CHUNG & VANDER DOELEN ENGINEERING LTD.

Geotechnical Engineering, Construction Inspecting
& Testing, Environmental Services

311 Victoria Street North
Kitchener, Ontario, N2H 5E1
Telephone: 519-742-8979
Facsimile: 519-742-7739
E-Mail: info@cvdengineering.com

GEOTECHNICAL INVESTIGATION
PRELIMINARY ASSESSMENT
PROPOSED 5-STOREY APARTMENT BUILDING
PRESTON SPRING CONDOMINIUMS
102 FOUNTAIN STREET SOUTH
CAMBRIDGE, ONTARIO

Submitted to:

Haastown Group of Companies
170 West Beaver Creek Road, Unit 13
Richmond Hill, Ontario
L4B 1L6

Attention: Mr. Rob Gazzola

Submitted by:

CHUNG & VANDER DOELEN ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario
N2H 5E1

File No.: 13-09-K01
January 31, 2014

CHUNG & VANDER DOELEN ENGINEERING LTD.

Geotechnical Engineering, Construction Inspecting
& Testing, Environmental Services

311 Victoria Street North
Kitchener, Ontario, N2H 5E1
Telephone: 519-742-8979
Facsimile: 519-742-7739
E-Mail: info@cvdengineering.com

January 31, 2013
File No.: 13-09-K01

Haastown Group of Companies
170 West Beaver Creek Road, Unit 13
Richmond Hill, Ontario
L4B 1L6

Attention: Mr. Rob Gazzola

Re: GEOTECHNICAL INVESTIGATION
PRELIMINARY ASSESSMENT
PROPOSED 5-STOREY APARTMENT BUILDING
PRESTON SPRINGS CONDOMINIUMS
102 FOUNTAIN STREET SOUTH, CAMBRIDGE, ONTARIO

We take pleasure in enclosing three (3) copies of our Geotechnical Investigation Report carried out at the above-referenced Site.

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 CONTENT

LIST OF ENCLOSURES

1.0 INTRODUCTION

CHUNG & VANDER DOELEN ENGINEERING LTD. (CVD) has been retained by the Haastown Group of Companies to carry out a geotechnical investigation for the proposed Preston springs Condominiums project. The proposed development will consist of a 5-storey apartment building and a 2- level parking structure to be constructed on three (3) adjoining properties, consisting of 102 Fountain Street South, 134 and 144 Fountain Street North in Cambridge, Ontario. The 5-storey historic structure existed on 102 Fountain Street South will be renovated.

The site is located on the west side of Fountain Street North and South (Fountain Street), at the 'T' intersection with King Street West. It is understood that the site is located within the regulated zone under the jurisdiction of Grand River Conservation Authority (GRCA). GRCA guidelines were reviewed to obtain their requirement/procedure for carrying out this assessment. The proposed 5 -storey apartment building will have two (2) underground parking levels. Based on preliminary design concepts, the lower basement floor level will be at elevation ±282.75 m.

Details of the proposed structure, finished grades and location on the property have not been finalized at the time of report, and therefore this report is intended to provide an assessment of the general foundation design and construction considerations. CVD will be pleased to provide review of the proposed development plan when it is available, and complete further geotechnical investigation as required.

The purpose of the investigation was to determine the subsurface conditions at the site to assess the stability of the existing slope and, furthermore, based on the findings, make preliminary geotechnical recommendations for the design and construction of the foundations, excavation, shoring, floor slabs and pavement areas.

2.0 FIELD WORK

In order to investigate the subsurface conditions at the site, boreholes at seven (7) locations were put down from October 15 to 17, 2013 at the site to depths of 1.98 to 12.20 m below existing grades. The borehole sampling was completed under the supervision of a member of our engineering staff who logged the subsurface conditions in the field, effected the subsurface sampling and monitored the groundwater conditions. The locations of all the boreholes are shown on Drawing No. 1, Borehole Location Plan.

The boreholes were advanced using a power auger drilling rig equipped with continuous flight hollow stem augers and standard soil sampling equipment. Standard penetration tests (SPTs) 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.

Soil samples obtained from the in situ tests were examined in the field and subsequently taken to our laboratory for detailed description and moisture content determination. Two (2) grain size distribution analyses and an Atterberg Limits determination were performed on the major soil deposits.

Ground surface elevations at the borehole locations were surveyed by this office and are referenced to a temporary benchmark (TBM) established as:

TBM : Top of MH Cover on Fountain Street as shown on Drawing 1

Elevation : 277.53 m (Geodetic)

3.0 EXISTING SITE AND SLOPE CONDITIONS

The site has frontages on Abraham Street (west site limit) and Fountain Street (east site limit at the intersection with King Street West. The Abraham Street frontage is an undeveloped building lot. There is an existing five-storey building at the intersection of King Street West and Fountain Street. The building dates back to the end of the 19th century, with uses varying from spa resort to retirement residences. The north end of the site consists of two residential lots with existing two-storey residential dwellings.

The majority of the site consists of sloping terrains. There is tree cover over most of the site and the ground has vegetative ground cover varying from tended lawn to bushes. There is evidence of landscaping which has been neglected for several years such as small retaining walls and pathways. Minor ponding on the flatter areas and preferred drainage paths were observed on the slope during rainfall events. The site has ground surface elevations ranging from ±275 to 298 m, with up to ±23 m of grade difference over slope lengths of up to ±100 m.

This study area is limited to the edge of Fountain Street because the topography beyond this boundary is unlikely to affect slope stability on the site. Fountain Street descends ±9 m from the north over the ±156 m frontage of the site. At the north end of the site, there is drop in elevation of ±8 m (±10% grade) over a distance of approximately 76 m (to near the intersection of King Street West with Fountain Street).

South and east of Fountain Street lays the flood plain for the Speed River and a small tributary to the Speed River. At the south end of Fountain Street, the ground elevation drops approximately 3 to 5 m over the 70+ m to the edge of bank for the two water courses. The water surface of the Speed River and the tributary to the Speed River were 2 or more metres below the edge of bank at the time of the site visits in October 2013 and January 2014. The banks of the water courses are built up with landscaping and retaining walls which have been constructed over the past century. The flow of the Speed River is presently attenuated by a weir upstream of the site.

Selected photographs of the site and site condition are provided in Appendix "B".

4.0 SUBSURFACE CONDITIONS

The subsurface conditions encountered at the boreholes are detailed on the Borehole Log Sheets, Enclosures 1 to 7 of this report. The following notes are intended to amplify and comment on the subsurface data.

Beneath surficial asphalt or topsoil, loose to compact fill materials (consisting of silty sand and gravel with traces of topsoil) were contacted to depths of 0.5 to 2.44 m below ground surface at Boreholes 1A, 1B, 1C, 2 and 3. Below the fill materials, topsoil and pavement structures, the major native soil deposit consisted of stiff to hard, brown to grey, clayey silt till which overlay calcareous bedrock with an occasional compact to very dense sand and gravel layer between the bedrock and the clayey silt till.

The groundwater levels that were observed in the open borehole on completion of drilling and in the monitoring wells, were measured at elevations of ±271.5 to ±293.7 m, following the surficial topography of the slope. The groundwater was observed to be perched in fill and silty sand layers, occurred in saturated sand/silt seams within the clayey silt till and within fissures in the bedrock at depths of 0.9 to 3.5 m. It is noted that the observed groundwater table will fluctuate seasonally and in response to major weather events.

4.1 Pavement, Topsoil and Fill

Asphalt pavement, which was encountered at the ground surface of Boreholes 1A, 1B and 1C (located in an existing paved parking lot), consisted of typically 40 mm asphalt and up to 600 mm silty granular base. Boreholes 4 and 5 penetrated 200 to 700 mm of topsoil at the ground surface.

Fill materials, consisting of silty sand and gravel with traces of topsoil, were contacted to depths of 0.5 to 2.44 m below ground surface at Boreholes 1A, 1B, 1C, 2 and 3.

Standard penetration testing within the fill yielded "N"-values of 4 to 20 blows per 300 mm, thus indicating loose to compact state of compactness. Natural moisture contents varied between 9 and 20%, indicative of moist to saturated moisture conditions.

4.2 Silty Sand

Silty sand was encountered at Boreholes 3 below the upper 2.44 m thick fill materials and Borehole 4 below the surficial topsoil, and extended to respective depths of 4.10 and 1.52 m.

Standard penetration testing within the silty sand yielded "N"-values of 3 to 14 blows per 300 mm, thus indicating very loose to compact compactness condition. Natural moisture content was measured between 7 and 25%, indicative of moist to saturated moisture condition.

4.3 Clayey Silt Till

A major deposit of brown to grey clayey silt till was contacted below the upper fill, topsoil and silty sand layers. The clayey silt till extended to depths of 1.52 to 2.44 m at Boreholes 1A, 1B, 1C and 2. At Boreholes 3 and 4, the clayey silt till extended to the full depth of exploration of 11.7 and 7.92 m respectively.

The clayey silt till extended to a depth of 9.3 m at Borehole 5 and was further underlain by a 1.8 m thick sand and gravel deposit.

The clayey silt till contained traces of embedded sand and gravel with occasional saturated silt/sand layers. Two (2) grain size distribution analyses were performed and the results are presented graphically on Enclosures 8 and 9 of this report.

Standard Penetration testing within the clayey silt till yielded "N"-values from 5 to over 100 blows per 300 mm. The undrained shear strength determined by field pocket penetrometer on slightly disturbed SPT samples yielded values of 130 to over 250 kPa. Based on tactile examination, moisture content, the SPT "N"-values and the measured undrained shear strength, the bulk of the clayey silt till is considered to exhibit a hard consistency with the upper stratum showing stiff to very stiff consistency.

An Atterberg Limits determination was performed on the clayey silt till and the results are shown on the log of Borehole 3. The results yielded Liquid Limit of 29%, Plastic Limit 11% with Plasticity Index of 18%. The test results indicate an inorganic clay of low plasticity (CL). Natural moisture content generally varied between 6 and 28%, typically in the range of 8 to 14% and indicative of typically "drier than plastic limit" moisture condition. The clayey silt till is therefore considered to be "moderately consolidated" in the upper stiff to very stiff stratum to "heavily consolidated" in the lower hard stratum.

4.4 Sand and Gravel

A compact to very dense sand and gravel layer underlay the clayey silt till at Boreholes 1A, 1B, 1C and 5.

Standard Penetration testing within the clayey silt till yielded "N"-values from 10 to over 100 blows per 300 mm. The natural moisture contents were measured between 4 and 12%, indicative of moist to saturated moisture condition. Based on tactile examination, moisture content and the SPT "N"-values, the sand and gravel is considered to be in a compact to very dense state of compactness.

4.5 Bedrock

Bedrock was contacted at elevations of ±272 to 276 m adjacent to Fountain Street (eastern portion of the site) at Boreholes 1A, 1B, 1C and 2, and bedrock was contacted at depths of ±280 to 285 m at Boreholes 3 and 5, which are at higher elevations up the slope.

4.6 Groundwater Condition

The groundwater conditions were monitored during and following completion of borehole sampling. Groundwater in the uncased borehole at Borehole 4 was measured at a depth of 7.2 m depth on completion of sampling and at 6.4 m depth 3 hours after completion of sampling. No free water was observed in the uncased borehole at Boreholes 1A, 1B, and 5 on completion of sampling.

Monitoring Wells were installed in Boreholes 1C, 2 and 3 and water levels were measured at depths of 2.55 m, 3.52 m, and 3.20 m on October 21, 2014, corresponding to respective elevations of 276.54, 271.51 and 293.69 m.

During sampling, a wet to saturated condition was encountered locally in the upper silty sand layer and in the sand/silt seams within the clayey silt till. The following table summarizes the depth at which saturated native soils were first contacted and the respective elevation at each borehole location:

Borehole	Elevation (m)	Contact Depth (m)	Approximate Elevation (m)
1A	277.84	---	moist soils
1B	278.16	---	moist soils
1C*	279.09	2.55	276.54
2*	275.03	3.52	271.51
3*	296.89	3.20	293.69
4	291.17	3.80	287.37
5	290.87	0.87	290.00

* measured in installed monitoring well

Based on the observed groundwater level, observed soils that were saturated and measured natural moisture content, the groundwater level is expected to lie 0.9 to 3.8 m below existing grades, at elevations ±271.5 to ±293.7 m, following the surficial topography of the slope. The groundwater was observed to be perched in fill and silty sand layers, occurred in saturated sand/silt seams within the clayey silt till and within fissures in the bedrock at depths of 0.9 to 3.5 m.

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

5.0 DISCUSSION AND RECOMMENDATIONS

The proposed 5 -storey apartment building will have two (2) underground parking levels. Based on preliminary design concepts, the lower basement floor level will be at elevation ±282.75 m.

The majority of the site consists of sloping terrains. There is tree cover over most of the site and the ground has vegetative ground cover varying from tended lawn to bushes. There is evidence of landscaping which has been neglected for several years such as small retaining walls and pathways. Minor ponding on the flatter areas and preferred drainage paths were observed on the slope during rainfall events. The site has ground surface elevations ranging from ±275 to 298 m, with up to ±23 m of grade difference over slope lengths of up to ±100 m.

Beneath surficial asphalt or topsoil, loose to compact fill materials (consisting of silty sand and gravel with traces of topsoil) were contacted to depths of 0.5 to 2.44 m below ground surface at Boreholes 1A, 1B, 1C, 2 and 3. Below the fill materials, topsoil and pavement structures, the major native soil deposit consisted of stiff to hard, brown to grey, clayey silt till which overlay calcareous bedrock with an occasional compact to very dense sand and gravel layer between the bedrock and the clayey silt till.

The groundwater levels that were observed in the open borehole on completion of drilling and in the monitoring wells, were measured at elevations of ±271.5 to ±293.7 m, following the surficial topography of the slope. The groundwater was observed to be perched in fill and silty sand layers, occurred in saturated sand/silt seams within the clayey silt till and within fissures in the bedrock at depths of 0.9 to 3.5 m. It is noted that the observed groundwater table will fluctuate seasonally and in response to major weather events.

5.1 Slope Profiles

Slope profiles were examined at three (3) locations over the property to assess the present stability and the stability of the proposed building/structure.

Section A-A' (located near the south limit of the site) follows the slope from Abraham Street at the highest elevation on the property to Fountain Street at the lowest elevation on the property. The top of the slope is at an elevation of ±296.3 m and the bottom of the slope is at an elevation of ±275.0 m over a length of ±87 m. The slope profile contains alternating flat and steeper sections with an average slope of ±4.1 H:1 V. At Fountain Street, the embedded wall of the former garage acts as a 3 to 4 m high retaining wall at the toe of the slope.

Section B-B' intersects the proposed parking garage and the existing building which has a foundation which may extend to an elevation of ±269.8 m. The highest point is on the west property line at ±294.0 m and the ground level at Fountain Street is at ±276.0 m over a distance of ±76 m. The average slope is ±4.2 H:1 V. The slope profile contains alternating flat and steep sections and there is a series of retaining walls adjacent to the existing building which is the steepest section of the total slope.

Section C-C' intersects the proposed building and parking garage. With a slope starting at an elevation of ±293.0 m at the property line and lying at ±281.9 m at Fountain Street over a length of ±50 m the average slope is ±4.5 H:1 V. The existing slope is even and will be replaced with the building and parking area

5.2 Slope Stability Analysis

Stability analysis were conducted for the existing slope condition and for the post-construction slope condition. Results of the analyses are presented in Appendix "C". The methodology and results are summarized below.

The soil parameters used in the slope stability analyses were determined based on the field and laboratory test results of the present investigation and our past experience with similar soil types. The selected "effective stress - drained condition" soil parameters and groundwater condition were entered into the GEO-SLOPE slope stability computer program to perform the stability analyses.

The following soil strength parameters were used in the stability analysis, the coefficients for active pressure and passive resistance are included for the purpose estimating loads acting on the basement or retaining walls:

Soil Type	Unit Weight (kN/m^3)	Effective Angle of Internal Friction ø' (degrees)	Effective Cohesion c' (kPa)	Ka	Kp
1. Existing Fill	20	28°	n/a	0.36	2.8
2. Silty Sand	21	31°	n/a	0.32	3.1
3. Upper Clayey Silt Till	21	37°	10	0.25	4.0
4. Lower Clayey Silt Till	22	40°	15	0.22	4.6

The overall slopes average inclination is over 4.1 H: 1 V for all the profile sections and deep rotational failure over the whole slope is not an issue. The factor of safety against a rotational failure for the existing slope profiles and the factor of safety against a rotational failure of the proposed design at the most severe local steepening sections of the slope profiles were:

Section Profile	Factor of Safety for Existing Profile	Factor of Safety for Proposed Building/ Structure
A-A'	2.40	2.60
B-B'	1.55	1.55
C-C'	2.95	2.06

The existing retaining walls are assumed to be robust and adequate for the existing conditions. The condition of the existing structures which are integral to the present stable slope should be monitored to maintain a safe condition. The condition and suitability of the existing structures that are to be incorporated into the proposed development should be determined.

In summary, our stability calculations indicate that the existing slope condition has a sufficient global factor of safety (over 1.5) against slope failure. The long term stable top of bank is considered to be the present top of bank. The proposed residential development, provided it is properly designed and the construction properly staged, will not adversely affect the slope stability.

It is noted that the factor of safety does not include the stabilizing effect of the surface vegetation and trees which can increase the factor of safety by typically 15% against surface erosion and instability.

5.3 Foundations

The proposed basement floor level will be near elevation ±282.8 m and, consequently, footings will be founded near elevation ±281.6 m and lower (at the elevator and by providing 1.2 m frost cover). At the anticipated footing founding level, typically very stiff to hard brown to grey clayey silt till will be exposed as indicated by Boreholes 3, 4 and 5.

At lower ground surface elevations near Fountain Street, loose to compact fill materials and less competent native granular soil and clayey silt till could be expected near the ground surface.

In light of the limited number of the boreholes advanced in the proposed building area, further investigation is required to identify the founding subgrade elevation and possible bedrock elevations, especially at the locations approaching Fountain Street.

For preliminary design purposes, the footings can be designed using a Geotechnical Reaction at SLS of 400 kPa (8,000 psf). The SLS value given above is based on a maximum differential settlement of 25 mm. The Factored Geotechnical Resistance at ULS is 600 kPa (12,000 psf). 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.

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

Borehole	Elevation of Borehole (m)	Highest Depth Below Existing Grade (m)	Highest Footing Bearing Elevation (m)
1A	277.84	1.54	276.30
1B	278.16	1.56	276.60
1C	279.09	2.99	276.10
2	275.03	1.53	273.50
3	296.89	5.29	291.60
4	291.17	2.17	289.00
5	290.87	2.37	288.50

Spacing between adjacent footing steps should not be steeper than 10 horizontal to 7 vertical. 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 subgrade soils are considered to be frost-susceptible and 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.

5.4 Earthquake Consideration

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 soils condition encountered at the boreholes and our experience with the top 30 m of soil condition in the general area of the site, the site can be classified as a Site Class C as per OBC Table 4.1.8.4.A (Page B4-24).

5.5 Lateral Earth Pressure

The below grade 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 using granular backfill. The factored resistance can be calculated by using a factor of 0.8.

$$\mathbf{P = K(\gamma H + q)}$$

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 1). The backfill should be placed in thin layers and compacted to 95% SPMDD. Over-compaction should be avoided. Weeping tiles or weep holes should be installed to effect drainage behind the retaining wall.

The sliding resistance of the footings should be checked. The unfactored horizontal resistance against sliding between concrete and undisturbed native soil deposits can be calculated using a friction coefficient of 0.4 for the native soils. The unit weight of the native soil deposits are listed in Section 5.2 Slope Stability Analysis, and a unit weight of 21 kN/m^3 may be expected for the granular backfill compacted to 95% SPMDD.

5.6 Floor Slab Construction

The boreholes indicate that native soil deposits should be exposed at the anticipated garage floor slab subgrade. At the time of floor slab construction, the exposed subgrade should be proof-rolled with a heavy roller in conjunction with an inspection by the geotechnical engineer. Any soft and/or unstable areas detected should be replaced with granular fill which should be compacted to at least 95% SPMDD. On-site inorganic granular soils may be reused to raise grades where necessary provided its natural moisture content is near to the optimum moisture content. The backfill should be thoroughly compacted in thin lifts to at least 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. Alternatively, suitably densified 19 mm clear crushed stone can be used.

A modulus of subgrade reaction (k_s) of 25 MN/m^3 may be used for the design of the floor slabs, assuming a clayey silt subgrade.

The floor slabs 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.

Installation of footing drains, damp-proofing or water-proofing the below grade foundation walls are recommended. In addition, underfloor drains should be considered to ensure that surface water infiltration does not become a water penetration problem.

5.7 Asphalt Pavement

The subgrade for pavement construction is anticipated to consist of existing fill and backfill materials. Assuming the subgrade material likely to be clayey silt and the existence of adequate drainage, the following flexible pavement structures are recommended:

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

It is recommended that the pavement construction 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.

Prior to placement of the granular base, the subgrade should be stripped of any obvious deleterious materials and the exposed subgrade thoroughly recompacted with a heavy vibratory compactor and inspected by a qualified geotechnical inspector. Any soft spots encountered during the process should be excavated to the level of competent soil. The required grades can then be achieved by placing approved on-site soils in maximum 200 to 300 mm thick lifts which should be compacted to 95% SPMDD.

The Granular "A" and Granular "B" materials should meet the gradational requirements of OPSS Form 1010 and should be compacted to no less than 100% SPMDD. The placing and rolling of the asphalt mixture should conform to OPSS Form 310 or equivalent and should be compacted to between 92.0 and 96.5% of the Marshall density. 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 any trench backfill.

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 extending 3 m from the catch-basins.

5.8 Concrete Pavement

The subgrade clayey silt soils and silt rich granular soils are considered frost susceptible. 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.

5.9 Construction and Groundwater Control

Based on the recommended footing founding depths and elevations, the anticipated depths of excavation across the site will be in the order of 1.5 to 13 m. Locally deeper excavation at the elevator pit is anticipated. The excavations can be carried out using open cut with side slopes of 1H :1V throughout.

Shallow excavations will penetrate the loose fill materials, stiff to hard silty clay and loose to dense sand/silt soils. These soils are considered to be Type 3 Soils in accordance with the latest Occupational Health and Safety Act. Excavations in the Type 3 soils are expected to remain stable during the construction period provided that side slopes are cut to 1H : 1V throughout and the groundwater is adequately controlled. The slope surface should be suitably protected from erosion processes.

Deep excavations should be carried out with side slopes supported by shoring, the geotechnical design input is provided in the following section.

Groundwater could be encountered within the deeper excavations which are extended into the saturated sand/gravel layer, while minor seepage possibly from perched water within the upper fill materials and saturated lenses of granular material can occur randomly across the site. Any surface water runoff and seepage into the excavations may be handled by conventional sump pumping techniques, as and where required. The sump pits should be filtered.

A lean concrete mat may be required over the approved subgrade to prevent disturbance resulting from construction activities and the elements.

5.10 Shoring Requirement During Construction

A review of the critical areas of the proposed two-level underground parking was carried out based on the separation distances to the property lines and the elevation difference. The excavation conditions are summarized in the following table:

	Existing Ground Surface Elevation (m)	Distance from Underground Garage to Property Line (m)	Anticipated Depth of Excavation** (m)	Remarks
Along North Property Line	294.9 to 284.3 sloping down to the east	4.0 at the closest approach	8.9	shoring required
Along East Property Lines	283.8. to 274.9 sloping down to the south	15 to 20	n/a	shoring not required
Along South Property Line	299.47 to 274.9 sloping down to the east	6.0	8.4 to 11.9	shoring required
Along west Property Line	294.4 to 294.9 Level on slope	7.0	12.8	shoring required

Note: ** estimated footing founding depth at elevation 281.6 m, allowing 1.2 m depth below the finished basement floor of elevation 282.75 m for frost cover. Depth is from adjacent property line elevation.

Based on the above, a shoring system may be required for temporary excavation support during construction around sections of the perimeter of the site. However, it may be feasible to carry out sloped sided excavation at select locations by obtaining an agreement with the adjacent property owner to carry out the excavation past the property line.

If a shoring system is indeed required, it is generally designed and built by a specialized shoring contractor. Possible shoring supports include soldier pile and timber lagging system, caisson wall or soil nail retaining system with shotcrete facing. Alternatively, a gravity-type lean concrete retaining wall can be constructed to provide temporary support.

The shoring system will have to withstand the lateral earth pressure, buried services as well as the nearby building structures. Frost pressures on the shoring system should be considered if the system is exposed to winter freezing condition. The shoring system should be monitored to ensure deflection and lateral movements are acceptable, and modifications made to the shoring system if deflection and lateral movements become a problem.

The shoring system can be supported by soil anchor tie-backs and/or raker footings. Soil anchors will provide an unobstructed open space for construction whereas raker supports will obstruct the forming and construction of footings and foundation walls. Since soil anchors extend beyond the property lines, agreement with neighbouring property owner will have to be obtained. This also applies to the soil nail system.

Please review table in Section 5.2 Slope Stability Analysis for soil parameters of the soils that may be used in the design of shoring

6.0 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.

Hans Dworatzek P.Eng., CET.
Project Engineer

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

Appendix “A”

Statement of Limitations

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 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.

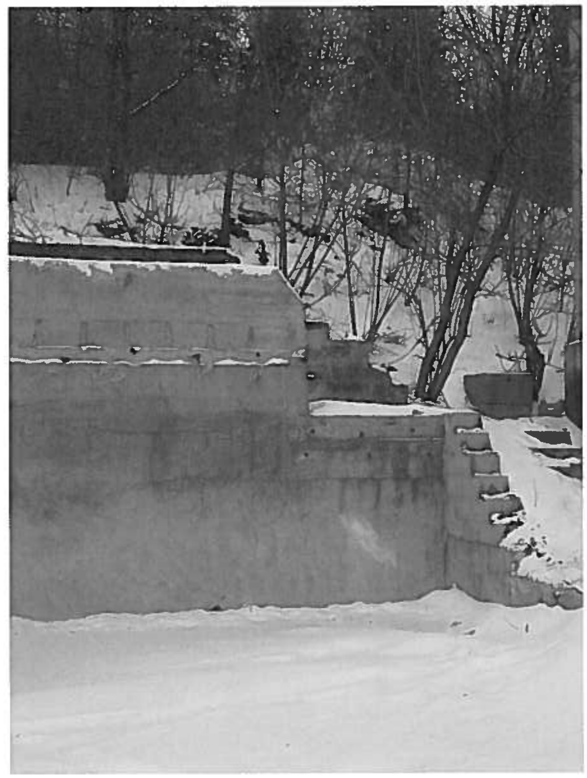
Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. CHUNG & VANDER DOELEN ENGINEERING LIMITED accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or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

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

Appendix "B"

Photographs of Slope

Existing Building



Section AA' Retaining Wall at Toe

Section BB' Retaining Walls near
Existing Building

Section CC' Toe - Slope Behind
Existing Building

Section BB' and Section CC'
Upper Elevations

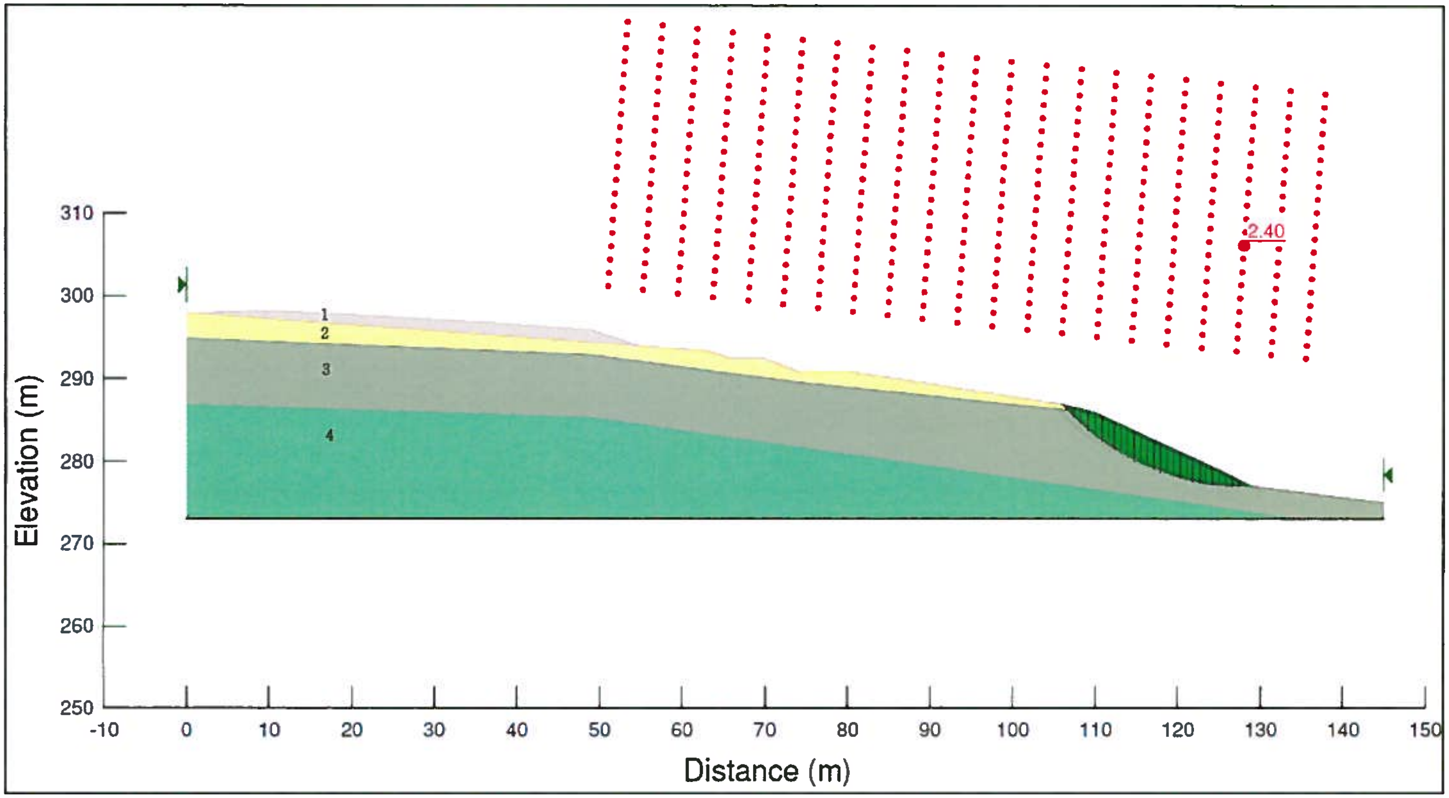
Section AA' Vacant Lot on
Abraham Street

Section AA' Multiple Banks on Total
Slope

Typical Property Line at Section BB'
and Section CC'

Appendix "C"

Slope Stability Analysis Results



Section A-A' Existing Condition

Soil Parameters

Soil Type	Unit Weight (kN/m³)	Effective Angle of Internal Friction ø' (degrees)	Effective Cohesion c' (kPa)
1. Existing Fill	20	28°	n/a
2. Silty Sand to Sandy Silt	21	31°	n/a
3. Dense Till	21	37°	10
4. Very Dense Till	22	40°	15

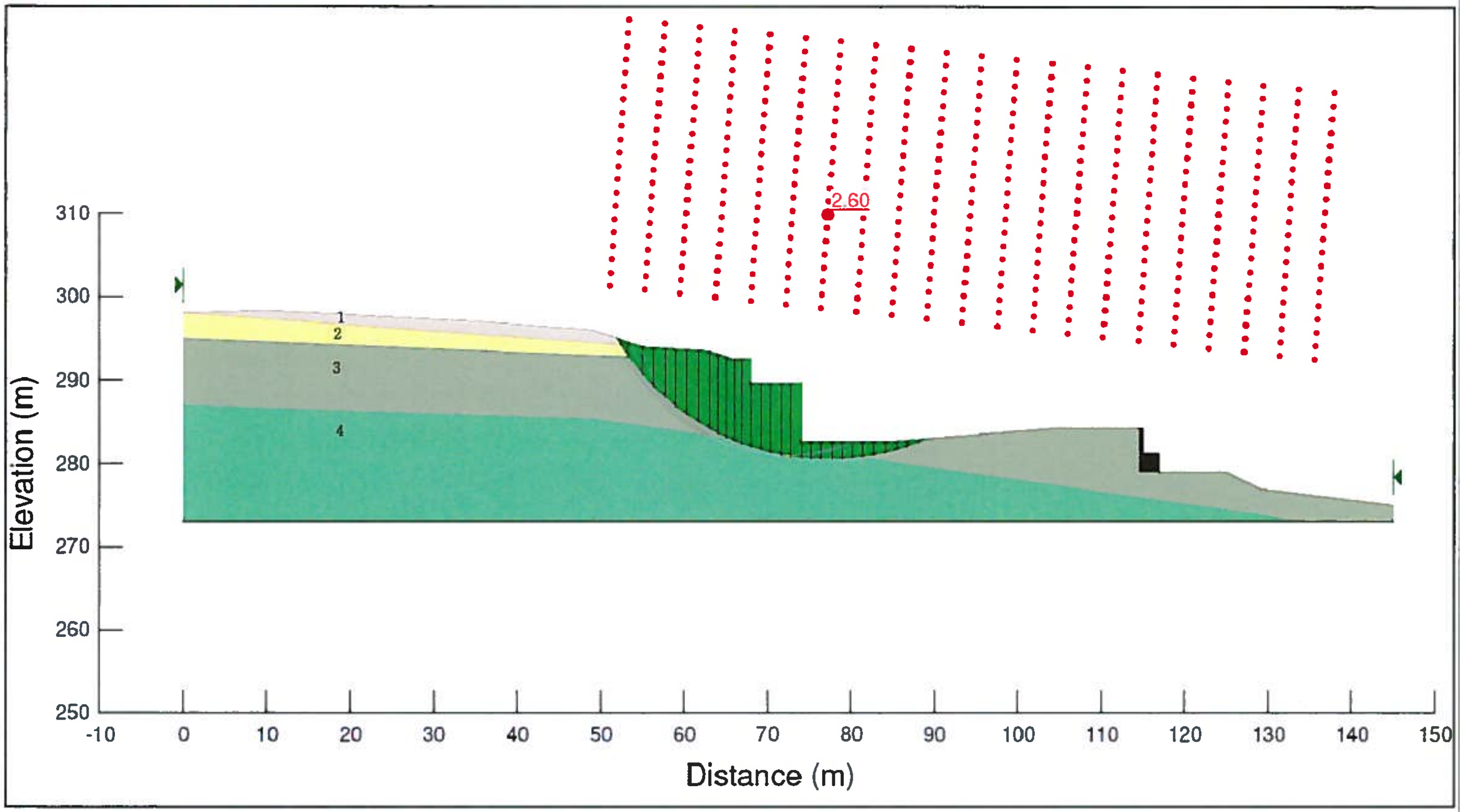
SLOPE STABILITY ASSESSMENT
Proposed Residence

GEOSLOPE SLOPE/W 5
Morgenstern/Price Analysis

CHUNG & VANDER DOELEN ENGINEERING LTD.

311 Victoria Street North
Kitchener, ON N2H 5E1
Phone:(519)742-8979 Fax:(519)742-7739
E-mail:info@cvdengineering.com

Drawn by: HD	Date: Jan. 2014	File No.: 13-09-K01
Checked by: EYC	Scale: As shown	Figure No.: 1



Section A-A' Proposed

Soil Parameters

Soil Type	Unit Weight (kN/m^3)	Effective Angle of Internal Friction ø' (degrees)	Effective Cohesion c' (kPa)
1. Existing Fill	20	28°	n/a
2. Silty Sand to Sandy Silt	21	31°	n/a
3. Dense Till	21	37°	10
4. Very Dense Till	22	40°	15

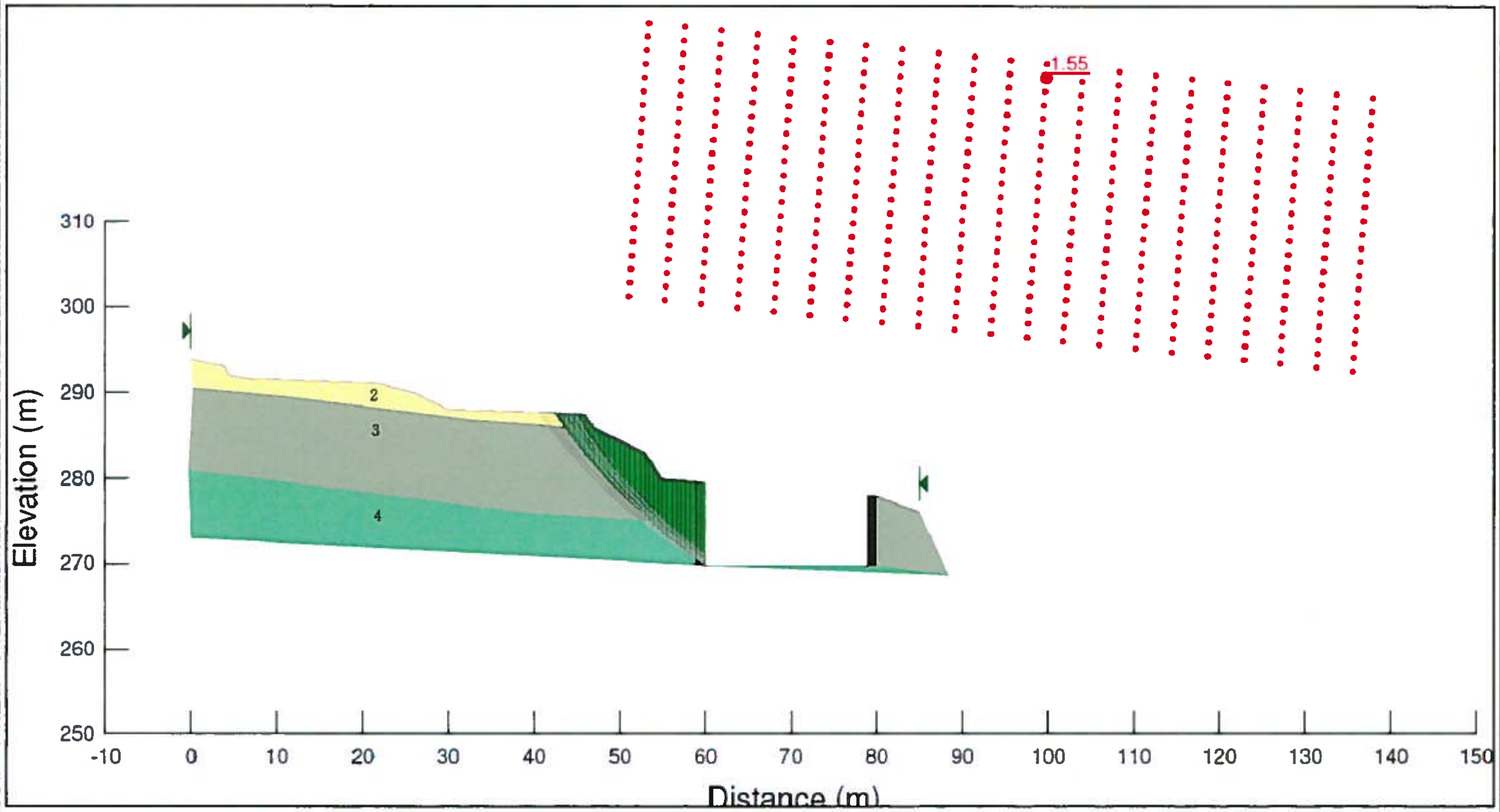
SLOPE STABILITY ASSESSMENT
Proposed Residence

GEOSLOPE SLOPE/W 5
Morgenstern/Price Analysis

CHUNG & VANDER DOELEN ENGINEERING LTD.

311 Victoria Street North
Kitchener, ON N2H 5E1
Phone:(519)742-8979 Fax:(519)742-7739
E-mail:info@cvdengineering.com

Drawn by: HD	Date: Jan. 2014	File No.: 13-09-K01
Checked by: EYC	Scale: As shown	Figure No.: 2



Section B-B' Existing Condition

Soil Parameters

Soil Type	Unit Weight (kN/m³)	Effective Angle of Internal Friction φ' (degrees)	Effective Cohesion c' (kPa)
1. Existing Fill	20	28°	n/a
2. Silty Sand to Sandy Silt	21	31°	n/a
3. Dense Till	21	37°	10
4. Very Dense Till	22	40°	15

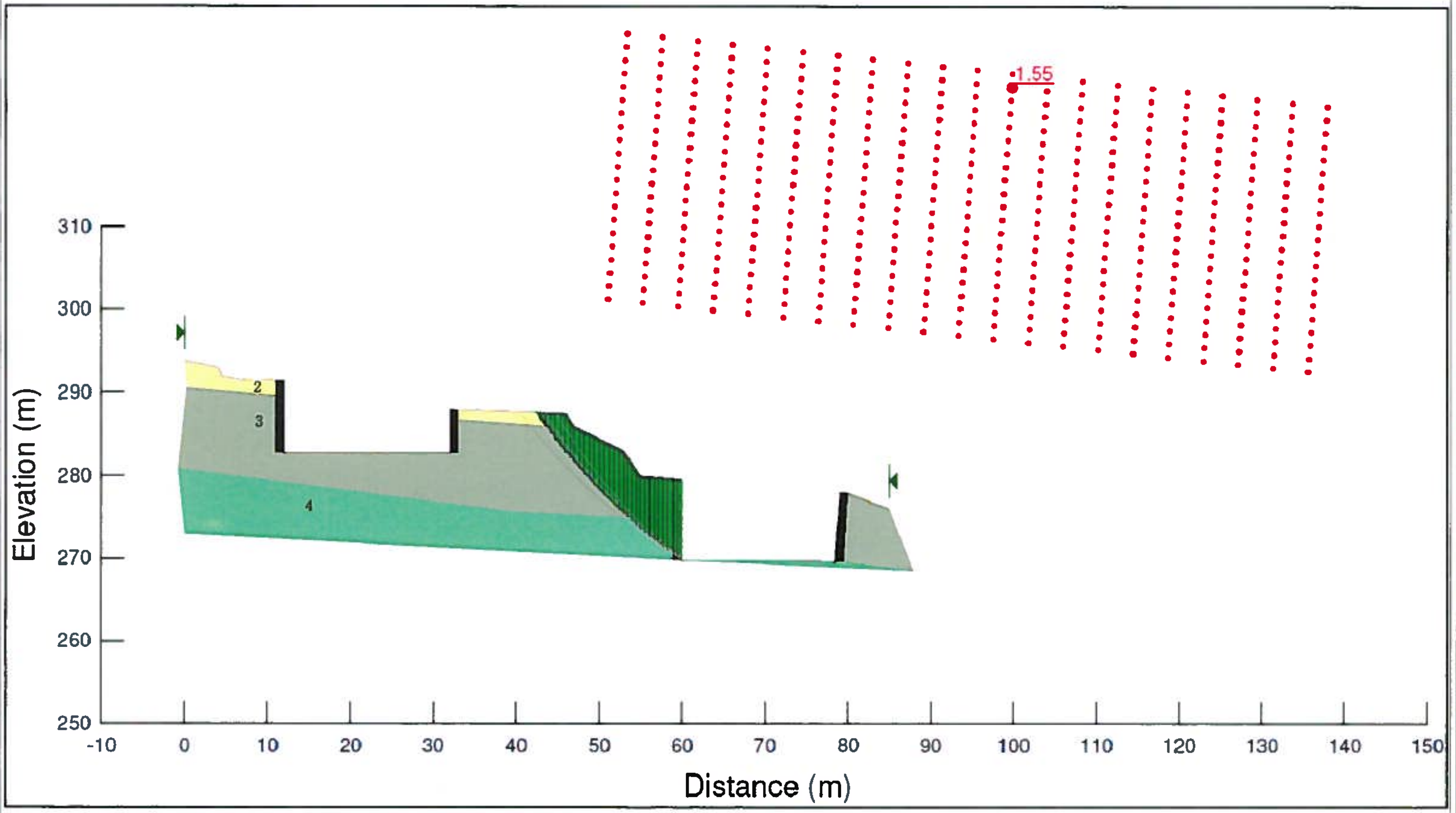
SLOPE STABILITY ASSESSMENT
Proposed Residence

GEOSLOPE SLOPE/W 5
Morgenstern/Price Analysis

CHUNG & VANDER DOELEN ENGINEERING LTD.

311 Victoria Street North
Kitchener, ON N2H 5E1
Phone:(519)742-8979 Fax:(519)742-7739
E-mail:info@cvdengineering.com

Drawn by: HD	Date: Jan. 2014	File No.: 13-09-K01
Checked by: EYC	Scale: As shown	Figure No.: 3



Section B-B' Proposed

Soil Parameters

Soil Type	Unit Weight (kN/m³)	Effective Angle of Internal Friction ø' (degrees)	Effective Cohesion c' (kPa)
1. Existing Fill	20	28°	n/a
2. Silty Sand to Sandy Silt	21	31°	n/a
3. Dense Till	21	37°	10
4. Very Dense Till	22	40°	15

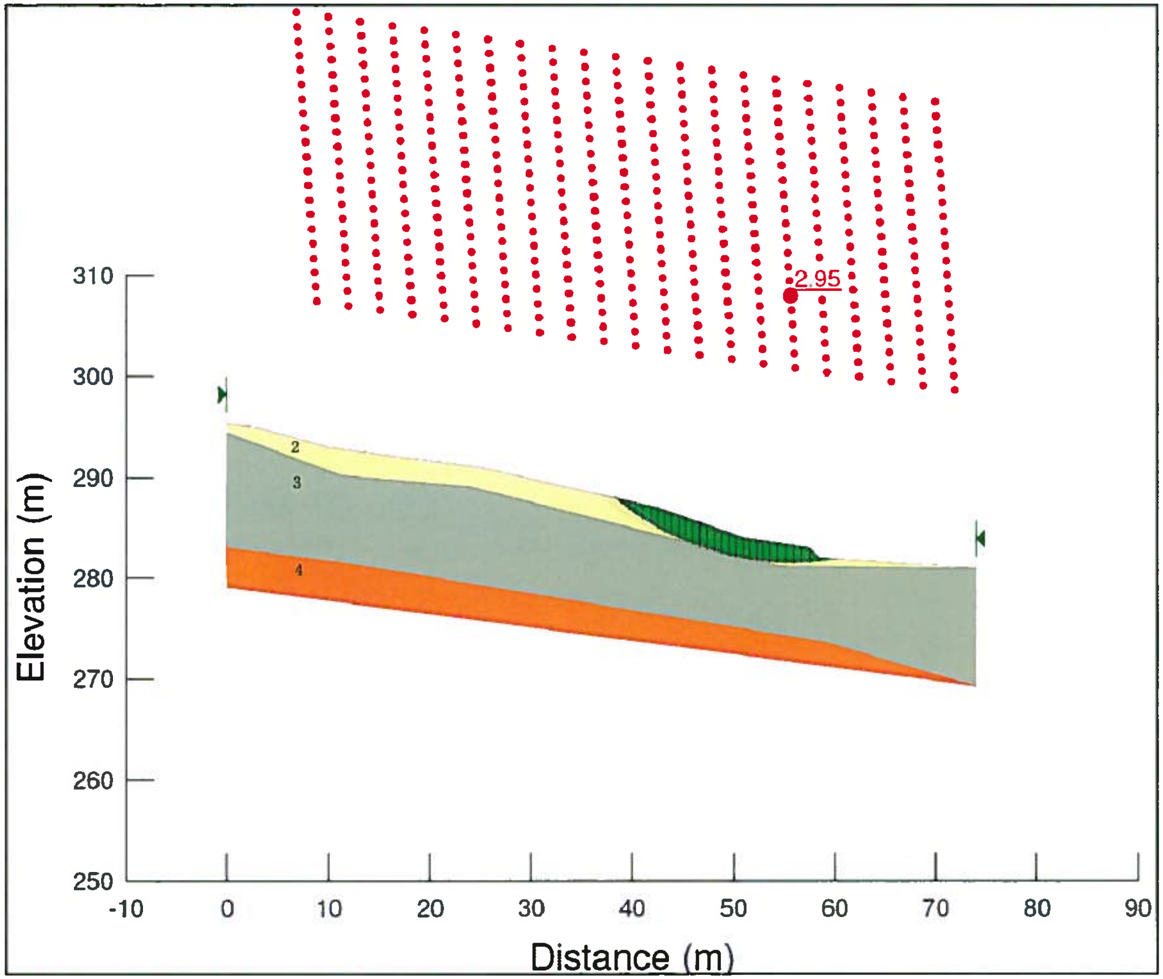
SLOPE STABILITY ASSESSMENT
Proposed Residence

GEOSLOPE SLOPE/W 5
Morgenstern/Price Analysis

CHUNG & VANDER DOELEN ENGINEERING LTD.

311 Victoria Street North
Kitchener, ON N2H 5E1
Phone:(519)742-8979 Fax:(519)742-7739
E-mail:info@cvdengineering.com

Drawn by: HD	Date: Jan. 2014	File No.: 13-09-K01
Checked by: EYC	Scale: As shown	Figure No.: 4



Section C-C' Existing Condition

Soil Parameters

Soil Type	Unit Weight (kN/m^3)	Effective Angle of Internal Friction ϕ' (degrees)	Effective Cohesion c' (kPa)
1. Existing Fill	20	28°	n/a
2. Silty Sand to Sandy Silt	21	31°	n/a
3. Dense Till	21	37°	10
4. Very Dense Till	22	40°	15

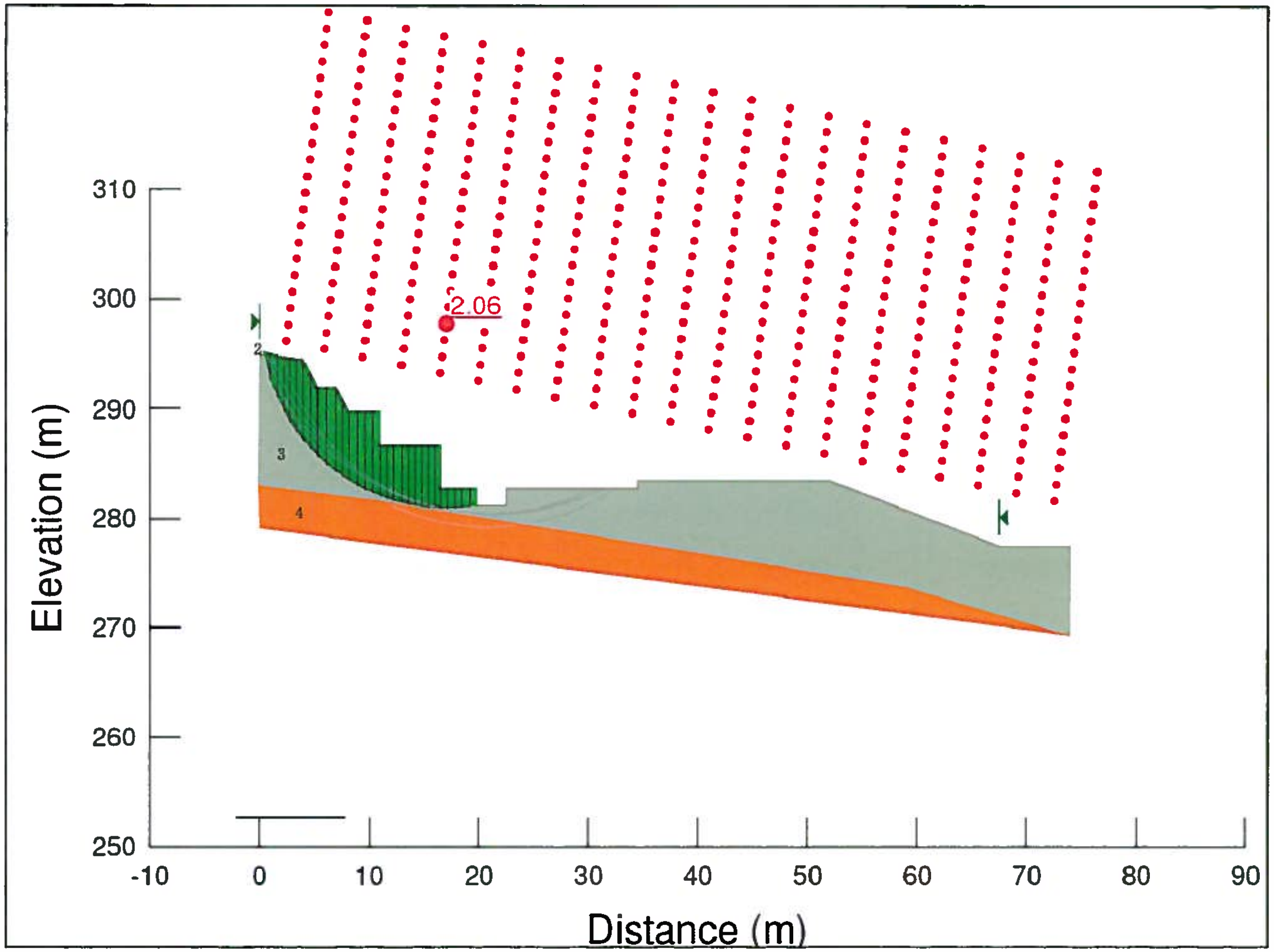
SLOPE STABILITY ASSESSMENT
Proposed Residence

GEOSLOPE SLOPE/W 5
Morgenstern/Price Analysis

CHUNG & VANDER DOELEN ENGINEERING LTD.

311 Victoria Street North
Kitchener, ON N2H 5E1
Phone:(519)742-8979 Fax:(519)742-7739
E-mail:info@cvdengineering.com

Drawn by: HD	Date: Jan. 2014	File No.: 13-09-K01
Checked by: EYC	Scale: As shown	Figure No.: **5**



Section C-C' Proposed

SOIL PARAMETERS

Soil Type	Unit Weight (kN/m^3)	Effective Angle of Internal Friction ø' (degrees)	Effective Cohesion c' (kPa)
1. Existing Fill	20	28°	n/a
2. Silty Sand to Sandy Silt	21	31°	n/a
3. Dense Till	21	37°	10
4. Very Dense Till	22	40°	15

SLOPE STABILITY ASSESSMENT
Proposed Residence

GEOSLOPE SLOPE/W 5
Morgenstem/Price Analysis

CHUNG & VANDER DOELEN ENGINEERING LTD.

311 Victoria Street North
Kitchener, ON N2H 5E1
Phone:(519)742-8979 Fax:(519)742-7739
E-mail:info@cvdengineering.com

Drawn by: HD	Date: Jan. 2014	File No.: 13-09-K01
Checked by: EYC	Scale: As shown	Figure No.: 6

Enclosures

FILE No: 13-09-K01

BOREHOLE No. 1 A

CHUNG & VANDER DOELEN CVD ENGINEERING LTD.

Client: **Haastown Group of Companies**

Project: **Preston Spring Condominiums**

Location: **102 Fountain Street South, Cambridge, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Hollow Stem Augers**
Size: **105 mm ID**
Date: **Oct 15 / 13** TO **Oct 15 / 13**

SOIL LITHOLOGY

Ground Elevation: **277.84 m**

ELEV./ DEPTH (m)	DESCRIPTION
	40 mm ASPHALT
	compact, brown FILL, silty sand and gravel moist ------ silty sand below 0.9 m depth
276.77 / 1.07	stiff, brown CLAYEY SILT TILL moist
276.32 / 1.52	compact, brown SAND and GRAVEL, wet
275.94 / 1.90	WEATHERED BEDROCK
275.71 / 2.13	End of Borehole at Auger and Spoon Refusal

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	20
2	SS	11
3	SS	20
4	SS	100/ 0cm

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

no free water at completion

PROJECT MANAGER: **EYC**

CHUNG & VANDER DOELEN ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario N2H 5E1
ph. (519) 742-8979, fx. (519) 742-7739

FILE No: 13-09-K01

BOREHOLE No. 1 B

CHUNG & VANDER DOELEN ENGINEERING LTD. CVD

Client: **Haastown Group of Companies**

Project: **Preston Spring Condominiums**

Location: **102 Fountain Street South, Cambridge, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Hollow Stem Augers**
Size: **105 mm ID**
Date: **Oct 15 / 13** TO **Oct 15 / 13**

SOIL LITHOLOGY | SAMPLE | 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) | REMARKS

Ground Elevation: **278.16 m**

ELEV./ DEPTH (m)	DESCRIPTION	DEPTH (m)	SYMBOL	SAMPLE ID	TYPE	N-VALUE	REMARKS
	40 mm ASPHALT						
	compact, brown FILL, silty sand and gravel moist ------ silty sand below 0.9 m depth	0.5					
277.09 1.07	stiff, brown CLAYEY SILT TILL moist	1.0					
276.64 1.52	compact, brown SAND and GRAVEL, wet	1.5					
276.31 1.85							
276.18 1.98	WEATHERED BEDROCK	2.0					no free water at completion
	End of Borehole at Auger Refusal						

Depth scale (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**

CHUNG & VANDER DOELEN
ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario N2H 5E1
ph. (519) 742-8979, fx. (519) 742-7739

FILE No: 13-09-K01

BOREHOLE No. 1 C

CHUNG & VANDER DOELEN CVD ENGINEERING LTD.

Client: **Haastown Group of Companies**

Project: **Preston Spring Condominiums**

Location: **102 Fountain Street South, Cambridge, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Hollow Stem Augers**
Size: **105 mm ID**
Date: **Oct 15 / 13** TO **Oct 15 / 13**

SOIL LITHOLOGY

Ground Elevation: **279.09 m**

ELEV./ DEPTH (m)	DESCRIPTION
	40 mm ASPHALT
	compact, brown FILL, silty sand and gravel moist ------ silty sand below 0.9 m depth
278.02 / 1.07	stiff, brown CLAYEY SILT TILL moist
277.57 / 1.52	compact, brown SAND and GRAVEL some silt wet to saturated
275.59 / 3.50	WEATHERED BEDROCK
275.13 / 3.96	End of Borehole at Auger Refusal

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	14
2	SS	10
3	SS	142/ 25cm

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

- Flush Mount Cover
- 0.5 Bentonite seal
- 2.4 m - 50 mm diameter PVC Riser
- 2.0 Filter sand
- 2.5 water level at 2.55 m in monitoring well BH1 on October 21, 2013
- 3.5 1.5 m - 50 mm diameter PVC Screen

PROJECT MANAGER: **EYC**

CHUNG & VANDER DOELEN
ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario N2H 5E1
ph. (519) 742-8979, fx. (519) 742-7739

Enclosure No.: 4
Sheet 1 of 1

FILE No: 13-09-K01 — BOREHOLE No. 2

CHUNG & VANDER DOELEN CVD ENGINEERING LTD.

Client:	**Haastown Group of Companies**
Project:	**Preston Spring Condominiums**
Location:	**102 Fountain Street South, Cambridge, Ontario**

EQUIPMENT DATA

Machine:	**Diedrich D50T**
Method:	**Hollow Stem Augers**
Size:	**105 mm ID**
Date:	**Oct 15 / 13** TO **Oct 15 / 13**

SOIL LITHOLOGY

Ground Elevation: **275.03 m**

ELEV./ DEPTH (m)	DESCRIPTION
	compact, brown FILL, sand and gravel, trace asphalt moist
274.53 / 0.50	very stiff to hard brown CLAYEY SILT TILL trace sand sand gravel moist
272.59 / 2.44	highly weathered fractured grey LIMESTONE BEDROCK wet
269.09 / 5.94	End of Borehole with Auger and Split Spoon Sampler Refusal

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	15
2	SS	18
3	SS	45
4	SS	124
5	SS	100/10cm
6	SS	50/5cm
7	SS	90/10cm
8	SS	25/0cm
9	SS	100/5cm

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

- Flush Mount Cover
- 2.4 m - 50 mm diameter PVC Riser
- Bentonite seal
- water level at 3.52 m in monitoring well BH2 on October 21, 2013
- Filter sand
- 1.5 m - 50 mm diameter PVC Screen

PROJECT MANAGER: **EYC**

CHUNG & VANDER DOELEN ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario N2H 5E1
ph. (519) 742-8979, fx. (519) 742-7739

FILE No: 13-09-K01 BOREHOLE No. 3

CHUNG & VANDER DOELEN CVD ENGINEERING LTD.

Client: **Haastown Group of Companies**
Project: **Preston Spring Condominiums**
Location: **102 Fountain Street South, Cambridge, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Hollow Stem Augers**
Size: **105 mm ID**
Date: **Oct 16 / 13** TO **Oct 16 / 13**

SOIL LITHOLOGY

Ground Elevation: **296.89 m**

ELEV./DEPTH (m)	DESCRIPTION
	loose to compact brown to dark brown FILL silty sand with occasional topsoil moist ------ concrete fragments ------ topsoil below 2.3 m depth
294.45 2.44	loose to very loose brown SILTY SAND moist to saturated
292.79 4.10	very stiff to hard brown to grey CLAYEY SILT TILL trace sand and gravel occ. silt/sand seams damp to moist

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	4
2	SS	15
3	SS	31
4	SS	3
5	SS	9
6	SS	6
7	SS	18
8	SS	43
9	SS	53

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
	Stick-up Well Protector
1.5	Bentonite seal
2.5	4.3 m of 50 mm diameter PVC Riser
3.0	saturated below 3 m depth
	water level at 3.20 m in monitoring well BH3 on October 21, 2013
	grey and dense below 5.2 m depth
	7.6 m - 50 mm diameter PVC screen
	saturated sand seam
	saturated sand seam
8.0	filter sand for screen

PROJECT MANAGER: **EYC**

CHUNG & VANDER DOELEN ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario N2H 5E1
ph. (519) 742-8979, fx. (519) 742-7739

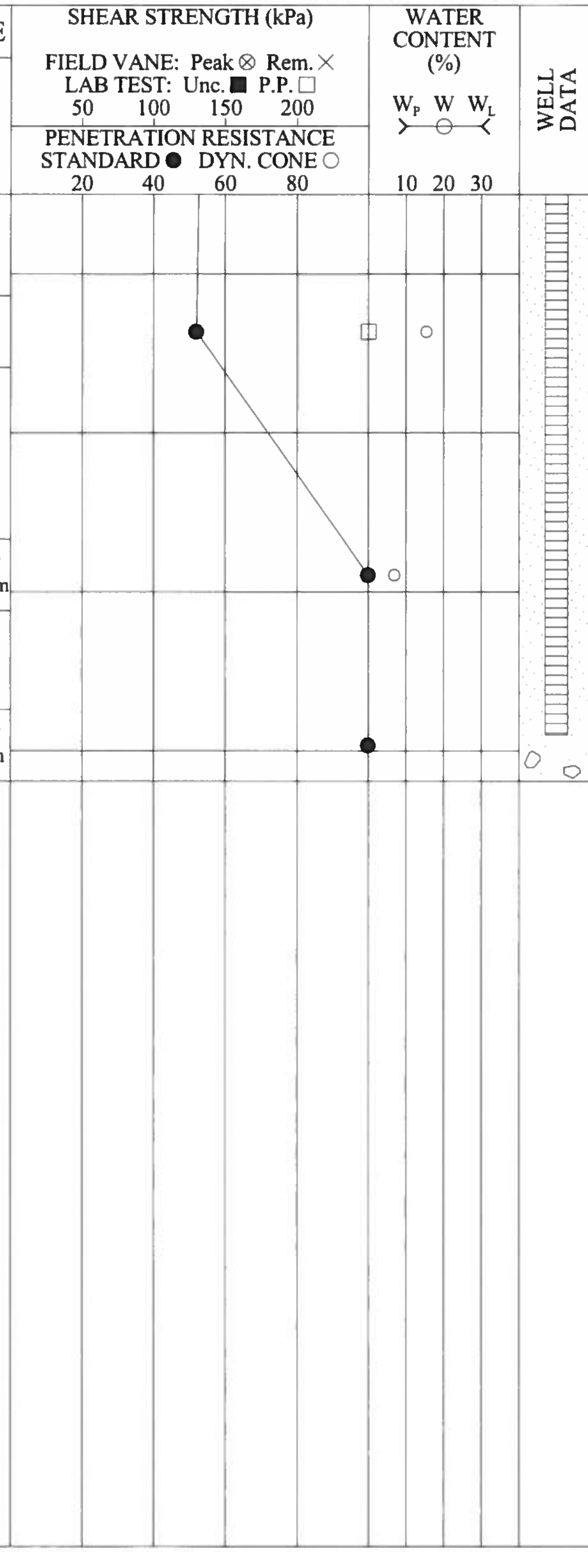
FILE No: 13-09-K01 BOREHOLE No. 3

CHUNG & VANDER DOELEN CVD ENGINEERING LTD.

Client: **Haastown Group of Companies**
Project: **Preston Spring Condominiums**
Location: **102 Fountain Street South, Cambridge, Ontario**

EQUIPMENT DATA
Machine: **Diedrich D50T**
Method: **Hollow Stem Augers**
Size: **105 mm ID**
Date: **Oct 16 / 13** TO **Oct 16 / 13**

ELEV./ DEPTH (m)	DESCRIPTION (continued)	DEPTH (m)	SAMPLE ID	TYPE	N-VALUE	REMARKS
	hard grey CLAYEY SILT TILL	9.0				saturated sand seam (9.0)
		9.5	10	SS	52	cobbles below 9.8 m depth
		11.0	11	SS	50/ 14cm	saturated sand seam (11.3)
285.19 11.70	WEATHERED BEDROCK	12.0	12	SS	50/ 0cm	
284.69 12.20	End of Borehole at Auger and Split Spoon Sampler Refusal					water level at 3.20 m in monitoring well BH3 on October 21, 2013



PROJECT MANAGER: **EYC**

CHUNG & VANDER DOELEN ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario N2H 5E1
ph. (519) 742-8979, fx. (519) 742-7739

Enclosure No.: 6
Sheet 1 of 1

FILE No: 13-09-K01 — BOREHOLE No. 4

CHUNG & VANDER DOELEN CVD ENGINEERING LTD.

Client:	**Haastown Group of Companies**
Project:	**Preston Spring Condominiums**
Location:	**102 Fountain Street South, Cambridge, Ontario**

EQUIPMENT DATA

Machine:	**Diedrich D50T**
Method:	**Hollow Stem Augers**
Size:	**105 mm ID**
Date:	**Oct 17 / 13 TO Oct 17 / 13**

SOIL LITHOLOGY

Ground Elevation: **291.17 m**

ELEV./DEPTH (m)	DESCRIPTION
	200 mm TOPSOIL loose to compact reddish brown to brown SILTY SAND moist
289.65 1.52	very stiff to hard brown to grey CLAYEY SILT TILL trace sand and gravel occ. silt/sand layers damp to moist
283.25 7.92	End of Borehole with Auger Refusal on Possible Bedrock

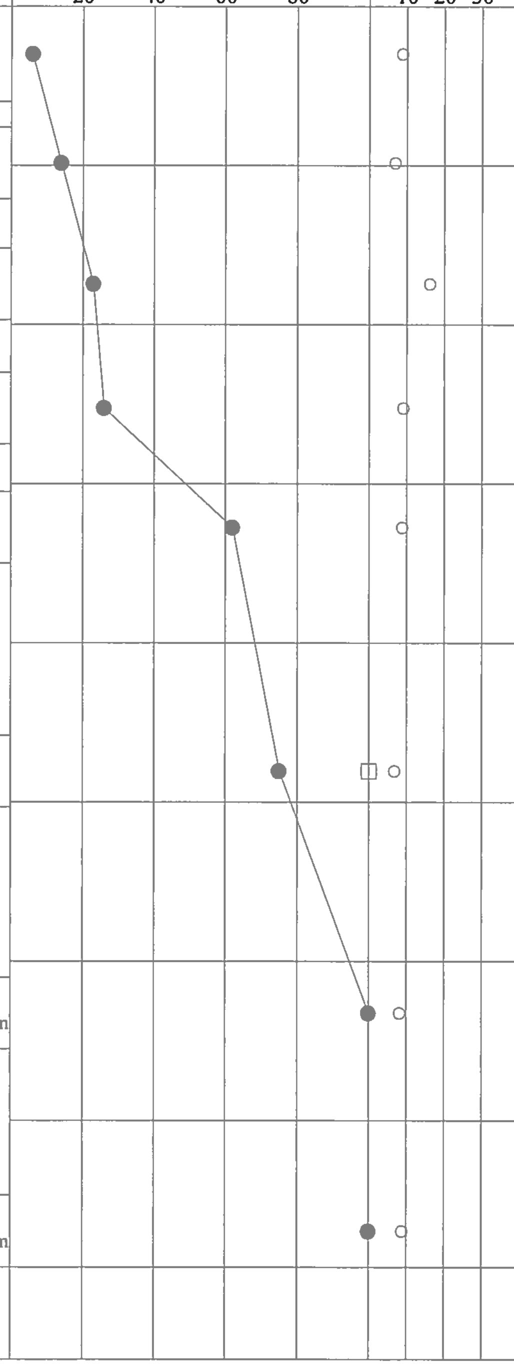
SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	6
2	SS	14
3	SS	23
4	SS	26
5	SS	62
7	SS	75
8	SS	60/15cm
9	SS	60/15cm

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

- saturated sand seam at 3.8 m depth
- grey below 4 m depth
- brown below 5.7 m depth
- water at 7.2 m depth in open borehole on completion of drilling, water at 6.4 m depth 3 hours after completion of drilling.
- cobbles and gravel at 6.7 to 7.2 m depth

PROJECT MANAGER: **EYC**

CHUNG & VANDER DOELEN ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario N2H 5E1
ph. (519) 742-8979, fx. (519) 742-7739

FILE No: 13-09-K01 BOREHOLE No. 5

CHUNG & VANDER DOELEN CVD ENGINEERING LTD.

Client: **Haastown Group of Companies**

Project: **Preston Spring Condominiums**

Location: **102 Fountain Street South, Cambridge, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Hollow Stem Augers**
Size: **105 mm ID**
Date: **Oct 17 / 13** TO **Oct 17 / 13**

SOIL LITHOLOGY

Ground Elevation: **290.87 m**

DESCRIPTION:

700 mm TOPSOIL

stiff to hard
brown to grey
CLAYEY SILT TILL
trace sand and gravel
occ. silt layers

damp to moist

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	5
2	SS	8
3	SS	13
4	SS	27
5	SS	45
7	SS	53/ 15cm
8	SS	74
9	SS	54/ 15cm

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:

saturated sand seam (at 1.0 m)

grey below 2.7 m depth

PROJECT MANAGER: **EYC**

CHUNG & VANDER DOELEN
ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario N2H 5E1
ph. (519) 742-8979, fx. (519) 742-7739

FILE No: 13-09-K01 BOREHOLE No. 5

CHUNG & VANDER DOELEN CVD ENGINEERING LTD.

Client:	Haastown Group of Companies
Project:	Preston Spring Condominiums
Location:	102 Fountain Street South, Cambridge, Ontario

EQUIPMENT DATA

Machine:	Diedrich D50T
Method:	Hollow Stem Augers
Size:	105 mm ID
Date:	Oct 17 / 13 TO Oct 17 / 13

SOIL LITHOLOGY / SAMPLE / 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

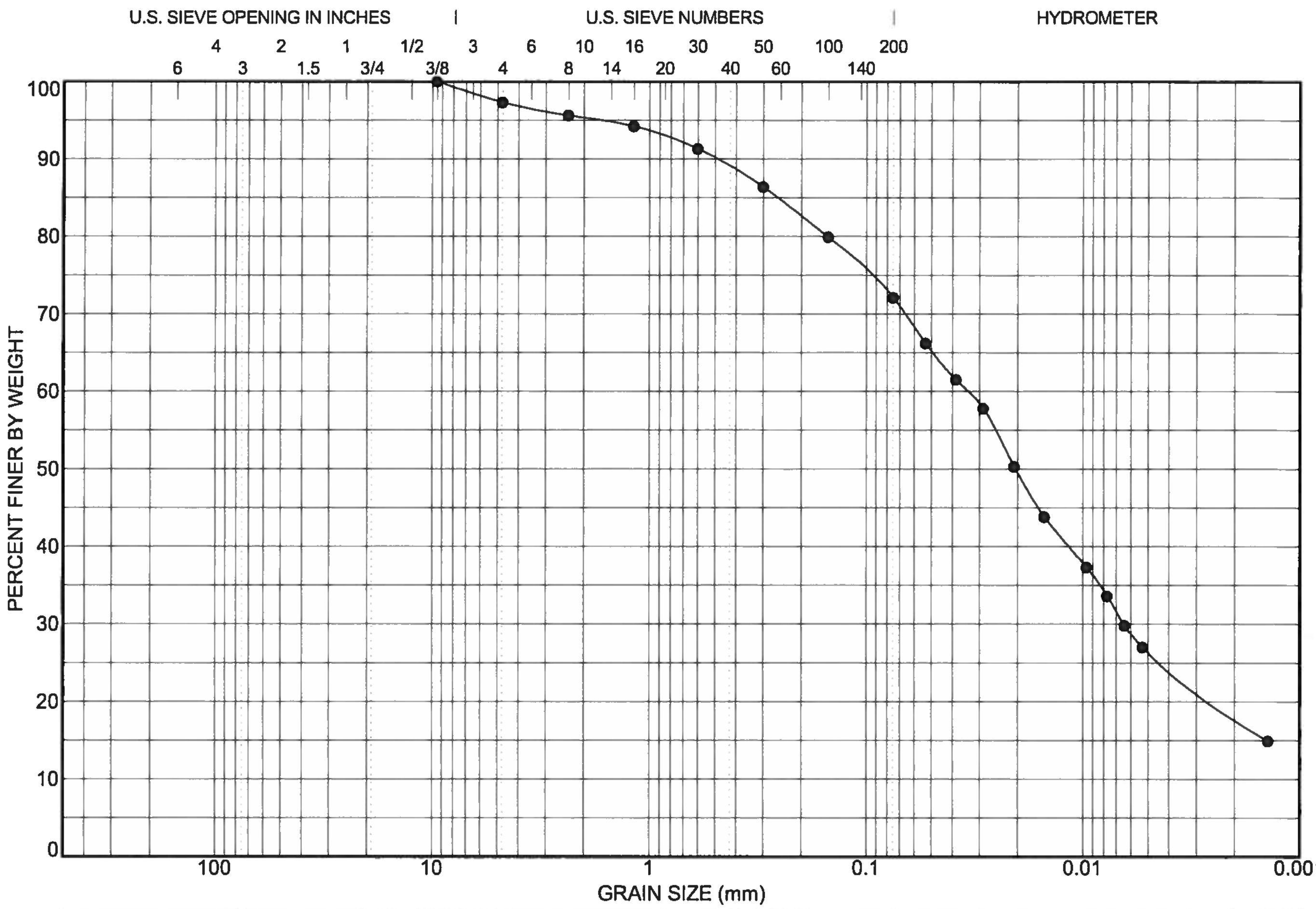
ELEV./ DEPTH (m)	DESCRIPTION (continued)	SAMPLE ID	TYPE	N-VALUE	REMARKS
	hard grey CLAYEY SILT TILL				
281.57 9.30	very dense grey to brown SAND and GRAVEL frequent cobbles moist	10	SS	50/ 8cm	cobbles (9.6 m)
		11	SS	50/ 8cm	
279.77 11.10	WEATHERED BEDROCK				
279.14 11.73	End of Borehole at Auger Refusal				no free water in open borehole on completion of drilling

DEPTH (m): 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0, 12.5, 13.0, 13.5, 14.0, 14.5, 15.0, 15.5, 16.0, 16.5

WELL DATA

PROJECT MANAGER: **EYC**

CHUNG & VANDER DOELEN ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario N2H 5E1
ph. (519) 742-8979, fx. (519) 742-7739



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

LL	PL	PI	Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
29	13				9.5	0.034	0.006		2.7	25.2	72.1	

Date: 11/15/2013
Client: Haastown Group of Companies
Contractor:
Source:
Sample No.: 3-9
Date Sampled: 16/10/2013
Sampled By: HD
Lab No.: 899
Date Tested: 23/10/2013
Type of Material: Clayey Silt Till

Sieve Size (mm)	Percent Passing	No Specifications

CHUNG & VANDER DOELEN ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario N2H 5E1
Telephone: 519-742-8979
Fax: 519-742-7739
e-mail: info@cvdengineering.com

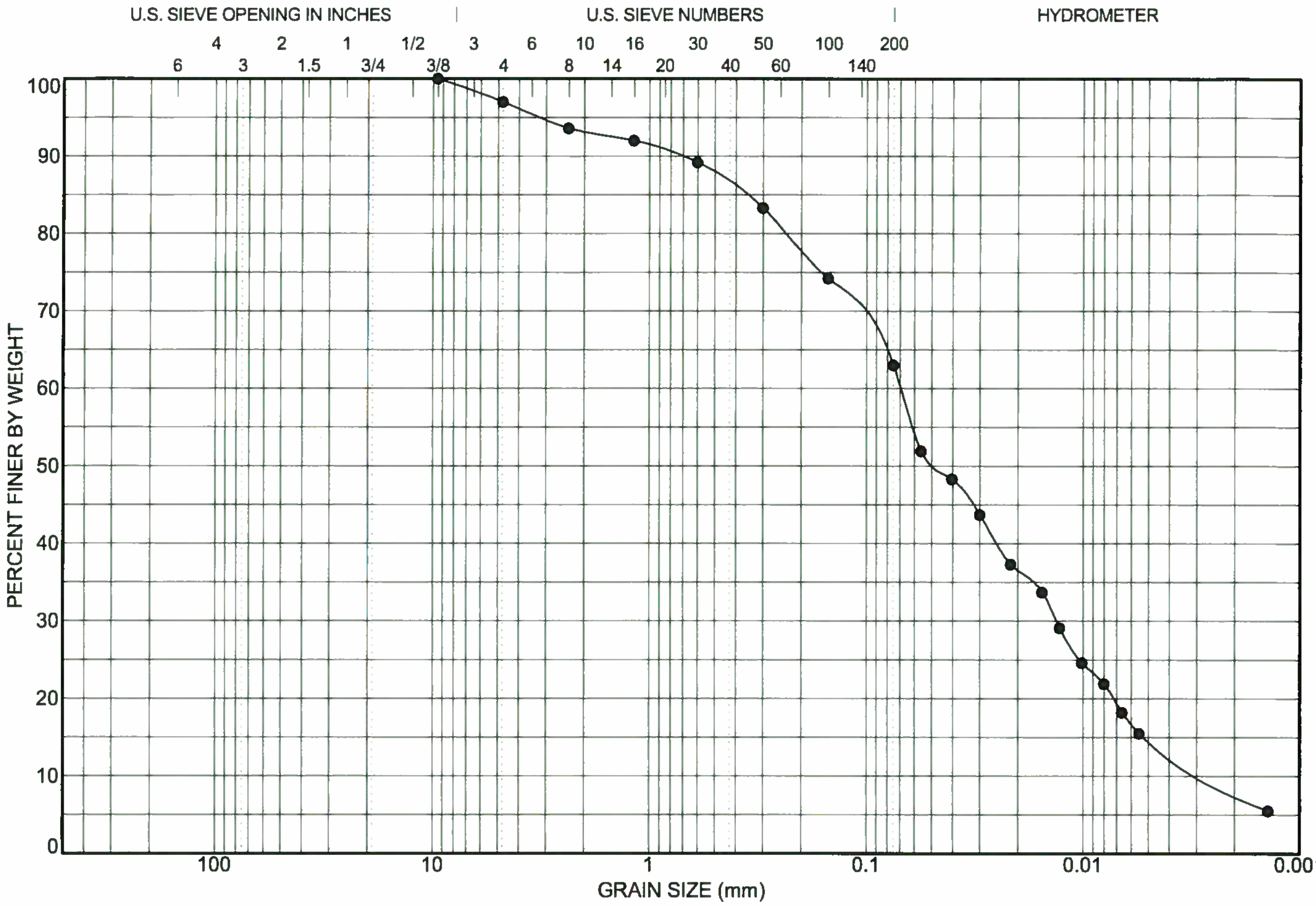
GRAIN SIZE DISTRIBUTION

Project: Preston Spring Condominiums

Location: 102 Fountain Street South, Cambridge, Ontario

File No.: 13-09-K01

Enclosure No.: 8



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

LL	PL	PI	Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
			0.98	26.72	9.5	0.069	0.013	0.003	3.0	34.0	63.0	

Date: 11/15/2013
Client: Haastown Group of Companies
Contractor:
Source:
Sample No.: 5-5
Date Sampled: 17/10/2013
Sampled By: HD
Lab No.: 901
Date Tested: 12/11/2013
Type of Material: Clayey Silt Till

Sieve Size (mm)	Percent Passing	No Specifications

CHUNG & VANDER DOELEN ENGINEERING LTD.
311 Victoria Street North
Kitchener, Ontario N2H 5E1
Telephone: 519-742-8979
Fax: 519-742-7739
e-mail: info@cvdengineering.com

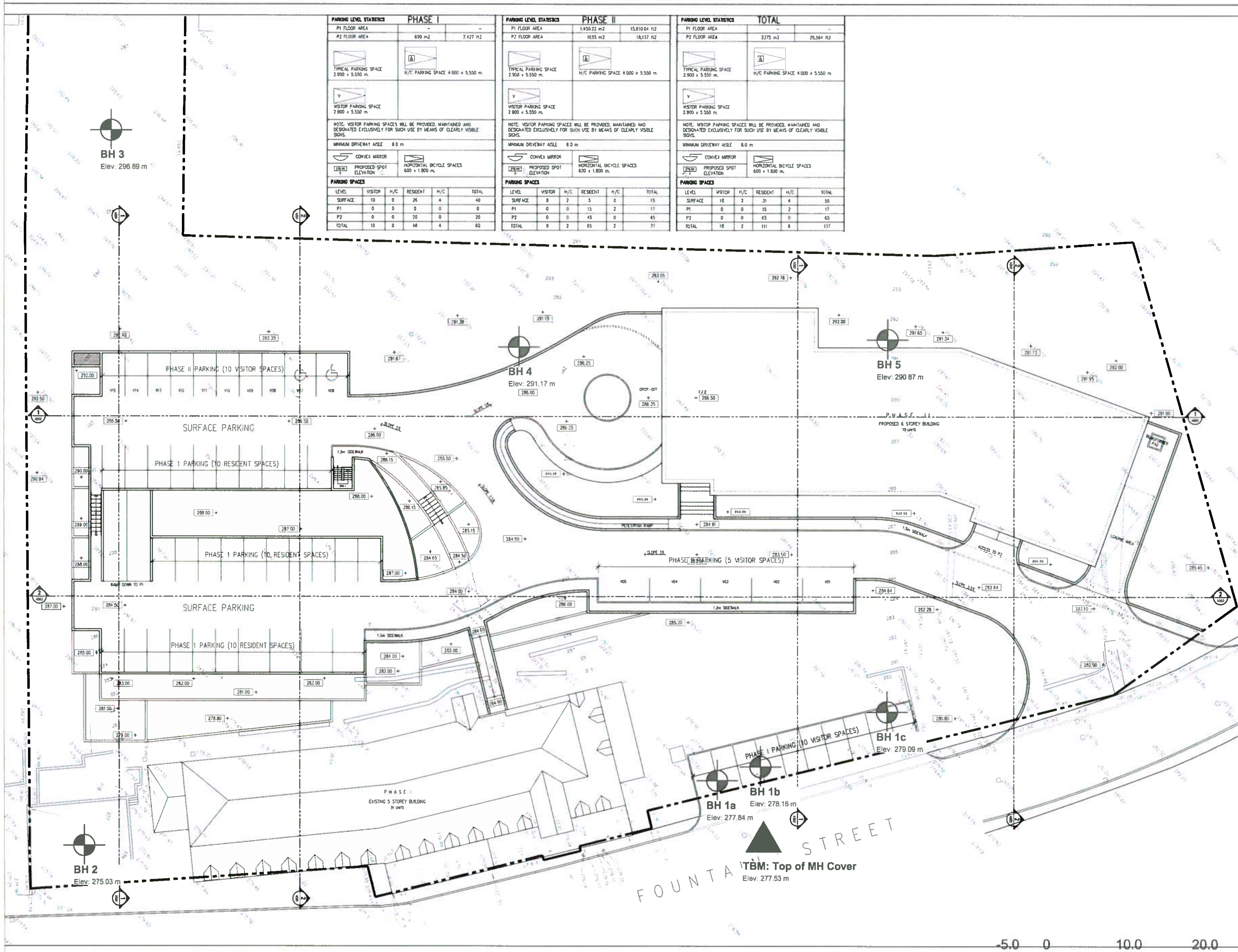
GRAIN SIZE DISTRIBUTION

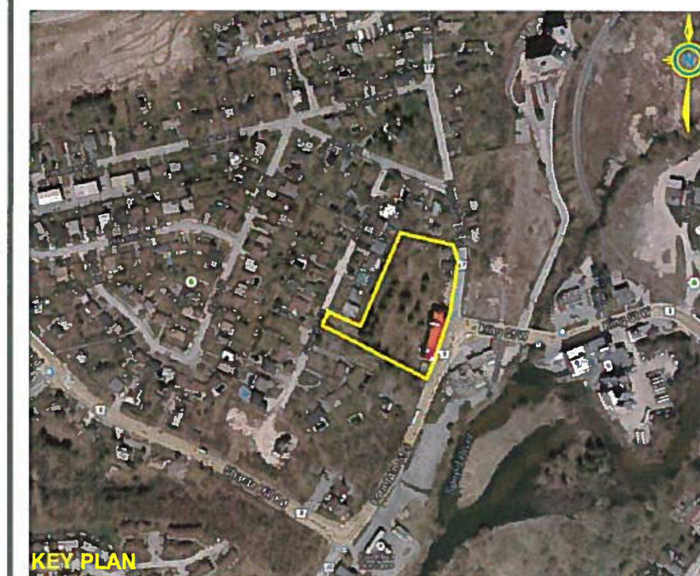
Project: Preston Spring Condominiums

Location: 102 Fountain Street South, Cambridge, Ontario

File No.: 13-09-K01

Enclosure No.: 9





TBM : Top of MH Cover
Elevation: 277.53 m

BOREHOLE LOCATION PLAN

Proposed Preston Spring Condominiums

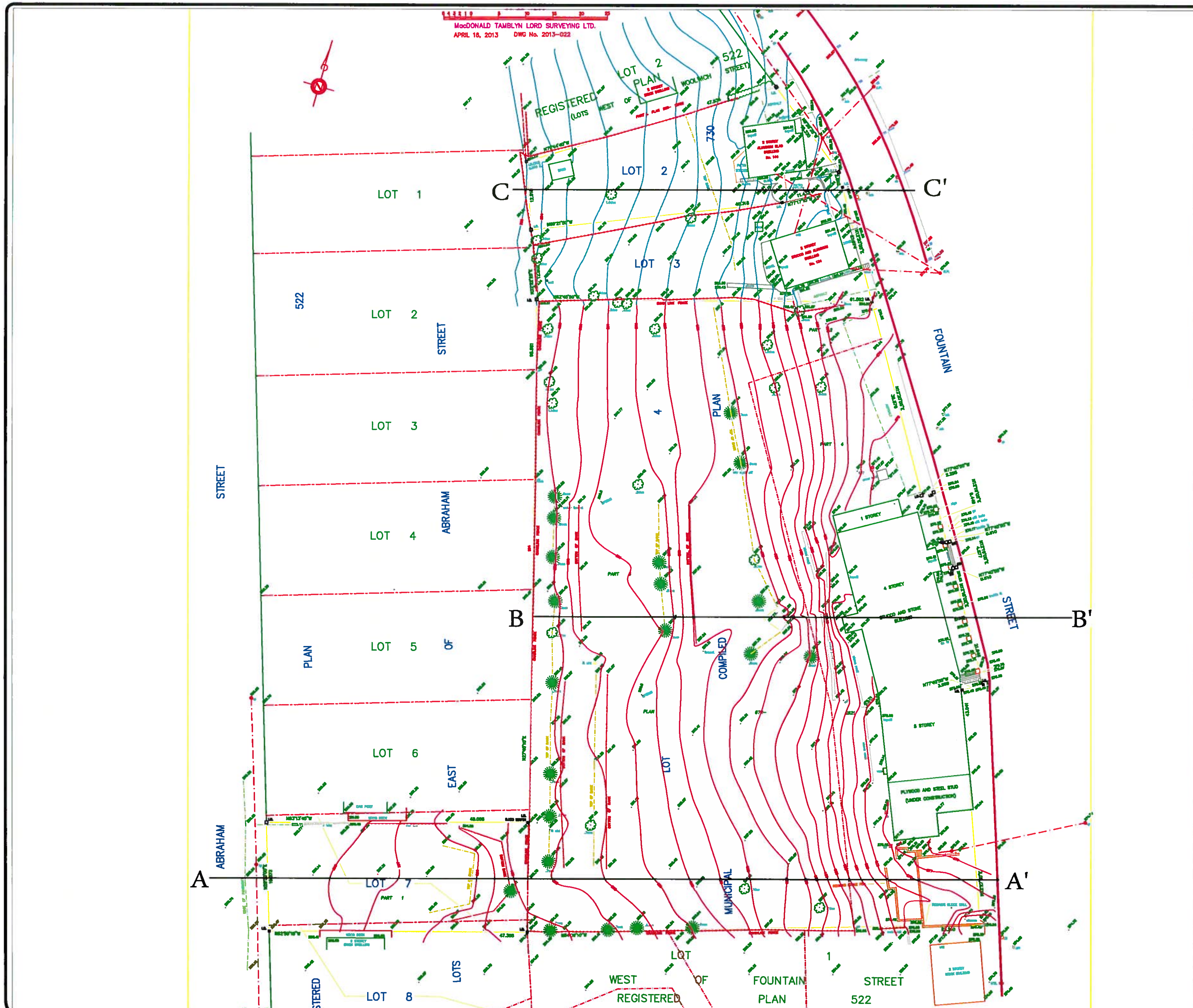
102 Fountain Street South

Cambridge, Ontario

CHUNG & VANDER DOELEN ENGINEERING LTD.

311 Victoria Street North
Kitchener, ON N2H 5E1
Phone:(519)742-8979 Fax:(519)742-7739
E-mail:info@cvdengineering.com

Drawn by: DVF	Date: January, 2014	File No.: 13-09-K01
Checked by: EYC	Scale: As Shown	Drawing No.: 1



Existing Conditions

Proposed Preston Springs Condominiums

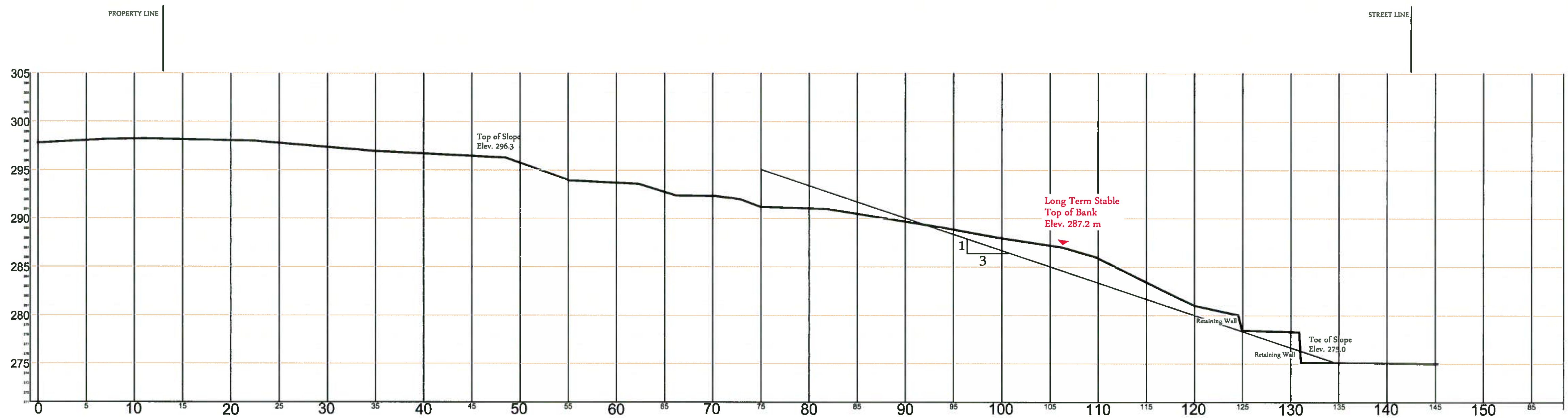
102 Fountain Street South
Cambridge, Ontario

CHUNG & VANDER DOELEN ENGINEERING LTD.

CHUNG & VANDER DOELEN
ENGINEERING LTD.

311 Victoria Street North
Kitchener, ON N2H 5E1
Phone:(519)742-8979 Fax:(519)742-7739
E-mail:info@cvdengineering.com

Drawn by: HD	Date: January 2014	File No.: 13-09-K01
Checked by: EYC	Scale: As shown	Drawing No.: 2



SECTION A - A'

CHUNG & VANDER DOELEN ENGINEERING LTD.

311 Victoria Street North
Kitchener, Ontario
N2H 5E1

Telephone: 519-742-8979
Facsimile: 519-742-7739
E-mail: info@cvdengineering.com

Project

Proposed Preston Springs Condominiums
102 Fountain Street South
Cambridge, Ontario

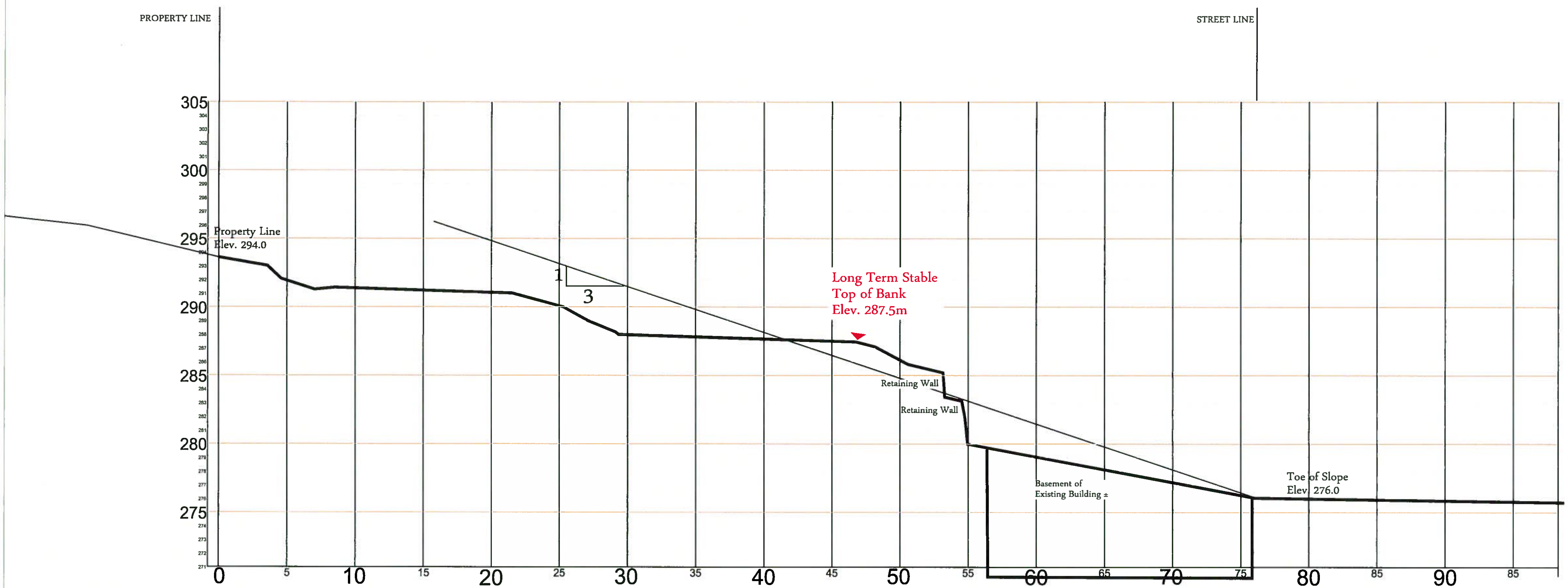
Drawing

SECTION PROFILE A - A'

REV. NO.	DESCRIPTION	DATE

Drawn by: HD
Checked by: EYC
File No.: 13-09-K01
Date: January, 2014
Scale: Not to Scale

Drawing No.
3



SECTION B - B'

CHUNG & VANDER DOELEN ENGINEERING LTD.

CHUNG & VANDER DOELEN
ENGINEERING LTD.

311 Victoria Street North
Kitchener, Ontario
N2H 5E1

Telephone: 519-742-8979
Facsimile: 519-742-7739
E-mail: info@cvdengineering.com

Project

Proposed Preston Springs Condominiums
102 Fountain Street South
Cambridge, Ontario

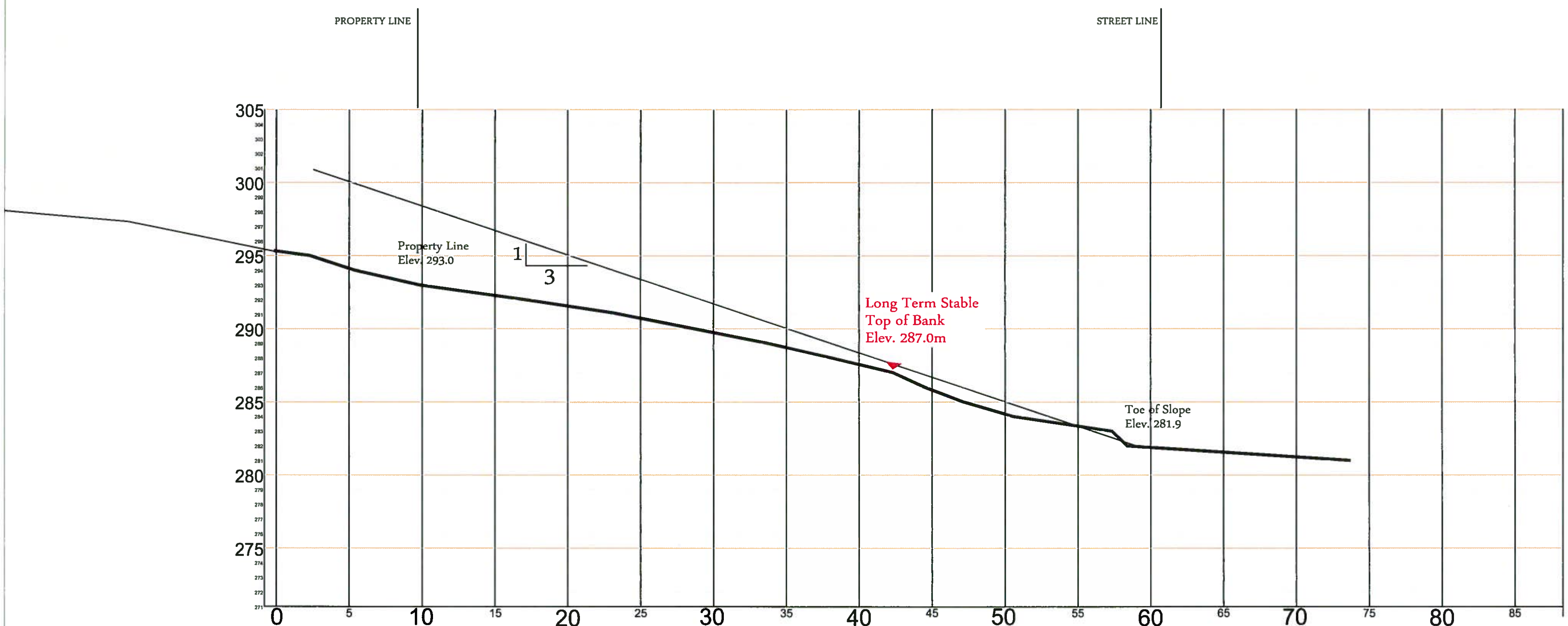
Drawing

SECTION PROFILE B - B'

REV. NO.	DESCRIPTION	DATE

Drawn by: HD
Checked by: EYC
File No.: 13-09-K01
Date: January, 2014
Scale: Not to Scale

Drawing No.
4



SECTION C - C'

CHUNG & VANDER DOELEN ENGINEERING LTD.

311 Victoria Street North
Kitchener, Ontario
N2H 5E1

Telephone: 519-742-8979
Facsimile: 519-742-7739
E-mail: info@cvdengineering.com

Project

Proposed Preston Springs Condominiums
102 Fountain Street South
Cambridge, Ontario

Drawing

SECTION PROFILE C -C'

REV. NO.	DESCRIPTION	DATE

Drawn by: HD
Checked by: EYC
File No.: 13-09-K01
Date: January, 2014
Scale: Not to Scale

Drawing No.
5