CHUNG & VANDER DOELEN
ENGINEERING LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
1016, 1018, 1024, 1030 & 1032 Doering Street
Wellesley, Ontario

SUBMITTED TO:
Jim Flynn
66 Schweitzer Crescent
Wellesley, Ontario
N0B 2T0

ATTENTION:
Mr. Jim Flynn

FILE NO /G18713/December 5, 2019

CHUNG & VANDER DOELEN
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Jim Flynn
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Attention: Mr. Jim Flynn

RE: Geotechnical Investigation
Proposed Residential Development
1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario

We take pleasure in enclosing one (1) copy of our Geotechnical Investigation Report carried out at the above-referenced Site. Soil samples will be retained for a period of three (3) months and will thereafter be disposed of unless we are otherwise instructed.

If you have any questions or clarifications are required, please contact the undersigned at your convenience.

We thank you for giving us this opportunity to be of service to you.

Yours truly,
CHUNG & VANDER DOELEN ENGINEERING LTD.

Eric Y. Chung, M. Eng., P.Eng.
Principal Engineer

TABLE OF CONTENTS

LIST OF ENCLOSURES

1.0 INTRODUCTION

CHUNG & VANDER DOELEN ENGINEERING LTD. (CVD) has been retained by Mr. Jim Flynn to carry out a geotechnical investigation for the proposed residential development proposed at 1016, 1018, 1024, 1030 & 1032 Doering Street in Wellesley, Ontario.

The existing residential dwellings at the site are not to be demolished as part of the development, however, three (3) sheds at Residences No. 1030 and 1032 are to be removed.

It is understood that the proposed development will consist of one (1) single detached home and eleven (11) townhome blocks up to 2-stories high with walkout/lookout basements. An internal roadway is proposed throughout the development and will connect to Doering Street at two (2) locations, at the eastern dead end and just west of the residence at address no. 1030. Two (2) visitor parking areas are proposed in the centre and northeast corner of the site. Finished first floor elevations at the various residential units are proposed between 351.38 and 357.82 m. Site grading and engineered fill placement will be required.

The purpose of this investigation was to determine the subsurface conditions at the site and, based on the findings, to make geotechnical recommendations for:

- Foundation design recommendations;
- Excavation condition;
- Groundwater control during construction;
- Basement condition;
- Site grading and engineered fill construction;
- Backfilling recommendations;
- Foundation soil classification seismic design per OBC 2012; and
- Slope stability assessment

2.0 FIELD WORK

In order to investigate the subsurface conditions at the site, eleven (11) boreholes were advanced to depths between 5.03 and 8.05 m below ground surface on January 23, 2019 as part of an initial investigation and on August 26 and 27, 2019 due to site expansion. The borehole locations are indicated on the Borehole Location Plan, Drawing No. 1.

The field work was carried out under the supervision of a member of our engineering team, who logged the boreholes in the field, effected the subsurface sampling, and monitored the groundwater conditions. The boreholes were advanced using a track-mounted drilling rig, supplied and operated by a specialized contractor. The drill rig was equipped with continuous flight augers and standard soil sampling equipment. Standard penetration tests (SPTs) in accordance with ASTM Specification D1586, were carried out at frequent intervals of depth, and the results are shown on the Borehole Logs as Penetration Resistance or “N”-values. The undrained shear strength of the cohesive soil deposit was determined on the slightly disturbed SPT samples using a field pocket penetrometer. The compactness condition or consistency of the soil strata has been inferred from these test results.

Monitoring wells were installed in Boreholes 2, 5, 6 and 10 to allow for long term monitoring of the groundwater condition. As part of a Phase II Environmental Site Assessment (ESA) investigation by CVD, a monitoring well (ESA-BH 7) was installed within one of the building footprints near Borehole 11.

Well response tests (slug tests) were completed on the five (5) monitoring wells to determine the hydraulic conductivity (or permeability) of the geologic materials located at the water table. The data was analyzed using Aquifer Test software and the data and graphical analyses are provided in Appendix C.

The borehole locations were established on site by CVD. Ground surface elevations at Boreholes 1 to 5 were surveyed by Van Harten Surveying Inc. as part of the topographic survey on January 17, 2019. Ground surface elevations at Boreholes 6 to 11 were surveyed by CVD and the elevations were referenced to a temporary benchmark (TBM) which is shown on Drawing No. 1and described below:

TBM:	Top of manhole in Doering Street in front of Residence No. 1024, as shown on Drawing No. 1
Elevation:	352.41 m (Geodetic)

3.0 LABORATORY TESTING

Soil samples obtained from the in-situ tests were examined in the field and subsequently brought to our laboratory for visual and tactile examination to confirm field classification. Moisture content determination of all retrieved samples occurred.

In addition, four (4) grain size distribution analyses and two (2) sets of Atterberg Limits were performed on the major soil deposits to confirm field identification and to provide information on the soil properties.

4.0 EXISTING SITE CONDITIONS

The site is located at the end of Doering Street in Wellesley, Ontario. The site is irregular in shape and is bound to the east by Firella Creek, to the west by existing residences, to the north by existing residences and Doering Street and to the south by green space. The site surrounds the residence at 1032 Doering Street to the north, south and east along the western property limit.

The site is occupied by three (3) single-storey residences along Doering Street located at Residences No. 1016, 1018 and 1030 and two (2) sheds at Residence No. 1032. An extended driveway is located along the western property line and extends to sheds at Residence No. 1032. There is an existing pond located immediately to the south of a canvas-covered shed. The remainder of the site is grass-covered with many mature trees scattered throughout.

The site is terraced with an approximate 1 to 2 m high curving bank which dissects the site along its length. The ground surface of the upper terrace slopes downwards in a southeasterly direction. The highest ground on the upper terrace is located in the northwestern portion of the site, in the area of Residence No. 1030, with a ground surface elevation at approximately 356.0± m and slopes down to approximately 348.5± m, the lowest elevation on the upper terrace. A second 1 to 2 m high steep bank runs along the bottom of the lower terrace and the bank of Firella Creek. The elevation at the bottom of the lower bank is between 345.8± and 346.3± m.

The ground surface elevations at the borehole locations ranged between 349.01 and 355.89 m.

5.0 SUBSURFACE CONDITIONS

The detailed subsurface conditions encountered in the eleven (11) boreholes advanced as part of this investigation are shown on the Borehole Log Sheets, Enclosures 1 to 11 and the borehole log from the Phase II ESA investigation is shown in Appendix B. The following sections provide descriptions of the major soil deposits encountered in the boreholes.

The stratigraphic boundaries shown on the borehole logs are inferred from non-continuous sampling conducted during advancement of the borehole drilling procedures and, therefore, represent transitions between soil types rather than exact planes of geologic change. The subsurface conditions will vary between and beyond the borehole locations.

5.1 Topsoil

A layer of topsoil was encountered at ground surface at Boreholes 1 to 11 with a measured thickness between 100 and 380 mm.

5.2 Fill

A layer of fill materials was encountered below the topsoil in Boreholes 2 to 9 and 11. The fill layer extended to depths between 0.36 to 3.36 m below ground surface. Fill materials may exist at greater depths in the areas of existing structures and foundations.

The non-cohesive fill materials at Boreholes 1, 2, 4 to 8, 10 and 11 were comprised of varying amounts of sand and silt in the range of sand with trace silt to sandy silt, with trace to some gravel and trace clay. Underlying the silty sand fill at Borehole 7 was a layer of sand and gravel fill with trace to some silt. Occasional silty layers and cobbles were observed within the sand and gravel fill at Borehole 7. Occasional clayey seams were observed at Borehole 8. The SPT “N”-values measured within the non-cohesive fill materials ranged from 3 to 26 blows per 300 mm of penetration, indicating a very loose to compact compactness condition.

The cohesive fill materials at Boreholes 3 and 9 were comprised of clayey silt with some sand and trace gravel. A layer of buried topsoil was found near the bottom of the fill materials in Borehole 3. The SPT “N”-values measured within the cohesive fill materials ranged from 3 to 10 blows per 300 mm of penetration, indicating a soft to stiff consistency.

Trace to some topsoil/organics were observed within the fill at Boreholes 3, 6 to 9 and 11. The measured water content of the samples collected ranged between 2 and 23%, thus indicating damp to saturated moisture condition.

5.3 Clayey Silt

A deposit of clayey silt was encountered below the topsoil at Boreholes 1 and 10 and underlying the fill materials at Boreholes 2 to 9 and 11. Where fully penetrated in Boreholes 1, 2, 10 and 11, the deposit extended to depths between 3.96 and 5.49 m below existing grades. Boreholes 3 to 9 were terminated within this deposit to depths between 6.55 and 8.05m below ground surface. The clayey silt deposit contained trace to some sand, trace gravel. Occasional to frequent silt seams were observed throughout the deposit. An intermittent silt layer was encountered within the clayey silt in Borehole 3 at depths between 2.1 and 2.9 m. Results of two (2) grain size distribution analyses from Boreholes 3 and 5 are shown graphically on Enclosures 14 and 15.

The SPT "N"-values measured within the deposit ranged from 6 to 29 blows per 300 mm of penetration. The undrained shear strength obtained on the retrieved samples ranged from 24 kPa to over 250 kPa. Based on the above test results and tactile examination, the clayey silt is considered to have a firm to very stiff consistency. The measured water content of the samples collected ranged between 16 and 29%, thus indicating a moist to wet moisture condition.

Appendix C provides analyses of well response test (slug test) data collected on November 15, 2019 from wells at Boreholes 2, 5, 6, 10 and ESA-Borehole 7. These analyses indicate the hydraulic conductivity (permeability) of the predominant clayey silt soil beneath the property is in the $3x10^{-7}$ to $6x10^{-6}$ cm/sec range. This indicates that groundwater flow through this material (both vertical and horizontal) will be very slow.

5.4 Silt

A silt deposit containing trace amounts of sand and clay was encountered underlying the clayey silt deposit at Borehole 2 and 10 and within the clayey silt deposit at Borehole 4. The deposit extended to depths of 5.49 and 6.86 m below ground surface at Boreholes 2 and 10, respectively. The silt deposit at Borehole 4 was encountered between 2.1 and 2.9 m depth. Occasional clayey seams were observed within the deposit at Boreholes 2 and 10. Results of one (1) grain size distribution analysis from Boreholes 2 is shown graphically on Enclosure 12.

The SPT "N"-values measured within the deposit ranged from 15 to 41 blows per 300 mm of penetration, indicating a compact to dense compactness condition. The measured water content of the samples collected ranged between 15 and 19%, thus indicating a saturated moisture condition.

5.5 Sand

A sand deposit containing some gravel and trace silt was encountered below the clayey silt deposit at Borehole 1. Borehole 1 was terminated within the sand deposit at a depth of 5.03 m.

The SPT "N"-value measured within this layer was 25 blows per 300 mm of penetration, indicating a compact compactness condition.

5.6 Sand and Silt Till

A deposit of sand and silt till was encountered below the silt deposit at Boreholes 2 and 10 and underlying the clayey silt deposit at Borehole 11. All three (3) Boreholes were terminated within the deposit at depths between 6.38 and 7.30 m below existing grades. The deposit contained trace to some gravel and trace clay. Occasional cobbles were observed within the deposit at Borehole 10. Results of one (1) grain size distribution analysis from Boreholes 1 is shown graphically on Enclosure 13.

The SPT "N"-values measured within the deposit ranged from 57 blows per 300 mm to 50 blows per 125 mm of penetration, indicating a very dense compactness condition. The measured water content of the samples collected ranged between 8 and 10%, thus indicating a moist moisture condition.

5.7 Groundwater

Groundwater conditions were monitored during and following completion of borehole sampling. Monitoring wells were installed in Boreholes 2, 5, 6 and 10. In addition, as part of a concurrent Phase II Environmental Site Assessment (ESA) investigation by CVD, a monitoring well (ESA-BH 7) was installed near Borehole 11. The table below summarizes the water level readings in the monitoring wells:

Borehole No.	Ground Surface Elevation (m)	Date	Water Level Below Existing Ground Surface (m)	Water Level Elevation (m)
2	349.52	January 30, 2019	0.32	349.20
		March 8, 2019	0.36	349.16
		September 4, 2019	0.74	348.78
		November 15, 2019	0.41	349.11
5	351.16	January 30, 2019	0.99	350.17
		March 8, 2019	0.88	350.28
		September 4, 2019	3.00	348.16
		December 4, 2019	1.70	349.46
6	355.89	September 4, 2019	5.15	350.74
		December 4, 2019	1.58	354.31
10	350.61	September 4, 2019	0.82	349.79
		November 15, 2019	0.56	350.05
ESA-7	352.08	September 4, 2019	1.26	350.82
		November 15, 2019	0.70	351.38

The water level measured in the monitoring wells ranged in depths between 0.32 and 5.15 m, corresponding to elevations 348.16 and 354.31 m. The water levels observed during and following the completion of drilling at Boreholes 1, 3, 4, 7 and 11 ranged between 2.7± and 6.1± m below ground surface and Boreholes 8 and 9 remained dry. The water levels observed in the open boreholes upon completion of drilling does not represent stabilized groundwater levels.

It is noted that the observed groundwater table will fluctuate seasonally and in response to major weather events.

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 General

The existing residential dwellings at the site are not to be demolished as part of the development, however, three (3) sheds at Residences No. 1030 and 1032 are to be removed.

It is understood that the proposed development will consist of one (1) single detached home and eleven (11) townhome blocks up to 2-stories high with walkout/lookout basements. An internal roadway is proposed throughout the development and will connect to Doering Street at two (2) locations, at the eastern dead end and just west of the residence at address no. 1030. Two (2) visitor parking areas are proposed in the centre and northeast corner of the site. Finished first floor elevations at the various residential units are proposed between 351.38 and 357.82 m. Site grading and engineered fill placement will be required.

In general, the surficial topsoil was underlain by fill materials followed by a major deposit of firm to very stiff clayey silt. The clayey silt was in turn underlain by compact to very dense silt, sand, and sand and silt till deposits.

The water level measured in the monitoring wells ranged in depths between 0.32 and 5.15 m, corresponding to elevations 348.16 and 354.31 m. The water levels observed during and following the completion of drilling at Boreholes 1, 3, 4, 7 and 11 ranged between 2.7± and 6.1± m below ground surface and Boreholes 8 and 9 remained dry. The water levels observed in the open boreholes upon completion of drilling does not represent stabilized groundwater levels.

Based on the proposed finished floor elevations of the residential units, the finished first floor levels will lie above the observed high groundwater table. Assuming a depth of 3.2± m for proposed walkout/lookout basements, basement finished floor elevations will lie 0.7± to 2.0± m below the seasonal high groundwater table in the areas of lots 3 to 19 and 29 to 48. However, due to the very low permeability of the native clayey silt soils, basements can still be constructed below the groundwater table with a conventional weeping tile and sump pump drainage system implemented at each townhome block.

It may be necessary to install an underfloor drainage system in these townhome blocks to efficiently convey the water to the sumps. It is recommended that at least one (1) sump pit should be installed in every 3 units. Positive grade adjacent to the basement will direct surface water away from the building, preventing surface water infiltration.

6.2 Footing Foundations

Conventional strip and spread footing foundations can be used to support the proposed low rise (up to 2 levels) residential building. Footings cast on or native stiff to very stiff clayey silt can be designed using a Geotechnical Reaction at SLS of 150 kPa. The SLS value given above is based on a maximum settlement of 25 mm under the footing foundations. The Factored Geotechnical Resistance at ULS is 250 kPa.

These soil bearing pressures can be achieved provided that the founding subgrade is undisturbed during construction. The majority of the settlements will take place during construction and the first loading cycle of the building.

Engineered fill may be used to replace the firm/loose native soils. Procedures for engineered fill placement are given in Section 6.3.

The following table summarizes the highest founding level and elevation for the footing at each borehole location:

Borehole No.	Existing Ground Elevation (m)	Highest Founding Depth (m)	Highest Founding Elevation (m)
1	349.01	1.01	348.00
2	349.52	1.02	348.50
3	350.71	2.41	348.30
4	350.90	1.60	349.30
5	351.19	1.16	350.00
6	355.89	1.09	354.80
7	354.07	4.57	349.50
8	350.59	0.89	349.70
9	352.47	1.87	350.60
10	350.61	1.01	449.60
11	349.19	1.49	347.70

It is recommended that a lean concrete mat be placed over approved footing subgrade in wet or saturated areas to prevent further disturbance to the bearing soils resulting from construction activities.

In addition, the footings should be founded below any existing fill materials, on native undisturbed soils. Spacing between adjacent footing steps should not be steeper than 10H to 7V.

The maximum total and differential settlements of footings designed to the above-recommended soil bearing pressure are expected to be less than 25 and 12 mm, respectively, and these are considered tolerable for the structure being contemplated.

Exterior footings and footings in unheated portions of the building should be provided with a soil cover of not less than 1.2 m or equivalent synthetic thermal insulation for adequate frost protection. The founding subgrade soils must be protected from frost penetration during winter construction.

It is recommended that the footing excavations be inspected by the geotechnical engineer to ensure adequate soil bearing and proper subgrade preparation.

6.3 Site Grading and Engineered Fill

It is anticipated that site grading procedure will be required across the site. It is recommended to construct engineered fill in areas to be raised in order to suitably support the future building foundations, floor slabs and pavement areas. Additional boreholes/test pits will be required to establish the vertical and horizontal extent of the loose fill materials, especially near Borehole 7 where 3.66 m deep fill materials were identified.

Prior to any new fill placement, the site needs to be appropriately prepared. For footings constructed on engineered fill, the existing topsoil, fill materials, organics and all loose/firm native soils need to be stripped/excavated to expose competent native subgrade soil.

The inorganic onsite clayey silt deposits are deemed suitable for site regrading operations. The moisture content of these excavated soils should be within 3% below the optimum moisture content in order to achieve the specified degrees of compaction. The excavated inorganic fine granular soils can be reused to construct the engineered fill provided that this fill material is not overly wet or dry.

The grading work should be carried out during relatively dry weather as the predominant clayey silt soils are sensitive to wetting and are difficult to handle when wet. Therefore, earthworks should be scheduled in the drier summer months. The native fine granular soils are susceptible to softening and deformation when exposed to excessive moisture and construction traffic. As a result, it is imperative that the grading/filling operations are planned and maintained to direct surface water run-off to low points and then be positively drained by suitable means. During periods of wet weather, construction traffic should be directed along the designated construction routes so as not to disturb and rut the exposed subgrade soil. Temporary construction roads consisting of clear crushed material (such as crushed stone or recycled concrete) may be required during poor weather conditions such as a wet Spring or Fall.

Should additional bulk fill require to be imported to the site for site grading purposes, it should be similar in gradation to the existing on-site granular soils or consist of OPSS Granular B Type I. It is recommended that any proposed borrow source materials be tested prior to importing, in order to ensure that the environmental quality of the fill meets all environmental approval standards and to ensure that the natural moisture content of the fill is suitable for compaction.

Backfilling local excavations (such as foundation walls, footings and trenches inside the building footprint) should be performed using imported OPSS Granular B Type I or approved on-site soil. Some of the onsite fill materials could be reused to construct the engineered fill, provided that they are free of organics and deleterious materials.

It is recommended that any off-site borrow source materials be tested prior to importing, in order to ensure that the environmental quality of the fill meets all environmental approval criteria and to ensure that the natural moisture content of the fill is suitable for compaction.

The engineered fill should be constructed in accordance with the following procedures in order to support building foundations, floor slabs and pavement areas:

1) All topsoil, fill materials, firm clayey silt soils and deleterious materials are to be stripped from building and pavement areas to expose competent native subgrade soils.

2) The exposed subgrade surface is to be thoroughly recompacted by large heavy compaction equipment (10 tonne compactor is recommended) and inspected by qualified geotechnical personnel. Any loose or soft areas identified should be excavated to the level of competent soil;

3) The required grades can then be achieved by placing approved onsite soils or imported sand and gravel (OPSS Granular “B” Type I), in maximum 300 mm thick lifts and compacted to at least 100% Standard Proctor maximum dry density (SPMDD) in areas to support building foundations and floor slabs. The specified degree of compaction may be reduced to a minimum of 95% SPMDD in roadway areas.

4) The moisture content of the fill materials is recommended to be within 3% below their optimum moisture contents in order to achieve the specified degrees of compaction;

5) Engineered fill must be placed such that the fill pad extends horizontally outwards from all footings at least the same distance as how thick the engineered fill pad will exist between the underside of future footings and the approved native earth subgrade;

6) Compaction above the footing foundations to the floor subgrade level (for support of the floor slab) is to be no less than 95% SPMDD;

7) All fill placement and compaction operations must be supervised on a full-time basis by qualified geotechnical personnel to approve fill material and ensure the specified degrees of compaction have been achieved.

During construction, vibration could be generated from various construction equipment, such as compactors and rollers which could be harmful to surrounding structures and buildings. Peak particle velocity (PPV) of ground motion is widely accepted as the best descriptor of potential for vibration damage to structures. The safe vibration limit can be set to 10 to 20 mm/s PPV, depending on the sensitive of surrounding structures to vibration.

Vibration monitoring can be carried out to measure the PPV of ground motion from vibration generated from typical compaction equipment at the beginning of the project in the potentially critical areas. This will set criteria and establish the type of equipment to be used for this project.

It is recommended that a pre-construction condition survey be conducted to document the condition of the existing structures within the possible zone of influence.

6.4 Earthquake Considerations

In accordance with The Ontario Building Code 2012 (OBC), the proposed structure should be designed to resist earthquake load and effects as per OBC Subsection 4.1.8.

Based on the anticipated condition of the underlying soil condition encountered at the boreholes, the site can be classified as a Site Class C as per OBC Table 4.1.8.4.A (Page B4-24).

6.5 Open Cut Excavation and Groundwater Control

Excavations are expected to be in the order of 2 to 4 m deep for engineered fill placement, foundation and site servicing. The excavations will penetrate loose to compact and soft to stiff fill materials and firm to very stiff clayey silt. These materials are considered to be Type 3 Soils in accordance with the latest Occupational Health and Safety Act.

Above the groundwater table, excavations in the Type 3 Soils are expected to remain stable during the construction period provided that side slopes are cut to 1H : 1V from the bottom of the excavation. Where seepage or perched groundwater is encountered, side slopes should be cut to more stable angles of 3H : 1V. The side slopes should be suitably protected from erosion processes.

Rainwater or local perched groundwater can be controlled by pumping from filtered sump pits as and where required. It is recommended that excavation for future development be done during the typically drier summer months when groundwater conditions would be expected to lie at lower elevations.

In wet to saturated subgrade condition, it will be necessary to excavate below founding level and pour a 75 mm thick mud slab of lean concrete to protect the founding soil from disturbance during the installation of reinforcing steel bars and formwork.

6.6 Floor Slab Construction

Based on the proposed finished floor elevations of the residential units, the finished first floor levels will lie above the observed high groundwater table. Assuming a depth of 3.2± m for proposed walkout/lookout basements, basement finished floor elevations will lie 0.7± to 2.0± m below the seasonal high groundwater table in the areas of lots 3 to 19 and 29 to 48. However, due to the very low permeability of the native clayey silt soils, basements can still be constructed below the groundwater table with a conventional weeping tile and sump pump drainage system implemented at each townhome block.

It may be necessary to install an underfloor drainage system in these townhome blocks to efficiently convey the water to the sumps. It is recommended that at least one (1) sump pit should be installed in every 3 units. Positive grade adjacent to the basement will direct surface water away from the building, preventing surface water infiltration.

Cognizant of the expected subgrade soil conditions at the finished basement floor levels, it is anticipated that a system of underfloor drains (at 6 m spacing) will be required and be connected to a positively drained sump(s) or permanently to the municipal sewer to locally control the groundwater table (and expected fluctuations) in order to keep the basement floors in a dry condition.

The exposed subgrade should be proof-rolled with a heavy roller in conjunction with an inspection by the geotechnical engineer at the time of floor slab construction. Excess moisture in the subgrade soil will render the material incompactable. Any soft and/or unstable areas detected should be replaced with imported Granular "B" Type I which should be compacted to 95% SPMDD.

Following the proof-rolling of the subgrade, it is recommended that a minimum 150 mm thick layer of OPSS Granular "A" be placed and compacted to at least 100% SPMDD beneath the concrete floor slabs to provide uniform support.

A modulus of subgrade reaction (k_s) of 30 MN/m^3 may be used for the design of the floor slabs, considering the floor subgrade will consist of predominantly clayey silt soils.

The floor slab should be separated structurally from the columns and foundation walls. Sawcut control joints should be provided at regular spacing (less than 30 times the concrete slab thickness) and to depths between one-third to one-quarter of the slab thickness.

Moisture migration from the underlying soils through the concrete slab-on-grade will take place via "capillary action" and "diffusion" (due to vapour pressure differential). Although the Granular "A" layer will provide a capillary break, the low permeance of the concrete slab and floor coverings will result in 100% humidity under the concrete slab and, consequently, the moisture in the concrete will increase over time. The potential effect of the soil moisture should be considered in selecting the floor coverings. A vapour retarder material (such as a 15 mil poly, ASTM E-1745) can be placed to reduce soil moisture migration. Reference is made to ACI 302.

6.7 Lateral Earth Pressure

The unbalanced foundation walls and any other soil retaining structures should be designed to resist the lateral earth pressure acting against these walls. The following formula may be used to calculate the unfactored earth pressure distribution. The factored resistance can be calculated by using a factor of 0.8.

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

where:

P =	lateral earth pressure	kPa
K =	earth pressure coefficient, 0.5 for non-yielding foundation wall earth pressure coefficient, 0.3 for yielding retaining wall	
γ =	unit weight of granular backfill, compacted to 95% SPMDD	21 kN/m^3
H =	unbalanced height of wall	m
q =	surcharge load at ground surface	kPa

The backfill for the foundation walls and retaining walls should be free-draining granular materials which should have less than 8% silt particles (OPSS Granular "B" Type I). The backfill should be placed in thin layers and compacted to 95% SPMDD. Over-compaction should be avoided. Weeping tiles leading to a frost-free outlet or weep holes should be installed to effect drainage behind the retaining wall.
The sliding resistance of the retaining wall footings should be checked. The unfactored horizontal resistance against sliding between cast-in-place concrete and the various soils can be calculated using a friction coefficient as follows:

- Firm to very stiff clayey silt: 0.40
- Granular engineered fill: 0.35

The unit weight of clayey silt is 19 kN/m^3 and a unit weight of the granular backfill compacted to 95% SPMDD is 21 kN/m^3.

6.8 Access Driveway and Paved Parking Areas

Based on the results of the field work, the predominant subgrade materials at the site will consist of native clayey silt or engineered fill.

The following flexible pavement structures are recommended based on the results of grain size distribution, assumed CBR values, groundwater table, frost susceptibility of subgrade soils and traffic volume.

Component	Light Duty Pavement (mm)	Heavy Duty Pavement (mm)
Asphaltic Concrete HL3 HL8	 40 40	 40 50
Granular “A” Base	150	150
Granular “B” Sub-base	300	400

Due to the frost susceptibility of the subgrade soils, the pavement design considers that pavement construction will be carried out during the drier time of the year and that the subgrade is stable, not heaving under construction equipment traffic. If the subgrade is wet or unstable, additional granular sub-base may be required.

The base and sub-base materials should be produced in accordance with the current OPSS specifications and placed and uniformly compacted to at least 100% SPMDD. The asphaltic concrete should be placed and compacted in accordance with OPSS Form 310 and to at least 92% of the Marshall Density (MRD). Frequent in situ density testing by this office should be carried out to verify that the specified degree of compaction is being achieved and maintained.

It should be noted that even well-compacted trench backfill could settle for a period of time after construction. In this regard, the surface course of the asphaltic concrete should be placed at least one (1) year after trench backfill is completed so as to allow any minor settlements to occur within the trench backfill. The incomplete pavement structure may not be capable of supporting construction traffic. Consequently, minor repairs of the sub-base, base and asphaltic concrete may be required prior to paving with the base course and/or the surface course asphaltic concrete.

The prepared earth subgrade and final pavement surfaces should be graded to direct water runoff away from buildings, sidewalks and other similar pertinent structures. Positive drainage outlets should be provided at all low points of the prepared earth subgrade, such as stub drains extended from the catch-basins.

6.9 Concrete Pavement

The existing clayey silt material is frost-susceptible and will be subject to frost heaving. All concrete sidewalks adjacent to the building entrances (where the insulating effect of snow cover is removed on a continuous basis) should be underlain by a minimum 1.2 m thick suitably compacted OPSS Granular “B” Type I in order to ensure that the underlying frost-susceptible silt soil does not cause differential heaving problems in the winter months. Alternatively, equivalent suitable thermal insulation can be provided beneath these areas.

If OPSS Granular “B” Type I materials are used to backfill the foundation wall at the entrances, frost heaving of the concrete sidewalks or pavers will not be a concern.

6.10 Slope Stability Assessment

The site is irregular in shape and runs lengthwise along Firella Creek. The site is terraced and an approximate 1 to 2 m high curving bank dissects the site along its length.

The upper slope (bank) has a slope inclination of approximately 5H:1V. The ground surface of the upper terrace slopes downwards in a southeasterly direction. The highest ground on the upper terrace is located in the northwestern portion of the site with a ground surface elevation at approximately 356.0± m and slopes down to approximately 348.5± m, the lowest elevation on the upper terrace. The lower terrace which is about 10 to 35 m wide is relatively flat-lying.

A second 1 to 2 m high steep bank runs along the bottom of the lower terrace and the bank of Firella Creek. The lower bank (slope) has a slope inclination of approximately 1.5H:1V. The elevation of the bottom of the lower bank is between 345.8 and 346.3 m.

Based on Grand River Conservation Authority (GRCA) Regulation No. 124-15, effective October 23, 2015, the site can be defined as an “Apparent Valley (Confined System – Steep But Stable) as the slope adjacent to the upper terrace is less than 3H:1V.

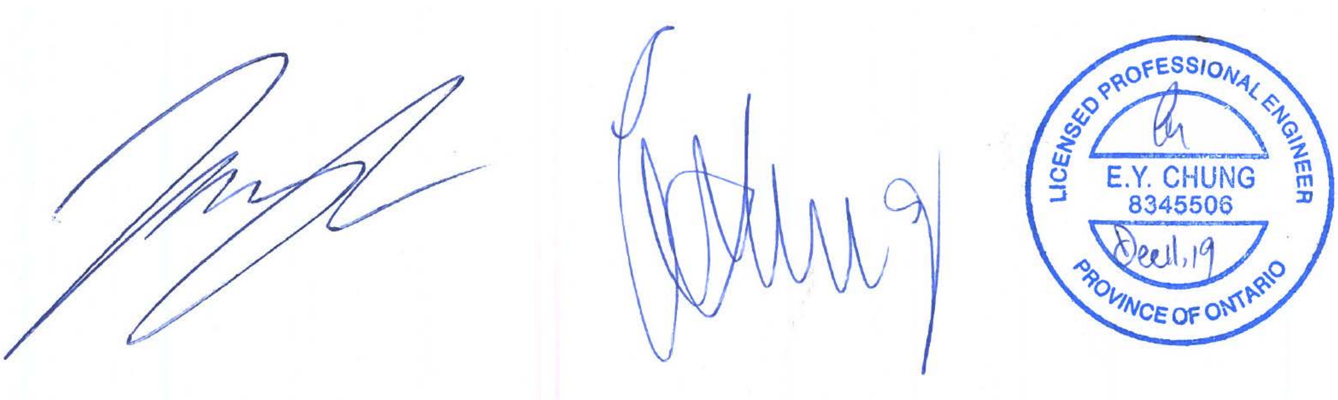
No stable slope allowance is required above the crest of the upper slope. The lower slope is located between 10 and 35 m from the toe of the upper slope which is an adequate distance for the toe erosion allowance. An erosion access allowance of 6 m is considered adequate as the site is readily accessible from both the top and the bottom of the upper slope.

7.0 CLOSURE

The Limitations of Report, as quoted in Appendix A, is an integral part of this report.

We trust that the information presented in this report is complete within our terms of reference. If there are any further questions concerning this report, please do not hesitate to contact our office.

Yours truly,
CHUNG & VANDER DOELEN ENGINEERING LTD.



Joseph van der Zalm, E.I.T.
Geotechnical Engineering Intern

Eric Y. Chung, M. Eng., P.Eng.
Principal Engineer

APPENDIX A

LIMITATIONS OF REPORT

APPENDIX “A”

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Soils Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes.

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 testholes and their respective depths 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 conclusion as to how the subsurface conditions may affect their work.

The benchmark and elevations mentioned in this report were obtained strictly for use in the geotechnical design of the project and by this office only, and should not be used by any other parties for any other purposes.

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. CHUNG & VANDER DOELEN ENGINEERING LIMITED accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

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. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

This report does not reflect the environmental issues or concerns unless otherwise stated in the report.

APPENDIX B

Previous Borehole Logs by Chung & Vander Doelen Engineering Ltd.

FILE No: E19807

BOREHOLE No. 7

Client: **Jim Flynn**

Project: **Phase II Environmental Site Assessment**

Location: **1016, 1018 & 1030 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D-50T**
Method: **Hollow Stem Auger**
Size: **108 mm I.D.**
Date: **Aug 28 - 19** TO **Aug 28 - 19**

Ground Elevation: **352.08 m**

ELEV./DEPTH (m)	DESCRIPTION	SAMPLE ID	TYPE	N-VALUE	REMARKS
	brown, mosit TOPSOIL with organics	1	SS	9	flushmount casing set in concrete; 0.0 ppm
351.78 / 0.30	brown FILL, silty sand moist to wet				bentonite seal/50 mm I.D. PVC riser
351.01 / 1.07	brown to grey CLAYEY SILT some gravel and cobbles moist to saturated	2	SS	5	water level in monitoring well at a depth of 1.26 m on September 4, 2019
		3	SS	14	3.0 m long, 50 mm I.D. PVC screen with sandpack
		4	SS	17	0.0 ppm
347.51 / 4.57	End of Borehole				monitoring well installed in BH7 at 4.57 m

SHEAR STRENGTH (kPa): FIELD VANE: Peak ⊗ Rem. ×; LAB TEST: Unc. ■ P.P. □; 50 100 150 200
PENETRATION RESISTANCE: STANDARD ● DYN. CONE ○; 20 40 60 80
WATER CONTENT (%): W_P W W_L; 10 20 30
WELL DATA
DEPTH (m): 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 5.0 5.5 6.0 6.5 7.0 7.5 8.0 8.5

PROJECT MANAGER: **JK**

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APPENDIX C

Well Response Test Analysis Charts

CHUNG & VANDER DOELEN
ENGINEERING LTD.
311 Victoria Street North
Kitchener / Ontario / N2H 5E1
519-742-8979

Slug Test - Water Level Data

Project: 1016, 1018, 1024, 1030 & 1032 Doering Street

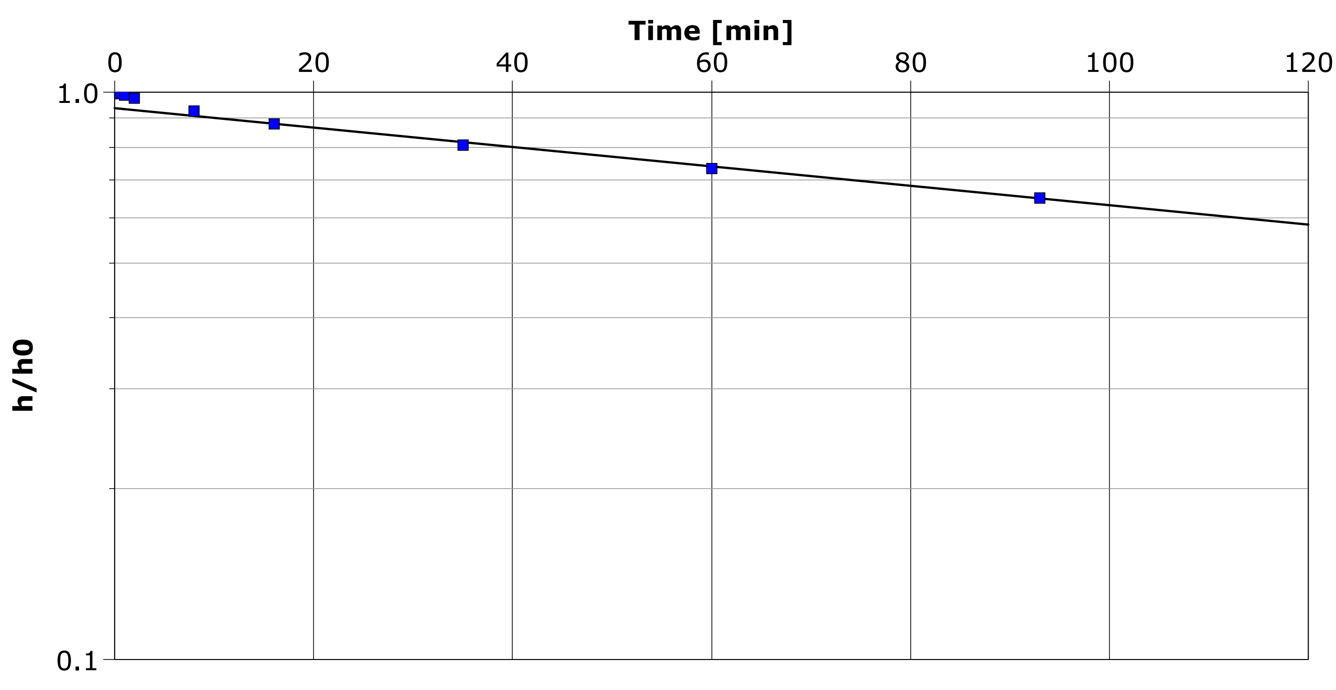
Number: G18713

Client: Mr. Jim Flynn

Location: Wellesley, ON	Slug Test: ESA - BH 7	Test Well: ESA - BH 7
Test Conducted by: D. White	Test Date: 15/11/2019	
Water level at t=0 [m]: 3.84	Static Water Level [m]: 0.70 T.O.P	Water level change at t=0 [m]: 3.14

	Time [min]	Water Level [m]	WL Change [m]
1	0	3.84	3.14
2	0.5	3.82	3.12
3	1	3.80	3.10
4	2	3.76	3.06
5	8	3.61	2.91
6	16	3.46	2.76
7	35	3.23	2.53
8	60	3.00	2.30
9	93	2.74	2.04

Aquifer Thickness: 3.87 m



Calculation using Bouwer & Rice

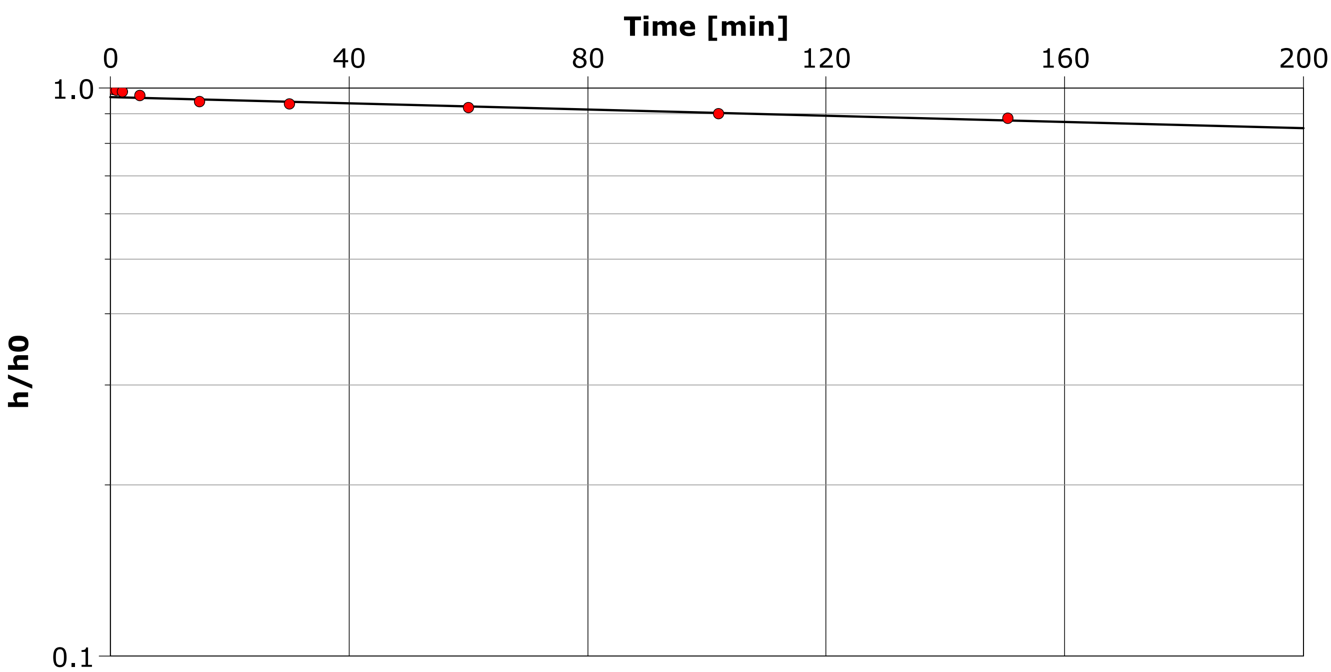
Observation Well	Hydraulic Conductivity [m/s]	
MW 7	2.07×10^{-8}	

CHUNG & VANDER DOELEN ENGINEERING LTD. 311 Victoria Street North Kitchener / Ontario / N2H 5E1 519-742-8979	**Slug Test - Water Level Data**	
	Project: 1016, 1018, 1024, 1030 & 1032 Doering Street	
	Number: G18713	
	Client: Mr. Jim Flynn	

Location: Wellesley, ON	Slug Test: BH 5	Test Well: BH 5
Test Conducted by: D. White	Test Date: 15/11/2019	
Water level at t=0 [m]: 5.77	Static Water Level [m]: 3.97 T.O.P	Water level change at t=0 [m]: 1.80

	Time [min]	Water Level [m]	WL Change [m]
1	0	5.77	1.80
2	0.5	5.76	1.79
3	1	5.755	1.785
4	2	5.74	1.77
5	5	5.715	1.745
6	15	5.675	1.705
7	30	5.66	1.69
8	60	5.63	1.66
9	102	5.59	1.62
10	150.5	5.56	1.59

Aquifer Thickness: 2.82 m



Calculation using Bouwer & Rice

Observation Well	Hydraulic Conductivity [m/s]	
BH 5	3.31×10^{-9}	

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Slug Test - Water Level Data

Project: 1016, 1018, 1024, 1030 & 1032 Doering Street

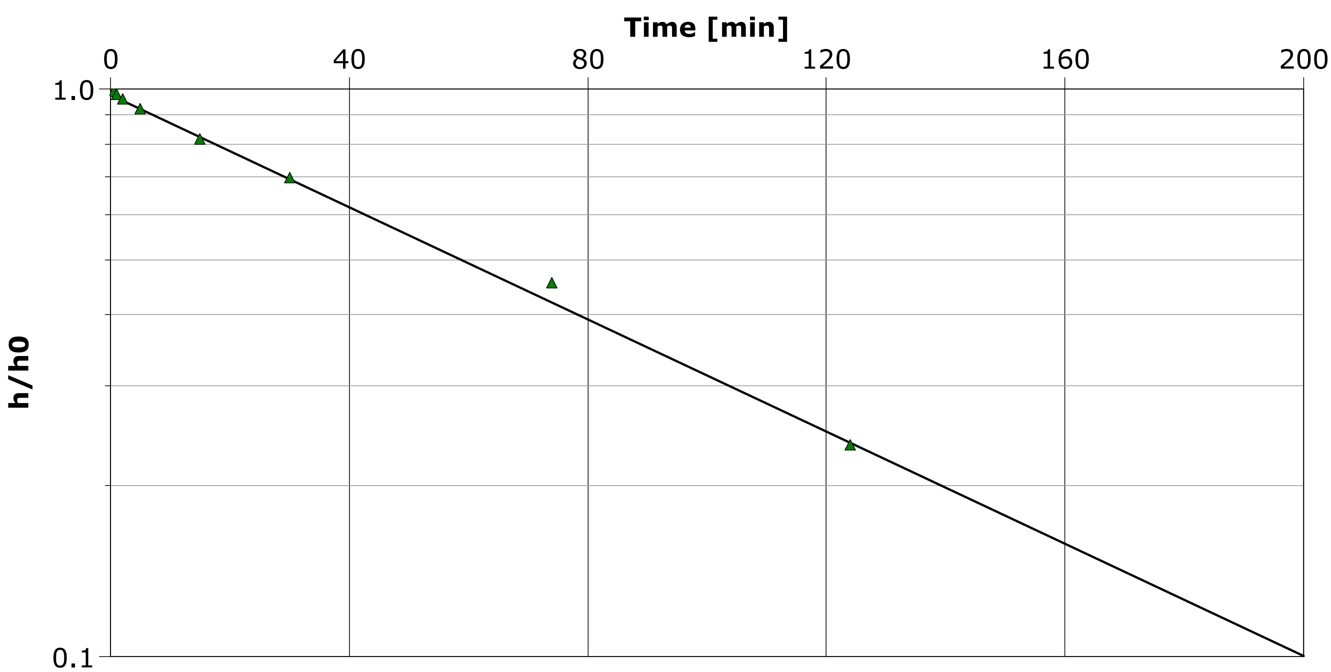
Number: G18713

Client: Mr. Jim Flynn

Location: Wellesley, ON	Slug Test: BH 2	Test Well: BH 2
Test Conducted by: D. White	Test Date: 15/11/2019	
Water level at t=0 [m]: 6.18	Static Water Level [m]: 1.18 T.O.P	Water level change at t=0 [m]: 5.00

	Time [min]	Water Level [m]	WL Change [m]
1	0	6.18	5.00
2	0.5	6.13	4.95
3	1	6.07	4.89
4	2	5.99	4.81
5	5	5.79	4.61
6	15	5.26	4.08
7	30	4.67	3.49
8	74	3.46	2.28
9	124	2.36	1.18

Aquifer Thickness: 5.69 m



Calculation using Bouwer & Rice

Observation Well	Hydraulic Conductivity [m/s]
BH 2	5.50×10^{-8}

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Slug Test - Water Level Data

Project: 1016, 1018, 1024, 1030 & 1032 Doering Street

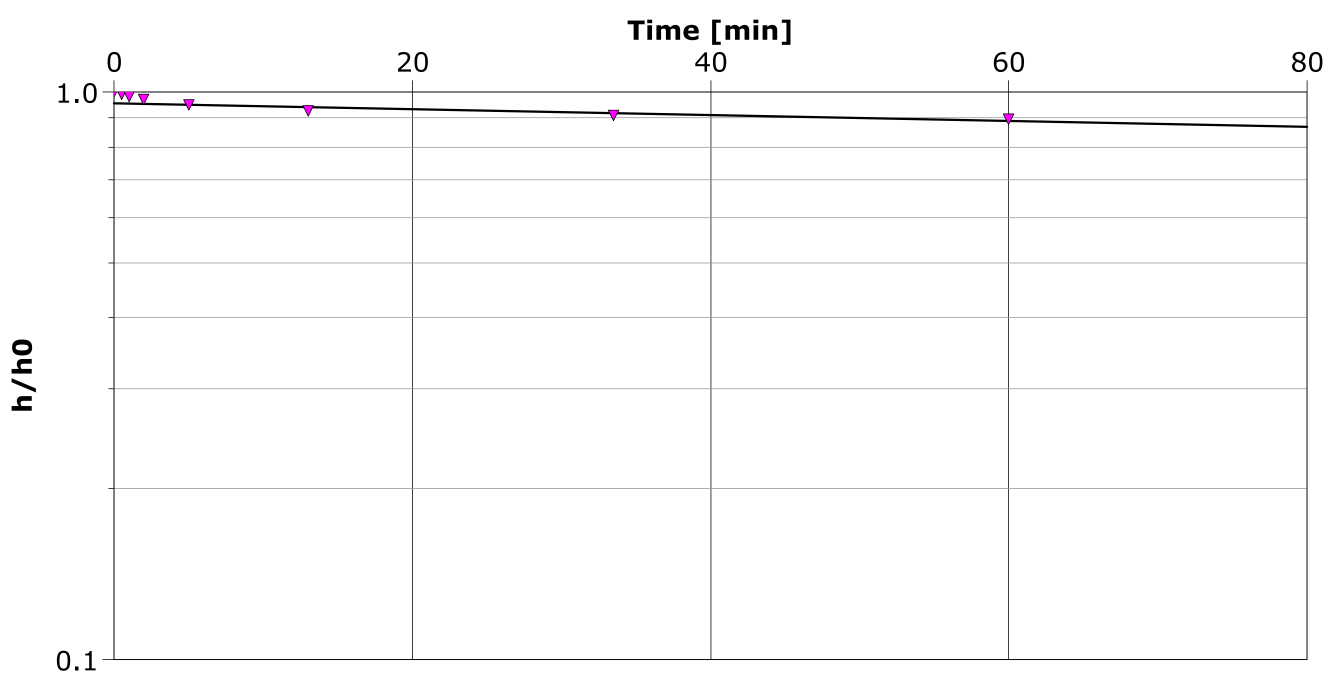
Number: G18713

Client: Mr. Jim Flynn

Location: Wellesley, ON	Slug Test: BH 6	Test Well: BH 6
Test Conducted by: D. White	Test Date: 15/11/2019	
Water level at t=0 [m]: 6.43	Static Water Level [m]: 1.98 T.O.P	Water level change at t=0 [m]: 4.45

	Time [min]	Water Level [m]	WL Change [m]
1	0	6.43	4.45
2	0.5	6.39	4.41
3	1	6.35	4.37
4	2	6.30	4.32
5	5	6.21	4.23
6	13	6.10	4.12
7	33.5	6.02	4.04
8	60	5.97	3.99

Aquifer Thickness: 4.97 m



Calculation using Bouwer & Rice

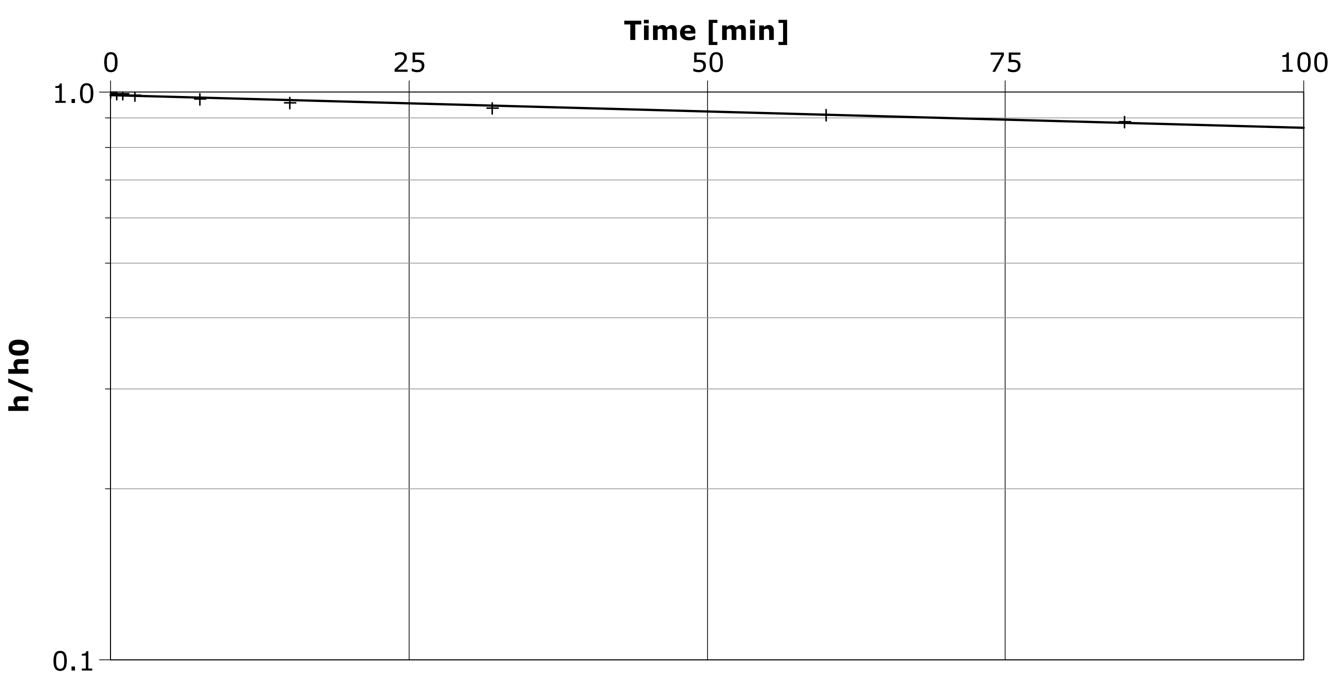
Observation Well	Hydraulic Conductivity [m/s]
BH 6	6.12×10^{-9}

CHUNG & VANDER DOELEN ENGINEERING LTD. 311 Victoria Street North Kitchener / Ontario / N2H 5E1 519-742-8979	**Slug Test - Water Level Data** Page 1 of 1 Project: 1016, 1018, 1024, 1030 & 1032 Doering Street Number: G18713 Client: Mr. Jim Flynn

Location: Wellesley, ON	Slug Test: BH 10	Test Well: BH 10
Test Conducted by: D. White	Test Date: 15/11/2019	
Water level at t=0 [m]: 6.87	Static Water Level [m]: 1.50 T.O.P	Water level change at t=0 [m]: 5.37

	Time [min]	Water Level [m]	WL Change [m]
1	0	6.87	5.37
2	0.5	6.84	5.34
3	1	6.83	5.33
4	2	6.805	5.305
5	7.5	6.73	5.23
6	15	6.65	5.15
7	32	6.53	5.03
8	60	6.39	4.89
9	85	6.27	4.77

Aquifer Thickness: 6.76 m

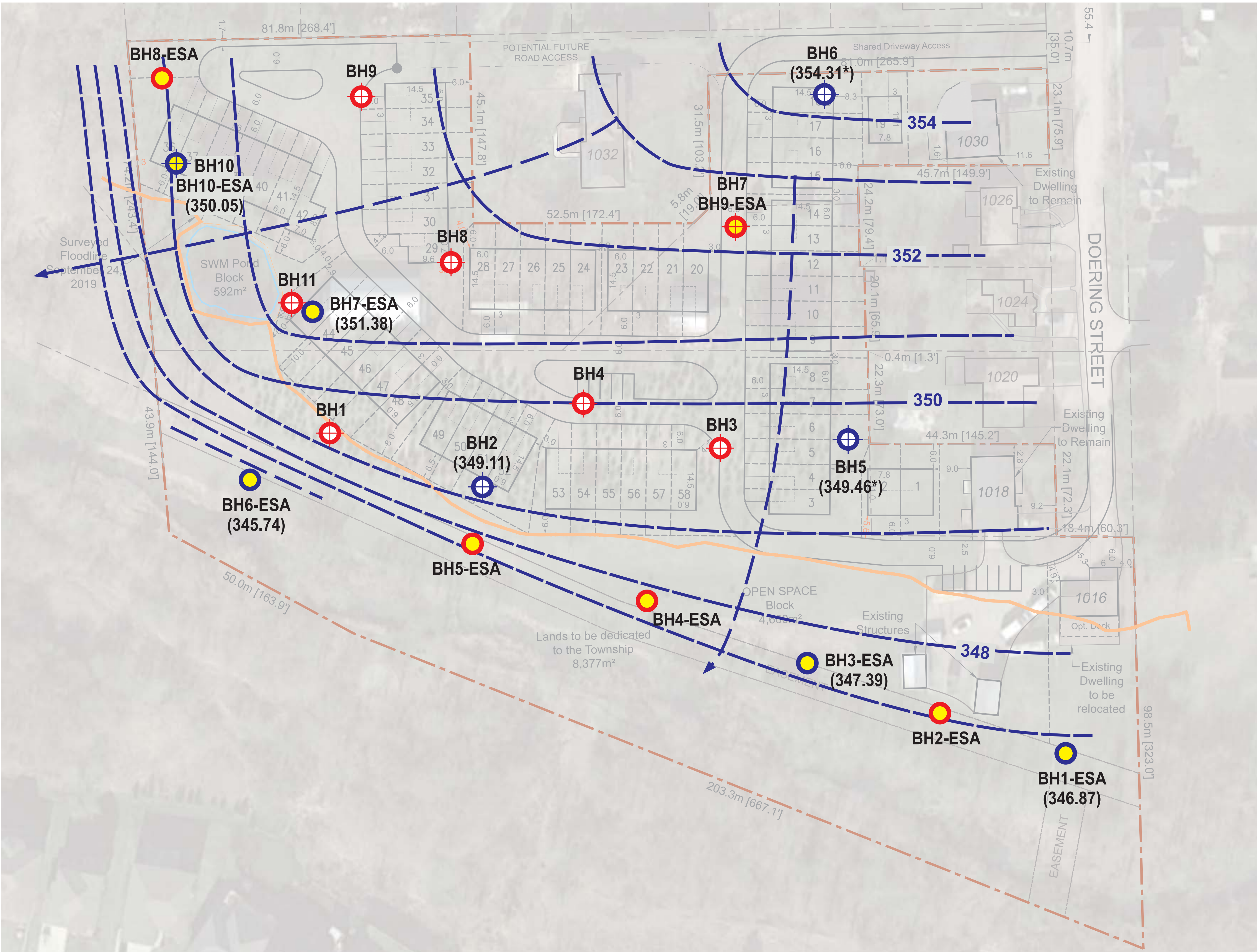


Calculation using Bouwer & Rice

Observation Well	Hydraulic Conductivity [m/s]	
BH 10	6.95×10^{-9}	

APPENDIX D

Groundwater Contour Drawing by Chung & Vander Doelen Engineering Ltd.



LEGEND

BH8-ESA — Monitoring Well / Borehole (for Phase 2 ESA)

BH8 — Monitoring Well / Borehole (for Geotech Investigation)

(352.71) — Water Table Elevation (mASL)
- November 15, 2019
- * December 4, 2019

352 — Interpreted Water Table Contour (mASL)

Interpreted Shallow Groundwater Flow Direction

0 10 20 m

Scale 1:800

Figure 3
Development Plan &
Water Table Interpretation
(November-December 2019)

Source Water Protection
Potential Contamination Study
Doering St. Development, Wellesley, ON

CHUNG & VANDER DOELEN ENGINEERING LTD.
311 VICTORIA STREET NORTH
KITCHENER / ONTARIO / N2H 2E1 / 519-742-8979

Drawn By: SA	Date: Dec 4 2019	File No.: H19108

ENCLOSURES

FILE No: G18713 — BOREHOLE No. 1

Client: **Mr. Jim Flynn**

Project: **Proposed Residential Development**

Location: **1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Solid Stem Auger**
Size: **152 mm O.D.**
Date: **Jan 23 - 19** TO **Jan 23 - 19**

Ground Elevation: **349.01 m**

SOIL LITHOLOGY

ELEV./ DEPTH (m)	DESCRIPTION
	380 mm TOPSOIL
348.63 / 0.38	stiff to very stiff, brown to grey, CLAYEY SILT, trace to some sand, trace gravel, occ. silt seams, moist
345.05 / 3.96	compact, brown, SAND, some gravel, trace silt, saturated
343.98 / 5.03	End of Borehole

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	8
2	SS	11
3	SS	17
4	SS	19
5	SS	29
6	SS	25

SHEAR STRENGTH (kPa): FIELD VANE: Peak ⊗ Rem. × ; LAB TEST: Unc. ■ P.P. □ ; 50 100 150 200

PENETRATION RESISTANCE: STANDARD ● DYN. CONE ○ ; 20 40 60 80

WATER CONTENT (%): W_P W W_L ; 10 20 30

WELL DATA

DEPTH (m): 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 5.0 5.5 6.0 6.5 7.0 7.5 8.0

REMARKS

- water level at 2.9 m bgs upon completion of drilling
- borehole cave-in at 4.9 m bgs upon completion of drilling

PROJECT MANAGER: **EYC**

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FILE No: G18713 — BOREHOLE No. 2

Client: **Mr. Jim Flynn**

Project: **Proposed Residential Development**

Location: **1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Solid Stem Auger**
Size: **152 mm O.D.**
Date: **Jan 23 - 19** TO **Jan 23 - 19**

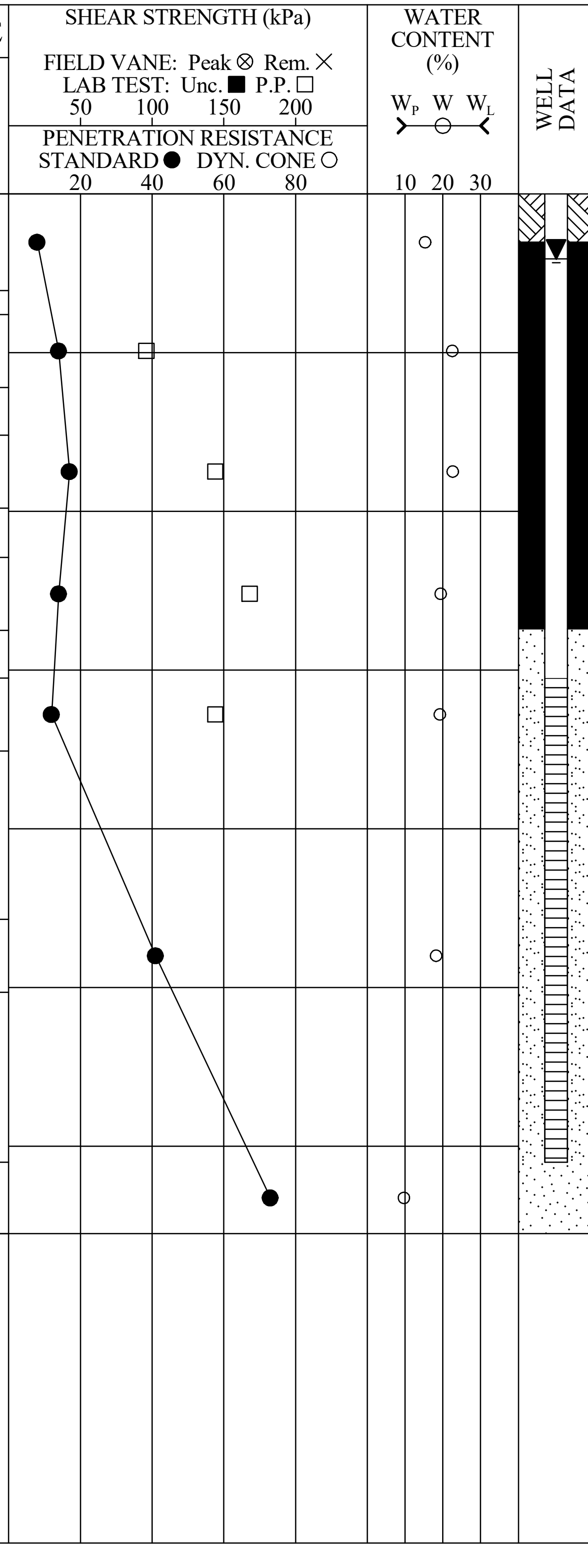
SOIL LITHOLOGY

Ground Elevation: **349.52 m**

ELEV./DEPTH (m)	DESCRIPTION
	300 mm TOPSOIL
349.22 / 0.30	loose, brown FILL, sandy silt, trace gravel wet
348.76 / 0.76	stiff to very stiff brown to grey CLAYEY SILT trace to some sand, trace gravel occ. silt seams moist
345.56 / 3.96	dense grey SILT trace sand, trace clay occ. clay seams saturated
344.03 / 5.49	very dense grey SAND AND SILT TILL trace to some gravel, trace clay moist
342.97 / 6.55	End of Borehole

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	8
2	SS	14
3	SS	17
4	SS	14
5	SS	12
6	SS	41
7	SS	73



REMARKS

DEPTH (m)	REMARKS
	steel casing set in concrete
2.0	bentonite seal/50 mm I.D. PVC riser
4.5	3.0 m long, 50 mm I.D. PVC screen with sandpack
6.5	water level in monitoring well at depth of 0.32 m bgs on January 30, 2019
7.0	water level in monitoring well at depth of 0.36 m bgs on March 8, 2019
7.5	water level in monitoring well at depth of 0.74 m bgs on September 4, 2019
8.0	water level in monitoring well at depth of 0.41 m bgs on November 15, 2019

PROJECT MANAGER: **EYC**

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FILE No: G18713 — BOREHOLE No. 3

Client:	**Mr. Jim Flynn**
Project:	**Proposed Residential Development**
Location:	**1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

Machine:	**Diedrich D50T**
Method:	**Solid Stem Auger**
Size:	**152 mm O.D.**
Date:	**Jan 23 - 19** TO **Jan 23 - 19**

SOIL LITHOLOGY

Ground Elevation: **350.71 m**

ELEV./DEPTH (m)	DESCRIPTION
	190 mm TOPSOIL
350.52 / 0.19	very loose to loose dark brown FILL, clayey silt some sand, trace gravel, some organics; moist to wet; ----- 0.5 m buried topsoil
348.42 / 2.29	firm to very stiff brown to grey CLAYEY SILT trace to some sand, trace gravel; occ. silt seams; moist
344.16 / 6.55	End of Borehole

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	8
2	SS	4
3	SS	3
4	SS	10
5	SS	10
6	SS	21
7	SS	6
8	SS	15
9	SS	24

SHEAR STRENGTH (kPa): FIELD VANE: Peak ⊗ Rem. ✕; LAB TEST: Unc. ■ P.P. □ (50, 100, 150, 200)

PENETRATION RESISTANCE: STANDARD ● DYN. CONE ○ (20, 40, 60, 80)

WATER CONTENT (%): W_P W W_L (10, 20, 30)

WELL DATA

REMARKS

- water level at 2.7 m bgs upon completion of drilling
- borehole open upon completion of drilling

PROJECT MANAGER: **EYC**

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Kitchener, Ontario N2H 5E1
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FILE No: G18713 BOREHOLE No. 4

Client: **Mr. Jim Flynn**

Project: **Proposed Residential Development**

Location: **1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Solid Stem Auger**
Size: **152 mm O.D.**
Date: **Jan 23 - 19** TO **Jan 23 - 19**

Ground Elevation: **350.90 m**

ELEV./DEPTH (m)	DESCRIPTION	SAMPLE ID	TYPE	N-VALUE	REMARKS
	300 mm TOPSOIL	1	SS	3	
350.60 / 0.30	very loose to compact dark brown to brown FILL, sandy silt trace clay moist	2	SS	15	
349.38 / 1.52	very stiff, brown CLAYEY SILT trace sand wet	3	SS	15	
348.77 / 2.13	compact grey SILT trace to some sand, trace clay saturated	4	SS	15	water level at 2.7 m bgs upon completion of drilling
348.00 / 2.90	stiff grey CLAYEY SILT trace to some sand, trace gravel occ. silt seams moist to wet	5	SS	15	
		6	SS	13	borehole cave-in at 5.3 m bgs upon completion of drilling
		7	SS	14	
344.35 / 6.55	End of Borehole				

SHEAR STRENGTH (kPa): FIELD VANE: Peak ⊗ Rem. × ; LAB TEST: Unc. ■ P.P. □ ; 50 100 150 200

PENETRATION RESISTANCE: STANDARD ● DYN. CONE ○ ; 20 40 60 80

WATER CONTENT (%): W_P W W_L ; 10 20 30

WELL DATA

PROJECT MANAGER: **EYC**

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Kitchener, Ontario N2H 5E1
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FILE No: G18713

BOREHOLE No. 5

Client: **Mr. Jim Flynn**

Project: **Proposed Residential Development**

Location: **1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

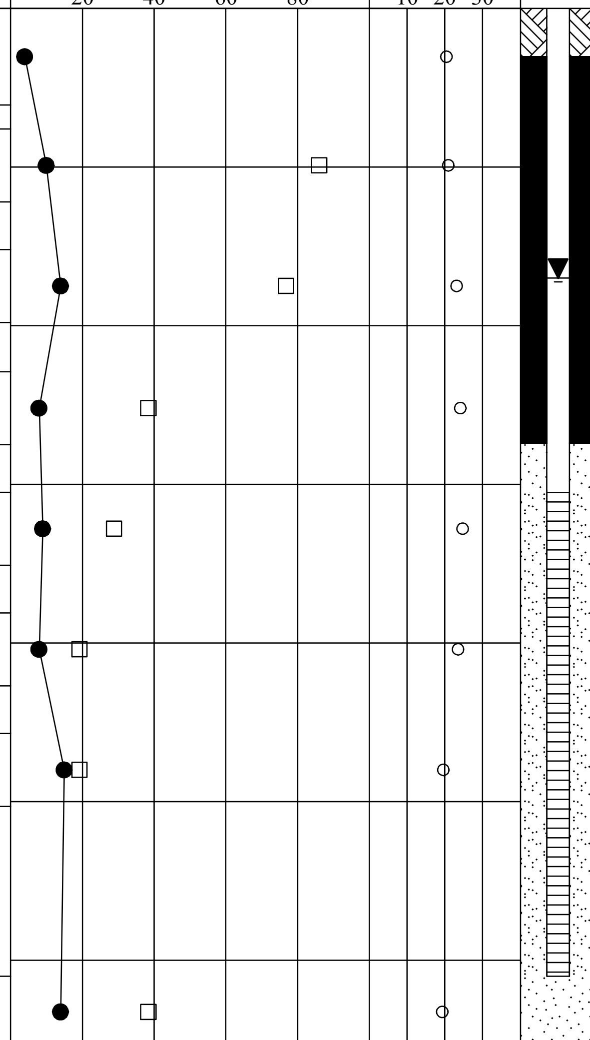
Machine: **Diedrich D50T**
Method: **Solid Stem Auger**
Size: **152 mm O.D.**
Date: **Jan 23 - 19** TO **Jan 23 - 19**

SOIL LITHOLOGY

Ground Elevation: **351.16 m**

ELEV./ DEPTH (m)	DESCRIPTION	SAMPLE ID	TYPE	N-VALUE
	250 mm TOPSOIL			
350.91 / 0.25	loose to compact brown FILL, sandy silt trace gravel moist	1	SS	4
350.16 / 1.00	stiff to very stiff brown to grey CLAYEY SILT trace to some sand, trace gravel; occ. silt seams; moist to wet	2	SS	10
		3	SS	14
		4	SS	8
		5	SS	9
		6	SS	8
	----- freq. silt seams	7	SS	15
		8	SS	14
344.61 / 6.55	End of Borehole			

SHEAR STRENGTH (kPa): FIELD VANE: Peak ⊗ Rem. × ; LAB TEST: Unc. ■ P.P. □ ; 50 100 150 200
PENETRATION RESISTANCE: STANDARD ● DYN. CONE ○ ; 20 40 60 80
WATER CONTENT (%): W_P W W_L ; 10 20 30
WELL DATA



REMARKS

- steel casing set in concrete
- bentonite seal/50 mm I.D. PVC riser (2.0 m)
- 3.0 m long, 50 mm I.D. PVC screen with sandpack (4.5 m)
- water level in monitoring well at depth of 0.99 m bgs on January 30, 2019
- water level in monitoring well at depth of 0.88 m bgs on March 8, 2019
- water level in monitoring well at depth of 3.00 m bgs on September 4, 2019
- water level in monitoring well at depth of 1.70 m bgs on December 4, 2019

PROJECT MANAGER: **EYC**

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Kitchener, Ontario N2H 5E1
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FILE No: G18713

BOREHOLE No. 6

Client:	**Mr. Jim Flynn**
Project:	**Proposed Residential Development**
Location:	**1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

Machine:	**Diedrich D50T**
Method:	**Hollow Stem Auger**
Size:	**108 mm I.D.**
Date:	**Aug 27 - 19** TO **Aug 27 - 19**

SOIL LITHOLOGY — Ground Elevation: **355.89 m**

ELEV./DEPTH (m)	DESCRIPTION	SAMPLE ID	TYPE	N-VALUE	Pocket Pen. (P.P.)	Water Content W (%)
355.79 / 0.10	100 mm TOPSOIL					
355.53 / 0.36	compact, brown FILL, silty sand, some gravel, trace to some topsoil/organics, moist	1	SS	11		
	stiff, brown to grey CLAYEY SILT, trace to some sand, trace gravel, occ. silt seams, moist	2	SS	11		
		3	SS	12		
		4	SS	13		
		5	SS	14		
		6	SS	12		
		7	SS	9		
349.34 / 6.55	End of Borehole					

Shear strength (kPa): FIELD VANE: Peak ⊗ Rem. ×; LAB TEST: Unc. ■ P.P. □; scale 50, 100, 150, 200
PENETRATION RESISTANCE: STANDARD ● DYN. CONE ○; scale 20, 40, 60, 80
WATER CONTENT (%): W_P W W_L; scale 10, 20, 30

WELL DATA / REMARKS:

DEPTH (m)	REMARKS
0.0	steel casing set in concrete
2.0	bentonite seal/50 mm I.D. PVC riser
4.5	3.0 m long, 50 mm I.D. PVC screen with sandpack
7.5	water level in monitoring well at depth of 5.15 m bgs on September 4, 2019
8.0	water level in monitoring well at depth of 1.58 m bgs on December 4, 2019

PROJECT MANAGER: **EYC**

CHUNG & VANDER DOELEN ENGINEERING LTD.
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FILE No: G18713

BOREHOLE No. 7

Client: **Mr. Jim Flynn**

Project: **Proposed Residential Development**

Location: **1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Hollow Stem Auger**
Size: **108 mm I.D.**
Date: **Aug 27 - 19** TO **Aug 27 - 19**

Ground Elevation: **354.07 m**

ELEV./ DEPTH (m)	DESCRIPTION	SAMPLE ID	TYPE	N-VALUE	REMARKS
	300 mm TOPSOIL	1	SS	11	
353.77 / 0.30	compact, brown FILL, silty sand, trace to some gravel, trace topsoil/organics, contains wood fragments, moist	2	SS	14	
352.70 / 1.37	compact to very dense, brown FILL, sand and gravel, trace to some silt, trace topsoil/organics, occ. silty layers, occ. cobbles, damp to saturated	3	SS		inflated "N"-value due to cobbles
		4	SS	24	
		5	SS	20	
350.41 / 3.66	stiff, brown to grey CLAYEY SILT, trace to some sand, trace gravel, occ. silt seams/layers, moist	6	SS	9	
		7	SS	11	
		8	SS	9	water level at depth of 6.1 m upon withdrawal of drilling augers
		9	SS	14	
346.02 / 8.05	End of Borehole				borehole open upon withdrawal of drilling augers

SHEAR STRENGTH (kPa): FIELD VANE: Peak ⊗ Rem. ×; LAB TEST: Unc. ■ P.P. □ (50, 100, 150, 200)
PENETRATION RESISTANCE: STANDARD ● DYN. CONE ○ (20, 40, 60, 80)
WATER CONTENT (%): W_P W W_L (10, 20, 30)
WELL DATA
DEPTH (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0

PROJECT MANAGER: **EYC**

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FILE No: G18713 — BOREHOLE No. 8

Client:	**Mr. Jim Flynn**
Project:	**Proposed Residential Development**
Location:	**1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

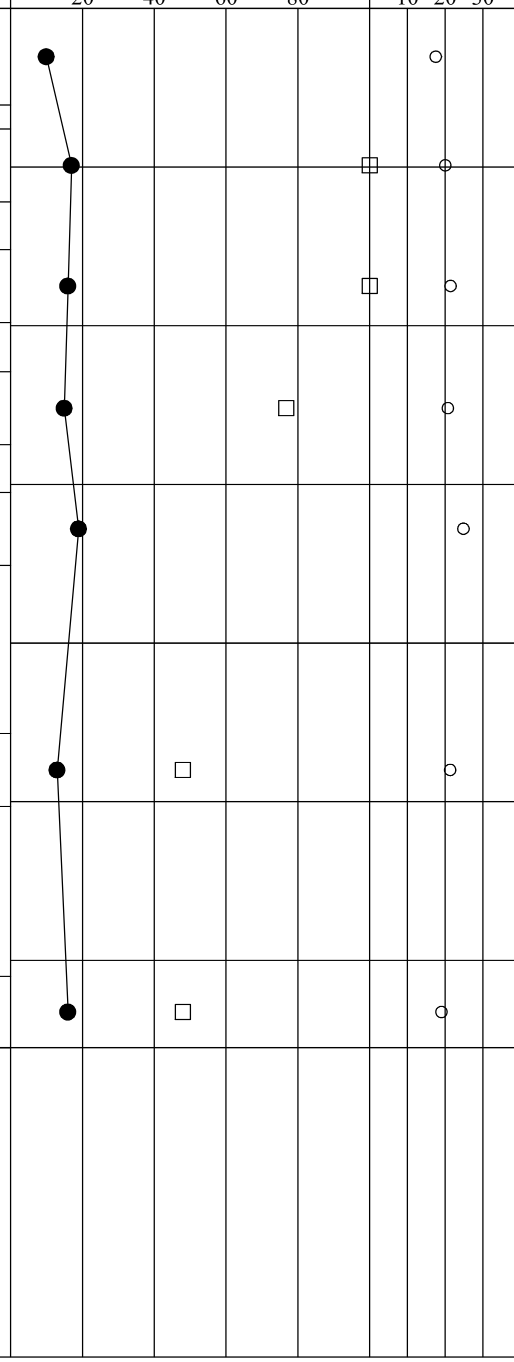
Machine:	**Diedrich D50T**
Method:	**Hollow Stem Auger**
Size:	**83 mm I.D.**
Date:	**Aug 26 - 19** TO **Aug 26 - 19**

Ground Elevation: **350.59 m**

ELEV./DEPTH (m)	SOIL LITHOLOGY – DESCRIPTION
	250 mm TOPSOIL
350.34 / 0.25	compact, dark brown FILL, sandy silt, trace gravel, trace clay, trace to some topsoil/organics, occ. clayey seams, moist
349.90 / 0.69	stiff to very stiff, brown to grey CLAYEY SILT, trace to some sand, trace gravel, occ. silt seams/layers, moist
344.04 / 6.55	End of Borehole

SAMPLE ID	TYPE	N-VALUE
1	SS	10
2	SS	17
3	SS	16
4	SS	15
5	SS	19
6	SS	13
7	SS	16

SHEAR STRENGTH (kPa): FIELD VANE: Peak ⊗ Rem. ×; LAB TEST: Unc. ■ P.P. □ (50, 100, 150, 200)
PENETRATION RESISTANCE: STANDARD ● DYN. CONE ○ (20, 40, 60, 80)
WATER CONTENT (%): W_P W W_L (10, 20, 30)



WELL DATA

REMARKS: borehole dry and open upon withdrawal of drilling augers (at 6.5 m)

PROJECT MANAGER: **EYC**

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FILE No: G18713 — BOREHOLE No. 9

Client: **Mr. Jim Flynn**

Project: **Proposed Residential Development**

Location: **1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Hollow Stem Auger**
Size: **83 mm I.D.**
Date: **Aug 26 - 19** TO **Aug 26 - 19**

SOIL LITHOLOGY

Ground Elevation: **352.47 m**

ELEV./DEPTH (m)	DESCRIPTION
	380 mm TOPSOIL
352.09 / 0.38	firm to stiff brown FILL, clayey silt some sand trace topsoil/organics moist
351.10 / 1.37	stiff to very stiff brown to grey CLAYEY SILT trace to some sand, trace gravel; occ. silt seams; moist
345.92 / 6.55	End of Borehole

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	10
2	SS	7
3	SS	8
4	SS	19
5	SS	18
6	SS	14
7	SS	11

SHEAR STRENGTH (kPa): FIELD VANE: Peak ⊗ Rem. ×; LAB TEST: Unc. ■ P.P. □ (50, 100, 150, 200)

PENETRATION RESISTANCE: STANDARD ● DYN. CONE ○ (20, 40, 60, 80)

WATER CONTENT (%): W_P W W_L (10, 20, 30)

WELL DATA

DEPTH (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0

REMARKS: borehole dry and open upon withdrawal of drilling augers

PROJECT MANAGER: **EYC**

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Enclosure No.: 10
Sheet 1 of 1

FILE No: G18713 BOREHOLE No. 10

Client:	**Mr. Jim Flynn**
Project:	**Proposed Residential Development**
Location:	**1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

Machine:	**Diedrich D50T**
Method:	**Hollow Stem Auger**
Size:	**108 mm I.D.**
Date:	**Aug 26 - 19** TO **Aug 26 - 19**

SOIL LITHOLOGY

Ground Elevation: **350.61 m**

ELEV./DEPTH (m)	DESCRIPTION
	230 mm TOPSOIL
350.38 / 0.23	stiff to very stiff, brown to grey, CLAYEY SILT, trace to some sand, trace gravel, occ. silt seams, moist
345.12 / 5.49	dense, brown, SILT, trace sand, trace clay, occ. clayey seams, saturated
343.75 / 6.86	very dense, grey, SAND AND SILT TILL, trace to some gravel, trace clay, occ. cobbles, moist
343.31 / 7.30	End of Borehole

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	8
2	SS	12
3	SS	18
4	SS	17
5	SS	13
6	SS	13
7	SS	38
8	SS	57

SHEAR STRENGTH (kPa): FIELD VANE: Peak ⊗ Rem. ×; LAB TEST: Unc. ■ P.P. □ (50, 100, 150, 200)

PENETRATION RESISTANCE: STANDARD ● DYN. CONE ○ (20, 40, 60, 80)

WATER CONTENT (%): W_P W W_L (10, 20, 30)

WELL DATA

REMARKS

DEPTH (m)	REMARKS
0.0	steel casing set in concrete
2.0	bentonite seal/50 mm I.D. PVC riser
6.0	3.0 m long, 50 mm I.D. PVC screen with sandpack
7.5	water level in monitoring well at depth of 0.82 m bgs on September 4, 2019
8.0	water level in monitoring well at depth of 0.56 m bgs on November 15, 2019

PROJECT MANAGER: **EYC**

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Enclosure No.: 11
Sheet 1 of 1

FILE No: G18713 — BOREHOLE No. 11

Client:	**Mr. Jim Flynn**
Project:	**Proposed Residential Development**
Location:	**1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario**

EQUIPMENT DATA

Machine:	**Diedrich D50T**
Method:	**Hollow Stem Auger**
Size:	**83 mm I.D.**
Date:	**Aug 26 - 19** TO **Aug 26 - 19**

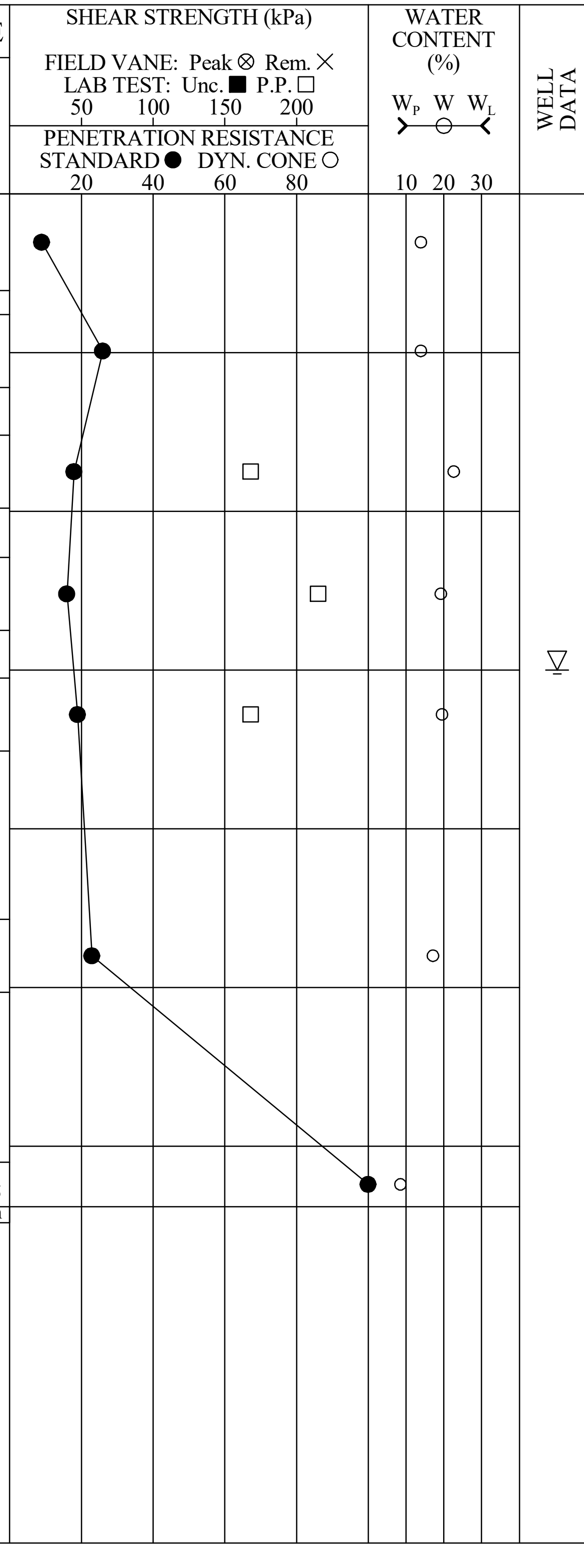
SOIL LITHOLOGY

Ground Elevation: **349.19 m**

ELEV./DEPTH (m)	DESCRIPTION
	180 mm TOPSOIL
349.01 / 0.18	loose, dark brown FILL, sandy silt, trace to some gravel, trace topsoil/organics, contains asphalt fragments, moist
348.50 / 0.69	compact, brown FILL, sand, trace to some gravel, trace silt, saturated
347.82 / 1.37	very stiff, brown to grey CLAYEY SILT, trace to some sand, trace gravel; occ. silt seams/layers; moist
343.70 / 5.49	very dense, grey SAND AND SILT TILL, some gravel, trace clay, moist
342.81 / 6.38	End of Borehole

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	9
2	SS	26
3	SS	18
4	SS	16
5	SS	19
6	SS	23
7	SS	50/125 mm



REMARKS

- water level at depth of 3.0 m upon withdrawal of drilling augers
- borehole cave-in at depth of 5.64 m upon withdrawal of drilling augers

PROJECT MANAGER: **EYC**

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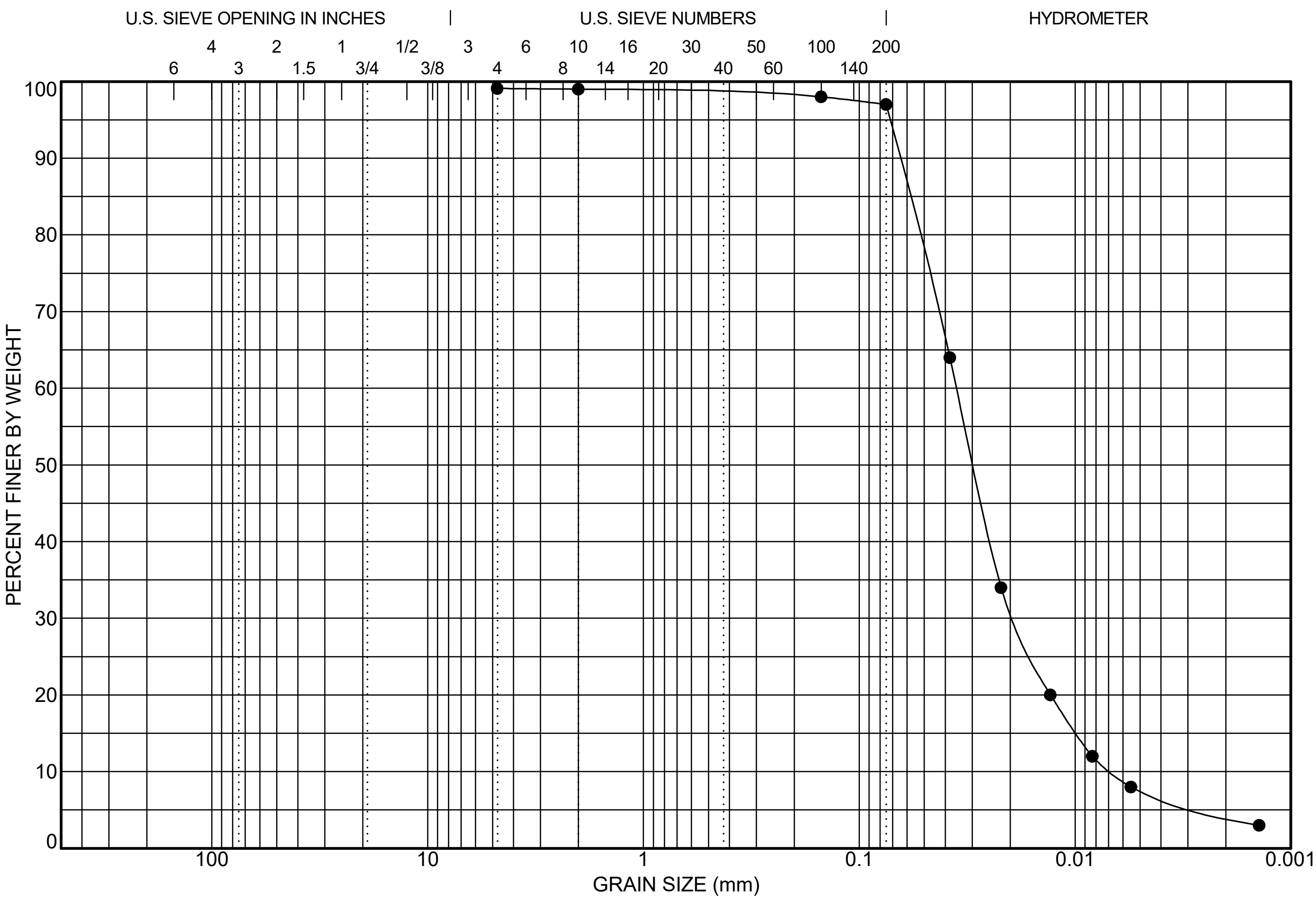

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

LL	PL	PI	Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
			1.50	5.23	4.75	0.035	0.019	0.007	0.0	2.1	97.0	

Date: Mar. 12 - 2019
Client: Mr. Jim Flynn
Contractor:
Source:
Sampled From: BH 2, SA 6, 4.57-5.03 m depth
Sample No.: 2-6
Date Sampled: Jan. 23 - 2019
Sampled By: NZ
Lab No.: 54
Date Tested: Jan. 30 - 2019
Type of Material: Silt, trace sand, trace clay

Sieve Size (mm)	Percent Passing	No Specifications

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e-mail: info@cvdengineering.com

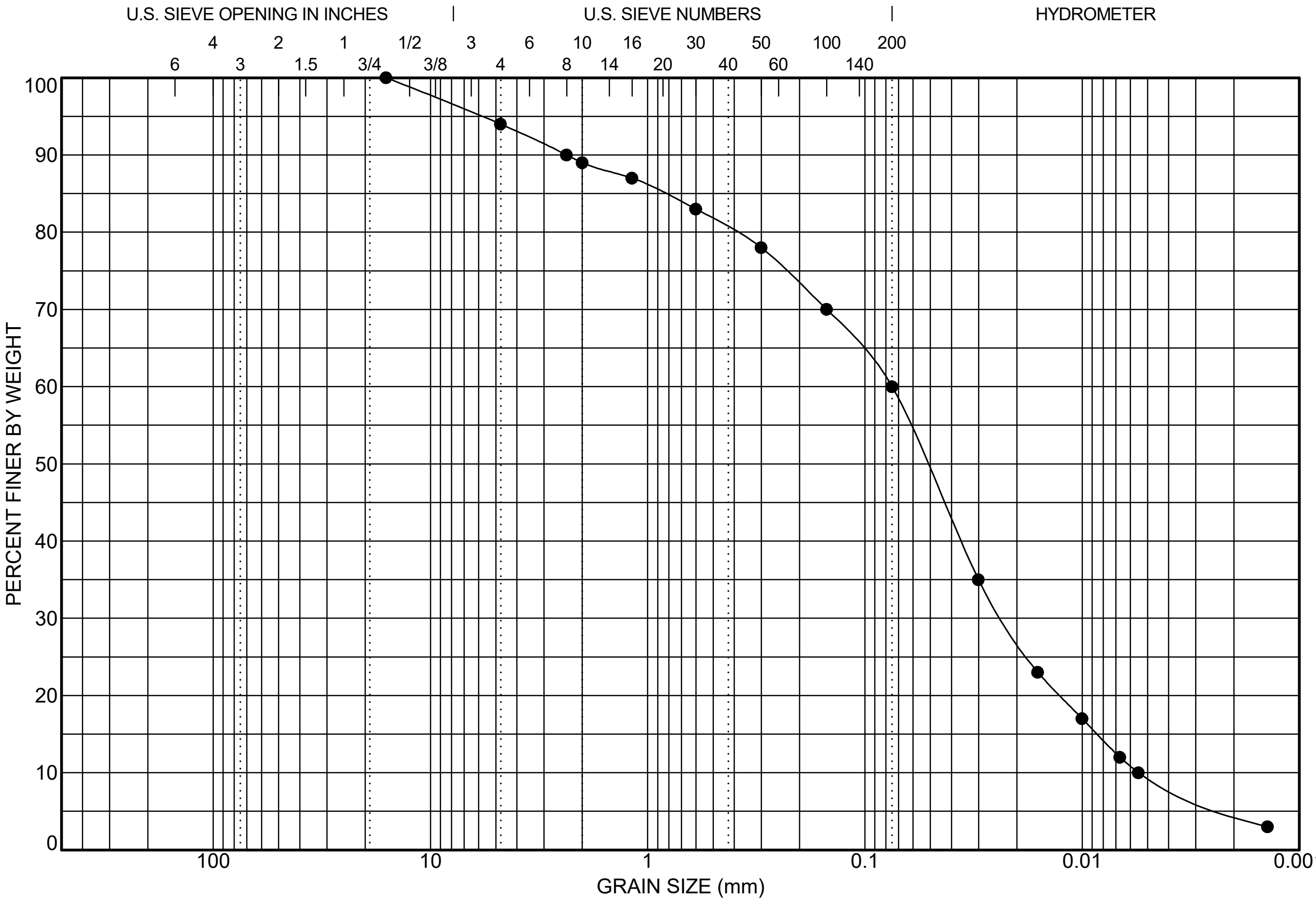
GRAIN SIZE DISTRIBUTION

Project: Proposed Residential Development

Location: 1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario

File No.: G18713

Enclosure No.: 12



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

LL	PL	PI	Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
			1.29	13.64	16	0.075	0.023	0.006	6.0	34.0	60.0	

Date: Mar. 12 - 2019
Client: Mr. Jim Flynn
Contractor:
Source:
Sampled From: BH 2, SA 7, 6.10-6.55 m depth
Sample No.: 2-7
Date Sampled: Jan. 23 - 2019
Sampled By: NZ
Lab No.: 55
Date Tested: Jan. 30 - 2019
Type of Material: Sand and Silt Till, trace gravel, trace clay

Sieve Size (mm)	Percent Passing	No Specifications

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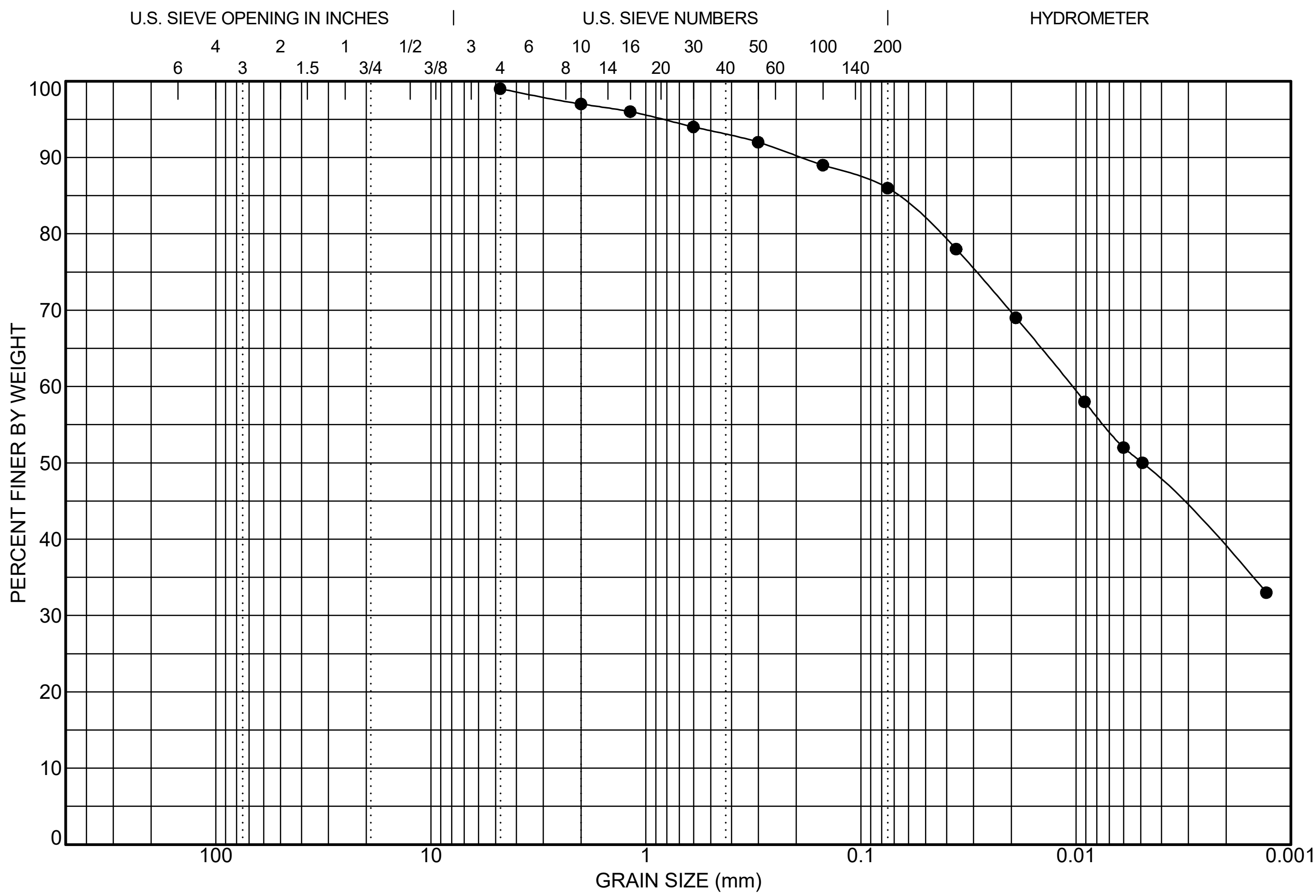
GRAIN SIZE DISTRIBUTION

Project: Proposed Residential Development

Location: 1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario

File No.: G18713

Enclosure No.: 13



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

LL	PL	PI	Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
					4.75	0.01			0.0	13.0	86.0	

Date: Mar. 12 - 2019
Client: Mr. Jim Flynn
Contractor:
Source:
Sampled From: BH 3, SA 8, 5.33-5.79 m depth
Sample No.: 3-8
Date Sampled: Jan. 23 - 2019
Sampled By: NZ
Lab No.: 56
Date Tested: Jan. 30 - 2019
Type of Material: Clayey Silt, some sand, trace gravel

Sieve Size (mm)	Percent Passing	No Specifications

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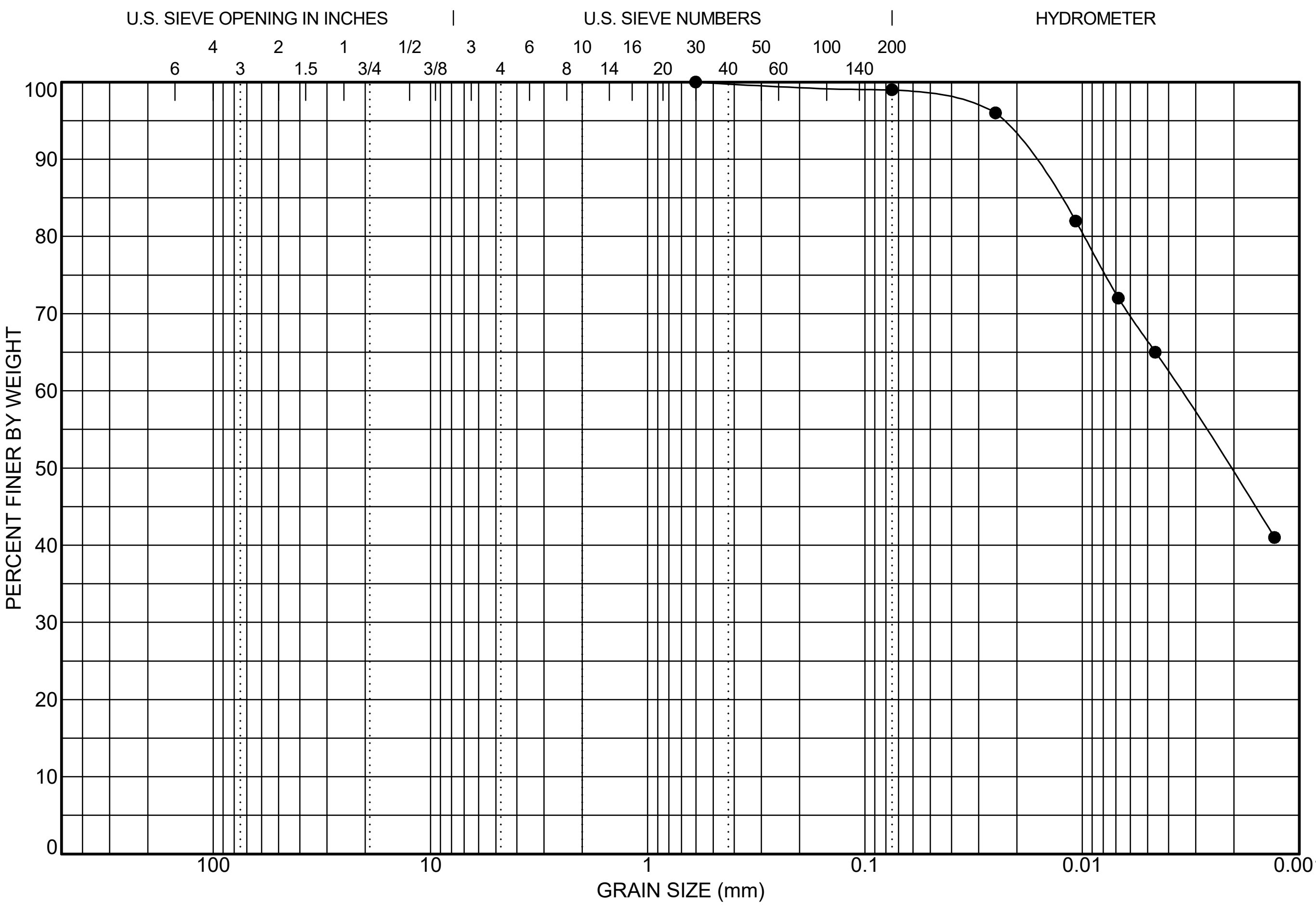
GRAIN SIZE DISTRIBUTION

Project: Proposed Residential Development

Location: 1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario

File No.: G18713

Enclosure No.: 14



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

LL	PL	PI	Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
					0.6	0.004			0.0	1.0	99.0	

Date: Mar. 12 - 2019
Client: Mr. Jim Flynn
Contractor:
Source:
Sampled From: BH 5, SA 6, 3.81-4.27 m depth
Sample No.: 5-6
Date Sampled: Jan. 23 - 2019
Sampled By: NZ
Lab No.: 57
Date Tested: Jan. 30 - 2019
Type of Material: Clayey Silt, trace sand

Sieve Size (mm)	Percent Passing	No Specifications

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e-mail: info@cvdengineering.com

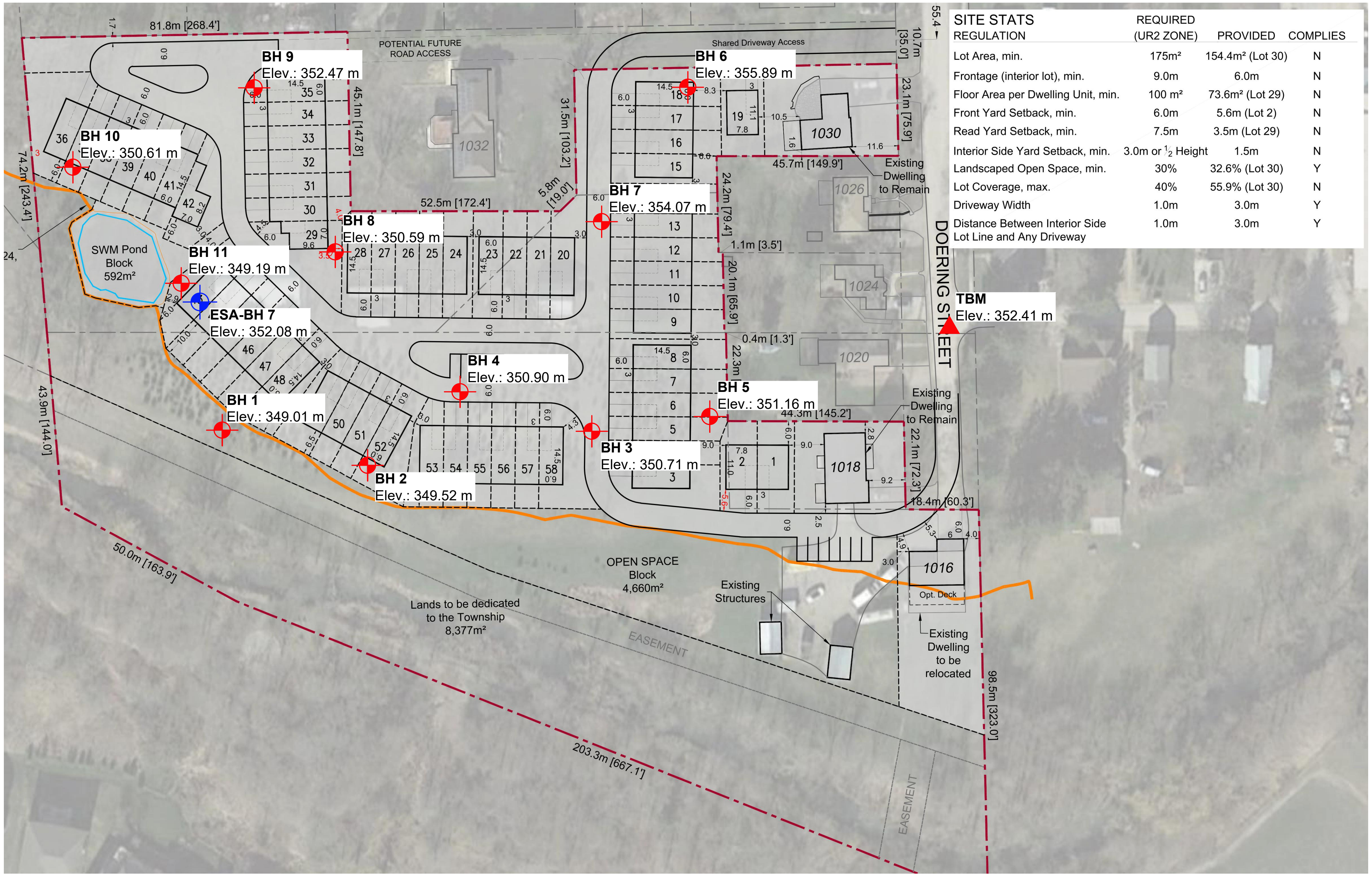
GRAIN SIZE DISTRIBUTION

Project: Proposed Residential Development

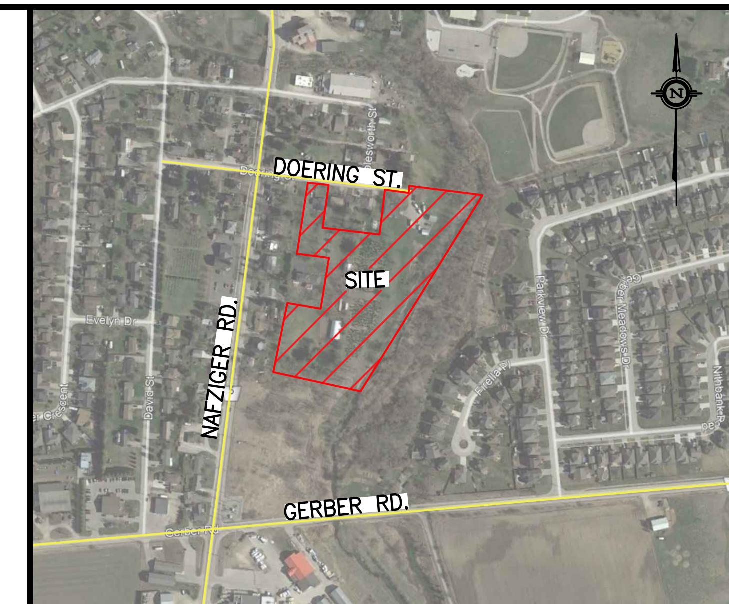
Location: 1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley, Ontario

File No.: G18713

Enclosure No.: 15



SITE STATS REGULATION	REQUIRED (UR2 ZONE)	PROVIDED	COMPLIES
Lot Area, min.	175m²	154.4m² (Lot 30)	N
Frontage (interior lot), min.	9.0m	6.0m	N
Floor Area per Dwelling Unit, min.	100 m²	73.6m² (Lot 29)	N
Front Yard Setback, min.	6.0m	5.6m (Lot 2)	N
Read Yard Setback, min.	7.5m	3.5m (Lot 29)	N
Interior Side Yard Setback, min.	3.0m or $^1/_2$ Height	1.5m	N
Landscaped Open Space, min.	30%	32.6% (Lot 30)	Y
Lot Coverage, max.	40%	55.9% (Lot 30)	N
Driveway Width	1.0m	3.0m	Y
Distance Between Interior Side Lot Line and Any Driveway	1.0m	3.0m	Y



KEY PLAN **SOURCE:** Google Earth

LEGEND

TBM: Top of manhole in Doering Street in front of Residential No. 1024 Elev.: 352.41 m (Geodetic)

Borehole Location

Borehole Location - Phase II ESA

Dwg. Ref.: *GSP; "Development Concept"; 1016, 1018, 1024, 1030 & 1032 Doering Street, Wellesley; Proj. No.: 18275; September 25, 2019*

Elev. Ref.: *Van Harten Surveying Inc.; "Topographic Survey of Part of Lot61 Municipal Compiled Plan 1148"; Geographic Township of Wellesley*

BOREHOLE LOCATION PLAN

Proposed Residential Development

1016, 1018, 1024, 1030 & 1032
Doering Street
Wellesley, Ontario

CHUNG & VANDER DOELEN ENGINEERING LTD.

311 VICTORIA STREET NORTH
KITCHENER / ONTARIO / N2H 5E1 / 519-742-8979

Drawn By: NZ/JV	Date: October, 2019	File No.: G18713
Checked By: EYC	Scale: N.T.S	Drawing No.: 1