GEOTECHNICAL INVESTIGATION REPORT

SOUTHMINSTER CHURCH
1040 BANK STREET

OTTAWA, ONTARIO

Prepared for:

Windmill Development Group Ltd.

Date: May, 2017

Project No.: 161-17230-00

WSP Canada Inc.
2611 Queensview Drive,
Ottawa, ON K2B 8K2 Canada

Phone: 613-829-2800
Fax: 613-829-8299
www.wspgroup.com

TABLE OF CONTENTS

APPENDICES

1 INTRODUCTION

1.1 CONTEXT

WSP Canada Inc. (WSP) was retained by Windmill Development Group Ltd. to conduct geotechnical and environmental investigations at the Southminster Church located at 1040 Bank Street, in Ottawa, Ontario. The purpose of these investigations was to obtain subsurface information at the site by means of exploratory boreholes.

This report presents the findings of the investigation and provides comments and recommendations related to the geotechnical aspects of the project. Concurrent with the geotechnical investigation Phased Environmental Site Assessments were also completed, the results of which are submitted under separate cover.

1.2 SITE DESCRIPTION AND PROJECT UNDERSTANDING

1.2.1 SITE DESCRIPTION

The site is located in the northwest quadrant of the intersection of Bank Street and Aylmer Avenue in a mixed residential, commercial and parkland area of the City of Ottawa. The site location is shown on Drawing Nos. 1 and 2.

The site is irregular in shape with approximately 50 metres of frontage along Galt Street to the west, 90 metres of frontage along Aylmer Avenue to the south and 15 metres along Bank Street to the east. The site area is approximately 3,300 square metres (0.33 hectares).

There are two connected structures presently on the Site. These include the original church building located in approximately the centre of the site, as well as a more recent addition on the west along Galt Street (see Drawing No. 2). The majority of the area around the buildings is grassed (landscaped) with concrete pathways leading to the entrances. Asphalt surfaces are present in a parking area along Galt Street, as well as in the northern portion of the site behind the addition on the west side of the site (which is currently used as a play area for a daycare operated at the Church).

The majority of the site is relatively flat, at approximately the same elevation as the surrounding roads (Bank St., Aylmer Ave. and Galt St.). The northern edge of the property is located at the top of a slope which dips towards the Rideau Canal to the north. Based on topographic surveys provided by the City, the slope drops a total of approximately 10 m (from the edge of the subject property to the bottom of the retaining walls which form the edge of the Rideau Canal). The overall slope angle is on the order of 10 degrees. This overall angle, however, includes several flat sections including a City pathway, Echo Drive and Colonel By Drive. Between these flat areas the slope is locally steeper. The majority of the slope is treed and/or landscaped with the remaining portions being hard-surfaced roads and pathways.

1.2.2 PROJECT UNDERSTANDING

The overall project includes redevelopment of the west portion of the site along Galt St. This redevelopment will include demolition of the existing addition to the church in the west portion of the site, and construction of new residential buildings.

The new buildings will include a mid-rise multi-unit building (currently proposed to be 6 storeys) with one floor of below-grade parking located in the northwest corner of the site, as well as a row of three storey townhomes in the southeast corner of the site.

Preliminary design drawings for the proposed development are included in Appendix B.

1.3 OBJECTIVES AND LIMITATIONS

The current report was prepared at the request and for the sole use of Windmill Development Group Ltd. Corp. according to the specific terms of the mandate given to WSP. The use of this report by a third party, as well as any decision based upon this report, is under this party’s sole responsibility. WSP may not be held accountable for any possible damages resulting from third party decisions based on this report.

Furthermore, any opinions regarding conformity with laws and regulations expressed in this report are technical in nature; the report is not and shall not, in any case, be considered as a legal opinion.

Information in this report is only valid for the borehole locations as described.

Reference should be made to the Limitations of this Report, attached in Appendix G, which follows the text but forms an integral part of this document.

2 SITE INVESTIGATION

2.1 SCOPE OF WORK

The scope of work for this assignment included:

- A desk study and review of existing geotechnical information in the general area;
- Laying out the boreholes and obtaining utility locates at the project site;
- Drilling five exploratory boreholes;
- In-situ soil sampling and testing, including Standard Penetration Testing (SPT);
- Obtaining soil samples for additional review and laboratory testing;
- Laboratory testing;
- Geotechnical analysis; and
- Preparation of this report which presents the results of the investigation and provides geotechnical recommendations related to the design and construction of the proposed development.

2.2 INVESTIGATION PROCEDURES

The geotechnical investigation was carried out in December 2016.

2.2.1 DESK STUDY

Surficial geology maps indicate that the area is underlain by Offshore Marine deposits consisting of sensitive clay, silty clay and silt with minor sand deposits which are in turn underlain by stone-poor, sandy silt to silty sand glacial till deposits. Bedrock geology maps indicate the bedrock in the general area consists of limestone, dolostone, shale, arkose and sandstone of the Shadow Lake Formation.

2.2.2 FIELD INVESTIGATION

The field investigation was carried out between December 20 and 23, 2016 and included the drilling of five boreholes at approximately the locations shown in Drawing No. 2.

The boreholes were advanced using a truck mounted drill rig supplied and operated by George Downing Estates Drilling (Downing) of Hawkesbury, Ontario. The boreholes were advanced using hollow-stem augers to auger refusal at a maximum depth of 16.0 m below the existing surface elevation. In borehole BH16-2 drilling was extended past the depth of refusal by NQ coring to a maximum depth of 21.9 m below the existing ground surface. Soil and rock samples retrieved during drilling were logged and visually classified in the field by a member of WSP’s geotechnical staff. In-situ tests including Standard Penetration Testing (SPT) were carried out at regular intervals.

Monitoring wells were installed in all five boreholes drilled at the site to allow for long-term groundwater level monitoring.

The borehole locations are shown in Drawing No. 2. Borehole logs are included in Appendix C of this report.

The ground surface elevation noted on the borehole logs for each borehole was based on a local benchmark, the top of the steel grated storm sewer catch basin located on the south side of Aylmer Avenue approximately 25 m west of the Aylmer Avenue and Bank Street intersection and was assigned a local elevation of 72.0 m. A detailed geodetic survey has not been completed as part of this investigation.

2.2.3 LABORATORY TESTING

Upon completion of drilling and in-situ testing, soil samples were returned to WSP’s laboratory for further examination, classification and testing. A laboratory testing program, which was carried out on selected representative soil samples, included the determination of natural water content, grain size distribution and chemical analyses of soil corrosivity.

The results of natural water content tests are included on the relevant borehole logs in Appendix C. The results of the grain size distributions are summarized on the individual borehole logs and presented in Appendix D. Chemical testing to determine sulphate content, chloride content, pH and resistivity was also carried out on selected soil samples obtained during drilling. The results of these tests are also included in Appendix D.

3 SUBSURFACE GEOTECHNICAL CONDITIONS

The subsurface conditions encountered within the boreholes at the site are discussed in the following sections. Detailed descriptions of the soil and groundwater conditions encountered at each of the borehole locations are included in the individual borehole logs in Appendix C.

3.1 SOIL CONDITIONS

3.1.1 PAVEMENT STRUCTURE

Boreholes BH16-1 and BH16-2 were drilled in asphalt surfaced areas (BH16-1 in the southeast corner of the parking area along Galt St. and BH16-2 in the asphalt surfaced area to the north of the building). The pavement structures encountered at these two locations included asphaltic concrete and crushed sand and gravel base, and are summarized below.

Table 1 – Results of Grain Size Analyses for Sandy Silt/Silty Sand Fill

Borehole No.	Asphaltic Concrete (thickness)	Granular Base (thickness)
BH16-1 (Parking Area)	20 mm	330 mm
BH16-2 (Paved Area North of Building)	50 mm	130 mm

At both locations the granular base material was underlain by sand and silt fill material.

3.1.2 TOPSOIL

Topsoil was present at the ground surface in Boreholes BH16-3, BH16-4 and BH16-5 (which were drilled in landscaped areas). The thickness of this topsoil layer ranged from 130 mm to 180 mm at the borehole locations.

3.1.3 SAND/SILTY SAND (FILL)

Underlying the granular base in Boreholes BH16-1 and BH16-2 and the topsoil in Boreholes BH16-3, BH16-4 and BH16-5 a deposit of sand and silty sand (with localized areas being sandy silt) was encountered. This layer extended to depths ranging from 3.8 m to 4.8 m below the existing ground surface at the various borehole locations. Significant portions of this material are likely to be fill.

Standard penetration tests carried out within this deposit gave SPT ‘N’ values ranging from 2 blows per 305 mm of penetration to 31 blows per 305 mm of penetration. These values indicate a loose to dense state of packing, though the majority are in the loose to compact range.

Grain size curves of selected sample of the sand/silty sand are included in Appendix D. A summary of this grain size distribution is also presented in the table below. It should be noted that grain size distribution testing was carried out on sample obtained through SPT testing, which does not recover coarse gravel, cobble and boulder sized particles.

Table 2 – Results of Grain Size Analyses for Sand/Silty Sand/Sandy Silt

Borehole No.	Sample No.	Grain Size Distribution		
		% Gravel	% Sand	% Fines
BH16-1	GS-2	8	77	15
BH16-1	SS3B	3	82	15
BH16-2	SS2	0	31	69
BH16-2	SS5	0	98	2
BH16-1	SS1B	0	21	79
BH16-3	SS4	0	99	1
BH16-3	SS5	0	97	3

The measured water content of the samples of the sand/silty sand/sandy silt were determined to range between 1 percent and 25 percent.

3.1.4 CLAYEY SILT

In Borehole BH16-3, underlying the sand/silty sand a deposit of clayey silt was encountered. This deposit was present from a depth of 4.9 m to 5.6 m below the existing ground surface in Borehole BH16-3 and was not encountered in any of the other boreholes drilled at this site.

3.1.5 GLACIAL TILL

Underlying the clayey silt in Borehole BH16-3 and the sand/silty sand in the other four boreholes glacial till was encountered. The glacial till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix ranging in consistency from sand to silty sand with trace to some clay. The glacial till extended to the depth of drilling (14.9 m to 21.9 m) in all the boreholes drilled at this site.

The depth of auger refusal ranged from ranging from 14.9 m to 16.0 m below the existing ground surface. In Borehole BH16-2 the borehole was extended beyond the refusal depth and cored using “N” sized diamond coring equipment to a depth of 21.9 m below the existing ground surface and from the depth of auger refusal to the depth of termination encountered cobbles and boulders.

Standard penetration tests carried out within this deposit of glacial till gave SPT ‘N’ values ranging from 7 blows per 305 mm of penetration to 50 blows for 50 mm of penetration. These values generally indicate a compact to very dense state of packing, however one SPT ‘N’ value of 7 indicates a loose state of packing. In some cases higher ‘N’ values likely reflect the presence of cobbles and boulders, rather than the state of packing of the soil matrix.

At Borehole BH16-2 auger refusal was encountered at a depth of 15.2 m below the existing ground surface. The borehole was then further advanced using diamond coring to a depth of 21.9 m, through till-like material with a significant number of cobbles and boulders.

Grain size curves of selected sample of the glacial till are presented in Appendix D. A summary of this grain size distribution is also presented in the table below. It should be noted that grain size distribution testing was carried out on a sample obtained through SPT testing, which does not recover coarse gravel, cobble and boulder sized particles. Because of this the grain size distribution obtained through drilling may be finer overall than some portions of the material in the field.

Table 3 – Results of Grain Size Analyses for Glacial Till

Borehole No.	Sample No.	Grain Size Distribution		
		% Gravel	% Sand	% Fines
BH16-1	SS7	0	93	7
BH16-1	SS10	2	94	4
BH16-1	SS12	39	53	8
BH16-1	SS11	45	50	5
BH16-2	SS10	16	80	5
BH16-2	SS14	29	64	7
BH16-2	SS18	28	61	11
BH16-3	SS11	14	85	1
BH16-3	SS19	48	48	4

The measured water content of the samples of the glacial till were determined to range between 2 percent and 20 percent.

3.1.6 AUGER REFUSAL/BEDROCK

Auger refusal was encountered in all of the boreholes drilled at the site and the depth of auger refusal ranged from 14.9 m to 16.0 m below the existing ground surface. Borehole BH16-2 was extended beyond the refusal depth and cored using “N” sized diamond coring equipment and to a depth of 21.9 m the coring encountering cobbles and boulders to a depth of 21.9 m below the existing ground surface. The depth of auger refusal at this site may represent the bedrock surface, however it is likely that refusal was due to boulders and cobbles within the glacial till layer (as was the case at BH16-2, which was cored past refusal).

A historical borehole record on the east side of the site (near the south abutment of the bridge across the canal on Bank St.) indicates the bedrock surface to be approximately 24 m below the ground surface.

3.2 GROUNDWATER CONDITIONS

Groundwater monitoring wells were installed in all of the boreholes advanced at the site to allow for measurement of stabilized groundwater levels and subsequent sampling of groundwater.

The groundwater level within the monitoring wells was measured between 3 and 6 days after installation (December 2016) and found to range from 12.7 m to 14.0 m below the existing ground surface. The low groundwater levels are likely due to natural drainage towards the north (where the land drops more than 10 m to the level of the canal).

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations as well as fluctuations in response to major weather events. It should also be noted that the canal was empty at the time of the investigation, and groundwater levels were near the elevation of the canal. It is likely, given the sandy nature of the soils, that the groundwater level could rise as the canal is filled in the spring.

Table 4 – Measured Groundwater Levels (December 2016)

Borehole No.	Approximate Ground Elevation (m)	Water Level (m)	Water Level Elevation (m)
BH16-1	73.8	12.7	61.1
BH16-2	72.4	12.8	59.6
BH16-3	72.3	13.0	59.3
BH16-4	74.0	14.0	60.0
BH16-5	71.9	13.0	58.9

The monitoring wells have been left in place after this investigation to allow for subsequent water level measurements and/or additional groundwater sampling and should be decommissioned during construction.

3.3 SUMMARY

A summary of the soil and groundwater conditions encountered at the Site is presented in the table below.

Table 5 – Simplified Stratigraphy and Groundwater Elevations

Borehole No.	Simplified Stratigraphy (Depth in metres)						Measured Ground Water Depth	Notes
	Pavement Structure		Topsoil	Sand/ Silty Sand and Fill	Clayey Silt	Glacial Till		
	Asphaltic Concrete	Granular Base						
BH16-1	0 – 20 mm	20 mm – 350 mm	--	350 mm – 4.6 m	--	4.6 – 15.7	12.7	Auger Refusal at 15.7 m
BH16-2	0 – 50 mm	50 mm – 180 mm	--	180 mm - 3.7 m	--	3.7 - 21.9	12.8	Drilling terminated at 21.9 m
BH16-3	--	--	0 - 180 mm	180 mm - 4.9 m	4.9 - 5.6	5.6 – 15.2	13.0	Auger Refusal at 15.2 m
BH16-4	--	--	0 - 150 mm	150 mm - 4.0 m	--	4.0 – 16.0	14.0	Auger Refusal at 16.0 m
BH16-5	--		0 - 130 mm	180 mm - 4.6 m	--	4.6 – 14.9	13.0	Auger Refusal at 14.9 m

4 RECOMMENDATIONS

4.1 GENERAL

This section of the report provides engineering guidelines related to the geotechnical design aspects of the project based on our interpretation of the available information described herein and project requirements. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities.

4.2 FROST PROTECTION

The depth of frost penetration for the site can be assumed to be 1.8 m. Exterior foundations of heated structures should be provided with a minimum of 1.5 m of soil cover (or equivalent insulation) for the purposes of protection from frost. Foundations of unheated structures should be provided with a minimum of 1.8 m of earth cover (or equivalent insulation).

4.3 SEISMIC CONSIDERATIONS

4.3.1 LIQUEFACTION POTENTIAL

The site stratigraphy includes an upper layer of loose to compact sand and silty sand (ranging from 3.7 m to 4.9 m deep at the borehole locations) underlain by compact to very dense glacial till (generally a silt and sand matrix with gravel, cobbles and boulders).

Measured groundwater levels range from 12.7 m to 14 m below the ground surface in the five boreholes installed at the site. The loose to compact sand and silty sand is present to depths of less than 5 m and is therefore above the groundwater level (even allowing for normal seasonal variations). This layer is therefore not considered to be susceptible to seismic liquefaction.

The soils below the groundwater level include predominantly very dense glacial till with SPT ‘N’ values generally in excess of 50. These soils would not be considered to be susceptible to liquefaction under a design earthquake event with a 2% probability of exceedance in 50 years (which is consistent with current building code requirements for seismic design).

4.3.2 SEISMIC SITE CLASSIFICATION

Based on the borehole information the site may be assumed to be Class ‘D’ for seismic site response.

It may be possible to provide a more favourable or refined site classification if site specific shear wave velocity measurements are obtained. A shear wave velocity sounding could be completed as part of detailed design if desired.

4.4 SITE GRADING

It is understood that the current design concept will not involve any significant changes to the site grading. Based on the soil present at the site, raising of the site grade would not be expected to cause any adverse effects due to settlement.

If the final design involves any raising of the grade along the north side (along the crest of the slope) then additional review of the stability of the small portion of the upper slope immediately to the north of the property should be undertaken during detailed design.

4.4.1 FOUNDATION OPTIONS

The proposed development is understood to include one mid-rise tower (six storeys above grade) with a single floor of underground parking (assumed to extend to approximately 3 m below grade) as well as a row of three storey townhomes.

The site is underlain by loose sand (at least portions of which are fill material) as well as compact to very dense glacial till. The loose fill material extends to depths of 3.7 m and 4.9 m in the two boreholes in the northwest corner of the site (where the six storey tower is proposed) and to 4.6 m in the southwest corner of the site (nearest to the row of townhomes).

Shallow foundations (either individual spread footings or a raft foundation) placed on undisturbed glacial till material may be feasible for the proposed buildings. Shallow foundations would, however, require excavation to depths of 4 m to 5 m (below existing grade) in order to ensure that they are placed below the loose sandy and fill material encountered in all of the boreholes. It is understood that the existing church building will remain on site, and may require underpinning or support during construction if large, deep excavations are completed immediately adjacent to the existing structure.

If shallow foundations are not feasible (either due to the depth of excavation required, or the bearing resistance which can be achieved – particularly for the six storey tower) then deep foundations (piles) may be considered.

4.4.2 SHALLOW FOUNDATIONS

Two shallow foundation options exist for the site, individual spread footings or raft foundations. Each of these is discussed in additional detail below.

SPREAD FOOTINGS

Individual spread footings may be placed on the compact to very dense glacial till which was encountered at depths of 3.7 m to 4.9 m below the existing ground surface. For typical square and strip footings founded within undisturbed native soils the following bearing resistances may be assumed:

Table 6 – Geotechnical Bearing Resistance for Spread Footings

Burial Depth[1]	**Geotechnical Bearing Resistance for Various Foundation Widths (kPa) (Factored ULS**[2] **/ SLS)**			
	1 m	2 m	3 m	≥ 4 m
0.5 m	125/80	165/110	210/140	250/165
1.0 m	210/140	250/165	295/195	370/220
≥1.5 m	280/170	320/200	360/220	400/240

---

[1] Below finished floor elevation (i.e. for underground parking the burial depth should be taken from the basement level, not from the ground surface).

[2] Includes a geotechnical resistance factor of 0.5.

Provided that the foundation subgrade is properly prepared, and not unduly disturbed by construction activities, total and differential settlements for shallow spread and strip footings constructed on undisturbed soil or properly placed and compacted engineered fill would be expected to be less than 25 mm and 20 mm, respectively under normal conditions.

All bearing surfaces should be checked, evaluated and approved by WSP at the time of construction prior to placement of any concrete.

RAFT FOUNDATION

If individual footings cannot accommodate the loading from the proposed structure then consideration can be given to a thickened raft foundation (again on undisturbed native soils present at 4 m to 5 m below the existing ground surface).

The geotechnical resistance of a raft foundation is based on the settlement characteristics of the soil below the slab, as well as the magnitude and geometry of loading and the stiffness of the raft itself. The geotechnical parameter typically used for analysis of settlement below a raft or slab is the vertical modulus of subgrade reaction. Based on the field investigation, a modulus of subgrade reaction (kv1) of 40 MPa/m may be used for a properly prepared subgrade.

The modulus of subgrade reaction is not a fundamental soil property, but is dependent upon the size and shape of the loaded area, soil type, relative stiffness of the raft and soil, duration of loading, etc. As a result, the modulus for a 300 mm square footing is typically used as a standard basis. For loaded areas greater than 300 mm square the above value should be multiplied as follows:

$$kvb = kv1\left[\frac{b + 0.3}{2b}\right]^2$$

where

k_{vb} = the modulus for actual loaded area of width b
b = width of the loaded area;

WSP can provide additional guidance related to the design of a raft foundation during detailed design, if required.

4.4.3 DEEP FOUNDATIONS

The most cost-effective type of pile is likely to be driven steel piles, either H-piles or closed ended pipe piles. Piles may be driven to rock or may be designed to provide compressive resistance through a combination of shaft friction and end bearing in glacial till.

4.4.3.1 COMPRESSIVE RESISTANCE

END-BEARING PILES DRIVEN TO ROCK

Steel piles driven to rock typically generate relatively high ultimate capacities, often equal to or in excess of the structural capacity of the steel section. For the purposes of preliminary design, steel piles driven to rock could be assumed to generate an ultimate geotechnical resistance equal to the structural capacity of the steel section. A geotechnical resistance factor of 0.4 should be applied to this value to obtain the factored geotechnical resistance.

Settlements for piles driven to rock are typically negligible, and the geotechnical resistance mobilized at 25 mm of settlement would be expected to exceed the factored ULS value. Therefore SLS considerations will not govern the overall design.

Piles will be driven through a thick deposit of stiff glacial till soils which likely contain cobbles and boulders. All piles should be fitted with appropriate driving shoes in order to protect the pile tip during driving. Any battered piles should be driven with rock points to avoid sliding of piles along the rock surface. In addition, a relatively heavy pile section will likely be required to resist driving stresses and prevent deflections during driving. Even with these measures, some allowance should be made for wasting of piles which become damaged or reduced design capacities for piles which cannot be driven to bedrock.

Piles driven to bedrock will need to be on the order of 25 m to 30 m long and driven through relatively dense overburden soils containing cobbles and boulders. Heavy piling equipment and large hammers capable of generating sufficient driving energy will be required to mobilize the full geotechnical resistance of the pile.

If piles are to be driven to bedrock, consideration should also be given to advancing additional boreholes within the footprint to further confirm/define the bedrock surface and driving conditions at depth.

COMBINED FRICTION AND END-BEARING H-PILES

As an alternative to end-bearing piles driven to rock, steel piles could be designed on the basis of a combination of friction and end-bearing resistance. Friction piles would generally have a somewhat lower capacity, but would not require driving to bedrock, which is a significant depth (through dense soils) at this particular site. This offers some advantages:

- The piles can be shorter, and can typically be driven with smaller equipment;
- There may be less risk of damage to piles due to heavy driving (particularly the driving needed to ensure the pile is adequately set in rock);

At this particular site the piles could be driven into dense glacial till below (but not necessarily driven to refusal on rock).

For preliminary design purposes the ultimate resistance of a pile founded in the dense till may be calculated based on contributions from both toe resistance and shaft resistance.

TOE RESISTANCE

The toe resistance of a pile driven into glacial the glacial till may be assumed to be:

$$R_t = N_t \sigma_v' A_t$$

Where:

R_t = the unfactored toe resistance of the pile (kN);
N_t = end bearing capacity factor (use 50 for glacial till);
σ_v' = vertical effective stress at the pile toe (for preliminary design use 0 kPa at the pile cap; 180 kPa at 10 m below existing ground surface and 320 kPa at 25 m depth, interpolating between for the actual proposed toe depth;
A_t = the pile toe area (m^2).

SHAFT RESISTANCE

The ultimate shaft resistance acting along the length of a pile driven into the glacial till may be assumed to be:

$$R_s = \beta\ \sigma_v'\ A_s$$

Where:

R_s = the unfactored pile shaft resistance (kN);
β = the shaft resistance coefficient (use 0.8);
σ_v' = vertical effective stress along the pile shaft (for preliminary design use 0 kPa at the pile cap; 180 kPa at 10 m below existing ground surface and 320 kPa at 25 m depth, interpolating between for the actual toe depth;
A_s = the pile shaft area (m^2).

RESISTANCE FACTORS

The above values (both for toe resistance and shaft resistance) are unfactored. A geotechnical resistance factor of 0.4 (static analysis only), 0.5 (static analysis and confirmation through PDA testing) or 0.6 (static analysis with confirmation through static load testing) should be applied to the geotechnical resistance.

The settlement of foundations supported by friction piles will be dependent upon the dimensions of the pile group, the length of the piles and the pile loading. However, friction piles in stiff soils typically develop their full geotechnical capacity at relatively small deflections. It is expected that a properly designed and constructed pile foundation would experience total and differential settlements of less than 25 mm and 20 mm respectively.

4.4.3.2 UPLIFT RESISTANCE

The uplift resistance of a pile will be as a result of skin friction acting along the surface area of the embedded pile.

The unfactored ultimate uplift resistance of a driven pile may be assumed to be equal to the ultimate shaft friction calculated as described above. A resistance factor of 0.3 should be applied to this value, to obtain the factored geotechnical uplift resistance. The dead weight of the pile itself (with an appropriate structural resistance factor for dead weight) may also be added to the geotechnical resistance in calculating the total uplift resistance.

The total uplift resistance of a pile group is the lesser of the sum of the individual pile resistances as described above, or the resistance of a single “block” of soil with a perimeter equal to the perimeter of the pile group (the mass of the soil inside the “block” may be included in the calculation; use a soil weight of 18 kN/m^3 above the water table and 8 kN/m^3 below).

4.4.3.2 LATERAL RESISTANCE

The lateral resistance of long piles is typically governed by limiting the deflection which will occur under loading to some acceptable level. The geotechnical parameter most commonly used to determine lateral deflection of piles is the coefficient of horizontal subgrade reaction (k_s) which may be assumed to be:

$$k_s = \eta_h\ (z/d)$$

Where:
k_s = the modulus of subgrade reaction (kN/m^3);
η_h = horizontal subgrade reaction coefficient (kN/m^3 use 2,200 from the underside of pile cap to 5 m depth, 6,600 from 5 m depth to 10 m depth and 4,400 below 10 m depth);
z = depth to the point being considered;
d = pile diameter (or width).

This parameter is associated with acceptable deflections, and therefore represents an unfactored SLS value.

The value above is for a single pile. Group interaction must be considered when piles are spaced closely together. Group effects may be accounted for by reducing the coefficient of horizontal subgrade reaction (k_s) by an appropriate factor as follows:

Table 7 – Coefficient of Horizontal Subgrade Reaction Reduction Factors

Pile Spacing in Direction of Loading (d = pile diameter)	Reduction Factor
6d	1.0
3d	0.25

Values for other spacings may be interpolated from the above. No reduction is required for the first row of piles (i.e. the row which bears against undisturbed soil with no piles in front).

4.4.3.3 NEGATIVE SKIN FRICTION

The raising of the grade and/or permanent lowering of the groundwater table may cause settlement of the existing soils which will in turn cause negative friction or down drag on the piles. Under either of these conditions the potential exists to develop negative skin friction along the piles and this should be considered in the final design.

The magnitude of negative skin friction depends on the pile loading, dimensions and the final configuration of the site, as well as the details of the permanent below-grade portions of the building (in particular drainage) and will need to be confirmed during detailed design based on these factors. For preliminary design, however, the negative skin friction can be assumed to be equal to the shaft friction as calculated for uplift resistance above (the resistance factor of 0.3 should not be applied).

Negative friction is typically only considered in conjunction with dead and sustained live loads (not transient loads such as wind, earthquake and transient live loads) in evaluating the structural capacity of the pile. Negative friction does not impact the geotechnical resistance of the piles.

4.4.3.4 CONSTRUCTION CONSIDERATIONS

The piles will be driven to bedrock (which is expected to be 25 m to 30 m below the existing ground surface) or into very dense glacial till. Pipe piles (if used) should be driven closed-ended. Some allowance should be made for wasting of piles which become damaged or for reduced design capacities for piles which cannot be successfully driven to rock or through cobbles and boulders.

Appropriate piling equipment and hammers capable of generating sufficient driving energy will be required to drive the piles to rock and mobilize the full geotechnical resistance of the pile. Allowance should also be made for re-striking a portion of the piles a minimum of 2 days after initial driving to confirm that relaxation has not occurred.

The piling specifications should be reviewed by WSP prior to tender, as should the contractor's submission (i.e. shop drawings, equipment, procedures and preliminary set criteria) prior to construction. Preliminary pile driving criteria should be established prior to construction using wave equation analysis (WEAP or similar) or other approved means and confirmed through a program of dynamic testing (PDA Testing) carried out at an early stage in the piling program. Additional PDA testing should be used to confirm the pile capacities at regular intervals as the project progresses. A properly planned and implemented PDA testing program would also justify increasing the geotechnical resistance factor to 0.5 (see Section 4.4.3.1).

All piling operations should be supervised on a full-time basis by WSP to monitor pile locations, plumbness, pile set, re-striking, etc. and to confirm that the design and construction of the piles is as anticipated in preparing the recommendations included in this report.

4.5 SLABS ON GRADE

Concrete slabs-on-grade should be supported on at least 200 mm of compacted, free-draining, well graded crushed sand and gravel (OPSS Granular A). The crushed sand and gravel should be placed over a properly prepared subgrade or engineered fill (if used to raise the grade of the site) and compacted to 100% of the materials Standard Proctor Maximum Dry Density (SPMDD) using a heavy vibratory roller. All subgrades should be reviewed by WSP prior to placement of the Granular A and slab-on-grade.

4.6 LATERAL EARTH PRESSURES

The lateral earth pressure acting on below-grade walls, retaining walls, etc. may be calculated using the following expression:

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

Where

P = lateral earth pressure (kPa) acting at depth h

K = earth pressure coefficient; for unrestrained walls and structures where some movement is acceptable use a coefficient of active earth pressure (K_a) equal to 0.3, for restrained walls which cannot move use the coefficient of earth pressure at rest (K_0) equal to 0.5

γ = the density of the backfill; use 21 kN/m^3 for compacted granular backfill

h = the depth to the point of interest (m)

q = the magnitude of any design surcharge at the ground surface; a minimum nominal surcharge of 10 kPa is recommended, a higher value should be used if appropriate for the building/site design

A minimum lateral earth pressure of 12 kPa should be used to account for the effects of compaction-induced earth pressure (i.e. if the calculated earth pressure at a given point is less than 12 kPa, use 12 kPa).

Appropriate allowances should also be included for the loading of any adjacent structures on the proposed below-grade walls (for example the buildings to the west) and adequate foundation information for nearby buildings should be reviewed during detailed design.

The above values assume free-draining granular backfill will be used and drainage will be provided. If this is not the case then the above values may need to be adjusted based on the soil type used, and water pressures should be considered in the calculation of lateral pressures. WSP can provide additional guidance based on actual building plans if required.

Earth pressures will be higher under seismic loading conditions. In order to account for seismic earth pressures the total earth pressure during a seismic event (including both the seismic and static components) may be assumed to be:

$$\sigma_h(z) = K_a \gamma z + (K_{AE} – K_a) \gamma (H-z)$$

Where $\sigma_h(z)$ = the total earth pressure at depth z (kPa);

K_a = the active earth pressure coefficient (0.3);

γ = the unit weight of soil (21 kN/m^3 for granular fill or glacial till);

K_{AE} = the combined active earth pressure and seismic earth pressure coefficient (use 0.8);
H = the total height of the wall (m)
z = the depth below the top of the wall (m)

The above earth pressure values (both static and seismic) are unfactored values.

4.7 BASEMENT WALL BACKFILL AND DRAINAGE

The earth pressure values provided above assume the wall will remain in a drained condition.

If shoring is used to support temporary excavations, and sufficient space does not exist between the formwork and the shoring to allow for conventional backfilling and compaction, then the backfill may consist of clear stone placed using a chute or similar method. Where this clear stone could come into contact with soil it should be wrapped with a non-woven geotextile to prevent migration of fines into the stone. If basement walls are cast against a shoring wall then a suitable drainage board (such as Miridrain or DeltaDrain) should be installed to ensure the wall remains in a drained condition.

In any case the backfill should be provided with a perforated rigid pipe subdrain encased in 300 mm of clear stone, which is completely wrapped with a non-woven geotextile. All drains should provide positive drainage to the City sewer or a suitable sump. Typical damp-proofing should be provided for below-grade walls.

4.8 BACKFILLING AND COMPACTION

Backfill for below-grade walls, retaining walls, foundation excavations, etc. should comprise free draining OPSS Granular A or Granular B materials. Backfill should be placed in shallow lifts, not exceeding 300 mm loose thickness, and compacted to 100% SPMDD where it is placed below structures, or 98% in other areas.

To avoid damaging or laterally displacing the structures, care should be exercised when compacting fill adjacent to new structures. Where possible, backfilling should be carried out on both sides of buried structures simultaneously. Heavy equipment should be kept a minimum of 1 m away from the structure during backfilling. The 1 m width adjacent to the wall should be backfilled using hand-operated equipment unless otherwise authorized.

4.9 EXCAVATIONS AND GROUNDWATER CONTROL

It is understood that the currently proposed development will include a single storey of underground parking, as well as localized excavations for foundations, utilities, etc.

4.9.1 TEMPORARY EXCAVATIONS IN SOIL

All temporary excavations should be carried out in accordance with the most recent Occupation Health and Safety Act (OHSA). Part III of Ontario Regulation 213/91 deals with excavations. The soils which will be encountered in temporary consist primarily of sand/silty sand as well as glacial till. For the purposes of excavation planning these soils may be considered as Type 3 soil (i.e. 1:1 excavations above the water table or the depth of dewatering). The groundwater level measured at the site in December, 2016 varied from 12.7 m to 14.0 m below the existing ground surface and as such is below the anticipated depth of excavation. Soil classifications must be confirmed by qualified individuals as excavation proceeds and if necessary adjusted.

In areas which do not have sufficient space to accommodate sloped excavations a shoring system may be used to support the excavation. Shoring for this type of project would typically include tied back soldier pile and lagging or sheet pile walls, and would normally be designed and installed by a specialist contractor. Earth pressures acting on temporary shoring may be assessed as outlined in Section 4.7 above.

In addition to supporting the soils surrounding the excavation, the design of temporary support systems (and in particular the selection of the appropriate earth pressure coefficients and construction sequencing; higher design earth pressure coefficients may be required to limit deflection of the shoring) will need to consider the support requirements of adjacent structures (such as the adjacent church structure, which is likely founded on shallow foundations, as well as nearby roads, utilities and other infrastructure), the staging of shoring installation and the potential for deflection of shoring at various stages of construction.

Alternatively, consideration could be given to underpinning nearby structures to reduce the requirements for shoring to resist small ground movements and settlements associated with excavation.

4.9.2 GROUNDWATER CONTROL

Groundwater levels were measured at all five borehole locations (Boreholes 16-1 through 16-5) in December 2016. The groundwater level ranged from 12.7 m to 14.0 m below the existing ground surface. Groundwater levels can change and are subject to seasonal fluctuations as well as fluctuations in response to major weather events, however even allowing for raising of the groundwater due to seasonal variations and filling of the canal groundwater is likely to be below the depth of excavation for a single-storey below grade.

It is anticipated that localised seepage may be encountered due to perched water and during periods of heavy rainfall and/or snow melt. Seepage at these times would be expected to be manageable by pumping from suitable sumps.

The exact volume of water to be pumped depends not only upon the soil and groundwater conditions at the site, but on the size and depth of the proposed excavations. If excavations are kept to small, localized, relatively shallow areas (as is understood to be the case at this time given that the parking structure will not require any excavation below the water table) then groundwater quantities are not likely to be large.

Based on our current understanding of the site conditions and proposed development it is not expected the work would require registration under the MOECC Environmental Activity and Sector Registration (EASR) or a Permit to Take Water (PTTW). As the design progresses, WSP should review the proposed development plans for any changes and can provide additional recommendations related to the most appropriate permit (if any) for the project.

Discharging any pumped water into the City sewer will also require a discharge agreement and potentially treatment of the water (depending upon environmental impacts) to bring it within acceptable City standards.

4.10 SITE SERVICES

Water-bearing services should be placed a minimum of 2.4 m below grade to provide protection from frost (which is a typical City requirement). Alternatively, equivalent insulation cover may be provided in lieu of burial.

Details of the proposed site services are not available at this time, however it is assumed that they will include localized shallow trenches throughout the site. Trenches in soil can typically be temporarily supported using sloped excavations, (see Section 4.9.1) or trench boxes.

Bedding for site services should consist of a layer of OPSS Granular A compacted to 95% SPMDD which extends from 150 mm below the invert of the pipe to the spring line of the pipe. The use of clear stone as a bedding material is not recommended as the finer particles of the native soils and backfill may migrate into the voids of the clear stone, resulting in loss of pipe support. Cover material above the spring line should consist of OPSS Granular A or Granular B material with a maximum particle size of 25 mm. Cover material should be compacted to a minimum of 95% SPMDD (100% if below the building structures or slabs-on-grade).

Clay seals should be placed across utility trenches to prevent the trench acting as a conduit for groundwater flow.

4.11 PAVEMENT DESIGN

Detailed traffic loads have not been provided at this time. It is, however, our understanding that the site will only experience low-volume residential traffic. The following table provides typical pavement structures based on experience with similar projects.

Table 8 – Recommended Flexible Pavement Structure Thickness

Pavement Layer	Light Duty Parking (Cars)	Heavy Duty Parking (Delivery Trucks, Fire Route, Access Roads, etc.)
Asphaltic Concrete	40 mm HL 3 or SP 12.5 40 mm HL 8 or SP 19.0	40 mm HL 3 or SP 12.5 80 mm HL 8 or SP 19.0
OPSS Granular A Base	150 mm	150 mm
OPSS Granular B	300 mm	450 mm

A functional design life of eight to ten years has been used to establish the pavement recommendations. This represents the number of years to the first rehabilitation, assuming regular maintenance is carried out. If required, a more refined pavement structure design can be performed based on specific traffic data and design life requirements provided by the client.

Rigid pavements may also be considered in critical or adversely loaded areas (such as ramps, loading docks, etc.). Rigid pavements typically have better performance with less on-going maintenance than flexible pavements.

A typical rigid pavement structure for this type of development would include 200 mm of concrete overlying 400 mm of OPSS Granular A

It would be prudent to provide the same subgrade level (bottom of granular sub-base) across rigid and flexible pavement sections and thus prevent the need to construct frost tapers. If similar subgrade levels cannot be maintained then frost tapers should be added between the various granular thicknesses.

The concrete should satisfy the requirements of CAN/CSA A23.1 Class C-2 concrete with a minimum compressive strength of 32 MPa. The base should consist of granular base material and be compacted to 100 percent of its standard Proctor maximum dry density. Slab joints should be doweled for proper weight transfer between slabs.

The long term performance of the pavement is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure uniform subgrade moisture and density conditions are achieved. In addition, the need for adequate drainage cannot be over-emphasized. The finished pavement surface and underlying subgrade should be free of depressions and should be sloped to provide effective surface drainage toward catch basins. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas. Subdrains can also be placed at catch basins and along curb lines to further improve sub-surface drainage.

As part of the subgrade preparation, proposed parking areas and access roadways should be stripped of topsoil and other obvious objectionable material. Fill required to raise the grades to design elevations should conform to backfill requirements outlined in previous sections of this report. The subgrade should be properly shaped, crowned then proof-rolled in the full time presence of a representative of this office. Soft or “spongy” subgrade areas should be sub-excavated and properly replaced with suitable approved backfill compacted to 98% SPMDD. Base and sub-base layers should be compacted to 100% of SPMDD.

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

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

4.12 CORROSION AND CEMENT TYPE

Samples of the existing site soils were submitted to Exova Accutest for testing related to soil corrosivity and potential exposure of concrete elements to sulphate attack. The results of these tests are included in Appendix D and summarized in the table below.

Table 9 – Results of Soil Corrosivity Testing

Borehole/ Sample No.	Soil Type	Chloride (%)	pH	Electrical Conductivity (mS/cm)	Resistivity (ohm-cm)	Sulphate (%)
BH16-1/SS-3B	Sand Fill	0.026	8.7	0.74	1350	0.01
BH16-2/SS-2	Sandy Silt Fill	0.003	7.7	0.36	2780	0.03
BH16-2/SS-14	Glacial Till	0.008	8.9	0.35	2860	<0.01
BH16-3/SS-5	Sand	<0.002	8.5	0.08	12500	<0.01

The soil resistivity values measured in the silty clay suggest a corrosive environment for buried steel elements which should be considered in the design of any buried steel elements. The test results indicate a low to negligible soluble sulphate content and sulphate resistant Portland cement is not required.

4.13 SLOPE STABILITY

A preliminary slope stability assessment was carried out at this site to evaluate the stability of the existing slope, both in its current condition and after the proposed development. In general, slope failures can occur when the forces generated by the weight of the soil in a slope and external loads, such as foundation loads, seismic loads, weight from additional fill, exceed the shear strength of the soil within the slope.

4.13.1 SLOPE GEOMETRY

For this assessment, the slope geometries used in the analyses were based on the conceptual site plans provided as well as existing survey data obtained from the City of Ottawa.

The existing slope drops a total of approximately 10 m (from the edge of the subject property to the retaining walls which form the edge of the Rideau Canal). The overall slope angle is on the order of 10 degrees. This angle, however, includes flat sections including a City pathway, Echo Drive and Colonel By Drive. Between these flat areas the slope is locally steeper. The majority of the slope is treed and/or landscaped with the remaining portions being hard-surfaced roads and pathways.

4.13.2 GEOLOGICAL PROFILE

The subsurface profile and geology within the slope was inferred from the results of the boreholes from the current investigation and consisted of three main strata:

- An upper layer of loose to compact silt and sandy silt, extending to a depth of approximately 5 m, underlain by;
- A layer of compact to dense glacial till which was present in all boreholes, underlain by;
- A layer of dense to very dense glacial till with a larger proportion of cobbles and boulders which was present in all of the boreholes at varying depths (but has been assumed to begin at approximately 8 m depth based on BH16-1 which is closest to the slope).

Groundwater levels within the slope were inferred, in part, from the groundwater levels measured in the piezometers. The stability analyses for the conditions after the proposed development, however, also considered higher groundwater levels within the slope based on the assumption that the level near the toe will rise when the canal is filled and assuming some additional rise in groundwater levels could occur during periods of wet weather (i.e., the groundwater table has been conservatively assumed to be higher than observed during the investigation).

4.13.3 SOIL PARAMETERS AND LOADING

The shear strength of a soil is conventionally described using a Mohr-Coulomb criterion. This criterion describes the shear strength of a soil in terms of cohesive and frictional components. The magnitude of the frictional component depends on the stress acting perpendicular to the potential failure plane. From this criterion, the strength of a soil to resist shear stress (i.e., to resist sliding) is described by:

$$\tau = c' + \sigma' \cdot \tan \phi'$$

Where: τ = shear strength of the soil
c' = effective cohesion of the soil
σ' = effective normal stress (i.e., stress acting perpendicular to the shear plane)
ϕ' = effective internal friction angle

The static strength parameters assigned to the soils at these sites were based on the results of the in situ and laboratory testing as well as our experience with similar soils in eastern Ontario, and are as follows:

Table 10 – Summary of Soil Properties

Parameter Soil Type	Bulk Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Internal Friction Angle (degrees)
Loose to Compact Sand/Silty Sand	18	0	30
Compact Glacial Till	20	0	32
Very Dense Glacial Till	20	0	36

The analysis has been completed for both the static case, as well as under an assumed seismic loading (assuming a seismic event with a 2% chance of exceedance in 50 years, similar to the seismic requirements of the building code).

For the purposes of completing a preliminary analysis of the slope following the proposed development, the loading due to the new building was assumed to be applied as a uniform load at the base of the underground parking structure. This simplification is necessary at the preliminary design stage (and is conservative if either a raft foundation or piled foundation are adopted). If individual spread footings are adopted as a foundation scheme the stability analysis should be repeated using the actual geometry and loading of the foundation elements (though for preliminary assessment the simplifications are still believed to be a reasonable initial approximation).

4.13.4 RESULTS OF SLOPE STABILITY ANALYSIS

The static stability of the slopes was evaluated using limit equilibrium methods and SLOPE/W software to compute the factor of safety against instability. A slope with a factor of safety of less than 1.0 will likely fail and one with a factor of safety greater than one will likely stand. However, because the modelling is not exact and natural variations exist for all of the parameters affecting slope stability, a factor of safety of 1.5 is conventionally used to define an adequately stable slope, or alternatively to define the safe set-back distance from an unstable slope.

An analysis of the existing slope was completed for both the existing conditions as well as the proposed construction (incorporating a single-storey basement). Typical results for one section of the analysis (the most critical section as it is the area where the slope building/property line is closest to the crest of the slope) are included in Appendix E.

The results of the analysis are summarized in the Table below.

Table 9 – Summary of Soil Properties

Analysis	**Factor of Safety**	
	Static	**Seismic**
Initial Conditions – Overall Stability	2.44	1.56
Initial Conditions – Localized Stability	1.06	0.93
Proposed Construction	2.56	1.56

The existing slope was found to have a calculated Factor of Safety of 2.44 under normal static loading and 1.56 under seismic loading for large-scale, overall slope failures. The small portion of the upper slope between the subject site and the pedestrian pathway has a calculated Factor of safety of approximately 1, suggesting this localized area is only marginally stable. This area is, however, outside of the subject property and is not impacted by the development.

As can be seen from the table above, as well as the example results, the calculated Factors of Safety against instability of the slope following redevelopment are approximately 2.56 under static conditions for slip surfaces that would impact a building with one storey of underground parking, and greater than 1.56 under seismic loading. Generally these would be considered adequate.

Localized instability of the upper slope (between the property and the City pathway which runs along the upper portion of the slope) will remain after construction. This potential instability, however, does not impact the proposed development.

5 CLOSURE

The Limitations of Report, as presented in Appendix E, are an integral part of this report.

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

WSP Canada Inc.

Report prepared by:

by:

Daniel Wall

Daniel Wall, B. Eng.
Geotechnical E.I.T

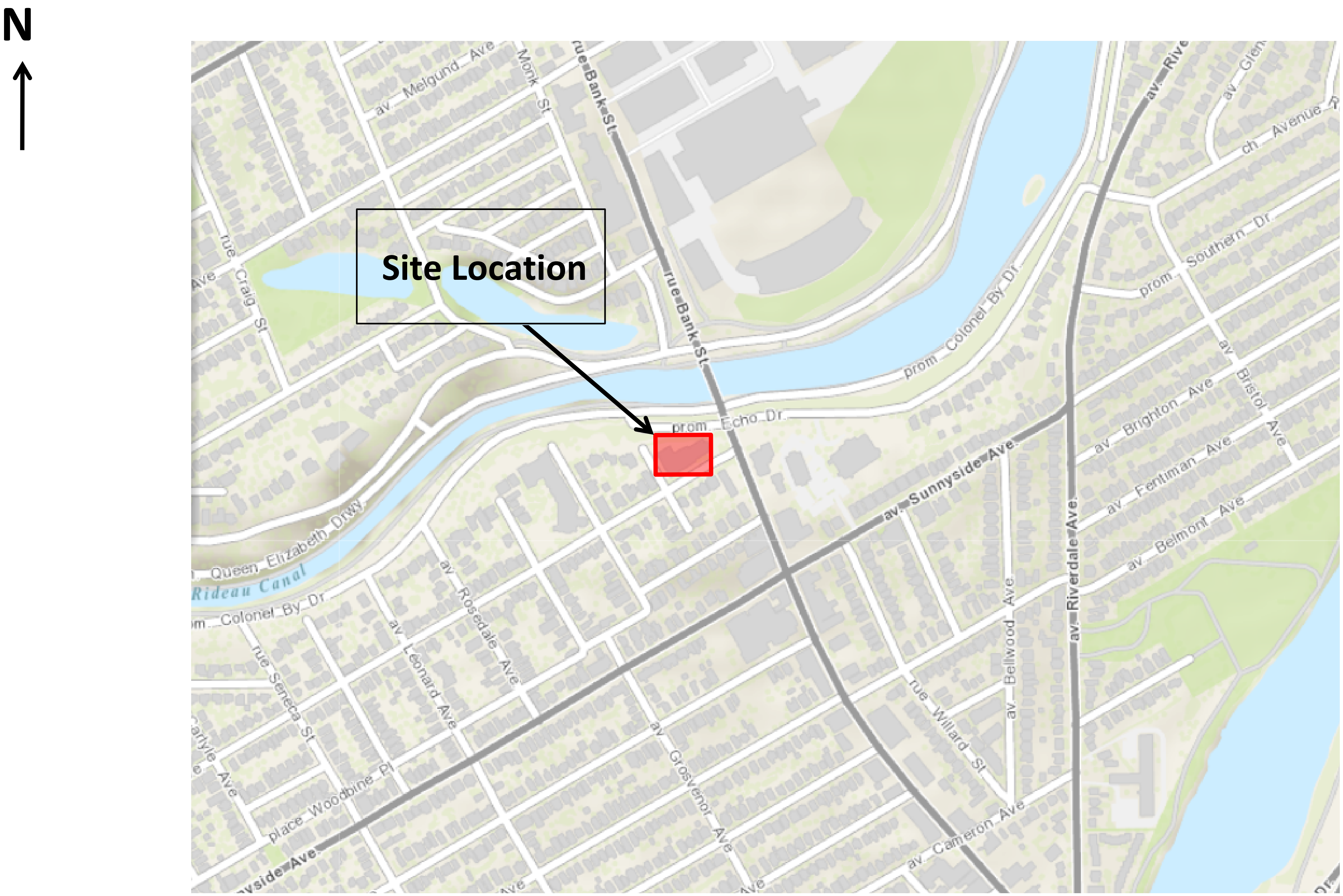
Reviewed

LICENSED PROFESSIONAL ENGINEER
C. G. HENDRY
100011328
PROVINCE OF ONTARIO

Chris Hendry, P. Eng., M. Eng.
Senior Geotechnical Engineer

Appendix A

DRAWINGS



Client:	Windmill Developments			Title:	Site Location Plan
Project#:	161-17230-00	DWG #:	1	Project:	Geotechnical Investigation - 1040 Bank Street., Ottawa, ON
Drawn:	DW	Approved:	CH		
Date:	Jan 2017	Scale:	N. T. S.		WSP
Size:	Letter	Rev:	0		

WSP
2611 QUEENSVIEW DRIVE, SUITE 300
OTTAWA, ONTARIO,
CANADA, K2B 8K2
WWW.WSPGROUP.COM

CLIENT:
WINDMILL DEVELOPMENT GROUP LTD.

PROJECT:
GEOTECHNICAL INVESTIGATION -
1040 BANK STREET, OTTAWA, ONTARIO

LEGEND
🟥 Site Boundary
◆ Monitoring Well

PROJECT NUMBER: 161-17230-00
DATE: JANUARY 2017
DRAWN BY: JS
CHECKED BY: KM
SCALE: 1:800

0 5 10 20 30 40 50 Meters
NAD1983 ZONE 18

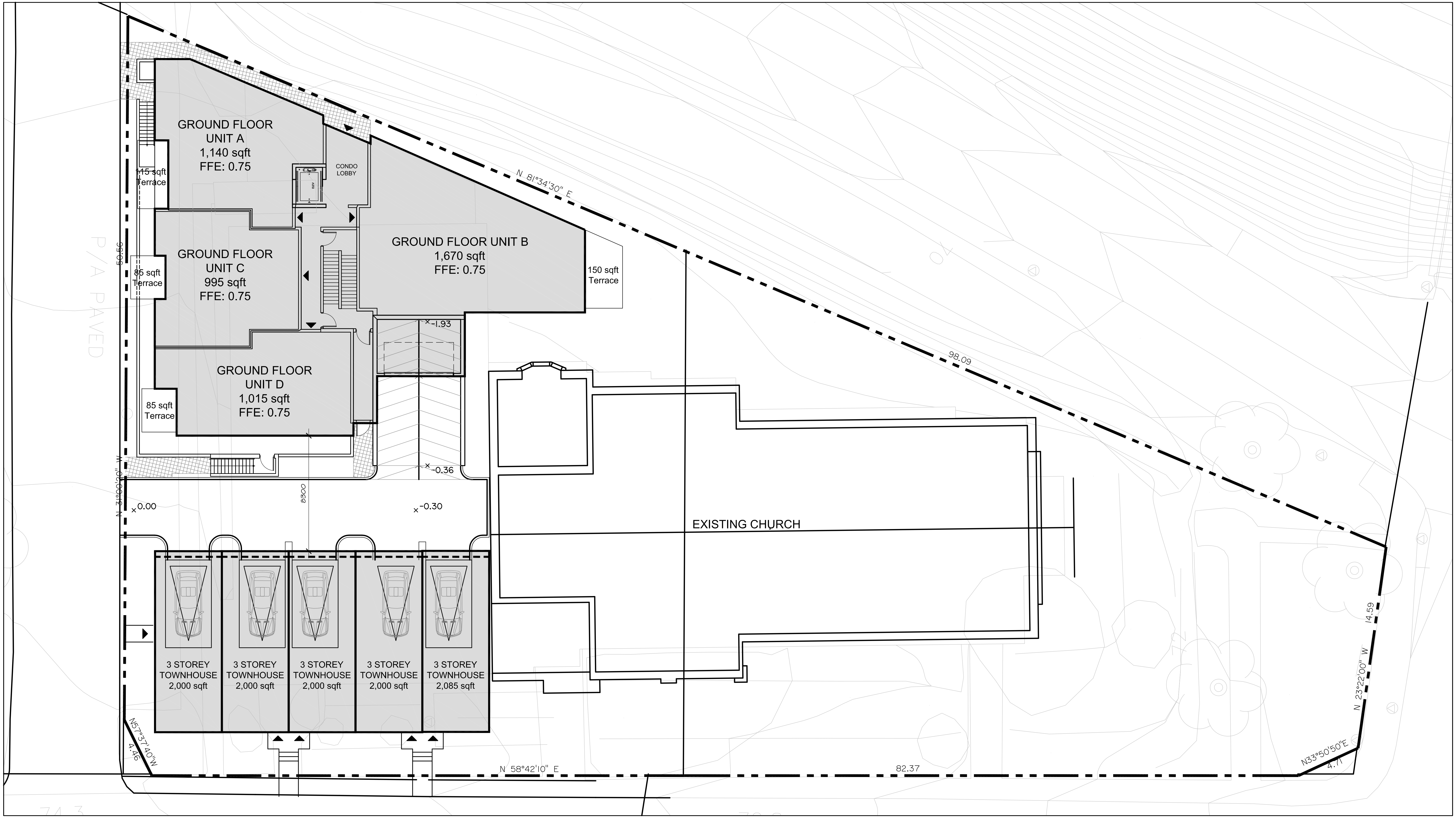
TITLE
BOREHOLE LOCATION PLAN

DRAWING NO. 2

Path: C:\Justin\Southminister Church\Mapping\MXD\Phase II ESA\FIGURE 2 - SITE PLAN.mxd

Appendix B

PRELIMINARY DESIGN INFORMATION



BARRY J.
HOBIN
& ASSOCIATES
ARCHITECTS
INCORPORATED

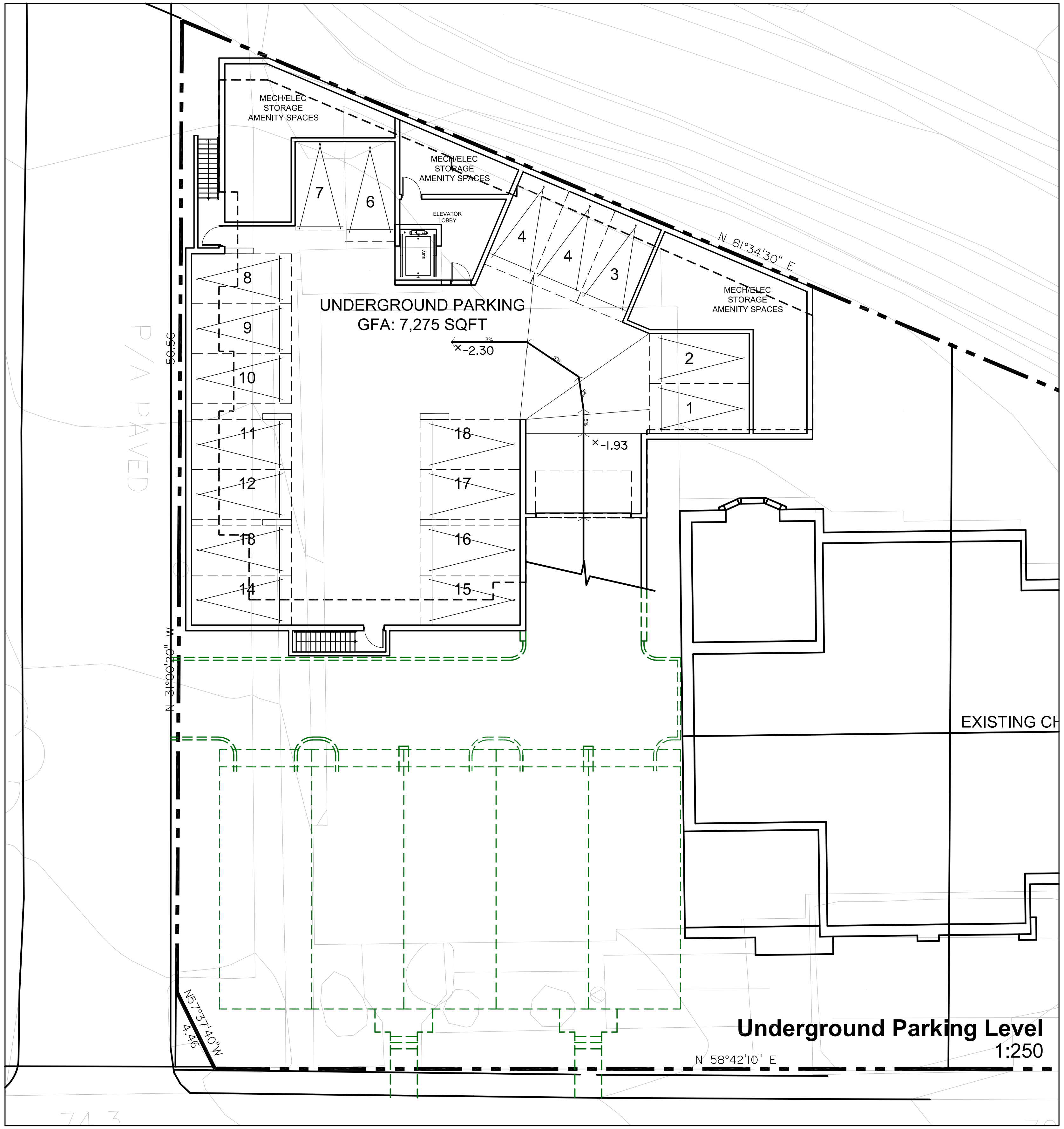
SOUTHMINSTER CHURCH

PROPOSED CONDOMINIUM AND TOWNHOUSES

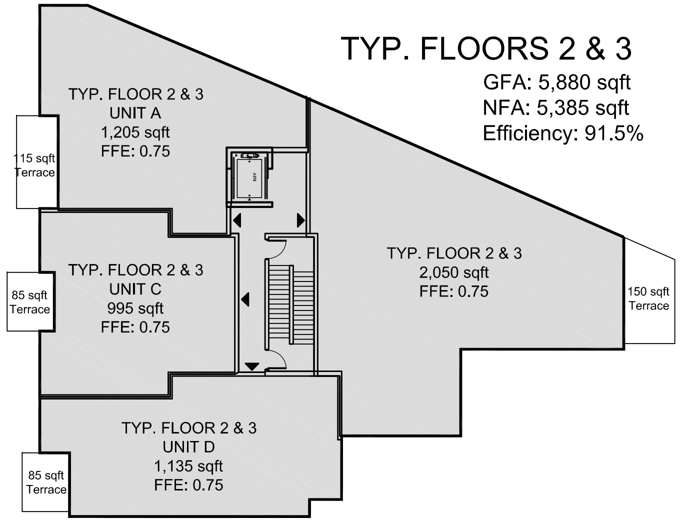
Site Plan

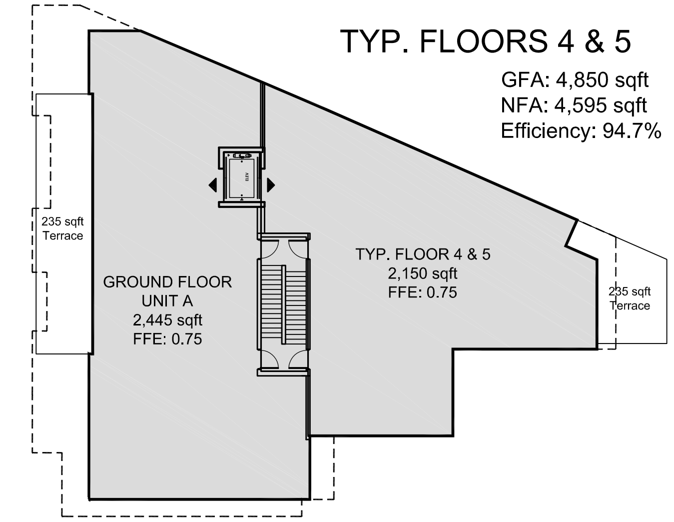
1:250

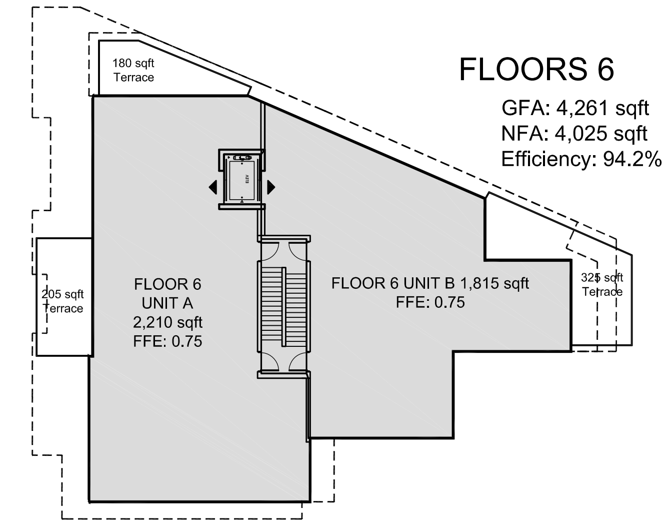
December 22, 2015



Underground Parking Level
1:250







STATS:

TOTAL UNITS	23
TOWNHOUSES	5 @ 2,000 SQFT
	(TOWNHOUSES TOTAL 10,000 SQFT)
CONDO GR-A	1,140 SQFT
CONDO GR-B	1,670 SQFT
CONDO GR-C	995 SQFT
CONDO GR-D	1,015 SQFT
CONDO 2&3-A	1,205 SQFT (x2)
CONDO 2&3-B	2,050 SQFT (x2)
CONDO 2&3-C	995 SQFT (x2)
CONDO 2&3-D	1,135 SQFT (x2)
CONDO 4&5-A	2,445 SQFT (x2)
CONDO 4&5-B	2,150 SQFT (x2)
CONDO 6-A	2,210 SQFT
CONDO 6-B	1,815 SQFT

(CONDO TOTAL 28,805 SQFT)

TOTAL SALEABLE AREA: 38,805 SQFT

Floors 2 - 6
1:400

BARRY J. HOBIN & ASSOCIATES ARCHITECTS INCORPORATED

SOUTHMINSTER CHURCH

PROPOSED CONDOMINIUM AND TOWNHOUSES

Floor Plans

December 22, 2015

Appendix C

BOREHOLE LOGS

LOG OF BOREHOLE BH/MH16-1

Project: Phase Two ESA and Geotech
Client: Windmill Developments
Project Location: 1040 Bank Street, Ottawa
Datum: Local
BH Location: See borehole location plan N 5027094 E 446364

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-17230-00
Date Started: 12/21/2016
Supervisor:KM
Reviewer: CH

SOIL PROFILE		SAMPLES				REMARKS AND GRAIN SIZE DISTRIBUTION (%)			
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	GR	SA	SI	CL
0.0	**Asphalt** - 20 mm								
	Gravel some sand to sandy, moist, brown (Granular Base)	1A	SS	12	Bentonite				
0.4	**Sand** some silt, trace gravel, trace roots, brown, moist, loose (FILL) - trace roots 0.4 m to 0.6 m -	1B	SS						
		2	SS	31		8	77	(15)	
1.7	**Sand**, some silt, brown, trace gravel, moist, loose (FILL)	3A	SS	9					
		3B	SS			3	82	(15)	
		4	SS	5					
	- brick fragment observed at approximatly 2.74 m in depth	5	SS	6					
		6	SS	8					
4.6	**Sand,** trace silt, brown, moist, compact (GLACIAL TILL)	7	SS	18		0	93	(7)	
		8	SS	25	Sand				
		9	SS	19					
		10	SS	23		2	94	(4)	
		11	SS	19					
8.4	**Sand and Gravel** trace silt,, grey, moist, very dense (GLACIAL TILL)	12	SS	80		39	53	(8)	
		13	SS	82					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE: 25 50 75 100 125
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT w LIQUID LIMIT W_L
WATER CONTENT (%): 25 50 75
POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (KN/m³)
STRATA PLOT
ELEVATION

Continued Next Page

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▽

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Sheet No. 1 of 2

LOG OF BOREHOLE BH/MH16-1

Project: Phase Two ESA and Geotech
Client: Windmill Developments
Project Location: 1040 Bank Street, Ottawa
Datum: Local
BH Location: See borehole location plan N 5027094 E 446364

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-17230-00
Date Started: 12/21/2016
Supervisor: KM
Reviewer: CH

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE: 25 50 75 100 125
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%): 25 50 75

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	WATER CONTENT (%)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
9.9	**Sand** some gravel, some silt, trace clay, brown, wet, compact (GLACIAL TILL)(Continued)	14	SS	14	Bentonite	○			
	- very dense below 10.7 m in depth	15A 15B	SS SS	54		○ ○			
		16	SS	>50/50 mm		○			
12.2	**Gravel** some sand, som esilt, brown, moist, very dense (GLACIAL TILL)	17	SS	61	Sand	○			45 50 (5)
					W. L. 12.7 mBGL Dec 26, 2016				
		18	SS	81		○			
		19	SS	78					
14.5	**Sand and Gravel** trace silt, brown, moist , very dense (GLACIAL TILL)	20	SS	72	Bentonite	○			
		21	SS	> 50\75 mm		○			
15.7	**END OF BOREHOLE**								

Notes

1) Auger refusal at 15.7 m below the existing ground surface
2) 50 mm monitoring well installed at 14.5 m below the existing ground surface
3) Date Groundwater Depth

Date	Groundwater Depth
12/26/2016	12.7 m

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

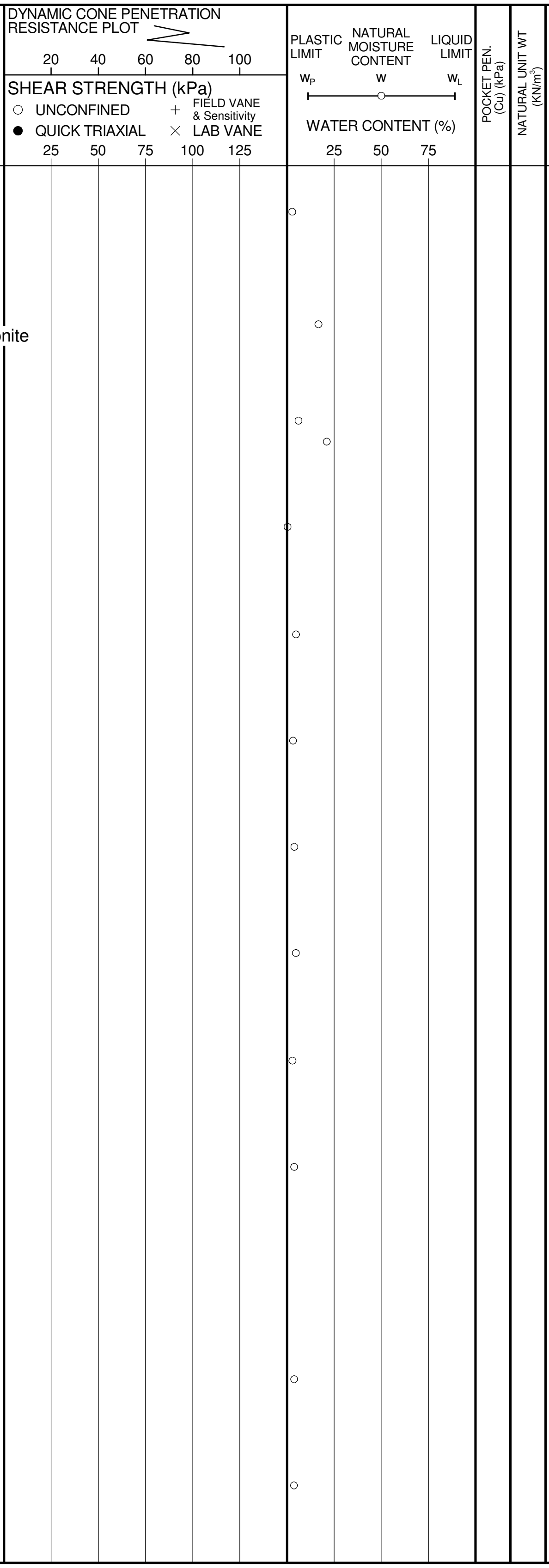
LOG OF BOREHOLE BH/MW16-2

Project: Phase Two ESA and Geotech
Client: Windmill Developments
Project Location: 1040 Bank Street, Ottawa
Datum: Local
BH Location: See borehole location plan N 5027121 E 446357

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-17230-00
Date Started: 12/22/2016
Supervisor: KM
Reviewer: CH

SOIL PROFILE		SAMPLES				REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	GR SA SI CL
0.0 0.1	**Asphalt - 50 mm**					
0.2	**Gravel** some sand to sandy, moist, grey brown (Granular Base)	1A 1B	SS SS	23	Sand	
	SANDY SILT brown, moist, loose to compact (FILL)	2	SS	10	Bentonite	0 31 (69)
1.5	**SAND** trace to some silt, trace to some clay, brown, moist, loose (FILL)	3A 3B	SS SS	7		
2.1	**SAND** trace silt, brown, moist, loose (FILL)	4	SS	9		
		5	SS	5		0 98 (2)
3.7	**SAND** brown, moist, loose to compact (GLACIAL TILL)	6	SS	11		
		7	SS	7		
		8	SS	14		
6.1	**SAND** some gravel, brown, moist, compact (GLACIAL TILL)	9	SS	20	Sand	
6.7	**SAND** some gravel, trace silt, brown, moist, compact (GLACIAL TILL)	10	SS	28		16 80 (5)
		11	SS	17		
8.4	**SILTY SAND** trace to some gravel, brown, moist. very dense (GLACIAL TILL)	12	SS	>50/125 mm		
		13	SS	>50/125 mm		



Continued Next Page

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH/MW16-2

Project: Phase Two ESA and Geotech
Client: Windmill Developments
Project Location: 1040 Bank Street, Ottawa
Datum: Local
BH Location: See borehole location plan N 5027121 E 446357

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-17230-00
Date Started: 12/22/2016
Supervisor: KM
Reviewer: CH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m		
9.9	**SAND** some gravel, trace silt, brown, moist, dense (GLACIAL TILL)(Continued)	14	SS	44		29 64 (7)
10.7	**GRAVELLY SAND** some silt, brown, moist, very dense (GLACIAL TILL)	15	SS	85		
		16	SS	>50/75 mm		
		17	SS	>50/125 mm		
	- becoming wet at 12.8 m in depth				W. L. 12.8 mBGL Dec 26, 2016	
		18	SS	97	Sand PVC Slotted Pipe	28 61 (11)
13.7	**SAND** with trace to some gravel, brown, wet, very dense (GLACIAL TILL)	19	SS	>50/125 mm		
		20	SS	>50/50 mm		
15.2	**COBBLES AND BOULDERS** (GLACIAL TILL)	1	RC			
		2	RC			
		3	RC			

STRATA PLOT | ELEVATION | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125 | PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT w LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75 | POCKET PEN. (Cu) (kPa) | NATURAL UNIT WT (KN/m³)

Continued Next Page

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Sheet No. 2 of 3

LOG OF BOREHOLE BH/MW16-2

Project: Phase Two ESA and Geotech
Client: Windmill Developments
Project Location: 1040 Bank Street, Ottawa
Datum: Local
BH Location: See borehole location plan N 5027121 E 446357

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-17230-00
Date Started: 12/22/2016
Supervisor: KM
Reviewer: CH

SOIL PROFILE			SAMPLES									
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT w LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	COBBLES AND BOULDERS (GLACIAL TILL)(Continued)		4	RC								
21.9	**END OF BOREHOLE** Notes 1) Auger refusal at 15.2 m below the existing ground surface. Switch to NQ coring. 2) Coring terminated at 21.9 below the existing ground surface. 2) 50 mm monitoring well installed at 14.9 m below the existing ground surface 3) Date Groundwater Depth 12/26/2016 12.8 m											

WSP SOIL LOG - OTTAWA BH LOGS - 15 AYLMER.GPJ SPL.GDT 2/2/17

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH/MW16-3

Project: Phase Two ESA and Geotech
Client: Windmill Developments
Project Location: 1040 Bank Street, Ottawa
Datum: Local
BH Location: See borehole location plan N 5027130 E 446374

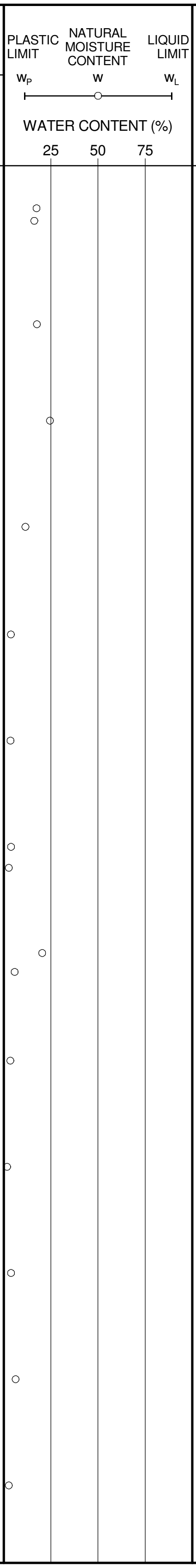
DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-17230-00
Date Started: 12/20/2016
Supervisor:KM
Reviewer: CH

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**TOPSOIL** - 180 mm					
0.2	**SANDY SILT** brown, moist, loose (FILL)	1A 1B	SS SS	7	Bentonite	0 21 (79)
	- trace roots at 0.9 m below the existing surface elevation	2	SS	5		
		3	SS	8		
2.4	**SAND** trace silt, brown, moist, loose (FILL)	4	SS	6		0 99 (1)
		5	SS	7		0 97 (3)
		6	SS	8		
4.9	**CLAYEY SILT** some sand, grey brown, moist, stiff - becoming wet at 5.3 m in depth	7A 7B	SS SS	10		
5.6	**SAND** trace gravel, light brown, moist, compact (GLACIAL TILL)	8A 8B	SS SS	16	Sand	
		9	SS	15		
6.9	**SAND** some gravel, trace silt, brown, moist, dense to very dense (GLACIAL TILL)	10	SS	29		
		11	SS	36		14 85 (1)
		12	SS	70		
		13	SS	>50/125 mm		
	- compact					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%)



POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (KN/m³)

Continued Next Page

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH/MW16-3

Project: Phase Two ESA and Geotech
Client: Windmill Developments
Project Location: 1040 Bank Street, Ottawa
Datum: Local
BH Location: See borehole location plan N 5027130 E 446374

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-17230-00
Date Started: 12/20/2016
Supervisor: KM
Reviewer: CH

(m) ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND some gravel, trace silt, brown, moist, dense to very dense (GLACIAL TILL)(Continued)		14	SS	23				○			
			15	SS					○			
							Bentonite					
11.3	**SILTY SAND** brown, moist, very dense (GLACIAL TILL)		16	SS	82				○			
12.2	**SAND** some gravel, brown, moist, very dense (GLACIAL TILL)		17	SS	82				○			
13.0	**SAND AND GRAVEL** trace silt, brown, wet, very dense (GLACIAL TILL)		18	SS	>50/75 mm	∇	W. L. 13.0 mBGL Dec 26, 2016 Sand PVC Slotted Pipe		○			
	- silty below 13.7 m in depth		19	SS	64				○			48 48 (4)
			20	SS	>50/100 mm							
15.2	**END OF BOREHOLE**											

Notes

1) Auger refusal at 15.2 m below the existing ground surface.
2) 50 mm monitoring well installed at 15.2 m below the existing ground surface
3) Date Groundwater Depth

Date	Groundwater Depth
12/26/2016	13.0 m

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ∇ ▼ Deep/Dual Installation ∇ ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH/MW16-4

Project: Phase Two ESA and Geotech
Client: Windmill Developments
Project Location: 1040 Bank Street, Ottawa
Datum: Local
BH Location: See borehole location plan N 5027114 E 446399

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-17230-00
Date Started: 12/21/2016
Supervisor: KM
Reviewer: CH

SOIL PROFILE			SAMPLES										
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
0.0	**TOPSOIL - 150 mm**		1A	SS	5								
0.2	**SAND** brown, moist, (FILL)		1B	SS		Bentonite							
0.6	**SILT** trace to some sand, brown, moist, very loose (FILL)		2	SS	3								
1.5	**SAND** brown, moist, loose (FILL)		3	SS	9								
2.1	**SILTY SAND** brown, moist, loose (FILL)		4	SS	7								
			5	SS	8								
4.0	**SAND** brown. moist, compact (GLACIAL TILL) - becoming wet at 4.1 metres below ground surface		6	SS	11								
			7	SS	18	Sand							
			8	SS	19								
			9	SS	19								
			10	SS	21								
7.6	**SAND** trace gravel, brown, moist, compact to dense (GLACIAL TILL)		11	SS	30								
			12	SS	35								
			13	SS	26								

Continued Next Page

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▽

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH/MW16-4

Project: Phase Two ESA and Geotech
Client: Windmill Developments
Project Location: 1040 Bank Street, Ottawa
Datum: Local
BH Location: See borehole location plan N 5027114 E 446399

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-17230-00
Date Started: 12/21/2016
Supervisor: KM
Reviewer: CH

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT w LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION					
	SAND trace gravel, brown, moist, compact to dense (GLACIAL TILL)(Continued)		14	SS	17							
10.4	**SAND** trace to some silt, trace to some clay, trace gravel, brown, wet (GLACIAL TILL)											
10.7	**SAND** trace gravel, moist, white-brown, very dense (GLACIAL TILL)		15	SS	60	Bentonite						
11.4	**SILTY CLAY** trace gravel, trace ashes, brown, moist, stiff (GLACIAL TILL)		16	SS	52							
11.7	**SAND** trace to some gravel, brown, moist, very dense (GLACIAL TILL)											
12.2	**SAND AND GRAVEL** trace to some silt, brown, moist, very dense (GLACIAL TILL)		17	SS	>50/75 mm							
			18	SS	>50/125 mm							
	- moist to wet below 13.7 m in depth		19	SS	>50/75 mm	W. L. 14.0 mBGL Dec 26, 2016 Sand						
14.3	**SAND** some gravel, brown, wet, very dense (GLACIAL TILL)		20	SS	55	PVC Slotted Pipe						
16.0	**END OF BOREHOLE**											

Notes

1) Auger refusal at 16.0 m below the existing ground surface.
2) 50 mm monitoring well installed at 16.0 m below the existing ground surface
3) Date Groundwater Depth

Date	Groundwater Depth
12/26/2016	14.0 m

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH/MW16-5

Project: Phase Two ESA and Geotech
Client: Windmill Developments
Project Location: 1040 Bank Street, Ottawa
Datum: Local
BH Location: See borehole location plan N 5027157 E 446423

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-17230-00
Date Started: 12/20/2016
Supervisor: KM
Reviewer: CH

SOIL PROFILE			SAMPLES									
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**TOPSOIL - 130 mm**											
0.1	**SILTY SAND** brown, moist, loose to very loose (FILL)		1A 1B	SS SS	4	Bentonite						
			2	SS	3							
			3	SS	2							
2.1	**SILTY SAND** with brick fragments, trace gravel, brown, moist, loose (FILL)		4	SS	4							
3.1	**SILTY SAND** moist to wet, compact, brown (FILL)		5	SS	11							
			6	SS	13							
4.6	**SAND** moist, compact, brown (GLACIAL TILL)		7	SS	13							
			8	SS	18	Sand						
			9	SS	20							
6.7	**SAND** with gravel, moist, dense to very dense, brown (GLACIAL TILL)		10	SS	40							
			11	SS	47							
			12	SS	50							
			13	SS	26							

Continued Next Page

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH/MW16-5

Project: Phase Two ESA and Geotech	**DRILLING DATA**	
Client: Windmill Developments	Rig Type:	Project No.: 161-17230-00
Project Location: 1040 Bank Street, Ottawa	Method: Hollow Stem Auger	Date Started: 12/20/2016
Datum: Local	Borehole Diameter: 203 mm	Supervisor: KM
BH Location: See borehole location plan N 5027157 E 446423	Core Diameter:	Reviewer: CH

(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS
	SAND with gravel, moist, dense to very dense, brown (GLACIAL TILL)(Continued)				
10.7	**GRAVEL** some sand to sandy, brown, moist, dense (GLACIAL TILL)	14	SS	36	Bentonite
					Sand
12.2	**SAND** brown, moist, compact (GLACIAL TILL)	15	SS	26	
12.8	**GRAVEL** some sand to sandy, some silt, brown, very dense (GLACIAL TILL)	16	SS	100	W. L. 13.0 mBGL Dec 26, 2016 PVC Slotted Pipe
13.6	**GRAVEL** some sand to sandy, brown, wet, very dense (GLACIAL TILL)	17	SS	>50/100 mm	
		18	SS	54	
14.9	**END OF BOREHOLE**				

Column headings for the plots, all of them empty:

- DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100
- SHEAR STRENGTH (kPa):
  - ○ UNCONFINED
  - + FIELD VANE & Sensitivity
  - ● QUICK TRIAXIAL
  - × LAB VANE
  - Scale: 25, 50, 75, 100, 125
- PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 25, 50, 75
- POCKET PEN. (Cu) (kPa)
- NATURAL UNIT WT (KN/m^3)
- REMARKS AND GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

Notes

1) Auger refusal at 14.9 m below the existing ground surface.
2) 50 mm monitoring well installed at 14.9 m below the existing ground surface
3) Date Groundwater Depth

Date	Groundwater Depth
12/26/2016	13.0 m

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

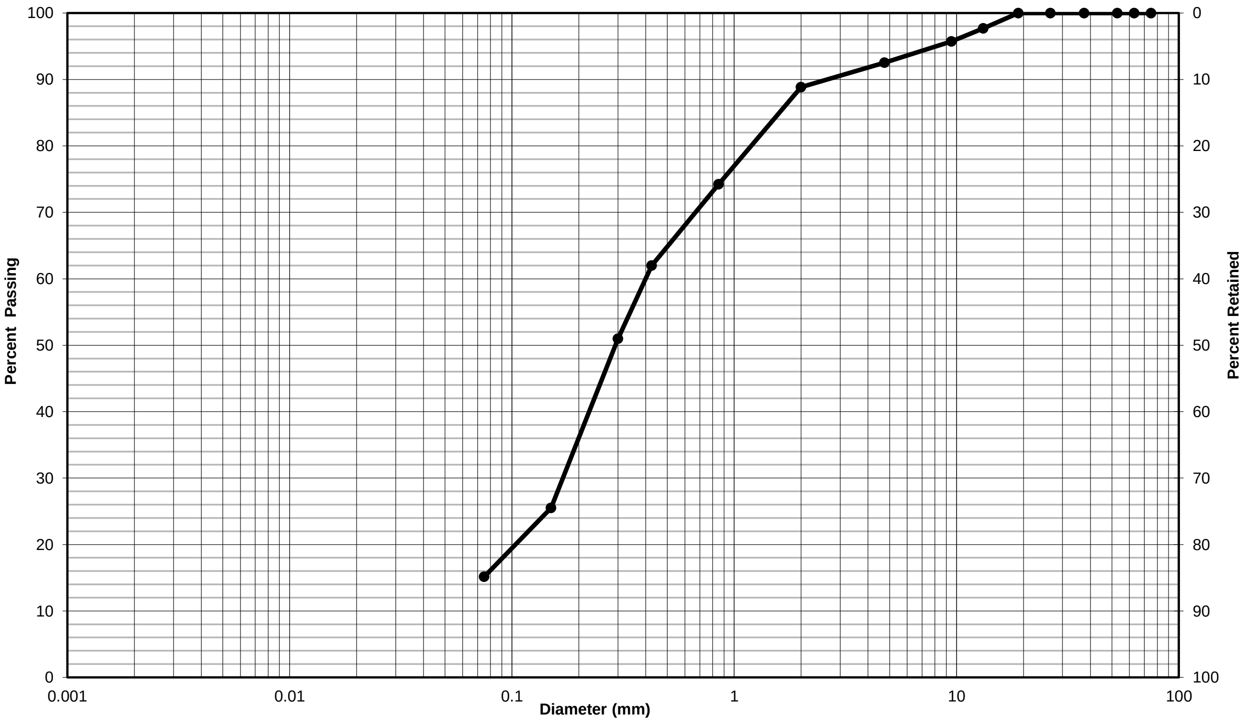
Appendix D

LABORATORY TESTING RESULTS

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-1
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.:	16-1	Sample no.:	SS2
Depth:	0.75-1.35m		



<table>
<tr><td rowspan="2">Clay & Silt</td><td colspan="3">Sand</td><td colspan="2">Gravel</td></tr>
<tr><td>Fine</td><td>Medium</td><td>Coarse</td><td>Fine</td><td>Coarse</td></tr>
<tr><td colspan="6">Unified Soil Classification System</td></tr>
</table>

Percent Retained	**Gravel**	**Sand**	**Clay & Silt**	**Silt**	**Clay**
	7.5	77.4	15.1	-	-

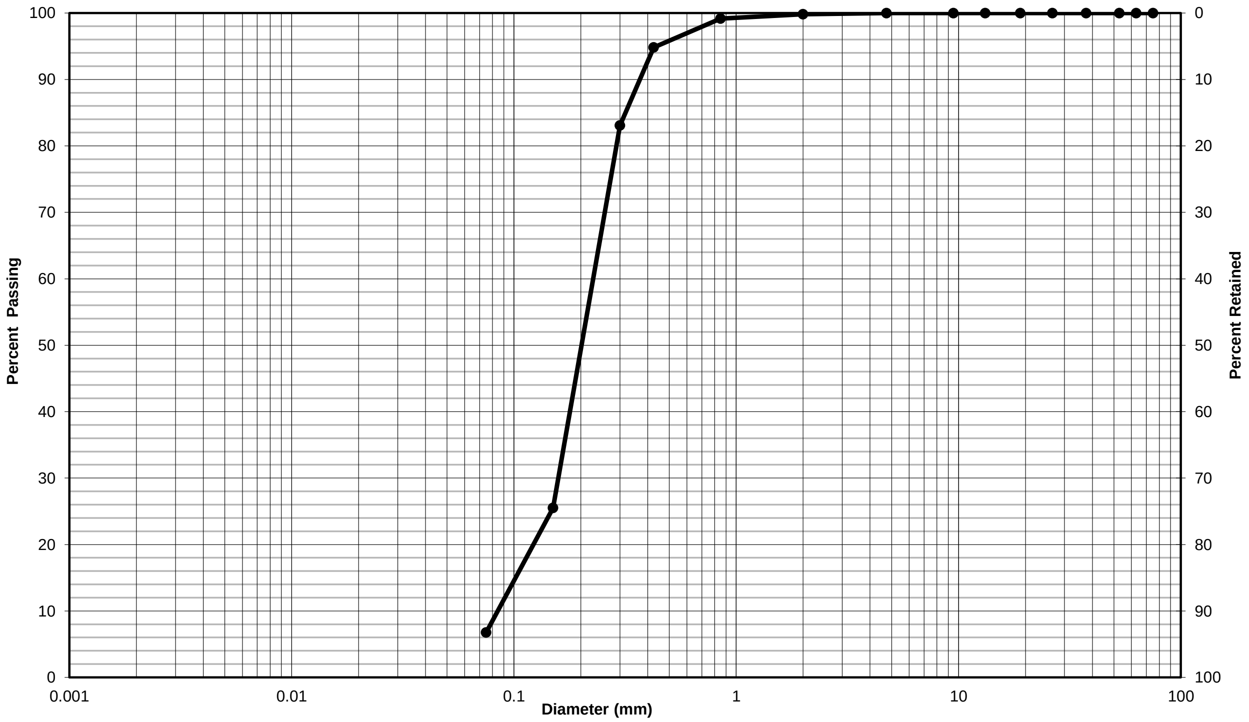
Remarks:

Performed by:	N.Krebs	**Date:**	February 1, 2017
Verified by:	N.Krebs	**Date:**	February 1, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-2
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.:	16-1	Sample no.:	SS7
Depth:	4.6-5.2m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	0.0	93.2	6.8	-	-

Remarks:

Performed by:	N.Krebs	**Date:**	February 1, 2017
Verified by:	N.Krebs	**Date:**	February 1, 2017

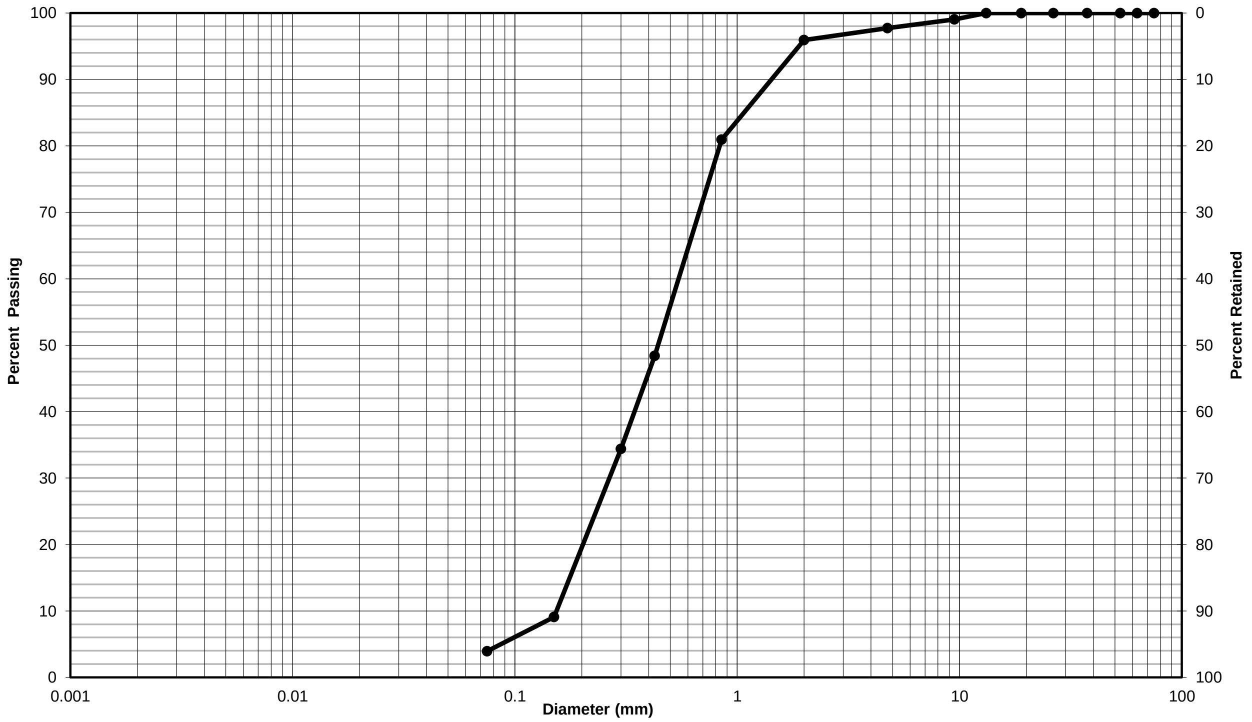
Particle-Size Analysis of Soils
(ASTM D422)

Client: Windmill Developments
Lab no.: OL149-3

Project/Site: Southminster Church
Project no.: 161-17230-00

Borehole no.: 16-1
Sample no.: SS10

Depth: 6.85-7.45m



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	2.3	93.8	3.9	-	-

Remarks:

Performed by: N.Krebs
Date: February 1, 2017

Verified by: N.Krebs
Date: February 1, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client: Windmill Developments

Lab no.: OL149-4

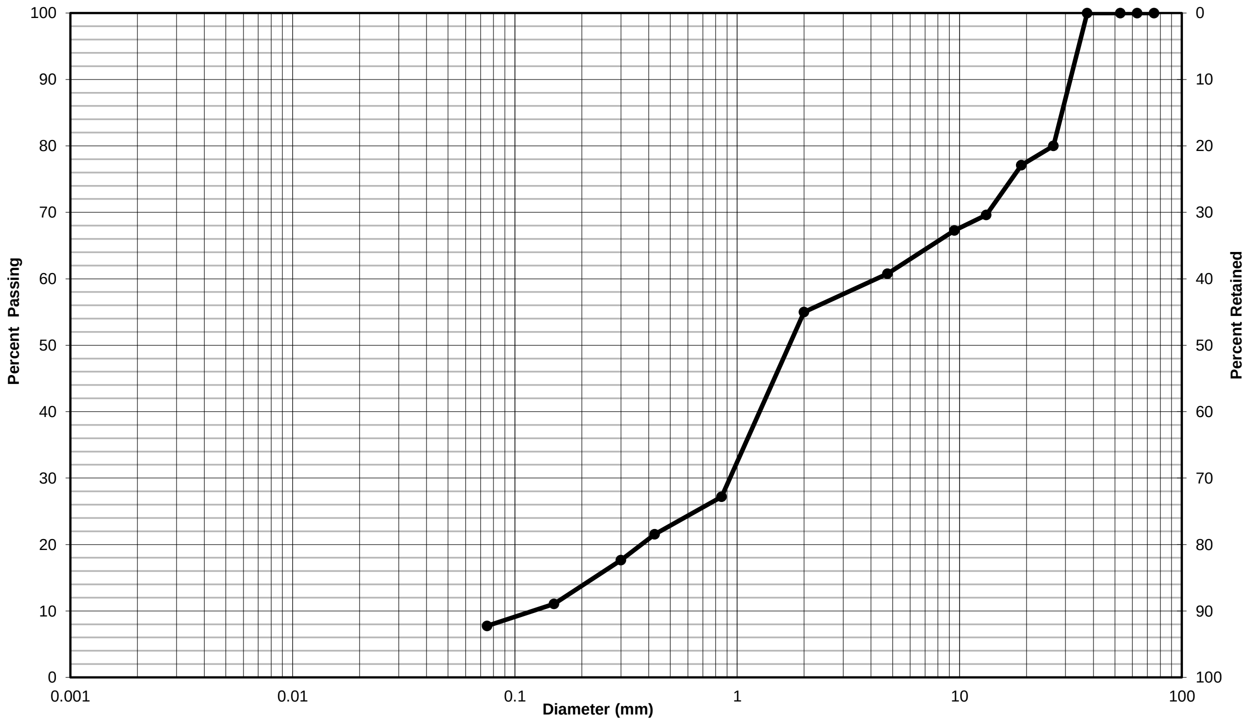
Project/Site: Southminster Church

Project no.: 161-17230-00

Borehole no.: 16-1

Sample no.: SS12

Depth: 8.35-8.95m



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	39.2	53.0	7.8	-	-

Remarks:

Performed by: N.Krebs

Date: February 1, 2017

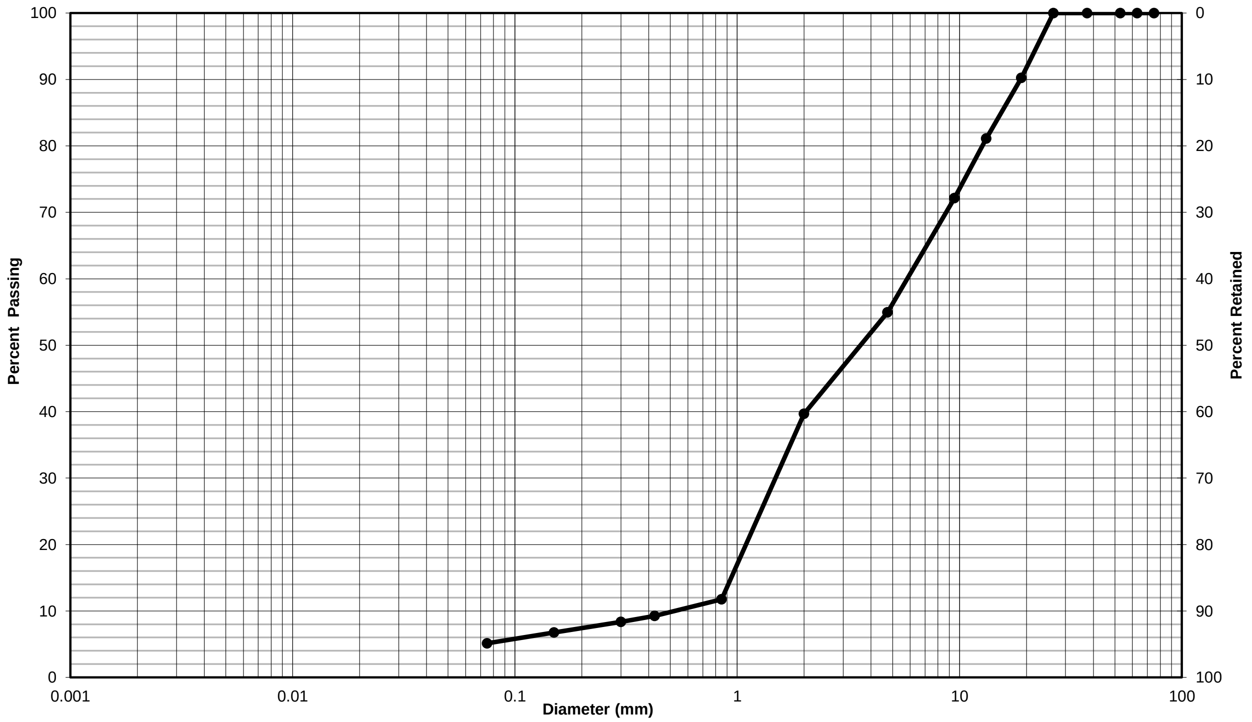
Verified by: N.Krebs

Date: February 1, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-5
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.:	16-1	Sample no.:	SS17
Depth:	12.2-12.8m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	45.0	49.8	5.1	-	-

Remarks:

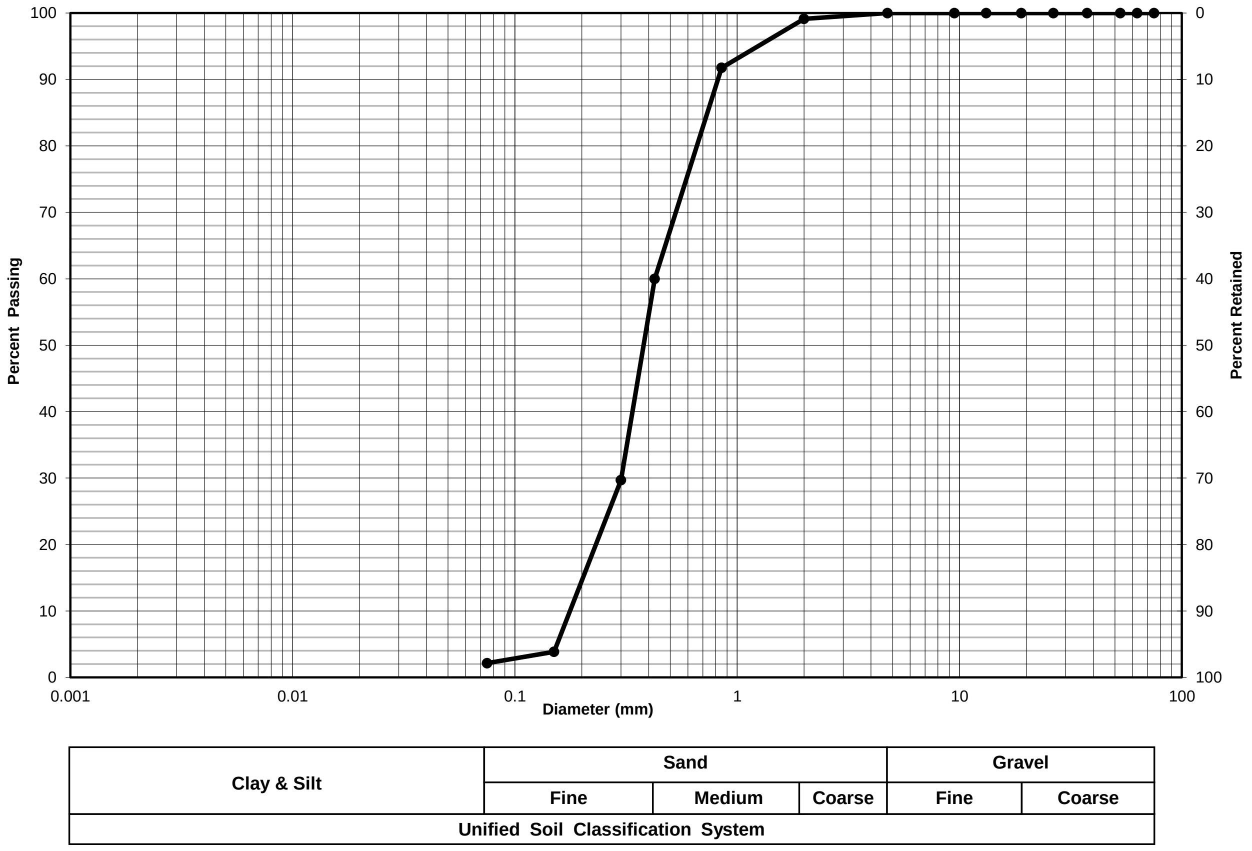
Performed by:	N.Krebs	**Date:**	February 1, 2017
Verified by:	N.Krebs	**Date:**	February 1, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-6
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.: 16-2 Sample no.: SS5

Depth: 3-3.7m



Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	0.0	97.9	2.1	-	-

Remarks:

Performed by:	N.Krebs	**Date:**	February 1, 2017
Verified by:	N.Krebs	**Date:**	February 1, 2017

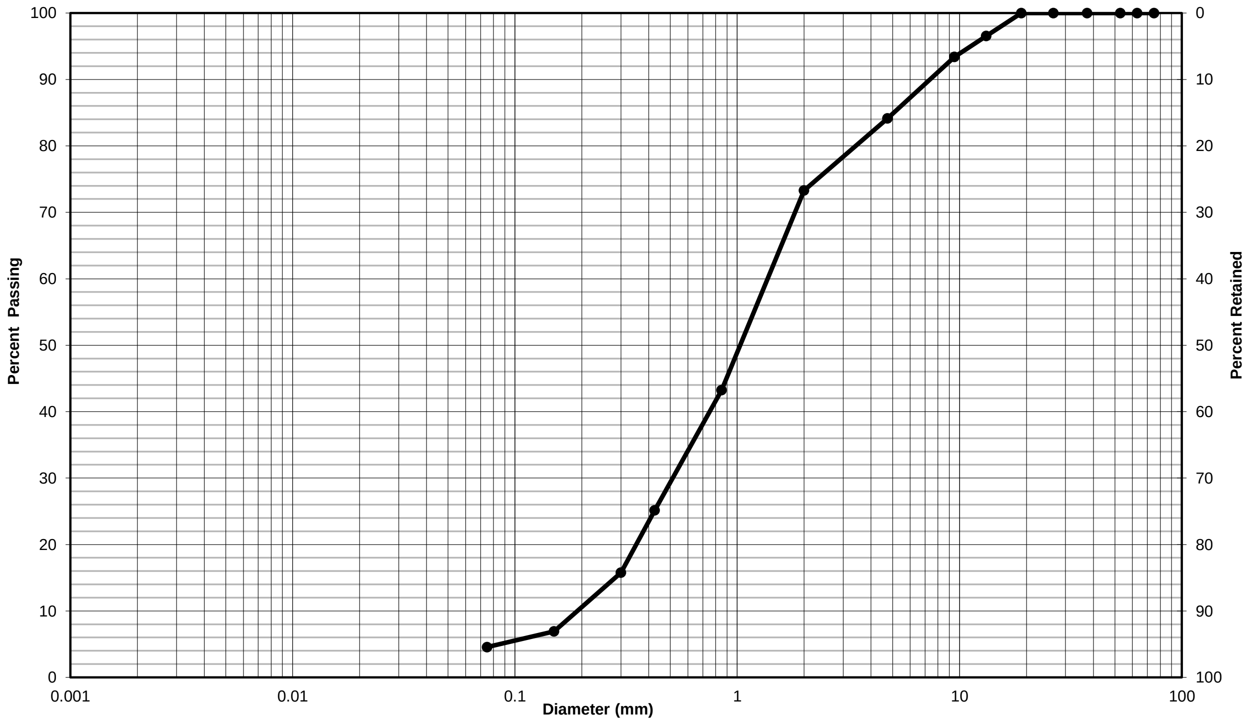
Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-7
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.: 16-2

Sample no.: SS10

Depth: 6.85-7.45m



Clay & Silt	**Sand**			**Gravel**	
	Fine	**Medium**	**Coarse**	**Fine**	**Coarse**
Unified Soil Classification System					

Percent Retained	**Gravel**	**Sand**	**Clay & Silt**	**Silt**	**Clay**
	15.8	79.6	4.6	-	-

Remarks:

Performed by:	N.Krebs	**Date:**	February 1, 2017
Verified by:	N.Krebs	**Date:**	February 1, 2017

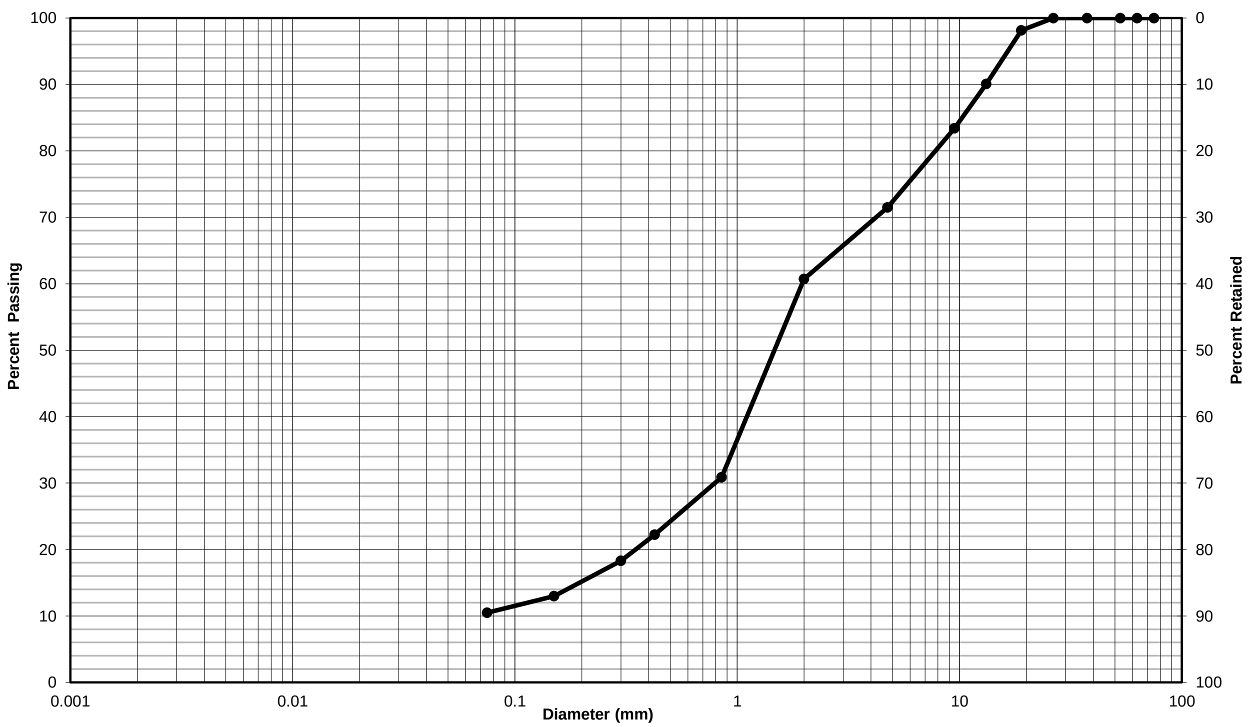
Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-8
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.: 16-2

Sample no.: SS18

Depth: 12.95-13.55m



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	28.5	61.0	10.5	-	-

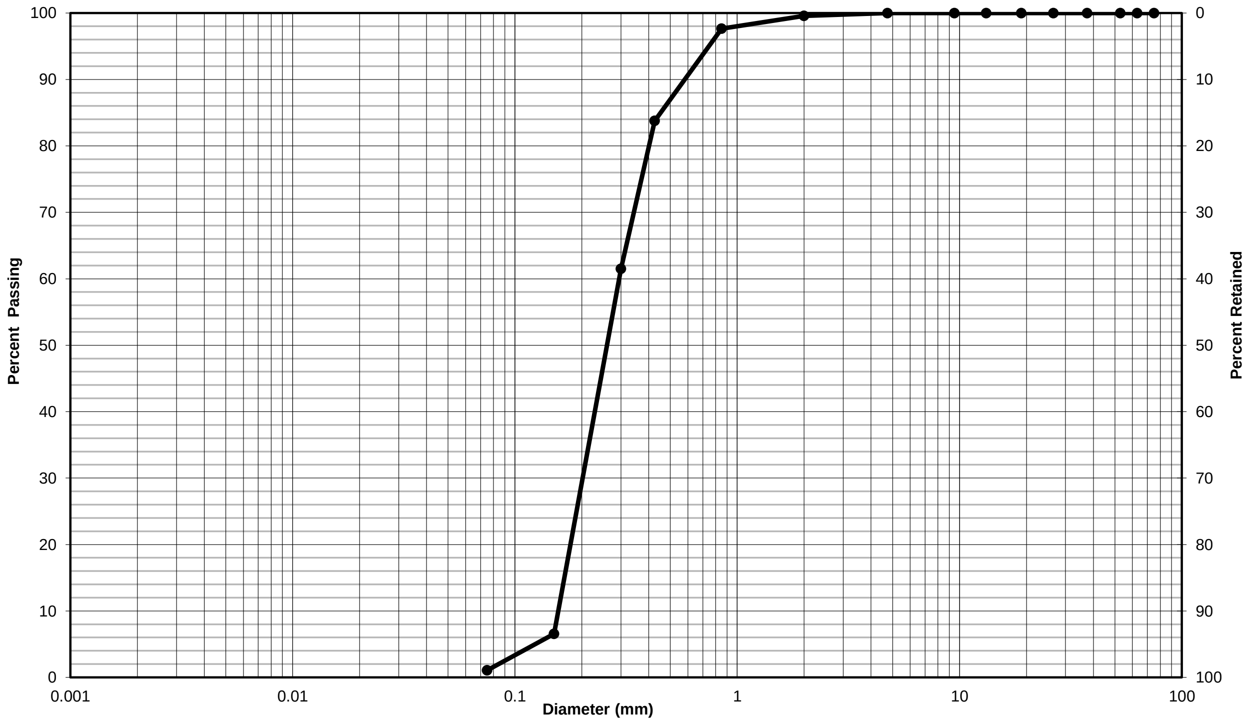
Remarks:

Performed by:	N.Krebs	**Date:**	February 1, 2017
Verified by:	N.Krebs	**Date:**	February 1, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-9
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.:	16-3	Sample no.:	SS6
Depth:	3.85-4.45m		



Clay & Silt	**Sand**			**Gravel**	
	Fine	**Medium**	**Coarse**	**Fine**	**Coarse**
Unified Soil Classification System					

Percent Retained	**Gravel**	**Sand**	**Clay & Silt**	**Silt**	**Clay**
	0.0	98.9	1.1	-	-

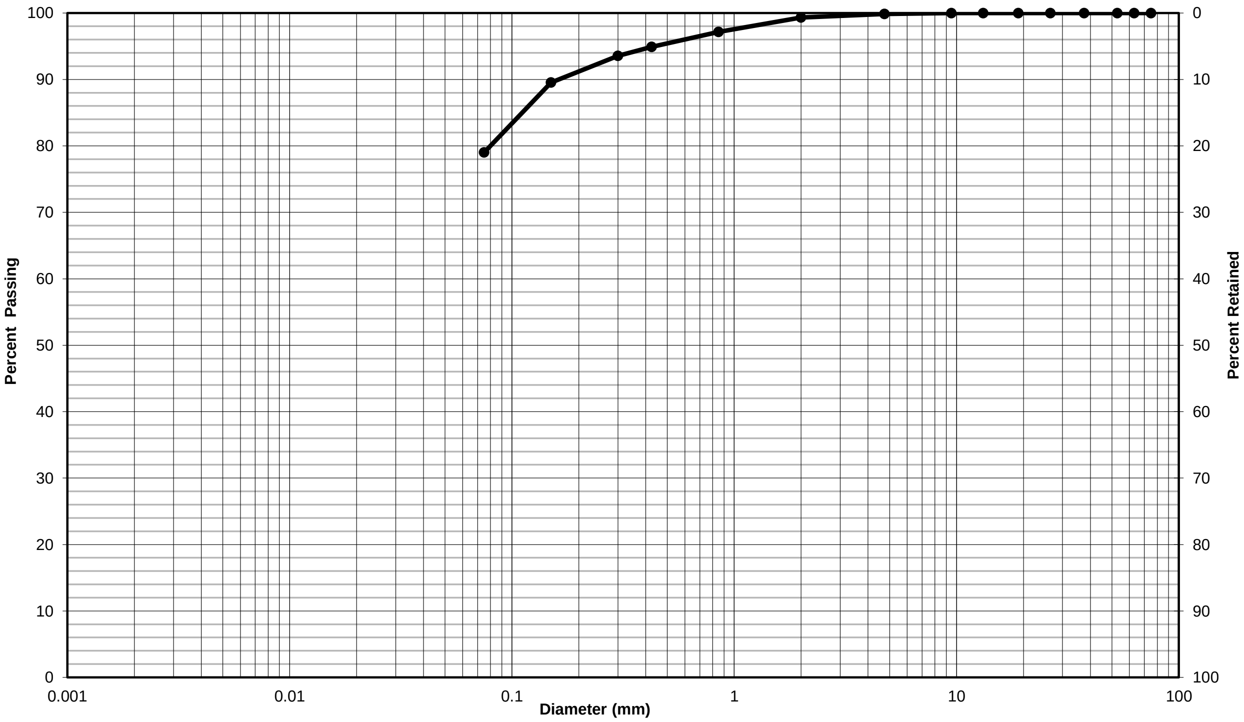
Remarks:

Performed by:	D.Robertson	**Date:**	February 1, 2017
Verified by:	N.Krebs	**Date:**	February 1, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-10
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.:	16-3	Sample no.:	SS2
Depth:	1.05-1.5m		



<table>
<tr><td rowspan="2">Clay & Silt</td><td colspan="3">Sand</td><td colspan="2">Gravel</td></tr>
<tr><td>Fine</td><td>Medium</td><td>Coarse</td><td>Fine</td><td>Coarse</td></tr>
<tr><td colspan="6">Unified Soil Classification System</td></tr>
</table>

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	0.1	20.9	79.0	-	-

Remarks:

Performed by:	D.Robertson	**Date:**	February 1, 2017
Verified by:	N.Krebs	**Date:**	February 1, 2017

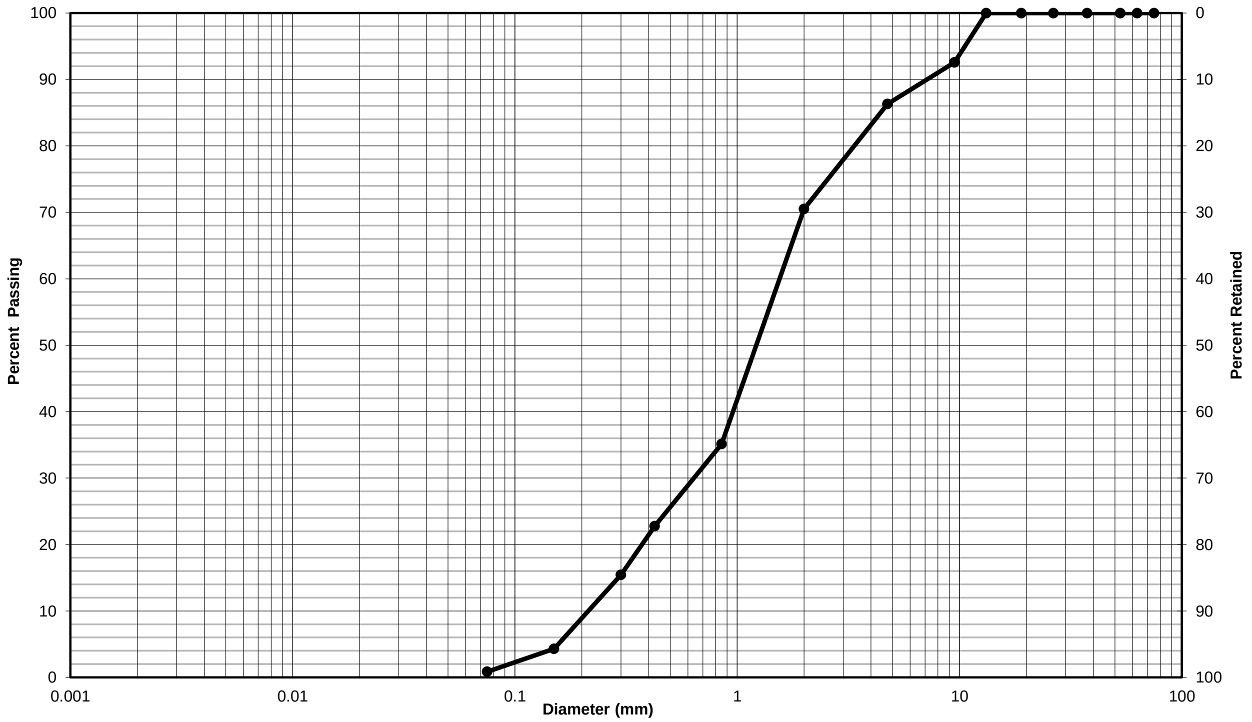
Particle-Size Analysis of Soils
(ASTM D422)

Client: Windmill Developments
Lab no.: OL149-11

Project/Site: Southminster Church
Project no.: 161-17230-00

Borehole no.: 16-3
Sample no.: SS11

Depth: 7.6-8.2m



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	13.7	85.5	0.8	-	-

Remarks:

Performed by: D.Robertson
Date: February 1, 2017

Verified by: N.Krebs
Date: February 1, 2017

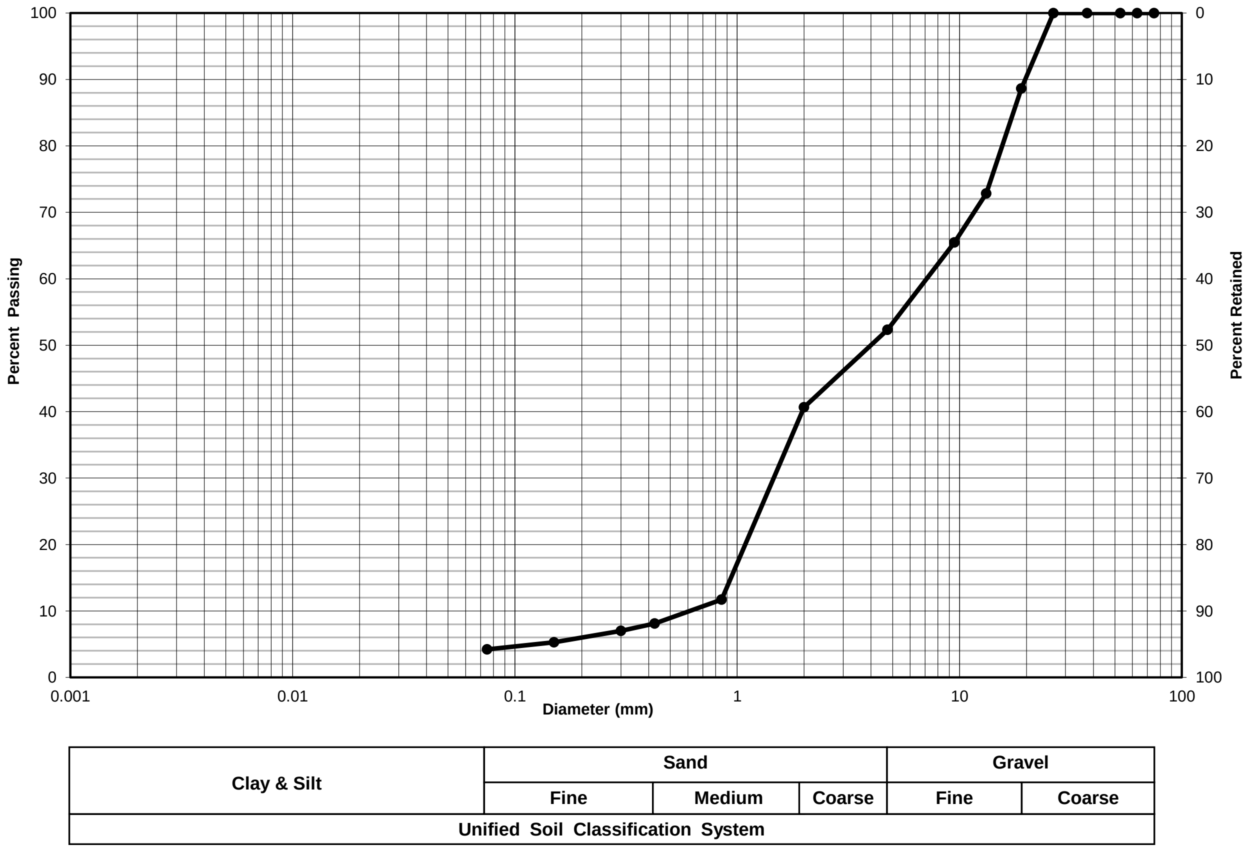
Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-12
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.: 16-3

Sample no.: SS19

Depth: 13.7-14.3m



Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	47.7	48.1	4.2	-	-

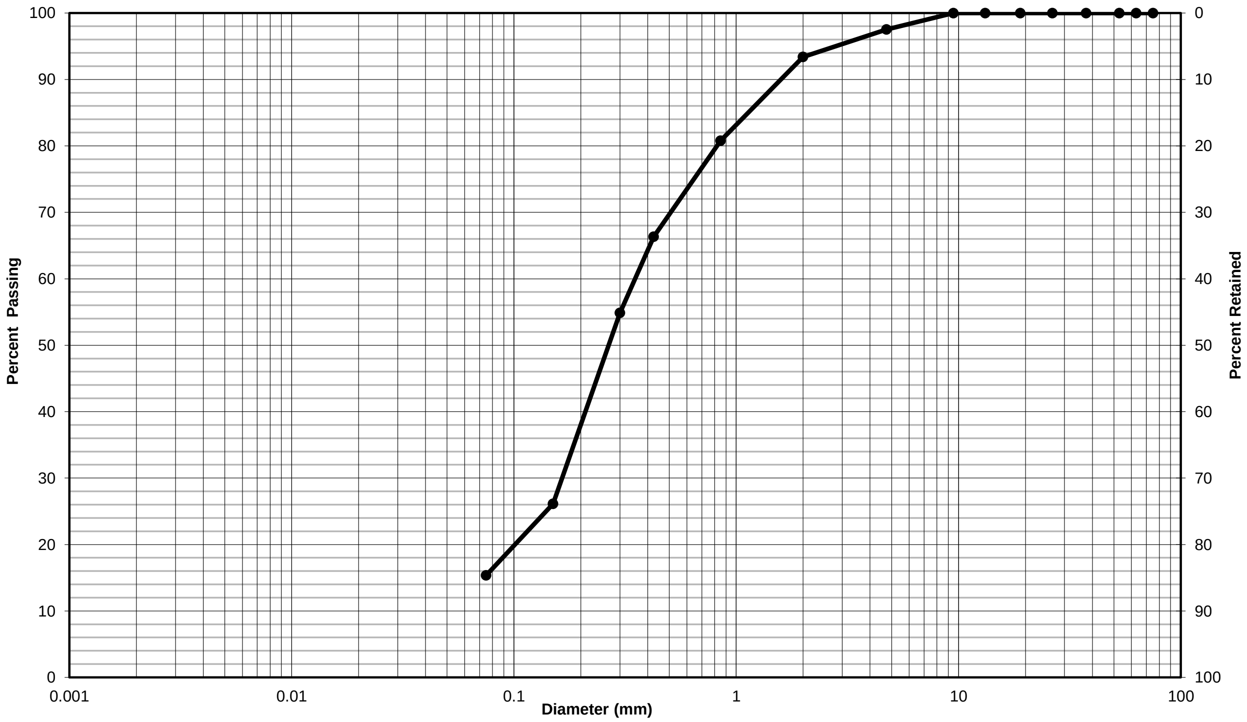
Remarks:

Performed by:	D.Robertson	**Date:**	February 1, 2017
Verified by:	N.Krebs	**Date:**	February 1, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-13
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.:	16-1	Sample no.:	SS3B
Depth:	1.5-2.1m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	**Gravel**	**Sand**	**Clay & Silt**	**Silt**	**Clay**
	2.5	82.2	15.4	-	-

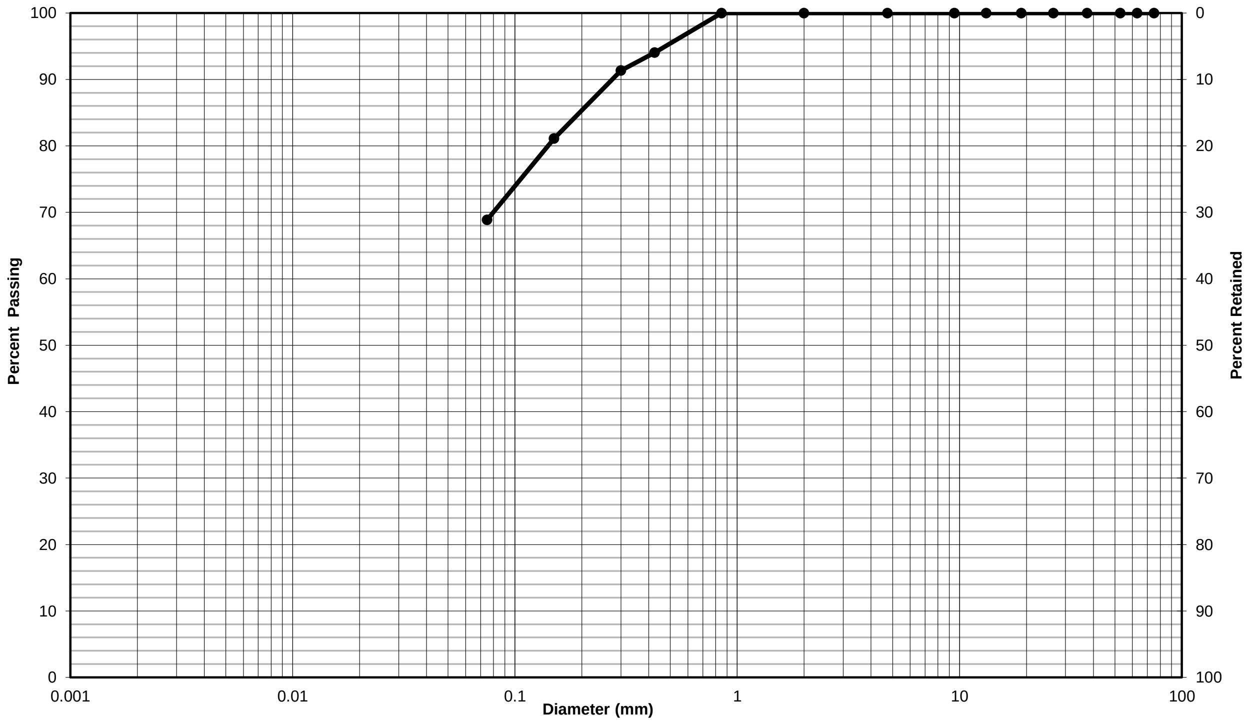
Remarks:

Performed by:	N.Krebs	**Date:**	February 1, 2017
Verified by:	N.Krebs	**Date:**	February 1, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client: Windmill Developments
Lab no.: OL149-14
Project/Site: Southminster Church
Project no.: 161-17230-00

Borehole no.: 16-2
Sample no.: SS2
Depth: 0.75-1.35m



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	0.0	31.1	68.9	-	-

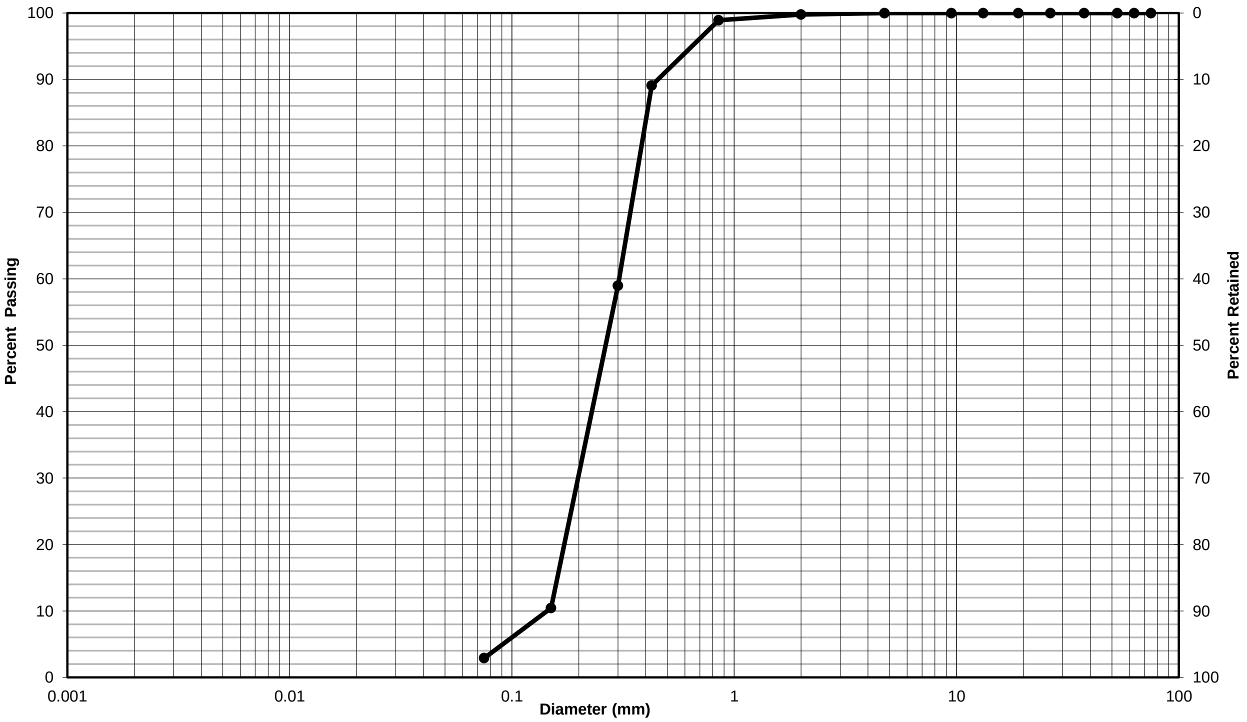
Remarks:

Performed by: D.Robertson
Date: February 1, 2017
Verified by: N.Krebs
Date: February 1, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-15
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.:	16-3	Sample no.:	SS5
Depth:	3-3.7m		



<table>
<tr><td rowspan="2">Clay & Silt</td><td colspan="3">Sand</td><td colspan="2">Gravel</td></tr>
<tr><td>Fine</td><td>Medium</td><td>Coarse</td><td>Fine</td><td>Coarse</td></tr>
<tr><td colspan="6">Unified Soil Classification System</td></tr>
</table>

	Gravel	**Sand**	**Clay & Silt**	**Silt**	**Clay**
Percent Retained	0.0	97.1	2.9	-	-

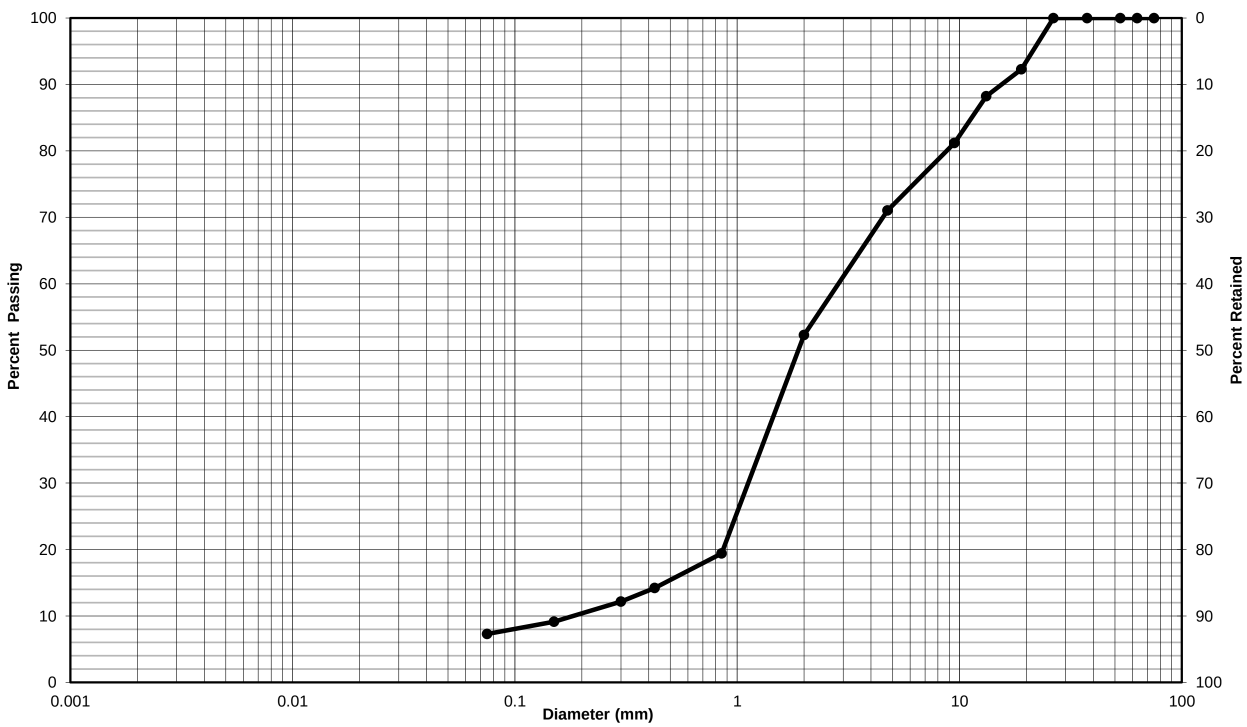
Remarks:

Performed by:	N.Krebs	**Date:**	February 2, 2017
Verified by:	N.Krebs	**Date:**	February 2, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Windmill Developments	**Lab no.:**	OL149-16
Project/Site:	Southminster Church	**Project no.:**	161-17230-00

Borehole no.:	16-2	Sample no.:	SS14
Depth:	9.95-10.55m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	29.0	63.8	7.3	-	-

Remarks:

Performed by:	D.Robertson	**Date:**	February 1, 2017
Verified by:	N.Krebs	**Date:**	February 2, 2017

Certificate of Analysis

Client:	WSP Canada Inc.(SPL) 146 Colonnade Rd., Unit 17 Ottawa, ON K2E 7Y1	Report Number:	1700781
		Date Submitted:	2017-01-17
		Date Reported:	2017-01-19
		Project:	161-17230-00 Southminster
		COC #:	184499
Attention:	Mr. Chris Hendry		
PO#:			
Invoice to:	WSP Canada Inc.		

Dear Chris Hendry:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: ______________________________

Shyla Monette
Team Leader, Inorganics

All analysis is completed in Ottawa, Ontario (unless otherwise indicated).

Eurofins Ottawa is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on our CALA scope of accreditation. It can be found at http://www.cala.ca/scopes/2602.pdf.

Eurofins(Ottawa) is certified and accredited for specific parameters by OMAFRA, Ontario Ministry of Agriculture, Food and Rural Affairs (for farm soils). Licensed by Ontario MOE for specific tests in drinking water.

Eurofins(Mississauga) is accredited for specific parameters by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required.

eurofins | Environment Testing

Certificate of Analysis

Client:	WSP Canada Inc.(SPL) 146 Colonnade Rd., Unit 17 Ottawa, ON K2E 7Y1
Attention:	Mr. Chris Hendry
PO#:	
Invoice to:	WSP Canada Inc.

Report Number:	1700781
Date Submitted:	2017-01-17
Date Reported:	2017-01-19
Project:	161-17230-00 Southminster
COC #:	184499

				Lab I.D.	1277723	1277724	1277725	1277726
				Sample Matrix	Soil	Soil	Soil	Soil
				Sample Type				
				Sampling Date	2016-12-20	2016-12-20	2016-12-20	2016-12-20
				Sample I.D.	BH 16-1 SS3B	BH 16-2 SS2	BH 16-2 SS14	BH 16-3 SS5
Group	**Analyte**	**MRL**	**Units**	**Guideline**				
Agri. - Soil	pH	2.0			8.7	7.7	8.9	8.5
General Chemistry	Cl	0.002	%		0.026	0.003	0.008	<0.002
	Electrical Conductivity	0.05	mS/cm		0.74	0.36	0.35	0.08
	Resistivity	1	ohm-cm		1350	2780	2860	12500
	SO4	0.01	%		0.01	0.03	<0.01	<0.01

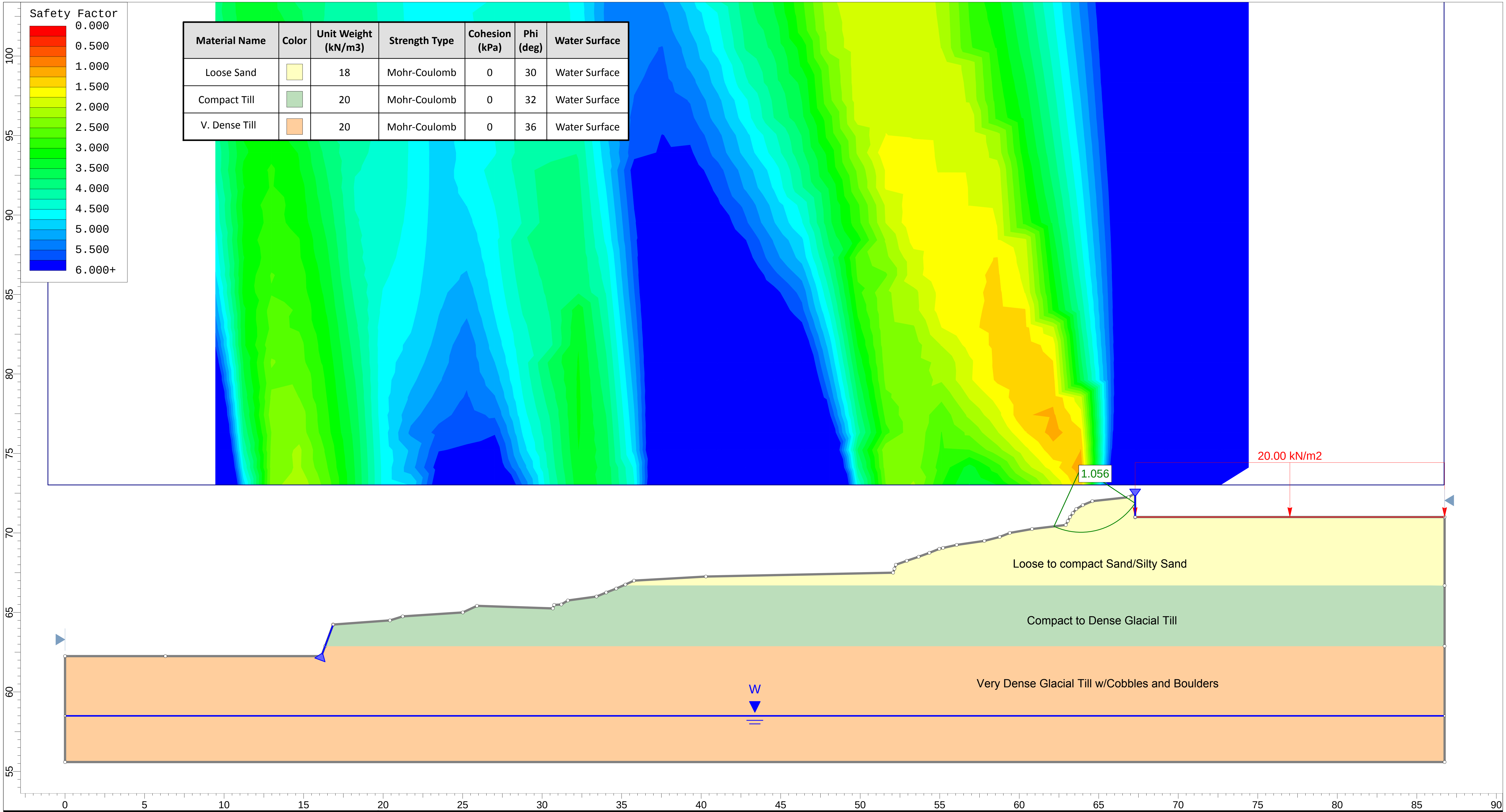
Guideline = *** = Guideline Exceedence**

All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in Mississauga, Ontario).
Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Appendix E

SLOPE STABILITY ANALYSIS



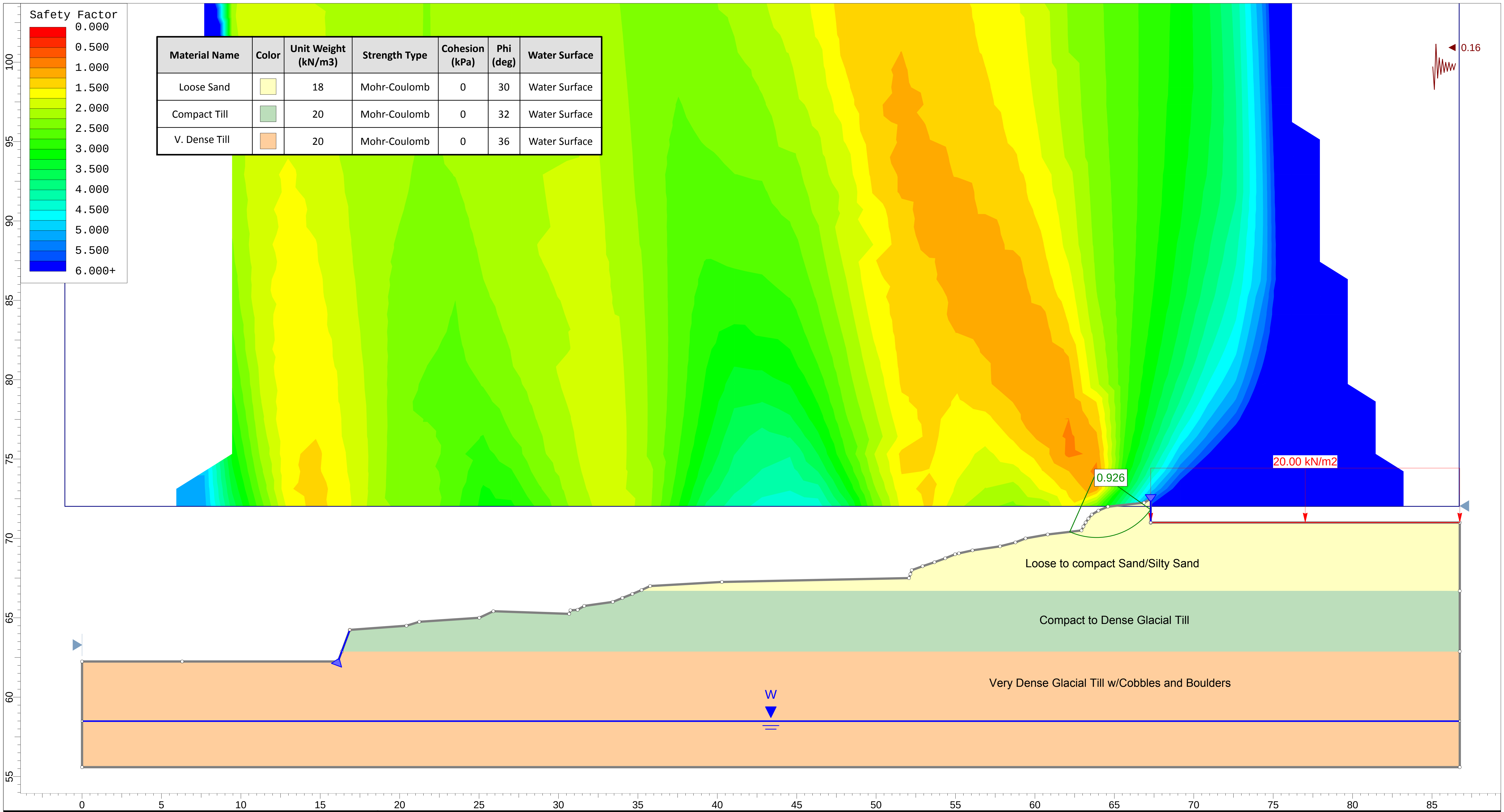
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface
Loose Sand	🟨	18	Mohr-Coulomb	0	30	Water Surface
Compact Till	🟩	20	Mohr-Coulomb	0	32	Water Surface
V. Dense Till	🟧	20	Mohr-Coulomb	0	36	Water Surface

SLIDEINTERPRET 6.039

Project
161-17230-00 Southminster Geotech and ESA

Analysis Description
Section 1: Existing Conditions

Drawn By		*Scale*	1:225	*Company*	WSP Canada Inc.
Date	2/14/2017, 11:15:48 AM			*File Name*	Localized Instability (Static)

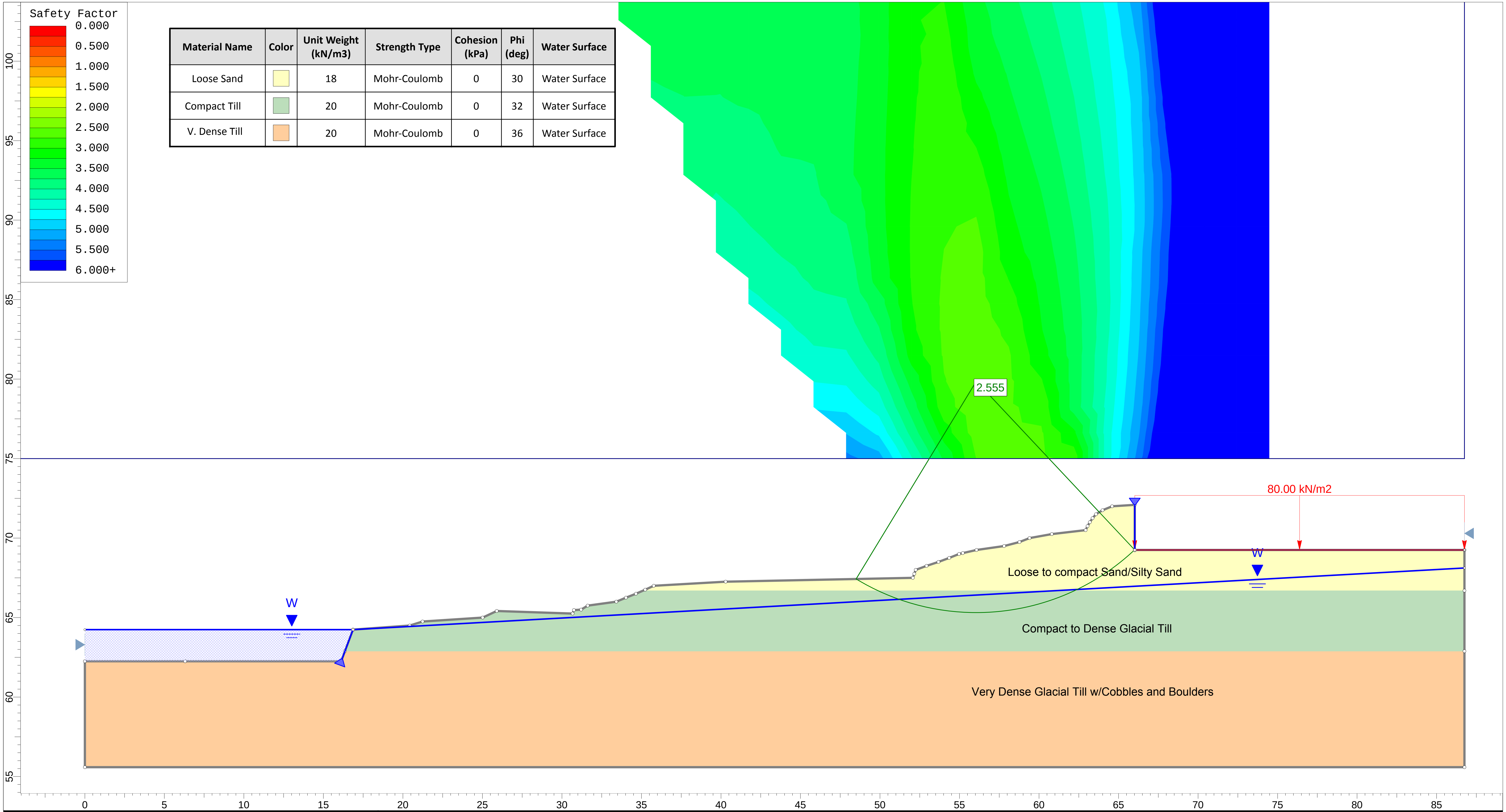


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface
Loose Sand	🟨	18	Mohr-Coulomb	0	30	Water Surface
Compact Till	🟩	20	Mohr-Coulomb	0	32	Water Surface
V. Dense Till	🟧	20	Mohr-Coulomb	0	36	Water Surface

SLIDEINTERPRET 6.039

Project	161-17230-00 Southminster Geotech and ESA
Analysis Description	Section 1: Existing Conditions

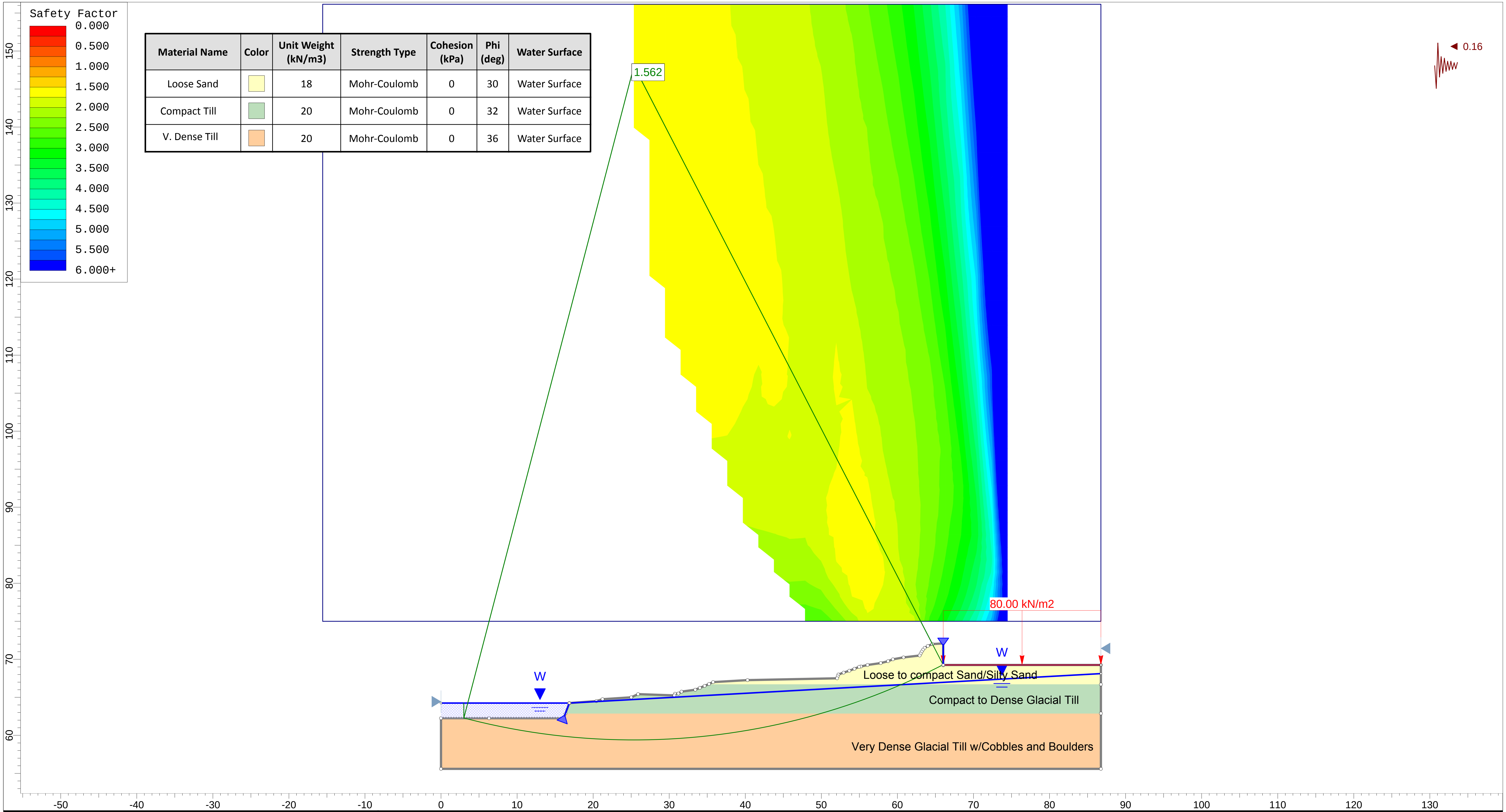
Drawn By	DW	*Scale*	1:225	*Company*	WSP Canada Inc.
Date	2/14/2017, 11:15:48 AM			*File Name*	Localized Instability (Seismic)



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface
Loose Sand	🟨	18	Mohr-Coulomb	0	30	Water Surface
Compact Till	🟩	20	Mohr-Coulomb	0	32	Water Surface
V. Dense Till	🟧	20	Mohr-Coulomb	0	36	Water Surface

SLIDEINTERPRET 6.039

Project	161-17230-00 Southminster Geotech and ESA		
Analysis Description	Section 1: Proposed Construction		
Drawn By	DW	*Scale*	1:225
Company	WSP Canada Inc.		
Date	2/14/2017, 11:15:48 AM	*File Name*	Proposed (Static)



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface
Loose Sand	🟨	18	Mohr-Coulomb	0	30	Water Surface
Compact Till	🟩	20	Mohr-Coulomb	0	32	Water Surface
V. Dense Till	🟧	20	Mohr-Coulomb	0	36	Water Surface

SLIDEINTERPRET 6.039

Project	161-17230-00 Southminster Geotech and ESA
Analysis Description	Section 1: Proposed Construction
Drawn By	
Scale	1:470
Company	WSP Canada Inc.
Date	2/14/2017, 11:15:48 AM
File Name	Proposed (Seismic)

Appendix F

EXPLANATION OF TERMS USED IN THIS REPORT

Explanation of Terms Used in the Record of Boreholes

Sample Type

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Dimension type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Spoon sample
SH	Shelby tube Sample
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

Penetration Resistance

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) required to drive a 50 mm (2 in) drive open sampler for a distance of 300 mm (12 in).

WH – Samples sinks under “weight of hammer”

Dynamic Cone Penetration Resistance, N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) to drive uncased a 50 mm (2 in) diameter, 60° cone attached to “A” size drill rods for a distance of 300 mm (12 in).

Textural Classification of Soils

Classification	Particle Size
Boulders	> 200 mm
Cobbles	75 mm - 200 mm
Gravel	4.75 mm - 75 mm
Sand	0.075 mm – 4.75 mm
Silt	0.002 mm-0.075 mm
Clay	<0.002 mm

Coarse Grain Soil Description (50% greater than 0.075 mm)

Terminology	Proportion
Trace	0-10%
Some	10-20%
Adjective (e.g. silty or sandy)	20-35%
And (e.g. sand and gravel)	> 35%

Soil Description

a) Cohesive Soils(*)

Consistency	Undrained Shear Strength (kPa)	SPT “N” Value
Very soft	<12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very stiff	100-200	15-30
Hard	>200	>30

(*) Hierarchy of Shear Strength prediction

1. Lab triaxial test
2. Field vane shear test
3. Lab. vane shear test
4. SPT “N” value
5. Pocket penetrometer

b) Cohesionless Soils

Density Index (Relative Density)	SPT “N” Value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Soil Tests

w	Water content
w_p	Plastic limit
w_l	Liquid limit
C	Consolidation (oedometer) test
CID	Consolidated isotropically drained triaxial test
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement
D_R	Relative density (specific gravity, Gs)
DS	Direct shear test
ENV	Environmental/ chemical analysis
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified proctor compaction test
SPC	Standard proctor compaction test
OC	Organic content test
U	Unconsolidated Undrained Triaxial Test
V	Field vane (LV-laboratory vane test)
γ	Unit weight

Appendix G

LIMITATIONS OF THIS REPORT

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to WSP Canada Incorporated (WSP) at the time of preparation. Unless otherwise agreed in writing by WSP, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

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

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.