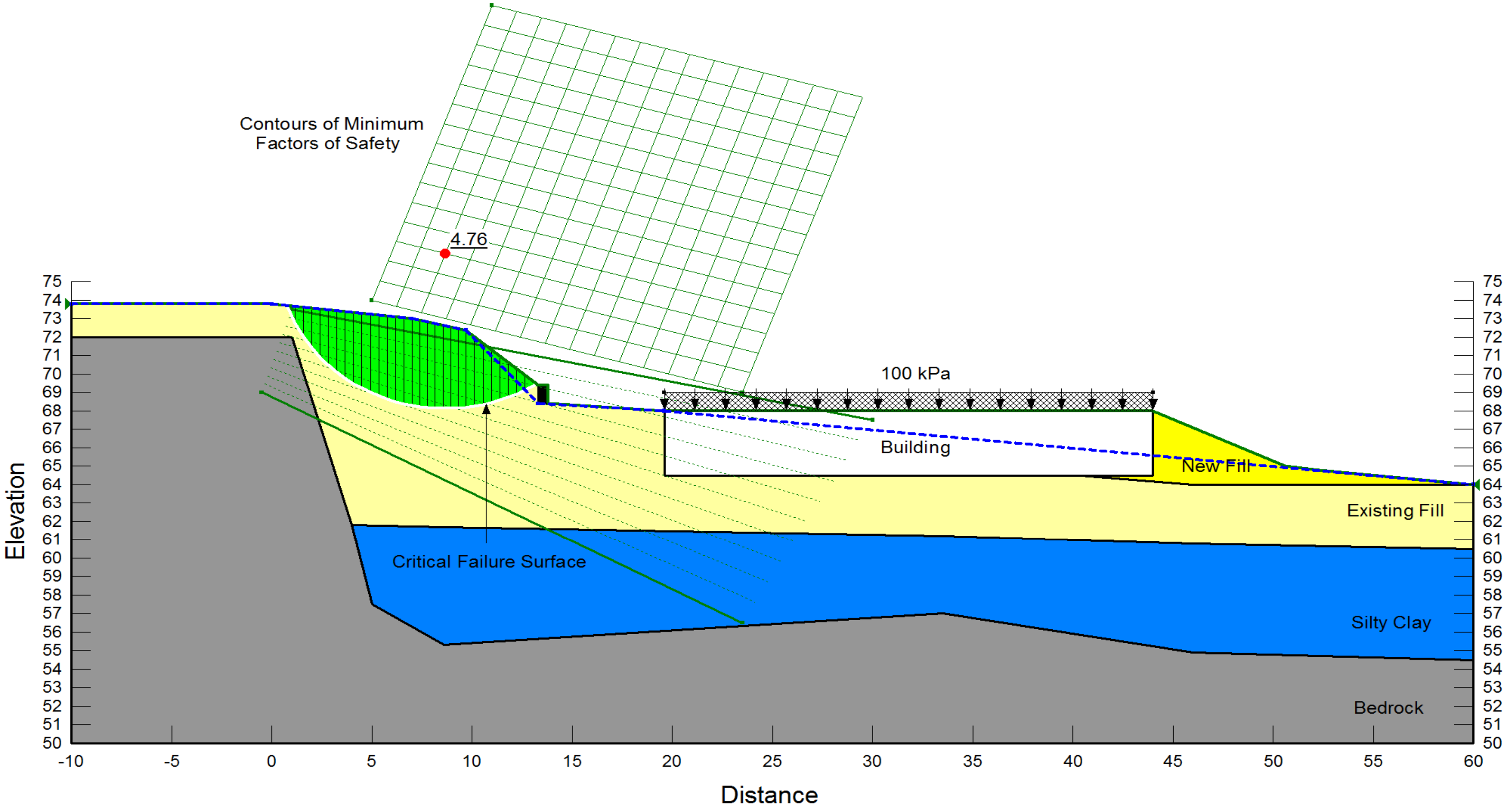
LEGEND

🟨 Existing Fill
🟦 Silty Clay
⬛ Bedrock

STATIC SLOPE STABILITY ANALYSIS **OTT-00241432-A0**

Hillside Terrace Condos

Ottawa, Ontario **Figure No. 46**



Section / Location : E-E

Original Slope : 3.3H:1V

Original Height : 8.8 m

Condition : End of Construction, Short Term

Water Table : At ground surface

Drained Condition : Undrained

Analysis Method : Morgenstern - Price

Surcharge : N.A.

LEGEND

🟨 New Fill

🟨 Existing Fill

🟦 Silty Clay

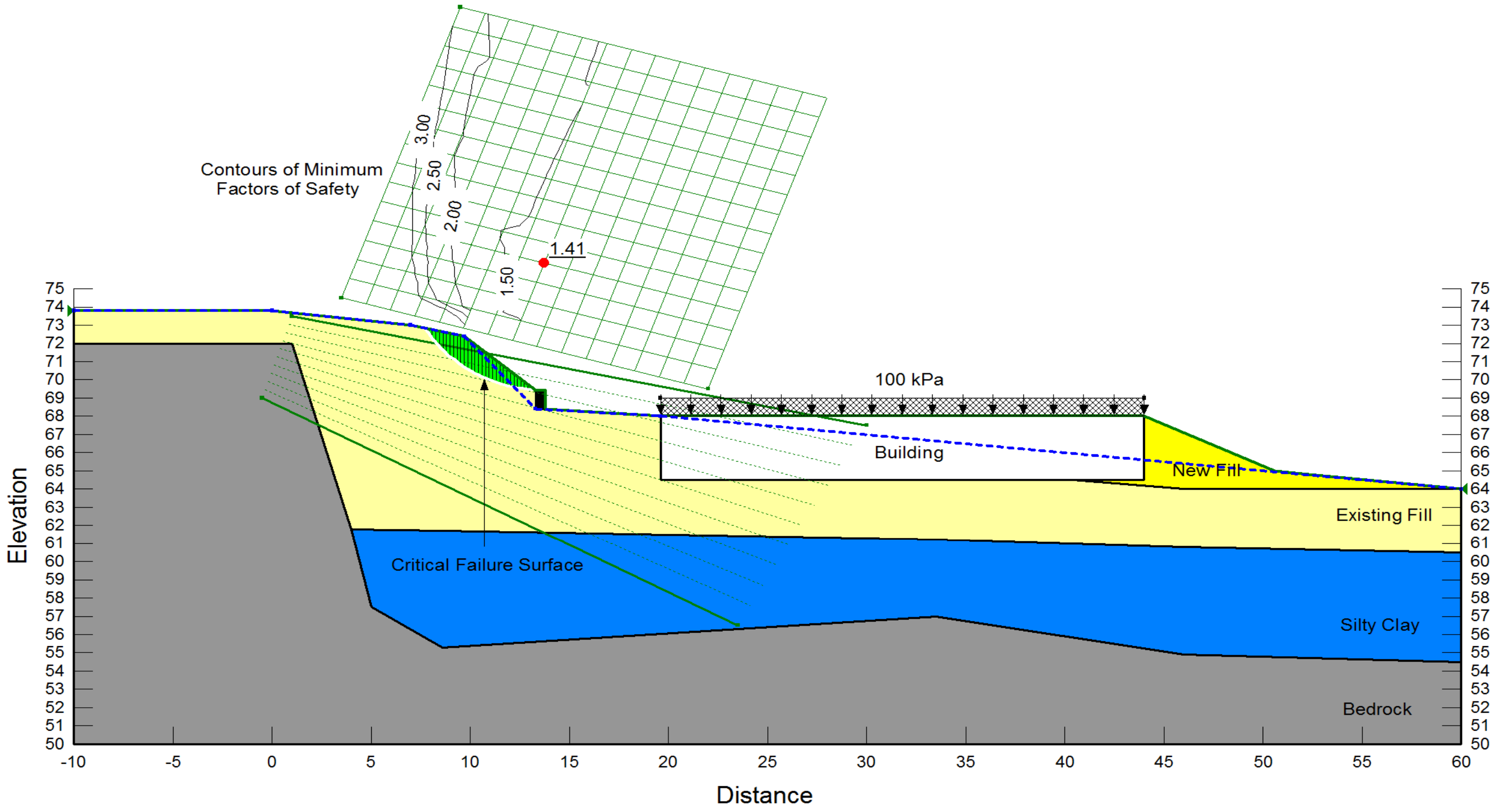
▨ Bedrock

⬛ Retaining Wall

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 47



Section / Location : E-E
Original Slope : 3.3H:1V
Original Height : 8.8 m
Condition : End of Construction, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

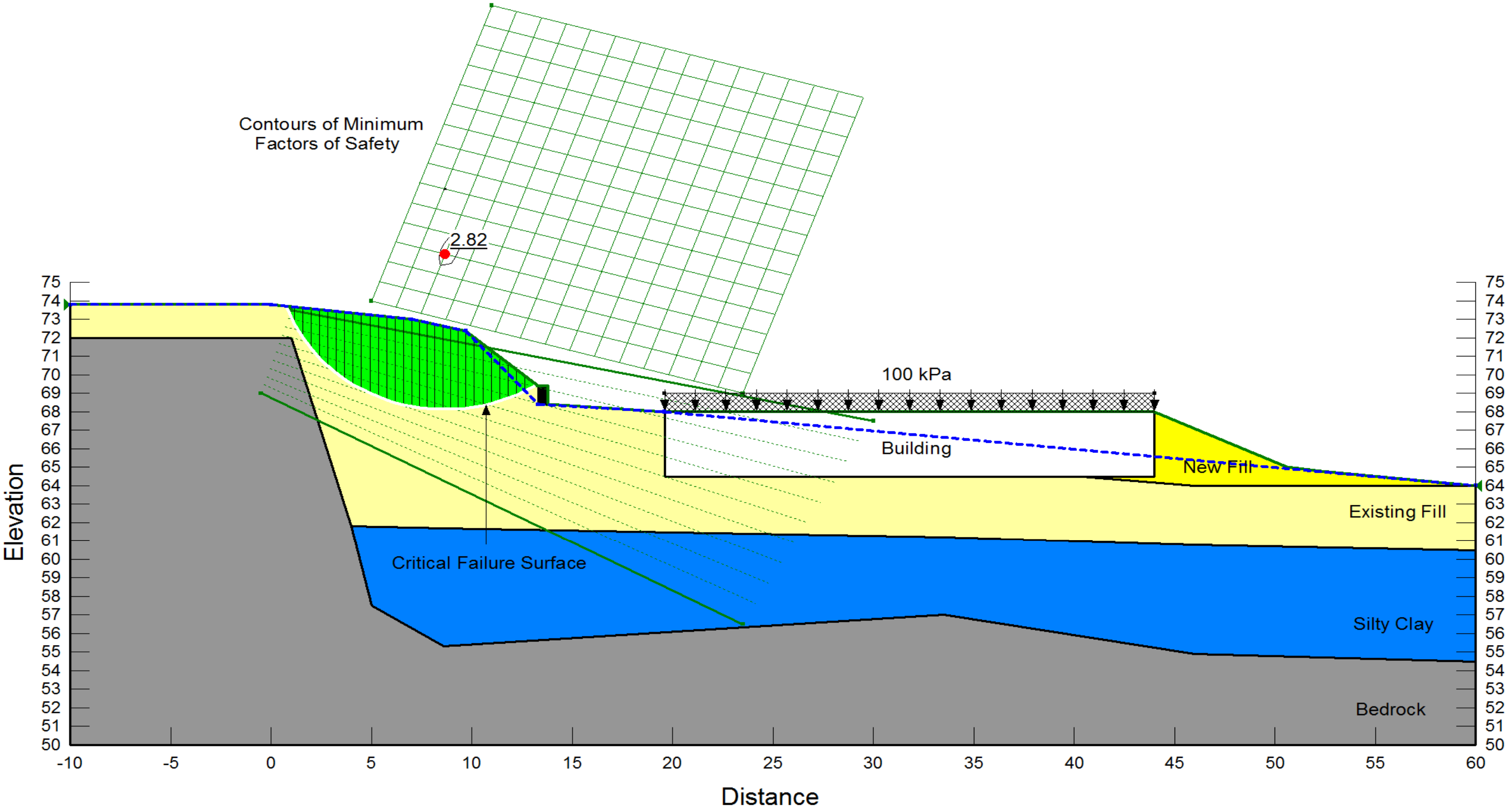
LEGEND

New Fill
Existing Fill
Silty Clay
Bedrock
Retaining Wall

PSEUDO-STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 48



Section / Location : E-E
Original Slope : 3.3H:1V
Original Height : 8.8 m
Condition : End of Construction, Short Term, Seismic
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

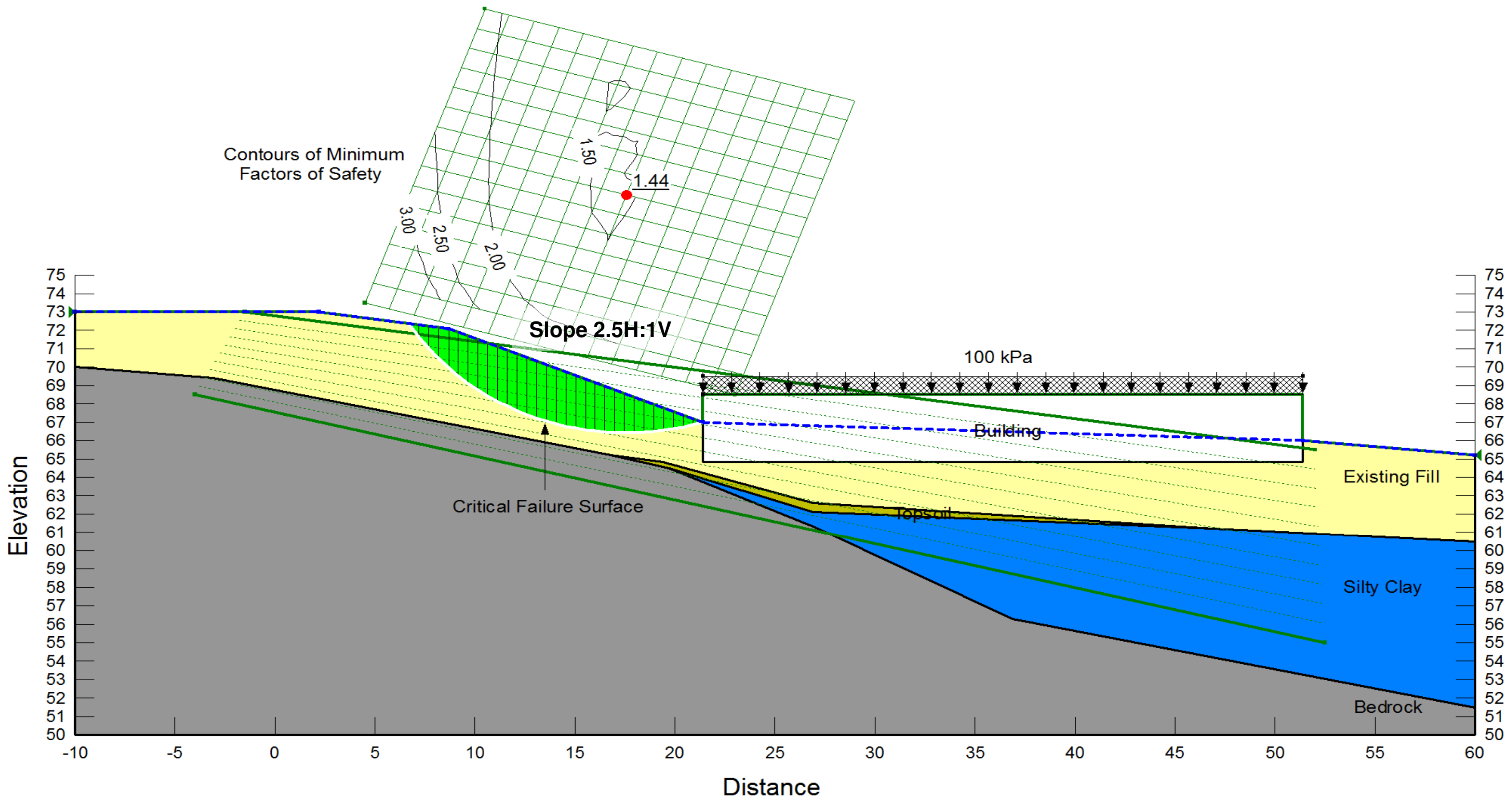
LEGEND

🟨	New Fill
🟨	Existing Fill
🟦	Silty Clay
⬛	Bedrock
⬛	Retaining Wall

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 49



Section / Location : C-C
Original Slope : 5.5H:1V
Original Height : 7.0 m
Condition : End of Construction, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

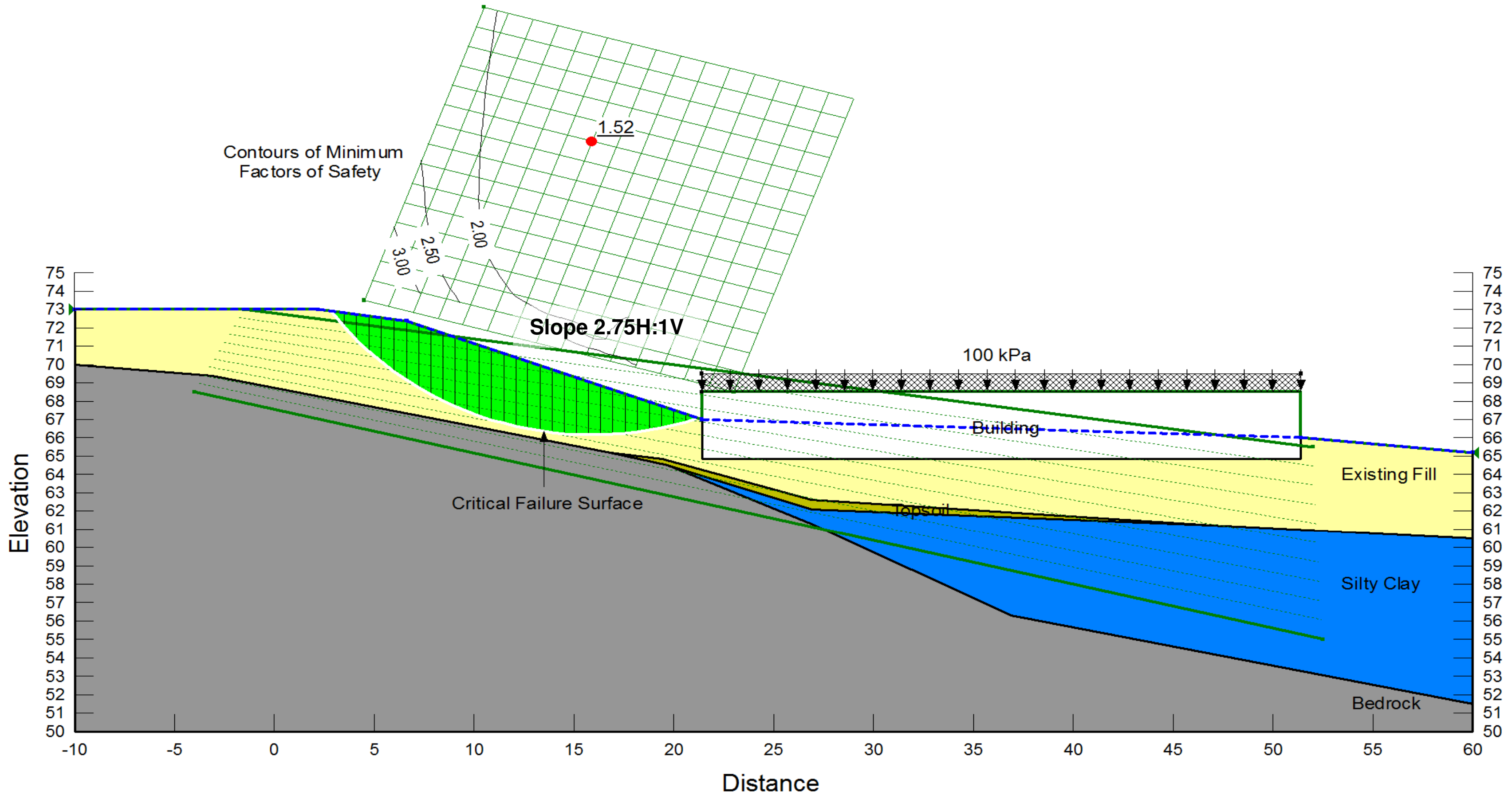
LEGEND

🟨 Existing Fill
🟫 Topsoil
🟦 Silty Clay
⬛ Bedrock

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 50



Section / Location : C-C
Original Slope : 5.5H:1V
Original Height : 7.0 m
Condition : End of Construction, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

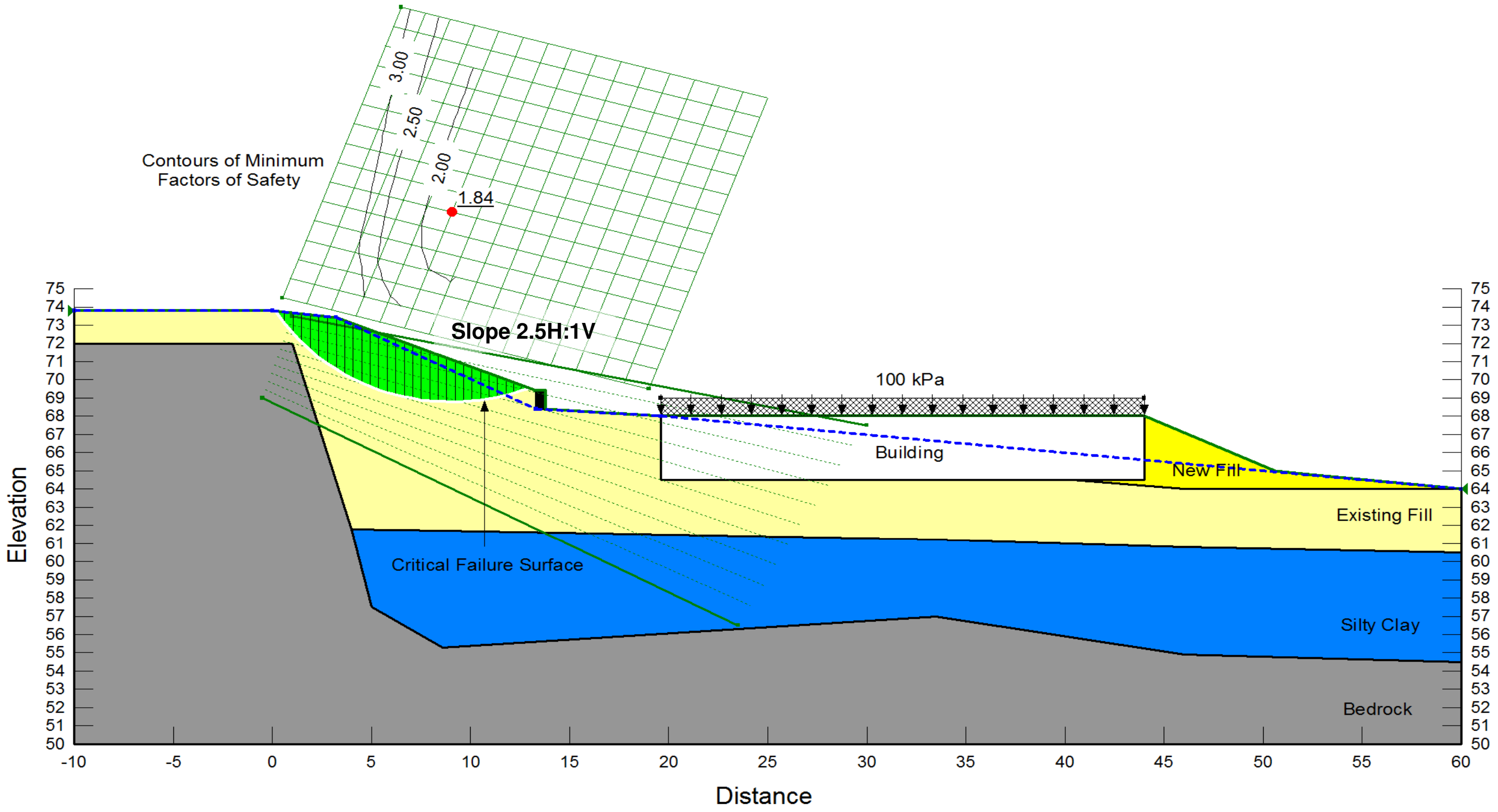
LEGEND

🟨 Existing Fill
🟫 Topsoil
🟦 Silty Clay
⬛ Bedrock

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 51



Section / Location : E-E
Original Slope : 3.3H:1V
Original Height : 8.8 m
Condition : End of Construction, Long Term
Water Table : At ground surface
Drained Condition : Drained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

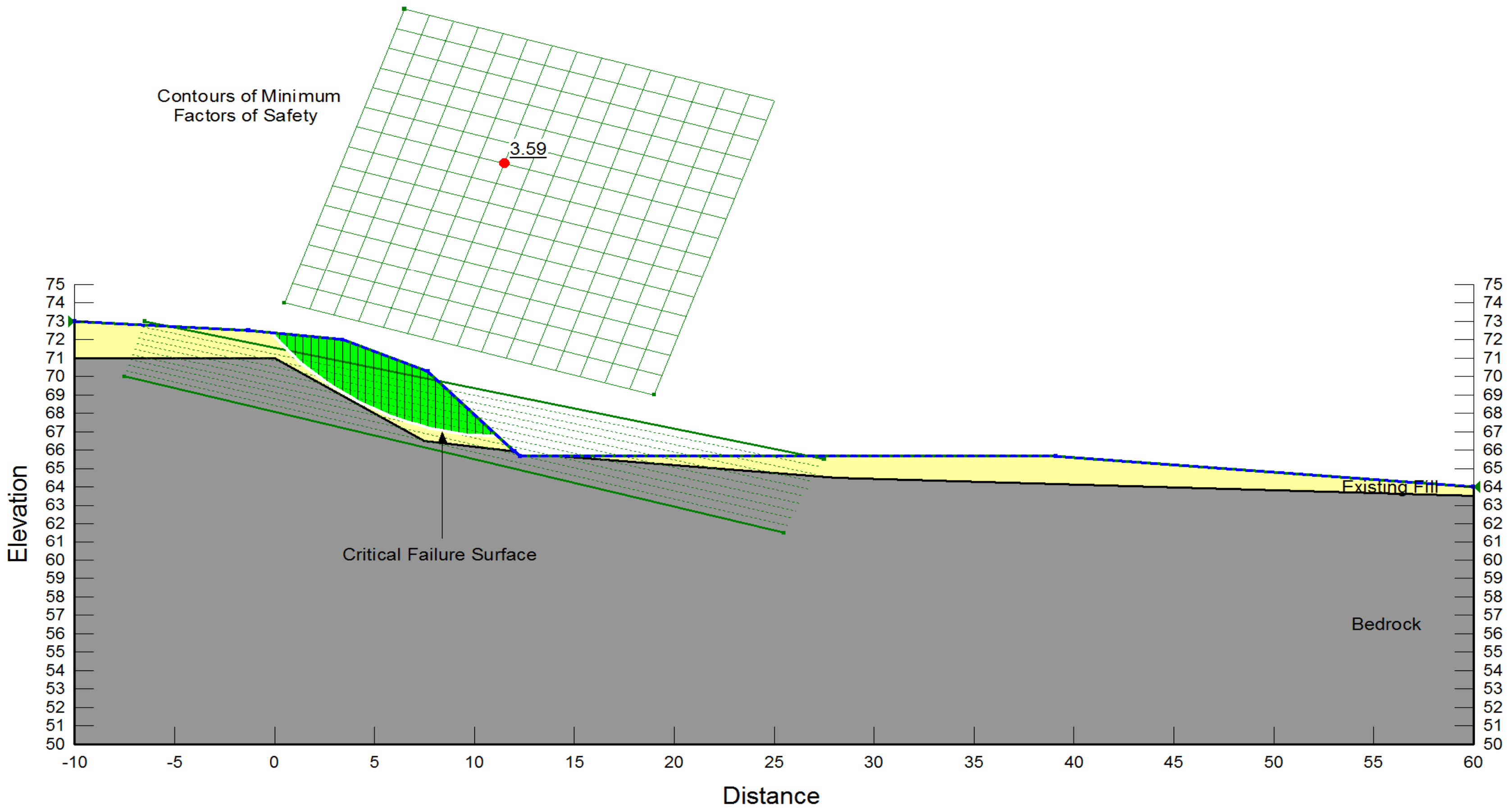
LEGEND

🟨	New Fill
🟨	Existing Fill
🟦	Silty Clay
⬜	Bedrock
⬛	Retaining Wall

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 52



Section / Location : A-A
Original Slope : 4.0H:1V
Original Height : 6.5 m
Condition : Excavation, Short Term
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

LEGEND

🟨	Existing Fill
⬛	Bedrock

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

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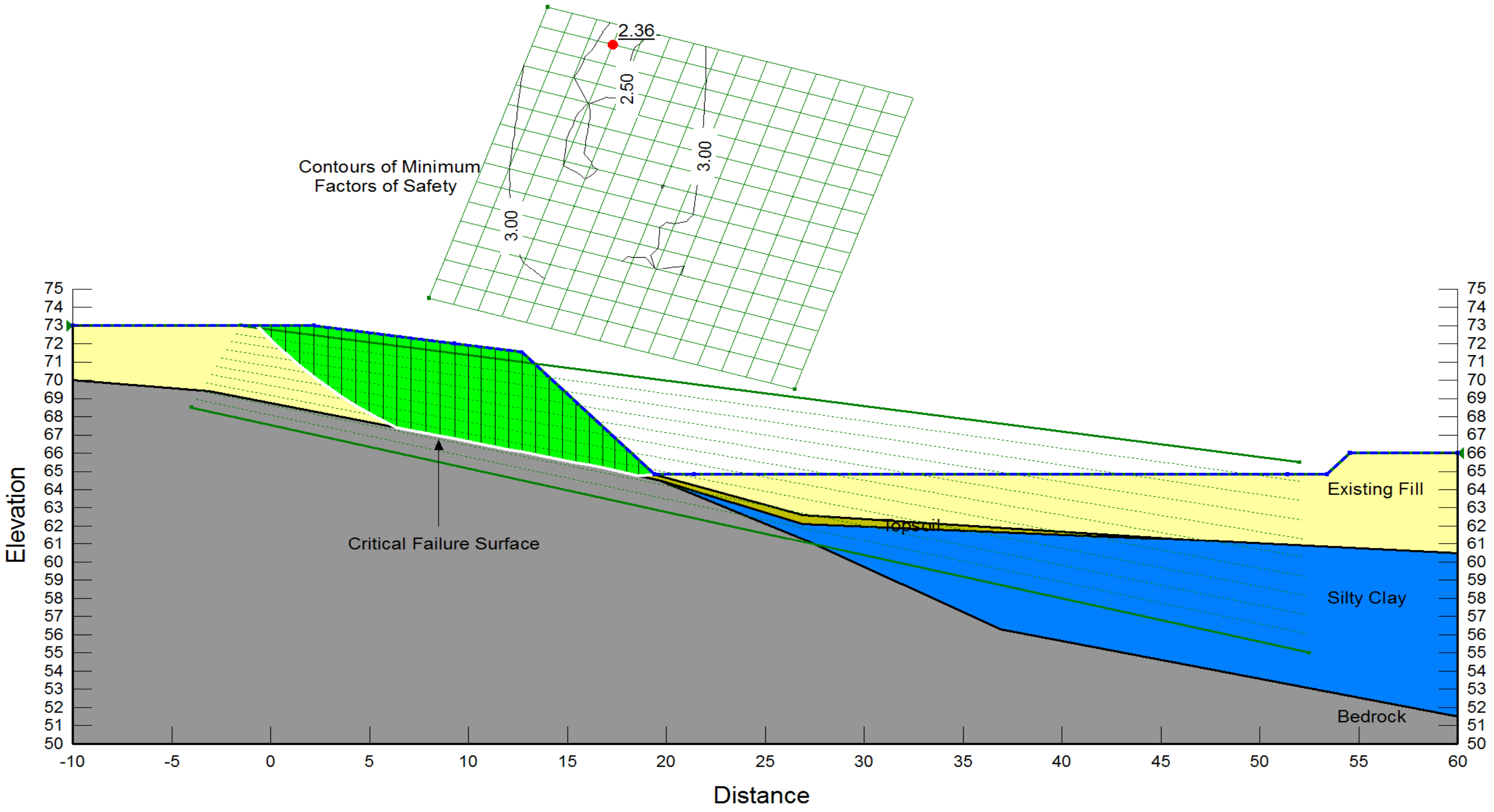
Figure No. 53



Section / Location : C-C
Original Slope : 5.5H:1V
Original Height : 7.0 m
Condition : Excavation, Short Term
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

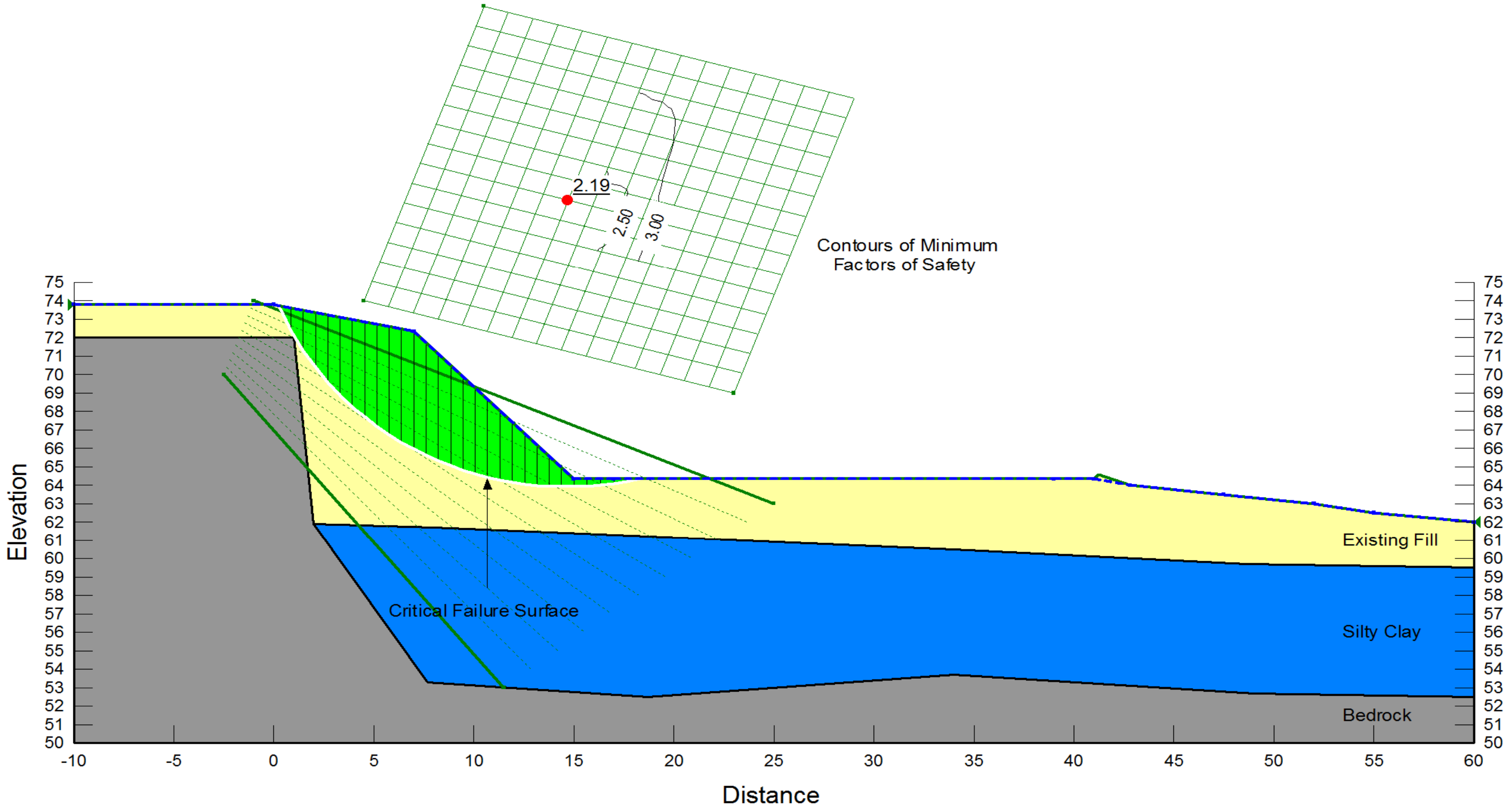
<u>LEGEND</u>

🟨 Existing Fill
🟫 Topsoil
🟦 Silty Clay
⬛ Bedrock

STATIC SLOPE STABILITY ANALYSIS
Hillside Terrace Condos
Ottawa, Ontario

OTT-00241432-A0

Figure No. 54

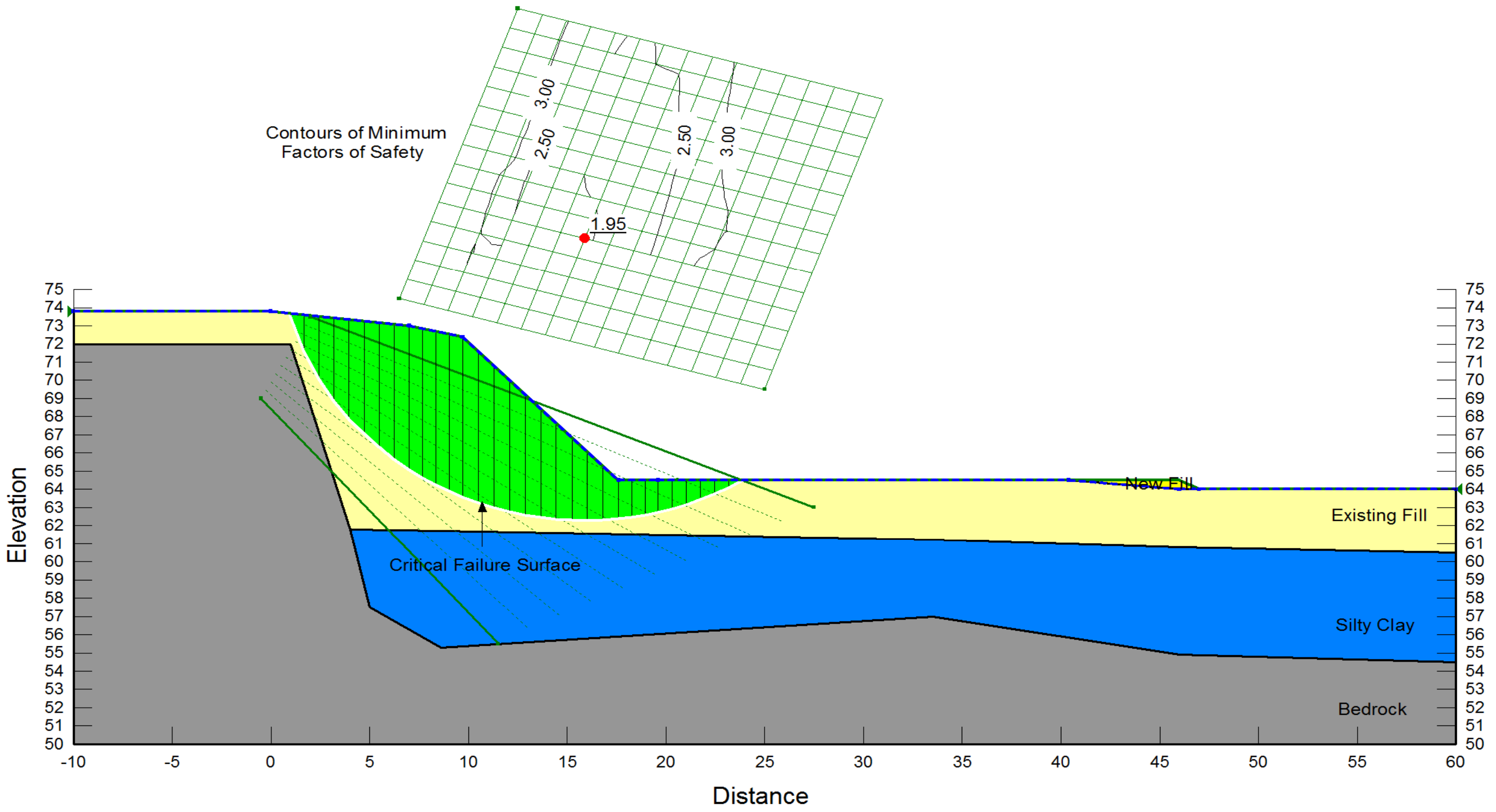


Section / Location : D-D
Original Slope : 4.4H:1V
Original Height : 9.8 m
Condition : Excavation, Short Term
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

LEGEND

🟨 Existing Fill
🟦 Silty Clay
⬛ Bedrock

STATIC SLOPE STABILITY ANALYSIS **OTT-00241432-A0**
Hillside Terrace Condos
Ottawa, Ontario **Figure No. 55**



Section / Location : E-E
Original Slope : 3.3H:1V
Original Height : 8.8 m
Condition : Excavation, Short Term
Water Table : At ground surface
Drained Condition : Undrained
Analysis Method : Morgenstern - Price
Surcharge : N.A.

LEGEND

🟨 Existing Fill
🟨 New Fill
🟦 Silty Clay
⬛ Bedrock

List of Distribution

Report Distributed To:

Mr. Michael Boucher - mboucher@phoenixhomes.ca

Mr. Greg Winters - G.Winters@novatech-eng.com

Mr. Drew Blair - D.Blair@novatech-eng.com