Geotechnical Investigation - Slope Stability Assessment

2164 Old Prescott Road,
Ottawa, ON

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1.0 INTRODUCTION

This report presents the results of a Slope Stability and Geotechnical Assessment carried out for the proposed residence at 2164 Old Prescott Road in Greely, ON. The location of the site is shown on the Key Plan, Drawing No. 1 in Appendix B.

The work was carried out in general accordance with Stantec Consulting Ltd. (Stantec) Proposal No. 160410204 dated April 10, 2018.

Limitations associated with the contents of this report are provided in the Statement of General Conditions included in Appendix A.

2.0 BACKGROUND

It is understood that the proposed development includes two structures; a residence and an office storage building on the property located at 2164 Old Prescott Road. The proposed building locations are on the north side of Old Prescott Road, there is a former sand pit located north of the site. The proposed footprint of the residence is approximately 495 m^2 and the proposed footprint of the office is approximately 265 m^2. It is assumed that the residence is a two-storey residential house with one below grade level and that the office is a single-storey structure with no below grade levels. It is assumed that both buildings will be designed to include either strip or spread footings.

The two structures are located near the crest of the slope that extends down to the pit. Based on the survey information, the elevation at the crest of the slope is approximately 97.0 m and the elevation at the toe of the slope near the water's edge is approximately 89.6 m.

As indicated in the 2005 historical air photo of the site, the site was previously used as the access to the sand pit. As shown on Drawing No. 4 in Appendix B, portions of the site have previously been excavated while the sand pit was in use and have since been backfilled.

3.0 SCOPE OF WORK

The scope of work for the geotechnical assessment included the following:

- Advancing four (4) boreholes; one in the footprint of each proposed structure, one near the crest of the slope and one near the toe of the slope.
- Installing one (1) monitoring well.
- Carrying out Standard Penetration Tests (SPT) at regular intervals in the boreholes to collect soil samples.
- Completing a geotechnical laboratory testing program to characterize the soil.

- Preparing a geotechnical investigation report with geotechnical engineering recommendations for the two proposed structures and the slope.

4.0 METHOD OF INVESTIGATION

Prior to carrying out the investigation, Stantec Consulting Ltd. (Stantec) personnel marked out the proposed borehole locations at the site. As a component of our standard procedures and due diligence, Stantec arranged to have the borehole locations cleared of both private and public underground utilities prior to drilling.

The field drilling program was carried out on May 29, 2018. The four boreholes were advanced at the locations shown on Drawing No. 2 in Appendix B, with a track mounted CME 75 drill rig. The subsurface stratigraphy encountered in each borehole was recorded in the field by experienced Stantec personnel while performing Standard Penetration Tests (SPT). Split spoon samples were collected at regular depth intervals in the boreholes. All recovered soil samples were stored in moisture-proof bags and returned to the Stantec Ottawa Laboratory for further classification and testing

One 50 mm diameter monitoring well was installed within MW18-2. The monitoring well consisted of a screen from 6.0 m to 3.0 m below ground surface, silica sand from 6.0 m to 2.6 m, followed by a bentonite seal to ground surface.

Following drilling, all boreholes were backfilled with auger cuttings. Samples were returned to the laboratory and subjected to detailed visual examination and additional classification by a geotechnical engineer.

4.1 SURVEYING

The ground surface elevation at each borehole location and the ground surface elevations across the site were surveyed by Stantec registered land surveyors. Geodetic elevations at the borehole locations and ground elevation contours are shown on Drawing No. 2 in Appendix B and on the Borehole Records in Appendix C.

4.2 LABORATORY TESTING

All samples returned to the laboratory were subjected to detailed visual examination and classification by a geotechnical engineer. Moisture content determination was conducted on all soil samples and select soil samples were also submitted for grain size analysis. The results of the laboratory tests are provided in the Borehole Records in Appendix C, and the figures included in Appendix D. The samples will be stored for a period of one (1) month after the issuance of this report, unless otherwise directed by the client.

5.0 RESULTS OF INVESTIGATION

5.1 SITE RECONNAISSANCE

A site visit was carried out to observe the condition of the slope.

The site photographs are presented in Appendix C, and site observations are summarized below.

- Long grass and shrubs are present on the slope near the proposed office.
- The slope immediately above the waterline, for a height of approximately 1.0 to 1.2 m, is steeper, poorly vegetated, and lined with sparsely placed boulders.
- The slope near the proposed residence is a grassed area with several trees near the toe of the slope.
- No evidence of slope failure was observed at the site.

5.2 SUBSURFACE INFORMATION

In general, the subsurface profile at the site consisted of a layer of fill over a sand deposit with varying amounts of silt and gravel over inferred bedrock.

The subsurface conditions and results of the laboratory tests performed on soil samples are presented in the Borehole Records provided in Appendix C. An explanation of the symbols and terms used in the Borehole Records is also provided.

A summary of the observed subsurface conditions is provided below.

5.2.1 Surficial Materials

Topsoil was encountered at ground surface, the topsoil varied from 100 mm to 150 mm in thickness.

5.2.2 Fill

Fill was encountered beneath the topsoil, the thickness of the fill varied from 0.5 m to 6.7 m. The fill consisted of silty sand with gravel to silty clay with gravel. Construction debris was observed in the fill in MW18-2. The deepest fill was observed in borehole BH18-3 which is at the toe of the slope near the former sand pit.

The SPT-N values varied from 4 to 26 indicating a loose to compact state.

The moisture content of the fill ranged from 9% to 26%.

One sample of the fill was chosen for grain size analysis and the results are summarized below. The grain size distribution curve is shown on Figure No. 1 in Appendix D.

- Gravel: 4-18%
- Sand: 61-86%
- Fines (silt and clay size particles): 11-21%

According to the Unified Soil Classification System (USCS), the fill can be classified as silty sand with gravel (SM) or poorly graded sand with silt (SP-SM).

5.2.3 Sand

A deposit of sand with varying amounts of silt and gravel was encountered beneath the fill.

The SPT-N values varied from 6 to 39, indicating a loose to dense state.

The moisture content of the sand ranged from 3% to 17%.

Eight representative samples of the material were chosen for grain size analysis; the results are summarized below. The grain size distribution curves are shown on Figure No. 2 in Appendix D.

- Gravel: 0-29%
- Sand: 53-90%
- Fines: 7-21%

According to the USCS, the soil can be classified as well-graded sand with silt (SW-SM), poorly graded sand with silt (SP-SM), silty sand (SM) and silty sand with gravel (SM).

5.2.4 Sandy Silt

A sandy silt deposit was encountered below the sand in borehole BH18-4. The silt was encountered at a depth of 8.5 m (elevation 88.4 m) and extended for 1.2 m before the borehole was terminated.

The SPT-N values varied from 5 to 22, indicating a loose to compact state.

The moisture content of the sandy silt ranged from 22% to 21%.

One representative sample of the material was chosen for grain size analysis; the results are summarized below. The grain size distribution curve is shown on Figure No. 3 in Appendix D.

5.2.5 Till

Till was encountered below the sand in borehole BH18-3. The till was encountered at a depth of 7.9 m (elevation 82.2 m) and extended to at least 1.1 m where the borehole was terminated.

The SPT-N values varied from 22 to 29, indicating a compact state.

The moisture content of the till ranged from 9% to 14%.

One representative sample of the material was chosen for grain size analysis; the results are summarized below. The grain size distribution curve is shown on Figure No. 4 in Appendix D.

5.3 GROUNDWATER

The groundwater level was measured at 5.2 m below ground surface, corresponding to an elevation of 89.2 m on June 22, 2018. This groundwater level corresponded closely to the open water level within the former sand pit.

Fluctuations in the groundwater level should be anticipated, due to seasonal variations or in response to a particular precipitation event.

6.0 SLOPE STABILITY ANALYSES

The stability analysis was carried out in general accordance with the "City of Ottawa Slope Stability Guidelines for Development Applications in the City of Ottawa" and included both static and seismic loading conditions.

The analysis was carried out using the GeoStudio 2016 SLOPE/W computer modeling software. The Morgenstern-Price method as presented in the SLOPE/W software was used for the stability modeling.

6.1 GEOMETRY & SOIL STRATIGRAPHY

A contour plan was provided for the site from which the ground slopes grades above the sand pit water level could be measured. Below the water level, the slope grade was assumed to be 32° which is significantly steeper than present above the water level. The underwater slope grade of 32° was selected assuming the following:

- The sand pit would have been mined to its maximum possible depth and lateral extent

- The underwater angle of repose for the sand at a mined face is estimated to be 32°

- The bottom of the sand extended down to the till layer

If information regarding the base elevation of the sand mining operations or if available underwater slope contours indicate that the sand mining activities were less extensive than assumed, the slope analysis could be recalculated based on the new data.

Two cross-sections were generated at the site. The cross-sections were developed based on survey data collected by Stantec registered land surveyors and the results of the boreholes. The two sections were designated A-A and B-B; A-A was generated through the location of the proposed residence and B-B was generated through the location of the proposed office structure. The location of these sections is shown on Drawing No. 2 and 3 in Appendix B. The cross-sections profiles are provided on the SLOPE/W models in Appendix E.

6.2 SOIL PARAMETERS

The soil parameters used in the stability models are shown in Table 6.1.

Table 6.1: Soil Parameters

Soil	Unit Weight (kN/m³)	Angle of Friction (°)
Fill	20	33
Sand	19.5	33
Sand (Saturated)	21.4	33
Silt	20	30
Till	22	34
Bedrock	-	-

6.3 SEISMIC LOADING

A seismic coefficient of 0.16g was used in the models to determine the factor of safety under seismic loading; this value corresponds to ½ of the PGA adjusted for seismic site Class D.

Consideration was given to soil liquefaction destabilizing the slopes. As discussed in Section 8.5, based on the N-values measured within the boreholes the native soils are not considered prone to liquefaction under the design peak ground acceleration (PGA) applicable to Ottawa.

6.4 GROUNDWATER REGIME

The phreatic surface (groundwater) was estimated based on the groundwater level readings within the monitoring well. The estimated phreatic surface is shown as a blue dashed line on the SLOPE/W output, found in Appendix E. The slope stability analyses models groundwater scenarios based on:

1. Groundwater levels measured in the monitoring well (existing groundwater level at the time of measurement).
2. A saturated condition after a heavy rainfall event (raised groundwater level).

6.5 SLOPE STABILITY RESULTS

For permanent structures or valuable infrastructures, the following factor of safety is considered appropriate: 1.5 for static conditions and 1.1 for seismic conditions. A factor of safety of 1.3 can also be considered for passive land use, such as roads, pathways or parkland. For this project, we do not recommend using a factor of safety of 1.3, a factor of safety of 1.5 for static conditions is appropriate given the proposed structures on site.

6.5.1 Existing Slopes

The results of the slope stability analysis for the existing slopes are presented in Appendix E and summarized in Table 6.2.

Table 6.2: 2164 Old Prescott Road Stability Analysis – Existing Slopes

Section	Groundwater Condition	Approximate Side Slope	Static Factor of Safety		Seismic Factor of Safety		Conclusion
			Value	Figure	Value	Figure	
A-A	Existing	5.1H: 1V	1.484	E1	1.136	E2	Unstable
B-B	Existing	2.9H: 1V	1.549	E3	1.044	E4	Unstable

The results of the analysis indicate that the existing slopes at A-A and B-B are unstable. The figures presented in Appendix E show the slip circle with the required factor of safety of 1.5 for static conditions and 1.1 for seismic conditions.

7.0 SLOPE STABILITY RECOMMENDATIONS

Stantec's site observations and slope stability analysis indicate the proposed buildings should be set back as shown on the Limit of Hazard Land plan provided in Appendix B and discussed below.

7.1 EXISTING STABILITY

The slopes at A-A and at B-B were determined to be unstable in their current state.

7.2 LIMIT OF HAZARD LAND

The Limit of Hazard Land is the land that is at risk of being impacted by geologic processes that results in the loss of land. For slopes, the Limit of Hazard land is the summation of the following three allowances or set-back distances:

Stable Slope Allowance:

Is the set-back distance beyond the crest of the slope for which there is an acceptable factor of safety against a slope failure occurring. For permanent structures of valuable infrastructure, a factor of safety of 1.5 for static conditions and a factor of safety of 1.1 for seismic conditions are appropriate. A Stable Slope Allowance with a factor of safety of 1.3 can also be considered for passive land use such as pathways or parkland. The Stable Slope Allowance is measured away from the top of the slope. For this project a factor of safety of 1.5 is appropriate.

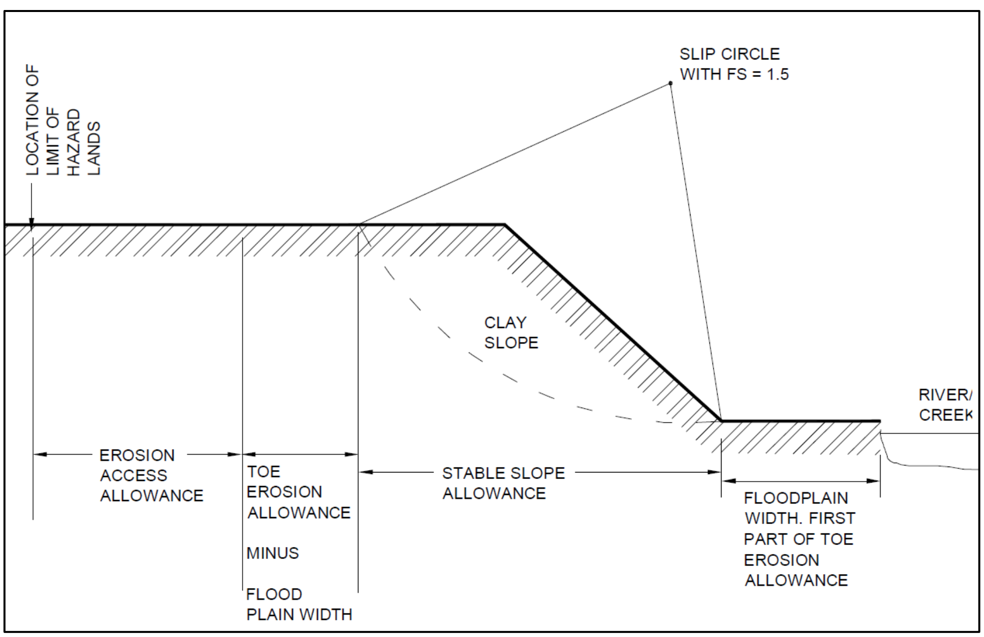
Toe Erosion Allowance:

Is a set-back distance which provides a safety margin to account for the future erosion of the toe of the slope. The Erosion Allowance is measured away from the Stable Slope Allowance.

Access Allowance:

Is a set-back distance which provides room for equipment to access the slope to carry out any future repairs or stabilization treatments. The Access Allowance is typically specified as a 6 m set-back measured away from the Erosion Allowance.

Figure from the City of Ottawa Slope Stability Guidelines displaying the location of the Limit of Hazard Lands



7.2.1 Stable Slope Allowance

The slope stability analyses carried out for each section indicated that the slopes have a factor of safety less than 1.5. The analysis was carried out to determine the slip circle for each slope where the factor of safety was 1.5 for static conditions and 1.1 for seismic conditions.

The SLOPE/W output for each of the cross-sections and corresponding factor of safety are presented in Appendix E. The Stable Slope Allowance is plotted on Drawing No. 3.

7.2.2 Erosion Allowance

The Ministry of Natural Resources guidelines suggest that an erosion allowance be included where the toe of the slope is adjacent to a river or stream where erosion may occur. The water present at the toe of the slope is water that has collected within the former sand pit. Since the water is not flowing it is reasonable to assume that very little erosion is taking place and thus an erosion allowance is not required in the Limit of Hazard Land. It is recommended that the erosion protection be increased at the site as a precaution, erosion at the toe of the slope should be monitored to ensure no erosion is taking place.

7.2.3 Access Allowance

It is recommended that an Access Allowance of 6 m be added to the proposed set-back line.

7.2.4 Conclusion

For the sites the Limit of Hazard Land was calculated to be 6 m from the crest of the slip circle. Drawing No. 3 shows the limit. Our analysis indicated that the current footprint of the office is within the recommended Limit of Hazard Land. The building footprint should be relocated to outside the Limit of Hazard Land.

8.0 DISCUSSION AND RECOMMENDATIONS

The following geotechnical issues should be considered during design activities:

- The building footprints should be located outside of the Slope Stability Limit of Hazard Lands as indicated on Drawing No. 3 in Appendix B.
- The building footprints should also be located away from the previously excavated area where deeper fills are anticipated. Three alternatives are available:
    1. Conventional spread footings founded on native material (sand) are appropriate for the design of structures at the site given that they are located outside of the previously excavated areas.
    2. If the building footprints are to be located within the previously excavated areas, pile foundations to bedrock would be appropriate for the design of the structures at the site.
    3. If the building footprints are to be located within the previously excavated areas, sub-excavation of fill to native material beneath the footprint and zone of influence

of the structures and placement of structural fill would be required before spread footings could be placed. The amount of fill is unknown and could extend deeper than 10 m. This alternative is not recommended.

- Groundwater was encountered at depths below the proposed depth of construction. It is anticipated that groundwater elevations will fluctuate throughout the year and could rise to the below grade level. The building design should include a perimeter and floor slab drainage system and damp-proofing.

- The recommended Site Classification for Seismic Site Response for the site is Site Class D in accordance with NBCC 2015.

8.1 SITE GRADING AND PREPARATION

8.1.1 Building Footprint

It is proposed to build two structures on the site, the house has a footprint of approximately 495 m^2 and the office building has a footprint of approximately 265 m^2. There are currently no underground services located within the footprints of the proposed buildings.

The area surrounding the site consists of a grassed area with trees on the south side of the site adjacent to Old Prescott Road. The site slopes down towards the former sand pit that is presently filled with water.

As shown on the Drawing No. 3 and 4 in Appendix B, portions of the site have previously been excavated and deeper fills are anticipated. The footprints of the two buildings should be located outside of the slope stability limit of hazard lands and outside of the previously excavated areas where deep fill deposits are anticipated. Alternatively, piled foundations on bedrock may be used where the deeper fills are anticipated.

All existing topsoil, fill and any deleterious materials should be removed from beneath the footprint of the building, the footings and the zone of influence of all footings. The zone of influence is defined by a line drawn at 1 horizontal to 1 vertical, outward and downward from the edge of the footings.

Prepared subgrade surface should be inspected by experienced geotechnical personnel prior to placement of either Structural Fill or concrete. All soft or disturbed areas revealed during subgrade excavation or inspection should be removed and replaced with approved Structural Fill, as defined below.

Structural Fill should conform to the requirements of OPSS Granular B Type II or OPSS Granular A. Structural Fill placed beneath the building should contain no recycled materials such as concrete or asphalt. It should be compacted in lifts no thicker than 300 mm to at least 100% Standard Proctor Maximum Dry Density (SPMDD). This material should be tested and approved by a Geotechnical Engineer prior to delivery to the site.

Earth removals should be inspected by a geotechnical engineer to ensure that all unsuitable materials are removed prior to placement of fill. Inspection and testing services will be critical to ensure that all fill used is suitable and is placed and compacted to that required degree.

For the case where piles are going to be used to support the house foundations, the fill at depth would remain in place. It is however recommended that to minimize basement slab settlements that the subgrade fills be sub-excated to 500 mm below the subgrade level (below the bottom level of the free draining granular layer). The exposed fill material should be surface compacted to 98% of its Standard Proctor maximum dry density, then Structural Fill should be placed to the top of subgrade.

8.1.2 Paved Areas

All vegetation, topsoil and other deleterious material should be removed from beneath pavement areas. The subgrade should be proof rolled in the presence of geotechnical personnel. All soft areas revealed during proof rolling or subgrade inspections should be excavated to a maximum depth of 500 mm and replaced with compacted Subgrade Fill.

8.2 FOUNDATIONS

8.2.1 Shallow Foundations

The foundations for the proposed buildings may be supported on spread footings provided that the foundation preparation work described in Section 8.1 is carried out. Spread footings should be placed in clean undisturbed native sand.

Table 8.1 provides Geotechnical Bearing Resistances for shallow foundations on sand. The values have been calculated assuming a footing embedment depth of 0.5 m.

Table 8.1: Geotechnical Resistance for Shallow Footings

Foundation Type	Footing Width (m)	ULS (kPa)	SLS (kPa)
Strip Footing	0.5 m	175	125
Strip Footing	1.0 m	175	125
Square Footing	1.0 m	225	150
Square Footing	2.0 m	225	150

The factored geotechnical bearing resistance at Ultimate Limit States (ULS) incorporates a resistance factor of 0.5. The geotechnical reaction at Serviceability Limit States (SLS) is the bearing pressure that corresponds to 25 mm of settlement or has been limited to the ULS resistance.

The design frost depth for this site is 1.8 m. All exterior spread footings and footings for unheated structures should be protected from frost action by a minimum soil cover of 1.8 m or equivalent insulation. Perimeter footings and interior footings within 1.5 m of perimeter walls of heated structures should be protected by a minimum soil cover of 1.5 m or equivalent insulation. Where proposed footings have insufficient soil cover for frost protection, the use of rigid insulation will be required; a geotechnical review of insulation designs is required.

The base of all footing excavations should be inspected by a geotechnical engineer prior to placing concrete to confirm the design pressures and to ensure that there is no disturbance of the founding soils.

Where construction is undertaken during winter conditions, all footing subgrades should be protected from freezing. Foundation walls and columns should be protected against heave due to soil adfreeze.

8.2.2 Deep Foundations

For building footprints located within the previously excavated area, pile foundations will be required. The piles should be end bearing on bedrock. The pile capacities are outlined in Table 8.2. Pile capacities should be reduced to account for down drag forces if a significant grade raise is carried out.

Table 8.2: Pile Capacities

Pipe Pile diameter (mm)	Wall Thickness (mm)	Maximum Factored Geotechnical Axial Resistance at ULS	Downdrag Force
323.9	11.13	1000 kN	N/A
273.1	9.53	750 kN	N/A

All piling activities should be monitored by trained geotechnical personnel.

Perimeter grade beams and pile caps should be provided with at least 1.5 m of soil cover to protect against frost action.

Where construction is undertaken during winter conditions, footing subgrades should be protected from freezing. Foundation walls and columns should be protected against heave due to soil adfreeze.

A resistance factor, ϕ, of 0.4 has been applied to ULS resistance.

8.2.2.1 Pile Installation

Native till was encountered in borehole BH18-3 at approximate elevation 81.0 m, it is assumed that bedrock is at an approximate elevation in the range of 80.0 m. Piles should be end bearing on competent bedrock.

Compatibility of the pile driving equipment, the soil conditions, and the pile type being driven are all essential items in achieving the required pile penetration and a satisfactory pile foundation.

Pile tips should be reinforced as per Ontario Provincial Standard Detail, OPSD-3000.100 Type I.

The sequence of driving piles in groups can affect the pile lengths and driving resistances due to ground densification. We recommend that the piles in the centers of a pile group be driven first. This procedure reduces pile drift and makes driving easier.

Pile penetration displaces the soil laterally and may cause surface heave during installation. The surface heave can cause adjacent piles to move upward. Level readings on the top of adjacent piles should be taken periodically to verify no significant heave is occurring. Additionally, care should be taken to keep construction equipment as far away as possible from driven piles. Heavy equipment traveling or operating too closely to piles can displace them laterally.

To the extent possible, the installation of piles should be a continuous operation without termination of driving until the point of acceptable resistance or embedment is achieved. If driving is interrupted, the pile should be driven at least 300 mm after driving is resumed, providing this will not overstress the piles.

Pile testing and all pile installations should be observed and documented by trained geotechnical personnel to confirm that piles are being installed in accordance with the pile driving criteria. Continuous driving and installation records should be maintained for all driven piles.

8.1 TEMPORARY EXCAVATIONS AND BACKFILLING

8.1.1 General Excavations

The native sand present at the site is considered a Type 3 soil in accordance with the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. Temporary excavations in the overburden may be supported or should be sloped at 1 horizontal to 1 vertical from the base of the excavation and as per the requirements of OHSA.

8.1.2 Foundation Backfill

Backfill within the footprint of the proposed buildings should consist of Structural Fill placed as described in Section 6.1. Exterior foundation backfill should consist of a material meeting the requirements of OPSS Granular B Type I.

The Subgrade Fill must be placed in lifts no thicker than 300 mm and compacted using suitable compaction equipment to at least 95% of SPMDD. Care should be taken immediately adjacent to the foundation walls to avoid over-compaction of the soil which could result in damage to the walls.

8.1.3 Pipe Bedding and Backfill

Bedding for utilities should be placed in accordance with the pipe design requirements. It is recommended that a minimum of 150 mm to 200 mm of OPSS Granular A be placed below the pipe invert as bedding material. Granular pipe backfill placed above the invert should consist of Granular A material. A minimum of 300 mm vertical and side cover should be provided. These materials should be compacted to at least 95% of SPMDD.

Backfill for service trenches in landscaped areas may consist of excavated material replaced and compacted in lifts. Where the service trenches extend below paved areas, the trench should be backfilled with subgrade fill material as defined in Section 6.1 from the top of the pipe cover to within 1.2 m of the proposed pavement surface, placed in lifts and compacted to at least 95% of SPMDD. The material used within the upper 1.2 m and below the subgrade line should be similar to that exposed in the trench walls to prevent differential frost heave, placed in lifts and compacted to at least 95% of SPMDD. Different abutting materials within this zone will require a 3 horizontal to 1 vertical frost taper to minimize the effects of differential frost heaving.

It should be noted that reuse of the site generated material will be highly dependent on the material's moisture content at time of placement.

Backfill should be compacted in lifts not exceeding 300 mm.

8.1.4 Groundwater and Dewatering

Groundwater was encountered during this geotechnical investigation below the depths of the anticipated excavations. The groundwater level was measured at 5.4 m below the ground surface, corresponding to elevation 89.0 m. However, groundwater elevations will fluctuate seasonally and may rise to the level of the basement.

Foundation walls should be protected with damp-proofing and backfilled with free-draining granular material such as OPSS Granular B Type I. The zone of free-draining backfill should extend a horizontal distance of at least 500 mm out from the foundation wall. It is recommended that a perimeter drain and underslab drainage system be installed. The drainage system should be designed to allow positive drainage to a frost-free outlet.

If dewatering is required during construction, it will likely be possible using conventional sump and pump techniques.

8.2 CONCRETE FLOOR SLABS

Conventional slab-on-grade units are suitable for use for the proposed structure provided the floor slab areas are prepared as outlined in Section 6.1. A layer of free-draining granular material such as OPSS Granular A, at least 200 mm in thickness should be placed immediately beneath the floor slab for leveling and support purposes. This material should be compacted to at least 100% SPMDD. The installation of a vapor barrier below the floor slab is recommended.

The floor slabs constructed as recommended above may be designed using a soil modulus of subgrade reaction, k, of 35 MPa/m, based on a loaded area of 0.3 m by 0.3 m. The slab-on-grade units should float independently of all load-bearing walls and columns.

8.3 CEMENT TYPE AND CORROSION POTENTIAL

Two representative soil samples were submitted to Paracel Laboratories Ltd. in Ottawa, Ontario, for pH, chloride, sulphate and resistivity testing. The test results are summarized in Table 8.3.

Table 8.3: pH, Sulphate, Chloride and Resistivity Analysis Results

Borehole/ Sample No.	Depth	pH	Sulphate (µg/g)	Resistivity (ohm.m)	Chloride (µg/g)
BH18-1/ SS4	2.3 m – 2.9 m	7.91	6	114	7
MW18-2/ SS5	3.0 m – 3.7 m	7.95	25	87.5	7
BH18-4/ SS4	2.3 m – 2.9 m	7.97	22	93.1	7

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The soluble sulphate results ranged from 6 to 25 µg/g. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH ranged from 7.91 to 7.97 which is within what is considered the normal range for soil pH of 5.5 to 9.0. The pH levels of the tested soil do not indicate a highly corrosive environment. The test results provided in the Table 8.2 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

8.4 PAVEMENT STRUCTURE RECOMMENDATIONS

It has been assumed that any parking areas will be used mostly by passenger vehicles.

The subgrade in paved areas should be prepared as described in Section 6.1 above. The minimum pavement recommendations for both the asphalt walkway and any standard parking areas are included in Table 8.4.

Table 8.4: Recommended Pavement Design

Material	Standard Duty Parking Area
SP 12.5 (surface course asphalt)	50 mm
Granular Base Course, OPSS Granular A	150 mm
Granular Subbase Course, OPSS Granular B Type II	300 mm

It is estimated that the service life prior to major rehabilitation for the above pavement structures is 15 years provided they are properly maintained. The pavement surface and the underlying subgrade should be graded to direct runoff water towards suitable drainage.

All granular materials should be tested and approved by a geotechnical engineer prior to delivery to the site. Both base and subbase materials should be compacted to at least 100% SPMDD. Asphalt should be compacted to at least 97% Marshal bulk density.

It is recommended that the lateral extent of the subbase and base layers not be terminated in a vertical fashion immediately behind the curb line. A taper with a grade of 5 horizontal to 1 vertical is recommended in the subgrade line to minimize differential frost heave problems under sidewalks.

8.5 SEISMIC SITE CLASSIFICATION

Liquefaction Induced Settlements

An assessment for seismic liquefaction has been carried out for this site. Seismic liquefaction is the sudden loss in stiffness and strength of soil due to the loading effects of an earthquake. Liquefaction can cause significant settlements and structural failure.

The analysis followed was the one set forth in the Canadian Foundation and Engineering Manual, 2006 (CFEM). For the analysis, a magnitude 6.2 design earthquake with a Peak Ground Acceleration of 0.311g, were assumed. Based on the SPT N for the soil, plots of Factor of Safety against Liquefaction (FSL) with depth were developed for the site.

Our analysis indicates that the site soil is not considered susceptible to liquefaction.

Seismic Site Classification

As outlined in the 2012 Ontario Building Code buildings, their foundations must be designed to resist a minimum earthquake force. In accordance with Table 4.1.8.4.A of the 2012 Ontario Building Code the seismic site response for the site is Class D – Stiff Soil. The site class is based on the Average Standard Penetration Resistance shown in Table 8.5.

Table 8.5: Parameters for Seismic Site Classification

Depth	Soil	N_{60} Value
2 m to 9 m	Sand	14
9 m to 13 m	Till	29
13 m to 32 m	Bedrock	100
Design N_{60}		36

8.6 LATERAL EARTH PRESSURES

The earth pressures recommended in Table 6.5 are based on the assumption that a permanent horizontal back slope will be utilized behind the wall. In order to use the coefficients of pressures for the granular materials, the granular backfill must be provided within a wedge extending from the base of the wall at 45 degrees (or smaller) to the horizontal. If a smaller wedge is used, the coefficients of earth pressures of the materials outside the backfill wedge must be used for lateral pressure design calculations.

For walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied structures, the at rest pressure should be used for design, unless the wall can deflect enough (approximately 0.05% of the wall height) to establish the active pressure.

Lateral earth pressures may be calculated using parameters provided in Table 8.6.

Table 8.6: Lateral Earth Pressure Parameters

Parameter	Fill	Sand	OPSS Granular A	OPSS Granular B Type II
Unit Weight (kN/m^3)	20.0	19.5	22.8	21.2
Angle of Internal Friction, Φ	33°	33°	35°	32°
Coefficient of Passive Earth Pressure, K_p	3.39	3.39	3.69	3.25
Coefficient of at Rest Earth Pressure, K_o	0.46	0.46	0.43	0.47
Coefficient of Active Earth Pressure, K_a	0.29	0.29	0.27	0.31

Sliding resistance can be calculated using the following unfactored friction coefficients, outlined in Table 8.7.

Table 8.7: Unfactored Friction Coefficients

Condition	Unfactored Friction Coefficient
Between Concrete and Structural Fill	0.55
Between Concrete and Native Soil	0.35

Seismic Design Parameters

For retaining structures total active and passive thrusts under earthquake conditions can be calculated using the following equations:

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_V)$$

$$P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_V)$$

where;

K_{AE} = active earth pressure coefficient (combined static and seismic)

K_{PE} = passive earth pressure coefficient (combined static and seismic)

H = height of wall

k_h = horizontal acceleration coefficient

k_v = vertical acceleration coefficient

γ = total unit weight

For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values (assumes Horizontal Backslope to wall).

Zonal Acceleration Ratio, A or PGA	0.33
Horizontal Acceleration Coefficient, k_h	0.17
Vertical Acceleration Coefficient, k_v	0.11

The above k_h value corresponds to ½ of the A value, and the k_v value corresponds to 0.67 of the k_h value. The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

Table 8.8: Combined Coefficients of Static and Seismic Earth Pressure

Parameter	Fill	Sand	OPSS Granular A	OPSS Granular B Type II
Bulk Unit Weight, γ (kN/m^3)	20.0	19.5	22.8	21.2
Effective Friction Angle	33°	33°	35°	32°
Angle of Internal Friction between wall and backfill	0 degrees	0 degrees	0 degrees	0 degrees
Yielding Wall				
Active Earth Pressure (K_{AE})	0.40	0.40	0.37	0.41
Height of Application of P_{AE} from base as a ratio of wall height (H)	0.402	0.402	0.404	0.401
Passive Earth Pressure (K_{PE})	3.08	3.08	3.36	2.95
Height of Application of P_{PE} from base as a ratio of wall height (H)	0.306	0.306	0.308	0.305

If the wall is designed as non-yielding wall it could be designed with the Wood (1973) method:

$$\Delta P_{eq} = \gamma H^2 \frac{a_h}{g} F_p$$

ΔP_{eq} = Steady state dynamic thrust
γ = 20 kn/m^3
H = Height of wall (m)
a_h = Amplitude of harmonic base acceleration = 0.42 m/s^2
g = Acceleration due to gravity (m/s^2) = 9.81 m/s^2
F_p = Dimensionless thrust factor □= 1.1
heq = 0.63H

9.0 CLOSURE

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of Justice Construction, who is identified as "the Client" within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec Consulting Ltd. should any these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying or unexpected site conditions
- Planning, design or construction

Respectfully submitted,

STANTEC CONSULTING LTD.

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APPENDIX A

Statement of General Conditions

STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Stantec Consulting Ltd.'s present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Stantec Consulting Ltd. is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec Consulting Ltd. will not be responsible to any party for damages incurred as a result of failing to notify Stantec Consulting Ltd. that differing site or sub-surface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec Consulting Ltd., sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Stantec Consulting Ltd. cannot be responsible for site work carried out without being present.

APPENDIX B

Drawing No. 1 – Key Plan

Drawing No. 2 – Borehole Location Plan

Drawing No. 3 – Limit of Hazard Land

Drawing No. 4 – 2005 Air Photo

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0 500 1,000 metres

1:20,000 (at original document size of 8.5x11)

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Notes
1. Coordinate System: NAD 1983 MTM Zone 9N.
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © Queen's Printer for Ontario, 2016.
3. Imagery provided by Frist Base Solutions 2018.

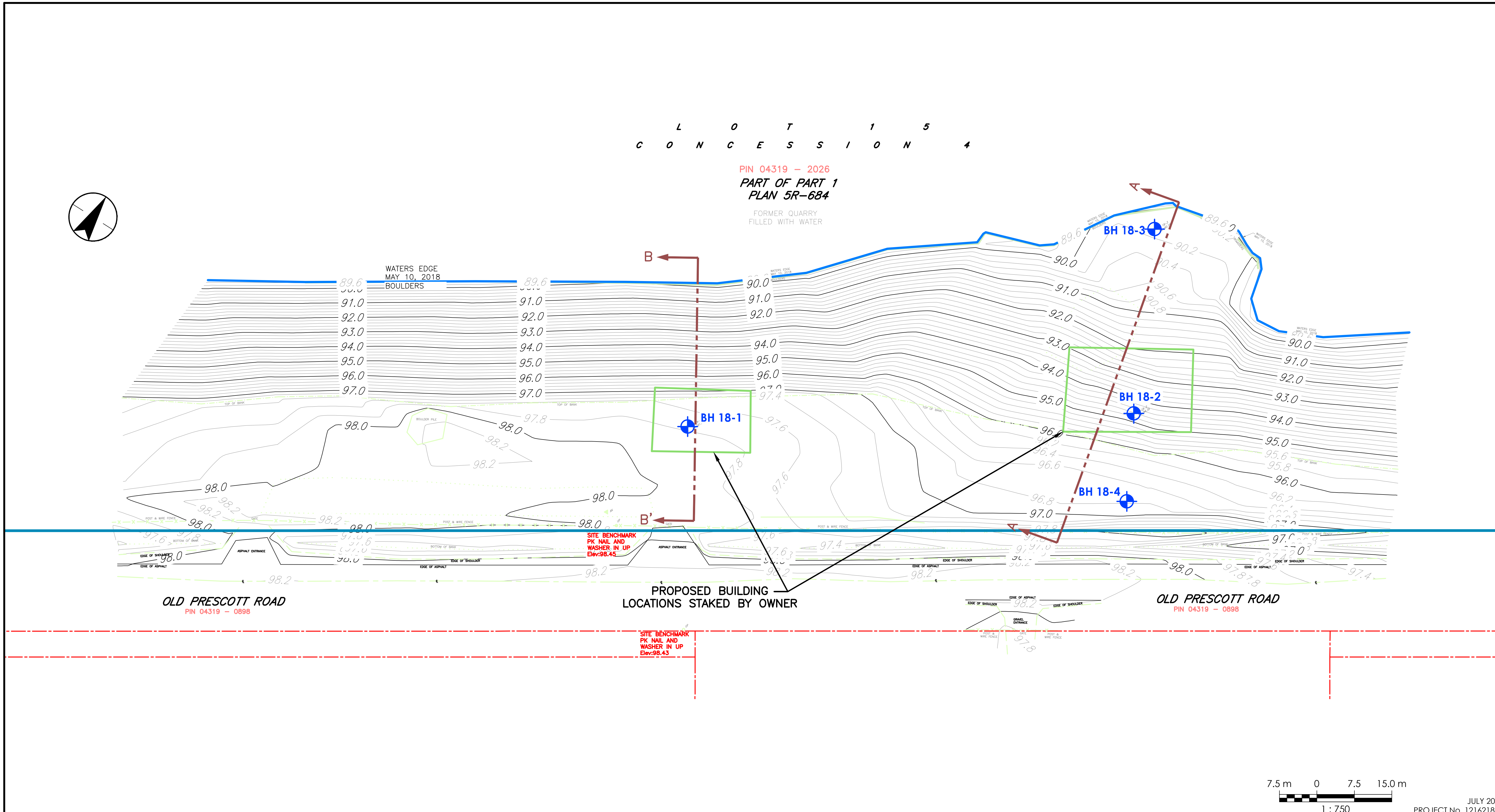
Project Location
2164 Old Prescott Road
Ottawa, Ontario

Project No. 163621894
Prepared by Gliceria Briones
on 2018-06-14

Client/Project
JUSTICE CONSTRUTION
GEOTECHNICAL INVESTIGATION
2164 OLD PRESCOTT ROAD

Drawing No.
1

Title
Key Plan



JULY 2018
PROJECT No. 121621894

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2018/07/11 2:51 PM By: Briones, Gliceria

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LEGEND

BOREHOLE

NOTES

1. COORDINATE SYTEM: NAD 1983 MTM ZONE 9N.
2. BASEPLAN PROVIDED BY STANTEC GEOMATICS LTD. FILENAME: 160410217-101_c3d_d1_2.DWG, DATED MAY 16, 2018.

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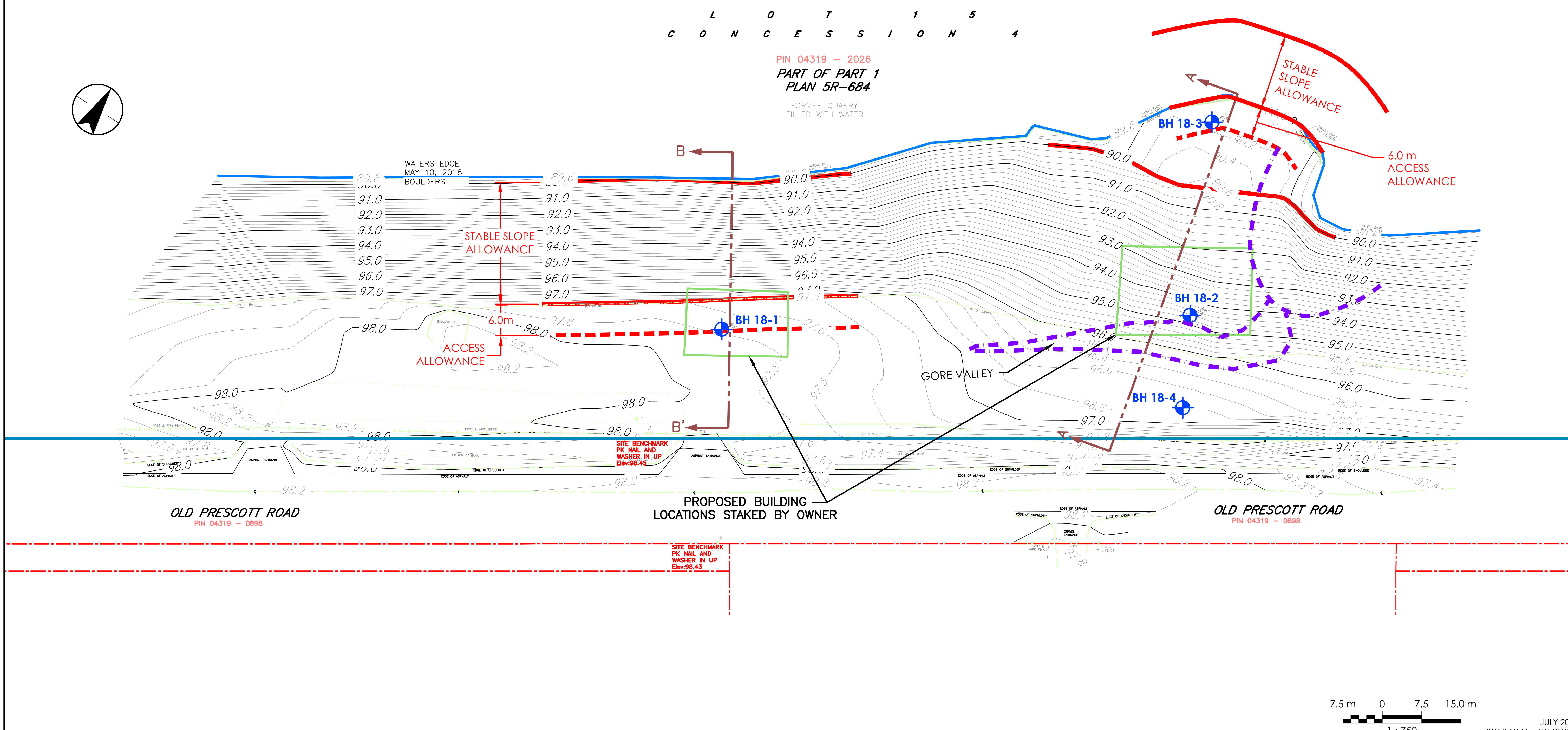
JUSTICE CONSTRUCTION
GEOTECHNICAL INVESTIGATION
2164 OLD PRESCOTT ROAD, OTTAWA, ONTARIO

Drawing No.

2

Title

BOREHOLE LOCATION PLAN



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2018/07/11 3:03 PM By: Briones, Gliceria

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LEGEND

- BOREHOLE
- LOCATION OF LIMIT OF HAZARD LANDS
- LIMITS OF HISTORICAL EXCAVATION (FROM 2005 AIR PHOTO)

NOTES

1. COORDINATE SYTEM: NAD 1983 MTM ZONE 9N.
2. BASEPLAN PROVIDED BY STANTEC GEOMATICS LTD. FILENAME: 160410217-101_c3d_d1_2.DWG, DATED MAY 16, 2018.

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JUSTICE CONSTRUCTION
GEOTECHNICAL INVESTIGATION
2164 OLD PRESCOTT ROAD, OTTAWA, ONTARIO

Drawing No.
3

Title
LIMIT OF HAZARD LANDS
SLOPE STABILITY EVALUATION

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2018/07/11 3:03 PM By: Briones, Gliceria

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PROJECT No. 121621894

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LEGEND

BOREHOLE

LIMITS OF HISTORICAL EXCAVATION
(FROM 2005 AIR PHOTO)

NOTES

1. COORDINATE SYTEM: NAD 1983 MTM ZONE 9N.
2. IMAGERY FROM GEOOTTAWA.

Client/Project

JUSTICE CONSTRUCTION

GEOTECHNICAL INVESTIGATION
2164 OLD PRESCOTT ROAD, OTTAWA, ONTARIO

Drawing No.

4

Title

2005 AIR PHOTO

APPENDIX C

Symbols and Terms Used on Borehole and Test Pit Records

Borehole Records

Site Photos

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Rootmat	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor Quality*
25-50	*Poor Quality*
50-75	*Fair Quality*
75-90	*Good Quality*
90-100	*Excellent Quality*

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	*Crushed*
Severely Fractured	*Shattered or Very Blocky*
Fractured	*Blocky*
Moderately Jointed	*Sound*
Intact	*Very Sound*

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.

Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Igneous Bedrock	Meta-morphic Bedrock	Sedi-mentary Bedrock

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

▼ measured in standpipe, piezometer, or well

▽ inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_U	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

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BOREHOLE RECORD

BH18-1

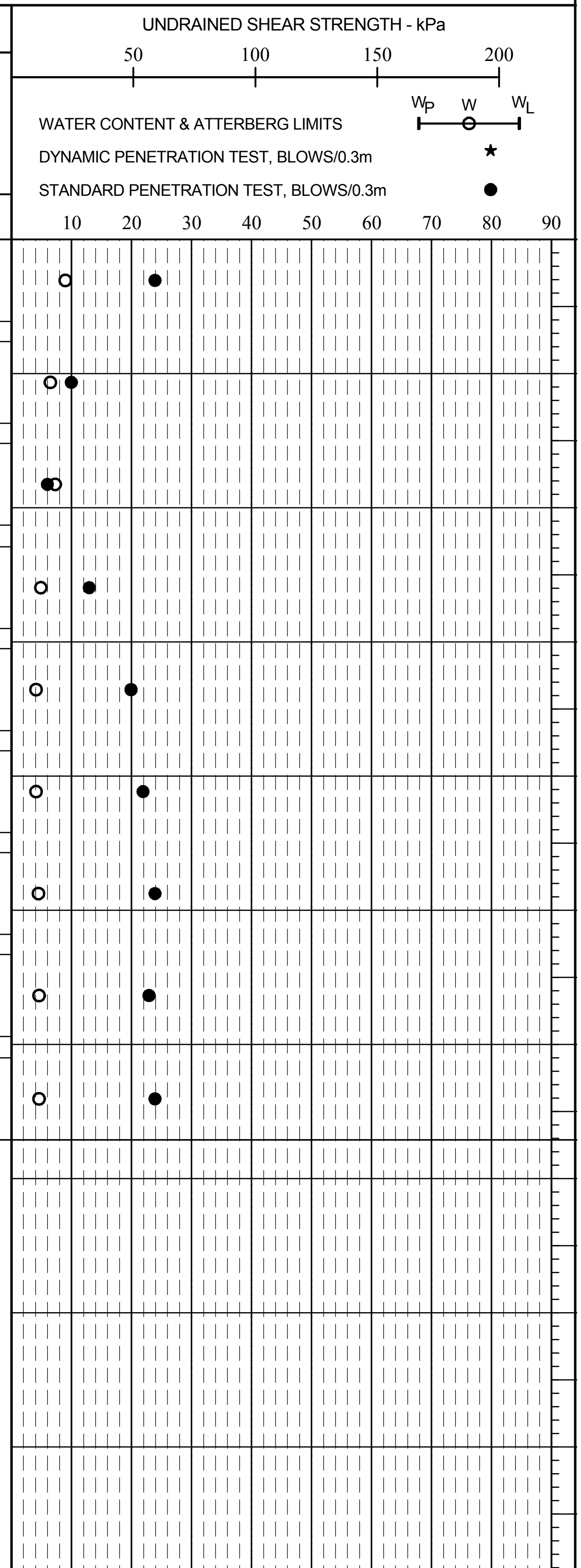
CLIENT: Justice Construction
LOCATION: 2164 Old Prescott Road
DATES: BORING May 29, 2018 WATER LEVEL

BOREHOLE No. BH18-1
PROJECT No. 121621894
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION
0	97.67	
	97.6	120 mm TOPSOIL
	97.1	FILL: Dark brown silty sand (SM) with gravel
		Loose to compact brown well graded SAND (SW-SM) with silt
	93.3	Compact brown well graded SAND (SW-SM) with silt
	91.0	End of Borehole

SAMPLES

TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
SS	1	500	24
SS	2	400	10
SS	3	400	6
SS	4	400	13
SS	5	450	20
SS	6	380	22
SS	7	420	24
SS	8	440	23
SS	9	400	24



Inferred Groundwater Level
Groundwater Level Measured in Standpipe

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

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MONITORING WELL RECORD

MW18-2

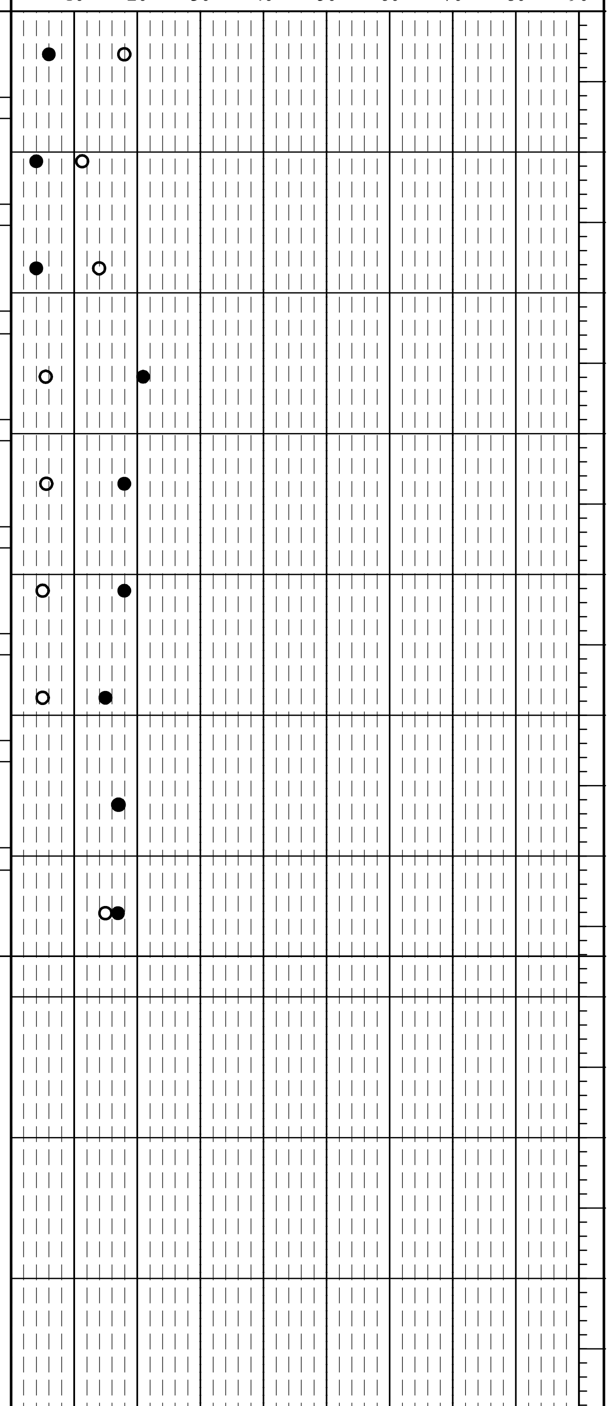
CLIENT: Justice Construction
BOREHOLE No.: MW18-2
LOCATION: 2164 Old Prescott Road
PROJECT No.: 121621894
DATES: BORING: May 29, 2018
WATER LEVEL: June 22, 2018
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	94.39							
	94.2	150 mm TOPSOIL						
		FILL: Grey/brown silty clay with gravel and construction debris			SS	1	330	6
1					SS	2	300	4
2					SS	3	50	4
	92.1	Compact brown silty SAND (SM)			SS	4	300	21
3					SS	5	400	18
4					SS	6	500	18
5					SS	7	520	15
	89.1	Compact brown well graded SAND (SW-SM) with silt - wet		∇	SS	8	450	17
6					SS	9	450	17
	87.7	End of Borehole Monitoring Well Installed						
7								
8								
9								
10								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90



∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

■ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

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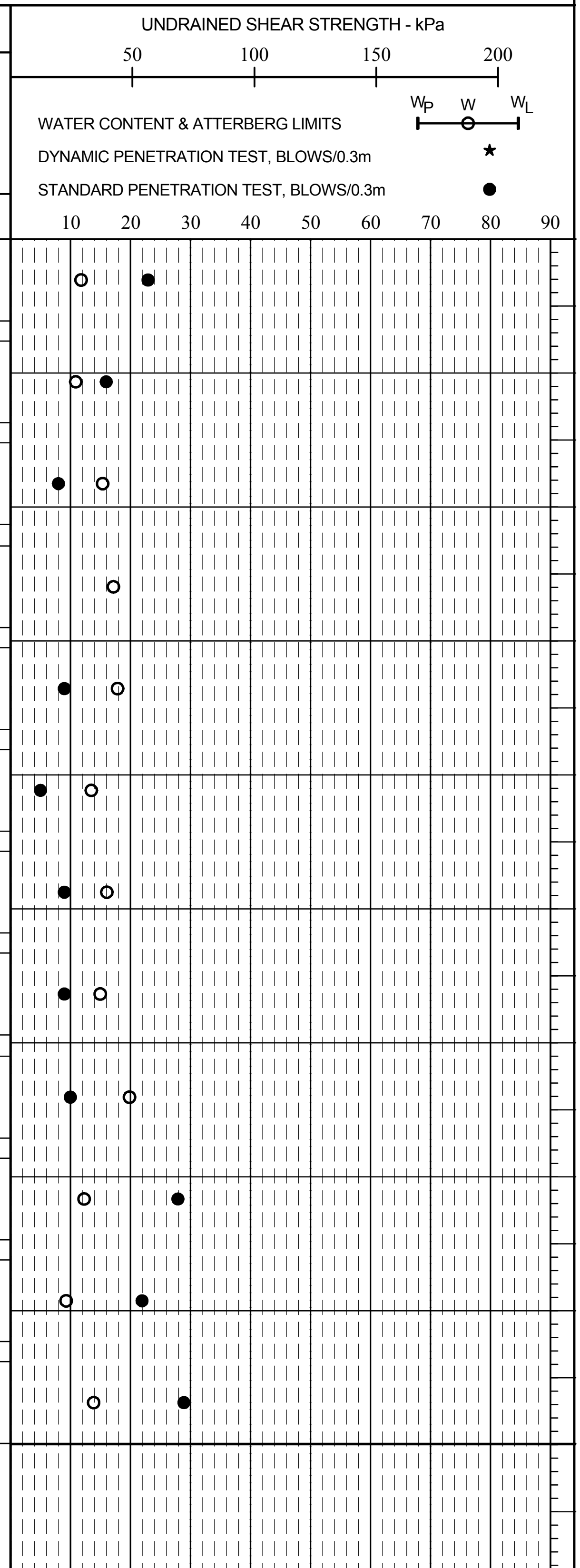
BOREHOLE RECORD

BH18-3

CLIENT Justice Construction
LOCATION 2164 Old Prescott Road
DATES: BORING May 29, 2018 WATER LEVEL
BOREHOLE No. BH18-3
PROJECT No. 121621894
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	90.14					
	90.0	120 mm TOPSOIL				
		FILL: Brown silty clay with sand and gravel	SS	1	520	23
1	89.3	FILL: brown poorly graded sand (SP-SM) with silt - wet	SS	2	400	16
2			SS	3	150	8
			SS	4	400	
3			SS	5	380	9
4	86.3	FILL: brown well graded sand (SW-SM) with silt and gravel	SS	6	320	5
5			SS	7	340	9
			SS	8	200	9
6			SS	9	200	10
7	83.4	Compact brown silty SAND (SM) with gravel - frequent cobbles and boulders	SS	10	180	28
8	82.2	Compact brown silty gravel (GM) with sand - frequent rock pieces	SS	11	220	22
	81.4	Compact grey silty sand (SM) with gravel TILL	SS	12	220	29
9	81.1	End of Borehole				
10						



$\underline{\nabla}$ Inferred Groundwater Level
$\blacktriangledown$ Groundwater Level Measured in Standpipe

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

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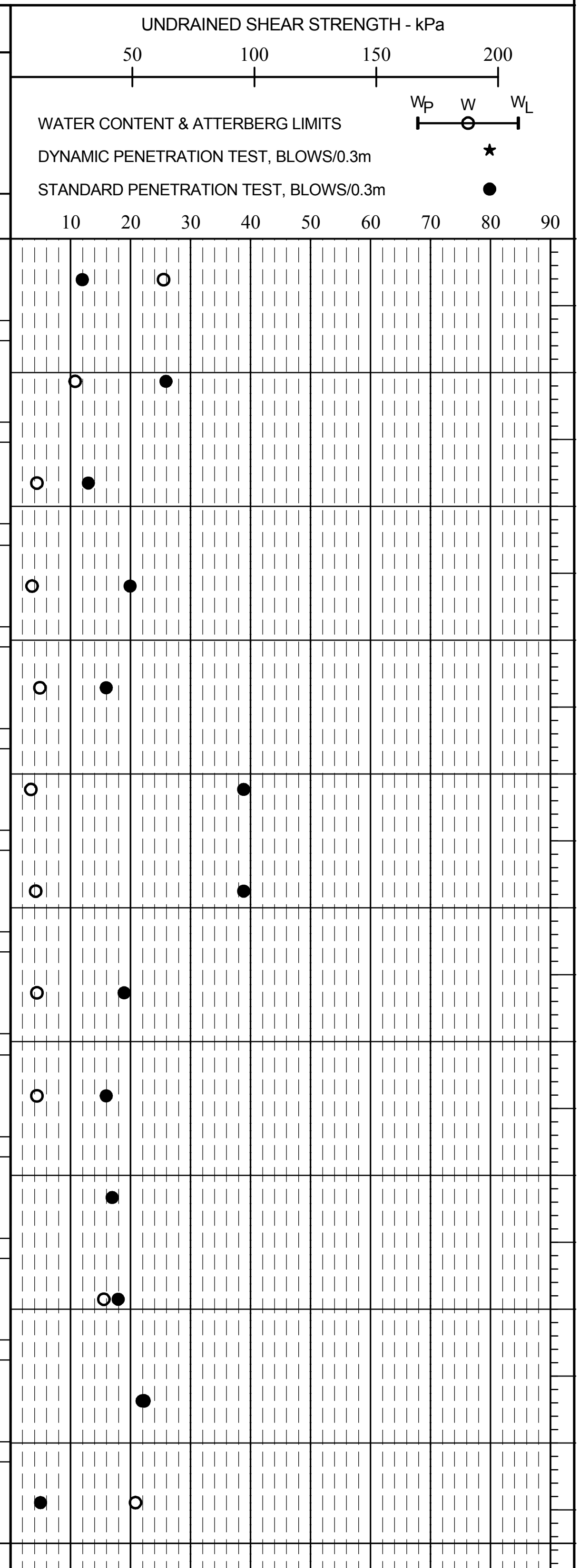
BOREHOLE RECORD

BH18-4

CLIENT Justice Construction BOREHOLE No. BH18-4
LOCATION 2164 Old Prescott Road PROJECT No. 121621894
DATES: BORING May 29, 2018 WATER LEVEL DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	96.90					
	96.8	100 mm TOPSOIL				
		FILL: Dark brown silty sand (SM) with gravel	SS	1	500	12
1			SS	2	500	26
	95.5	Compact brown well graded sand (SW-SM) with silt and gravel	SS	3	450	13
2			SS	4	440	20
3			SS	5	480	16
	93.1	Dense brown well graded SAND (SW-SM) with silt and gravel	SS	6	460	39
5			SS	7	380	39
	91.5	Compact brown poorly graded SAND (SP-SM) with silt - wet below 7.6 m	SS	8	500	19
6			SS	9	450	16
7			SS	10	400	17
8			SS	11	380	18
	88.4	Loose to compact light brown sandy SILT (ML)	SS	12	500	22
9			SS	13	400	5
	87.2	End of Borehole				
10						



Inferred Groundwater Level
Groundwater Level Measured in Standpipe

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

Photo No. 1: Looking east from footprint of office

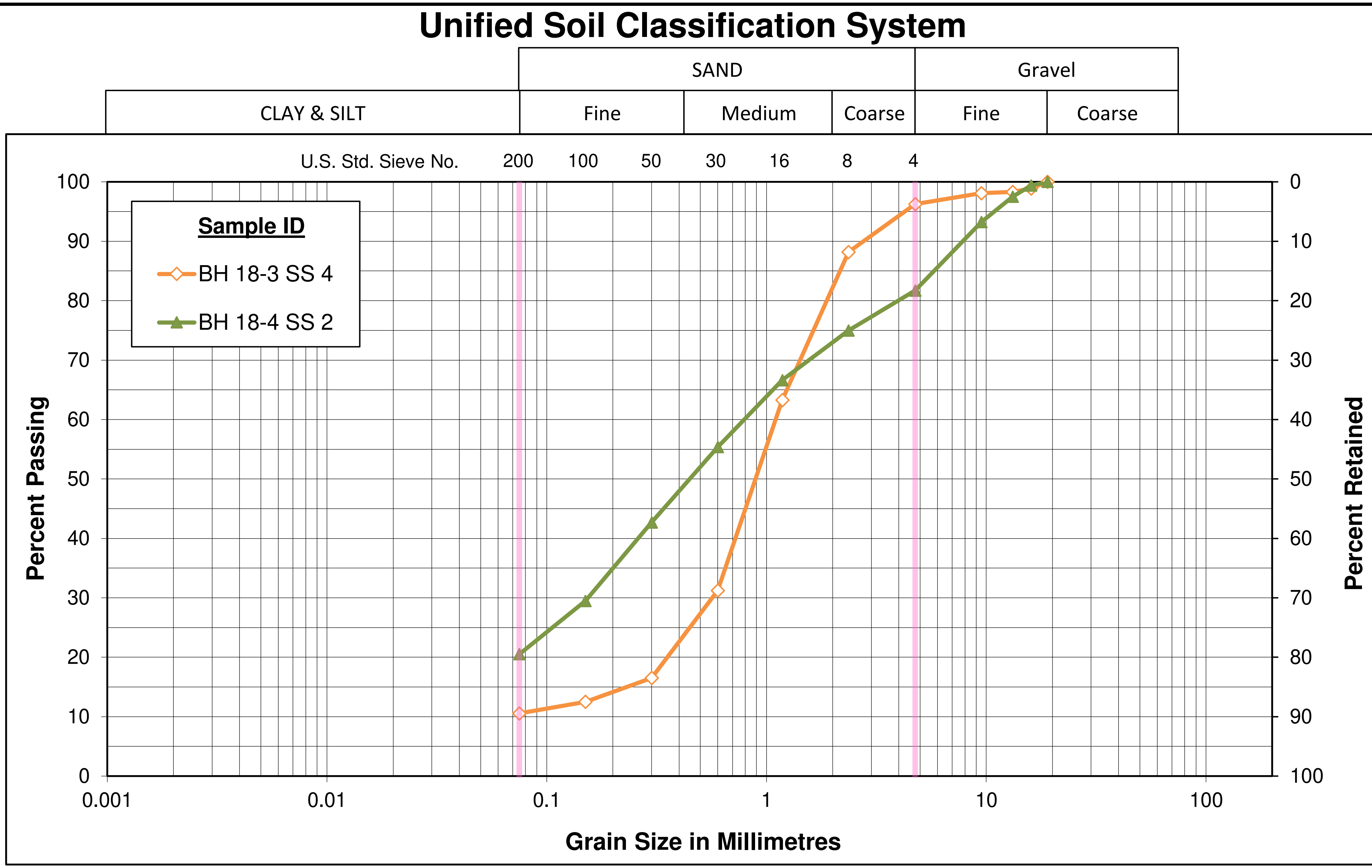
Photo No. 2: Looking south-west from BH18-3

Photo No. 3: Looking north from footprint of residence

Photo No. 4: Looking east from footprint of residence

APPENDIX D

Laboratory Test Results



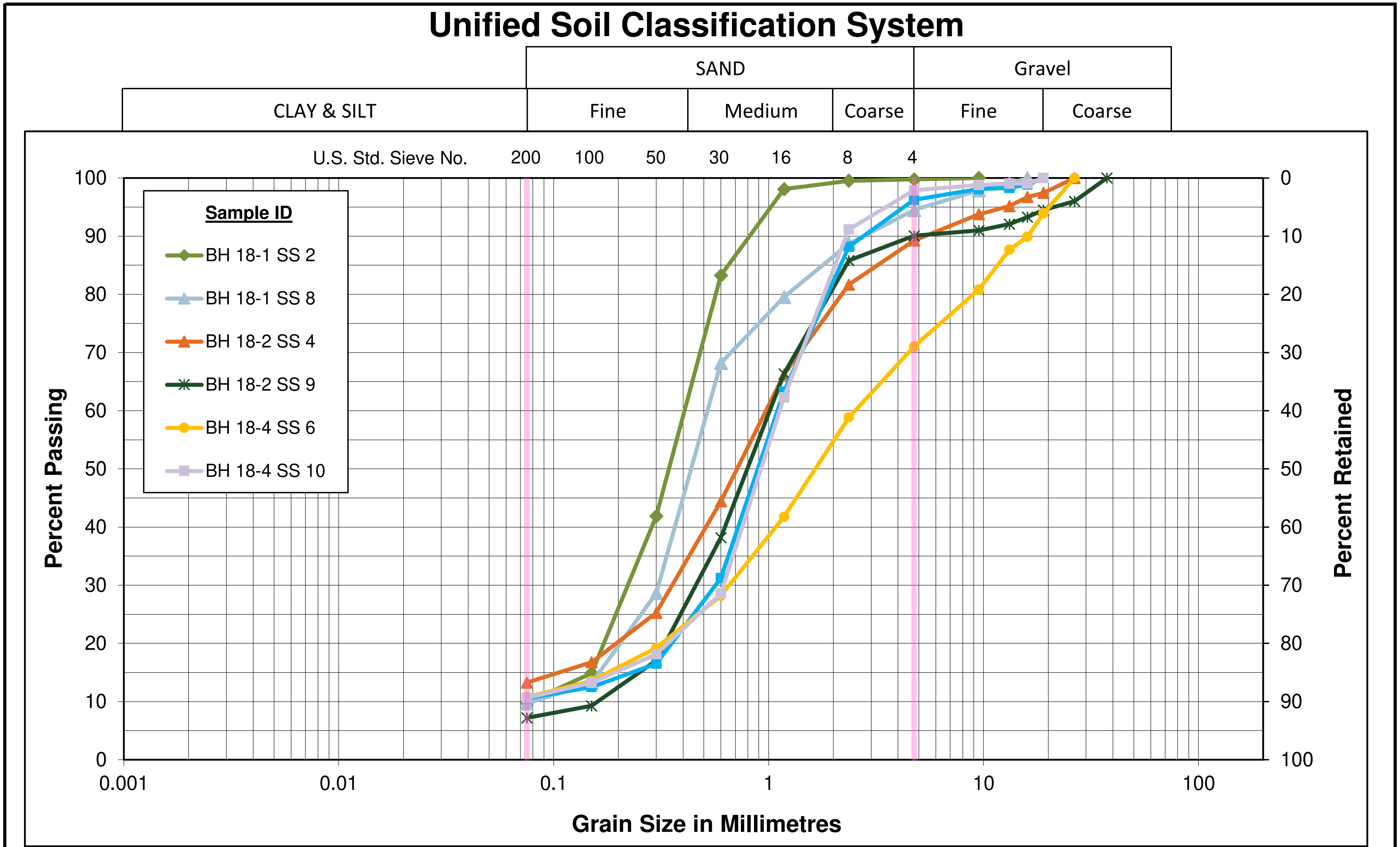
Stantec

GRAIN SIZE DISTRIBUTION

FILL: Silty Sand with Gravel (SM), Poorly Graded Sand with Silt (SP-SM)

Figure No. 1

Project No. 121621894.200



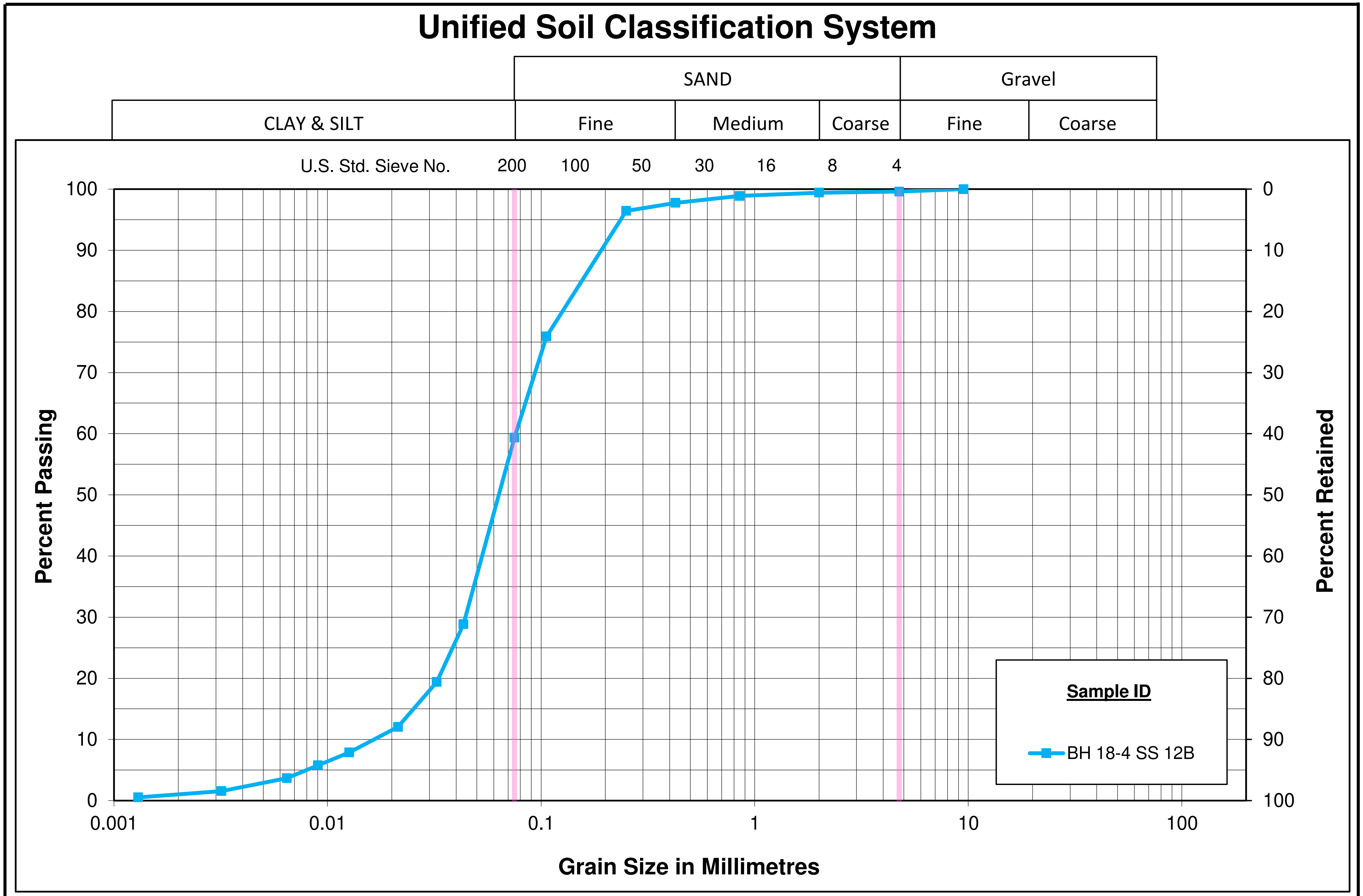
Stantec

GRAIN SIZE DISTRIBUTION

Silty SAND (SM), Poorly Graded SAND with Silt (SP-SM), Well Graded Sand with Silt and Gravel (SW-SM)

Figure No. 2

Project No. 121621894.200



Stantec

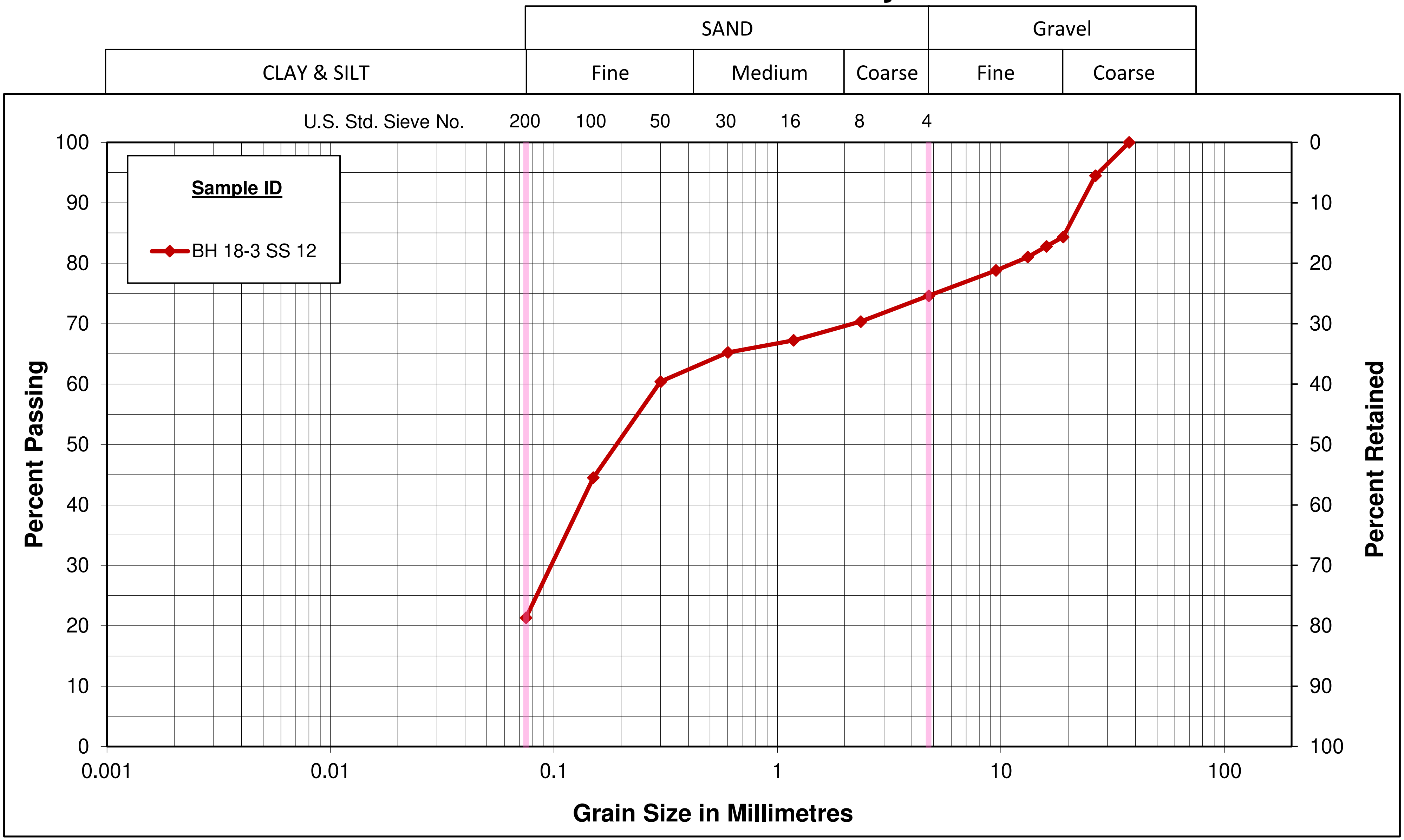
GRAIN SIZE DISTRIBUTION

Sandy SILT (ML)

Figure No. 3

Project No. 121621894.200

Unified Soil Classification System



Stantec

GRAIN SIZE DISTRIBUTION

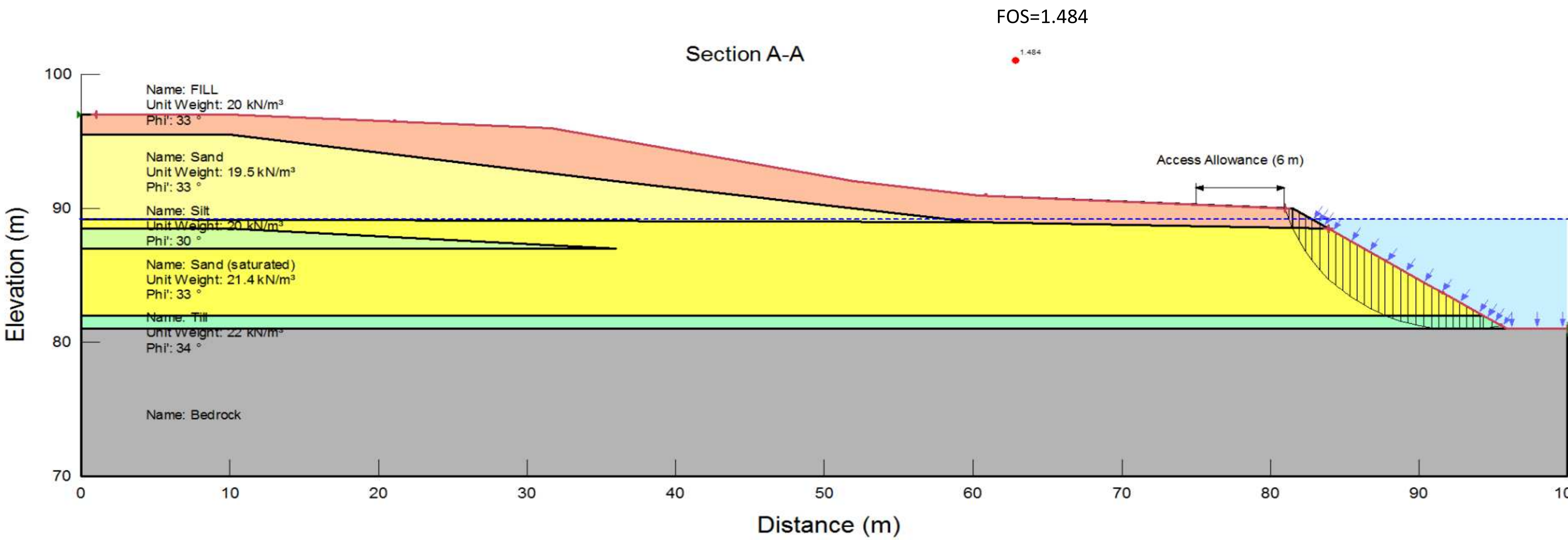
Silty Sand with Gravel (SM) TILL

Figure No. 4

Project No. 121621894.200

APPENDIX E

Output from Slope Stability Analyses



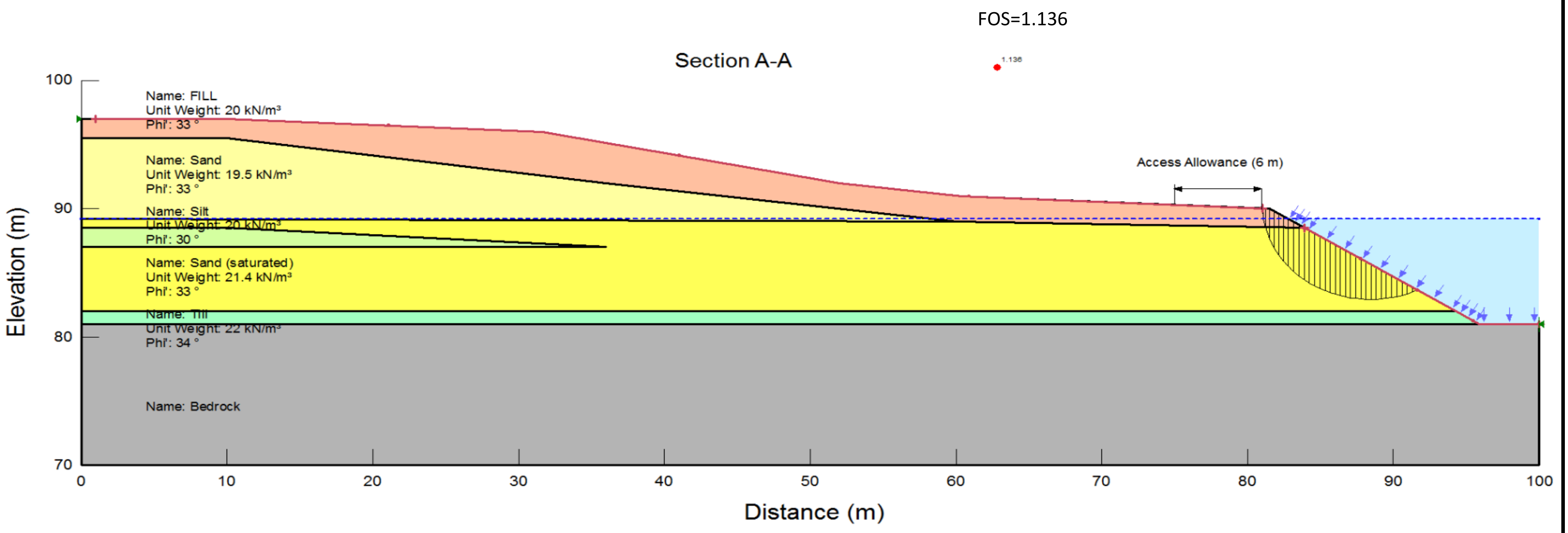
Stantec

Static Slope Stability Analysis (A-A)

2164 Old Prescott Road, Ottawa, ON

Figure E1

Project No. 121621894



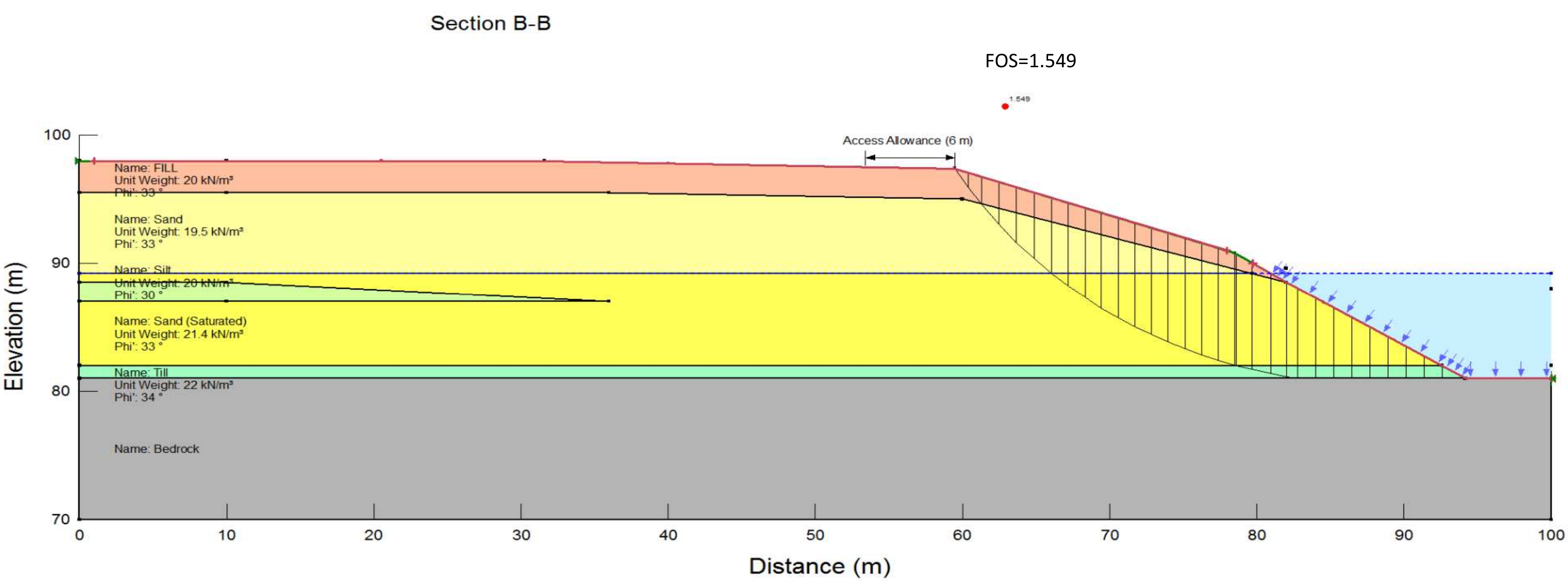
Stantec

Seismic Slope Stability Analysis (A-A)

2164 Old Prescott Road, Ottawa, ON

Figure E2

Project No. 121621894



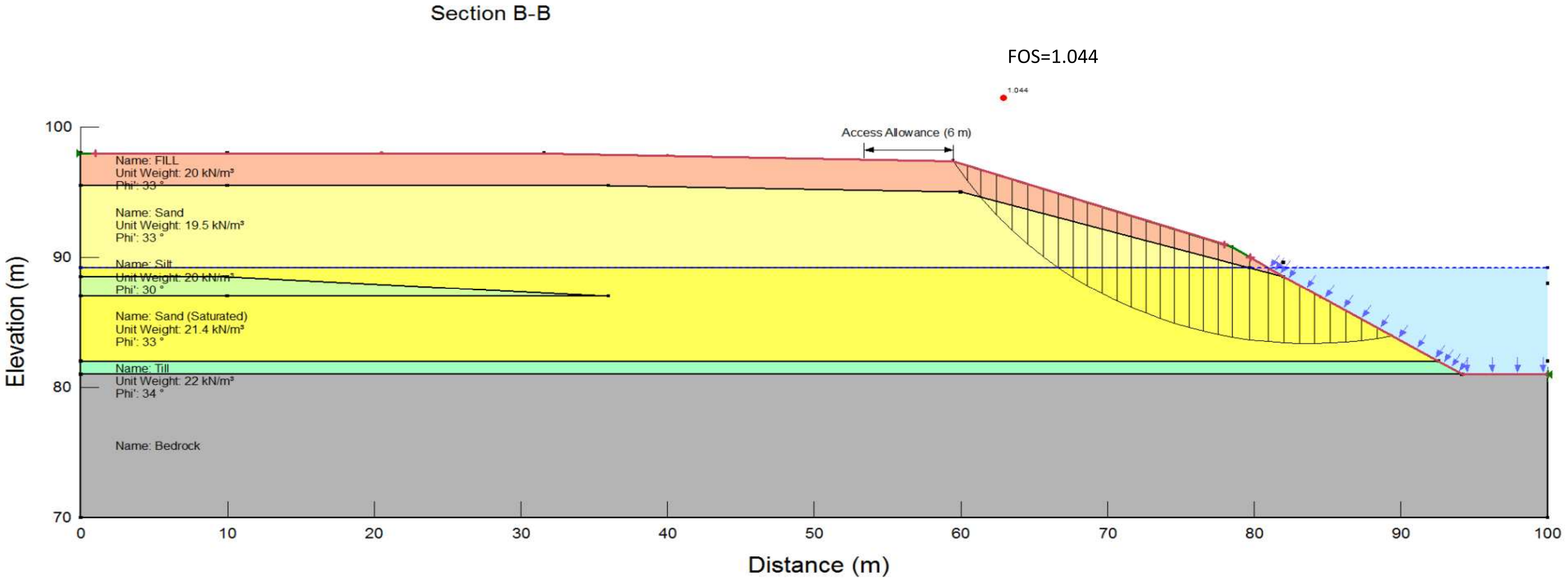
Stantec

Static Slope Stability Analysis (B-B)

2164 Old Prescott Road, Ottawa, ON

Figure E3

Project No. 121621894



Stantec

Static Slope Stability Analysis (B-B)

2164 Old Prescott Road, Ottawa, ON

Figure E4

Project No. 121621894