CHUNG & VANDER DOELEN
ENGINEERING LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
384 George Street
Port Stanley, Ontario

SUBMITTED TO:
Caliber Contracting
130 Shearson Crescent
Cambridge, Ontario
N1T 1J4

ATTENTION:
Mr. Matt Morningstar

FILE NO / G22383 / November 3, 2022

CHUNG & VANDER DOELEN
ENGINEERING LTD.

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

November 3, 2022
File No.: G22383

Caliber Contracting
130 Shearson Crescent
Cambridge, Ontario
N1T 1J4

Attention: Mr. Matt Morningstar

RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
384 George Street, Port Stanley, Ontario

We take pleasure in enclosing one (1) copy of our Geotechnical Investigation Report prepared for 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

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LIST OF ENCLOSURES

1.0 INTRODUCTION

CHUNG & VANDER DOELEN ENGINEERING LTD. (CVD) has been retained by Caliber Contracting to conduct a geotechnical investigation for a proposed residential development to be constructed at 384 George Street in Port Stanley, Ontario.

It is understood that the existing residential dwellings and cottages will be demolished, and the site will be redeveloped with eight (8) units of condos. The proposed condo units will be constructed in the northern and middle portions of the site as a buffer land will be required due to the existing slope located at the back of the property. The site grading plan and finished floor elevations were not available at the time of reporting. However, no basements are anticipated due to the shallow groundwater encountered at the site.

The purpose of the investigation was to determine the subsurface conditions at the site and, based on the findings, make geotechnical recommendations for the foundations, floor slabs and pavement areas. Estimates of infiltration rates of the onsite soil deposits will be provided. The stability of the existing slope at the southern (rear) portion of the site (and beyond) was assessed, and the setback for the proposed development from the toe of the slope is recommended.

2.0 FIELD AND LABORATORY WORK

Six (6) boreholes were advanced to depths between 6.55 and 9.15 m below existing grade on February 22, 2022. The borehole locations are illustrated on the Borehole Location Plan, Drawing No. 1, appended.

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. Public and private utilities were located prior to commencing the field investigation program.

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.

Dynamic Cone Penetration Tests (DCPTs) were also conducted at Boreholes 2 to 4 to further explore and confirm the subsurface conditions. 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 and consistency of the soil strata have been inferred from these test results.

Soil samples collected during the borehole investigation program were examined in the field and subsequently brought to CVD’s laboratory for tactile examination. Moisture content determination on all retrieved soil samples was performed. In addition, two (2) grain size distribution analyses were conducted on representative samples of the encountered soil deposits.

Groundwater conditions were monitored during sampling and upon removal of the drilling augers at all borehole locations.

The borehole locations and associated ground surface elevations were surveyed by CVD for the purpose of this report using a Leica ICON GPS 70T Rover Global Navigation Satellite System (GNSS) Receiver.

3.0 EXISTING SITE CONDITIONS

The site is bound to the north by George Street, to the west and east by residential houses, and to the south by a hill which is covered by mature trees. The site is currently occupied by two (2) residential dwellings and six (6) cottages with unpaved driveways around the existing buildings. Two (2) water ponds exist in the southeastern portion of the site. The remainder of the site is covered by grass and occasional mature trees.

The site is relatively flat and gently declines in a northerly direction towards George Street. Ground surface elevations at the borehole locations ranged between 179.67 and 180.09 m.

4.0 SUBSURFACE CONDITIONS

The subsurface conditions encountered at the boreholes are detailed on the Borehole Log Sheets, Enclosures 1 to 6 of this report. The following notes are intended to amplify and comment on the subsurface data.

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.

4.1 Topsoil and Pavement

Topsoil was encountered at the ground surface at Boreholes 1, 3, 5 and 6 with measured thicknesses between 100 and 200 mm.

Granular base materials were encountered at the ground surface at Boreholes 2 and 4 with measured thicknesses of 350 and 400 mm, respectively.

4.2 Upper Fine-Grained Soil Deposits

A layer of fine-grained soils varying in composition from sandy silt; to silt; to clayey silt; to organic silt was encountered below the surficial topsoil or pavement structure at all six (6) borehole locations and extended to depths between 2.15 and 7.0 m below existing grade. The upper fine-grained soil deposits generally contained trace to some organics and/or peat pockets/layers. Rootlets, wood and/or shell fragments were observed within the deposits at Boreholes 1 to 4. Results of one (1) grain size distribution analysis selected from the silt deposit are shown graphically on Enclosure 7.

Standard Penetration Testing within the upper fine-grained soil deposits generally yielded "N"-values between 0 and 14 blows per 300 mm, indicating a variable very loose to compact compactness condition or a very soft to stiff consistency. The undrained shear strength of the clayey deposit retrieved from Borehole 3 was 24 kPa. The undrained shear strength of the rest of the clayey soils was unable to determine due to the low consistency and saturated moisture condition. Natural moisture contents were measured between 19 and 65%, indicating a saturated moisture condition. Elevated moisture contents reflect the presence of organic matters.

4.3 Sand and Gravel

The upper saturated silt deposit at Borehole 1 was underlain by a layer of sand and gravel. The coarse granular deposit extended to a depth of 6.55 m below existing grade, the maximum depth of exploration. Trace silt was encountered within the deposit.

The SPT "N"-values measured within the coarse granular deposit were 12 and 14 blows per 300 mm, indicating a compact compactness condition. Based on the field observation, the sand and gravel deposit exhibited a saturated moisture condition.

4.4 Clayey Silt Till

Clayey silt till deposit was encountered underlying the fine-grained soil deposits at Boreholes 2, 3, 5 and 6. The cohesive deposit at Boreholes 2, 3 and 5 extended to at least the borehole termination depths between 6.55 and 8.1 m below existing grade. The clayey silt till extended to a depth of 2.9 m below existing grade at Borehole 6. The cohesive deposit contained trace gravel and sand. Occasional silt layers and seams were encountered within the cohesive deposit at Boreholes 5 and 6, respectively. A grain size distribution analysis was conducted on the representative sample retrieved from Borehole 6, and the results are illustrated graphically on Enclosure 8.

The SPT "N"-values measured within the cohesive deposit ranged from 11 to 30 blows per 300 mm of penetration. The undrained shear strength obtained on the retrieved samples ranged from 60 to 168 kPa. Based on the above test results, the cohesive deposit is considered to have a stiff to hard consistency condition. Natural moisture contents were measured between 16 and 25%, thus indicating a moist moisture condition.

4.5 Lower Silt

The clayey silt till at Borehole 6 was further underlain by a layer of silt deposit, extending to the borehole termination depth of 6.55 m below existing grade. Some sand, trace clay and occasional clayey seams were encountered within the deposit.

The SPT "N"-values measured within the cohesive deposit ranged from 51 blows per 300 mm of penetration to 50 blows per 125 mm of penetration, indicating a very dense compactness condition. Natural moisture contents were measured between 15 and 19%, indicating a moist to wet moisture condition.

4.6 Dynamic Cone Penetration Testing

DCPTs were conducted at Boreholes 2 to 4 below the conventionally drilled and sampled cohesive deposits to depths between 7.9 and 9.15 m below existing grade in order to further explore and confirm the subsurface conditions. The DCPTs yielded values between 13 and 80 blows per 300 mm of penetration inferred a stiff to hard consistency condition.

4.7 Groundwater

Groundwater conditions were monitored during sampling and upon removal of the drilling augers at all borehole locations. Upon withdrawal of drilling augers, groundwater was measured at depths between 0.1± and 0.6± m below existing grade, corresponding to elevations between 179.3± and 179.8± m.

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

5.0 DISCUSSION AND RECOMMENDATIONS

It is understood that the existing residential dwellings and cottages will be demolished, and the site will be redeveloped with eight (8) units of condos. The proposed condo units will be constructed in the northern and middle portions of the site as a buffer land will be required due to the existing slope located at the back of the property. The site grading plan and finished floor elevations were not available at the time of reporting. However, no basements are anticipated due to the shallow groundwater encountered at the site.

In general, the surficial topsoil and pavement structure at all six (6) borehole locations were underlain by a layer of very loose to compact or very soft to stiff fine-grained soil deposits with organic matters which extended to depths between 2.15 and 7.0 m below existing grade. Compact sand and gravel, stiff to hard clayey silt till and very dense silt deposits were encountered beneath the upper fine-grained soil deposits and extended to at least the borehole termination depths between 6.55 and 8.1 m below existing grade. DCPTs conducted below the conventionally drilled and sampled cohesive deposits at Boreholes 2 to 4 to depths between 7.9 and 9.15 m below existing grade yielded values between 13 and 80 blows per 300 mm of penetration which inferred a stiff to hard consistency condition.

Groundwater was measured at depths between 0.1± and 0.6± m below existing grade upon withdrawal of the drilling augers, corresponding to elevations between 179.3± and 179.8± m. It is noted that the groundwater table will fluctuate seasonally and in response to major weather events.

5.1 Foundation Options

If conventional strip and spread footing foundations are to be used to support the proposed buildings, the footings will need to be cast on the competent native soils which can be designed using a Geotechnical Reaction at SLS of 150 kPa (3000 psf). The Factored Geotechnical Resistance at ULS is 250 kPa (5000 psf).

The following table summarizes the highest founding level and elevation for conventional footing foundations on competent native soils encountered at each relevant borehole location:

Borehole No.	Existing Ground Elevation (m)	Highest Founding Depth (m)	Highest Founding Elevation (m)
1	179.67	5.67	174.00
2	179.84	7.04	172.80
3	179.74	6.04	173.70
4	179.91	6.61	173.30

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 maximum total and differential settlements of footings designed to the above recommended soil bearing pressure are expected to be less than 25 and 20 mm, respectively, and these are considered tolerable for the structure being contemplated.

Footings should be founded below any existing fill materials, previous foundation levels and existing adjacent utility trenches on competent native undisturbed soils, unless if engineered fill has been properly constructed. Spacing between adjacent footing steps should not be steeper than 10H to 7V.

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.

The subgrade soils at the founding grades could potentially be disturbed during construction, especially under wet conditions. In this regard, a 50 mm thick protective concrete slab could be poured and allowed to set on the prepared subgrade to further protect it from disturbance by construction traffic and the elements, if so required.

5.2 Site Grading and Engineered Fill Construction

Engineered fill can be used to replace the existing very loose to compact and very soft to stiff fine-grained soil deposits with organic matters in order to suitably support the future building foundations and floor slabs. Consequently, the footings can be placed at a higher, more desirable elevation.

Wherever beneath the future foundation and floor slabs, imported well-graded sand and gravel (similar to the gradational requirements of OPSS Granular B Type I) is recommended to be used to construct engineered fill. The moisture content of the soils should be within 3% below the optimum moisture content in order to achieve the specified degrees of compaction.

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 work should be carried out during relatively dry weather as the predominant clayey silt soil is sensitive to wetting and are difficult to handle when wet. Therefore, earthworks should be scheduled in the drier summer months. 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.

Engineered fill is to be constructed in accordance with the following procedures in order to support the future foundations and floor slabs, if adopted:

1. All topsoil, native loose/soft soils deposits, organic deposits and any otherwise deleterious materials are to be removed from the proposed building areas to expose the underlying competent native soil deposits;

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

3. It is anticipated that wet/saturated subgrade condition will be encountered. The initial lift to raise the grade will require coarse pit run sand and gravel, 450 to 600 mm thick and be statically rolled to stabilize and “bridge” the prepared subgrade;

4. The required grades can then be achieved by placing imported well-graded sand and gravel fill in maximum 300 mm thick lifts and compacted to at least 100% Standard proctor maximum dry density (SPMDD) beneath future foundations and compacted to at least 95% SPMDD beneath floor slabs, respectively, under controlled and supervised conditions. The moisture content of fill soil to construct engineered fill should be within 3% drier of the optimum moisture condition 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 receiving subgrade;

6. 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 degree of compaction has been achieved.

Footings constructed on approved engineered fill 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.

Vibration could be generated from various construction equipment, such as compactors and rollers which could be harmful to potential surrounding structures and buildings during construction. 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 sensitivity of any potential 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. A pre-construction condition survey could be conducted to document the condition of the existing structures within the possible zone of influence.

5.3 Helical Piles

An alternative to deep excavation for footing foundations is to use helical pile to transfer the building loadings to the lower competent soil strata. Helical piles are installed with minimal ground disturbance and require little to no excavation. The bearing capacity of a helical pile is a function of soil parameters, installation depth, and pile type. Each pile type has as an assigned maximum installation torque corresponding to an ultimate bearing capacity. Helical piles may be installed at a torque lower than the maximum installation torque if lower bearing capacities are desired.

Helical piles founded below the upper very loose to compact and very soft to stiff fine-grained soil layers within the competent soil deposits contacted at depths between 7.0± and 8.0± m below existing grade can be used to provide support for the future building foundations and floor slabs. Based on the results of the boreholes, Helical Pull-Down Micropiles (3 tier helixes of 200, 250 and 300 mm diameter and a 50 mm square shaft installed with grout) will typically provide service loads of up to 490 kN (110 kips) per pile. Helical Pull-Down Micropiles are installed by a specialty contractor to their detailed design and higher capacities may be possible.

Please refer to a specialty contractor for detailed design and installation procedures. A soil friction angle of 30° and unit weight of 21 kN/m^3 can be used for the design of the helical piles. Submerged unit weight should be used below the groundwater table.

Due to the presence of loose/soft and organic soil deposits, the floor slabs will need to be structurally reinforced and supported by the helical piles.

5.4 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 soil 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 D as per OBC Table 4.1.8.4.A (Page B4-24).

5.5 Access Driveway and Paved Parking Area

Based on the results of the field work, the predominant subgrade materials at the site will consist of fine-grained native soils with organic matters. 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 Asphalt Pavement (mm)	Heavy Duty Asphalt Pavement (mm)
Asphaltic Concrete HL3 Asphaltic Concrete HL8	40 40	40 50
Granular "A" Base	150	150
Granular "B" Sub-base	450	600

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. Alternatively, geotextile/geogrids can be used to strengthen the soil subgrade and the pavement structure.

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 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.

Considering the shallow groundwater table, sub-drains should be installed along the perimeter and in the interior of the parking lot to enhance the service life of the pavement.

5.6 On Site Infiltration

It is understood that the potential for at-source storm water management feature(s) is to be considered at the site.

The top of the infiltration feature(s) should be located below the footing drain/weeper and at least 5 m away from the proposed building footprints. The base of infiltration feature(s) should be located at least 1.0 m above the groundwater table and that a minimum of 15 mm/hr is required.

Grain size distribution analyses were conducted on representative samples of the native deposits and the results are graphically presented on Enclosures 7 and 8. Based on the results of grain size analyses and our experience, the hydraulic conductivity and infiltration rate of the native soil types encountered at the boreholes are estimated and provided in the following table:

MATERIAL	HYDRAULIC CONDUCTIVITY (K) (cm/sec)	INFILTRATION RATE (mm/hr)
Silt some sand, trace clay and organics (Enclosure 7)	1×10^{-6}	1
Clayey Silt Till trace sand, occ. silt seams (Enclosure 8)	$< 1 \times 10^{-6}$	< 1

Based on the results of the shallow groundwater table and the hydraulic conductivity of the native soil deposits, storm water management feature is **not** considered feasible at the site.

5.7 Slope Stability Assessment

5.7.1 Slope Condition and Inspection

A natural slope is located at the southern (rear) portion of the property. A slope condition reconnaissance/inspection was carried out on February 8, 2022 during the process of completing the private locates. The slope surface was fully covered by snow at the time of site reconnaissance. However, mature trees and shrubs were observed on the slope surface. Most of the trees were noted to be growing vertically, indicating minimal past slope instability. Selected photographs of the site and slope condition are shown in Appendix D.

The Ontario Ministry of Natural Resources Technical Guide provides a Slope Stability Rating Chart in Table 4.2, which has been used in assessing the slope condition. The completed Slope Stability Rating Chart which can be found in Appendix E, shows a rating value of 23, indicating low potential of instability.

According to the Flood Risk Topographic Mapping (dated 1985) obtained from Kettle Creek Conservation Authority (KCCA), the top of slope ranges in elevations between 205± and 208± m, and the bottom of slope ranges in elevations between 181± and 185± m. Thus, the slope height is approximately 20± to 25± m. The average slope inclination ranges between 2.0± to 3.2±H to 1V. Seepage zones, active slumping and erosions could not be observed as the ground and slope were covered by snow. However, active slumping and erosion are not expected since the slope surface is fully covered by shrubs and mature trees.

5.7.2 Slope Stability Analysis

A representative slope condition (Section A-A' in Appendix B) was selected from the existing slope to perform a slope stability analysis. The soil parameters used in the slope stability analysis were determined based on the field and laboratory test results of the present investigation and our experience with similar soil types. The selected "effective stress - drained condition" soil parameters and groundwater condition were used to perform the stability analyses with the use of modeling software Slide.

The following soil strength parameters were used in the stability analysis:

Soil Type	Unit Weight (kN/m^3)	Friction Angle (ϕ^0)	Cohesion (C', kPa)
very soft to firm/loose Silt and Organic Silt	18	25	0
stiff to hard Clayey Silt Till	20	29	10

The slope analysis was conducted based on a factor of safety of 1.5, and the results of the analysis are shown graphically on Figures No. 1 and 2 (Appendix F).

The bottom and top of slope were set at elevations 181 and 205 m, respectively. The slope inclination was at approximately 2.6H to 1V. The analysis shows that the overall slope has a factor of safety of greater than 1.5. Therefore, the natural slope is deemed stable.

5.7.3 Setback Considerations for Site Development

Reference is made to Appendix G, for the Erosion Hazard Limit of the Natural Hazards Training Manual (Policy 3.1). It is noted that there are three (3) setback allowances for developing adjacent to a natural slope, namely the Toe Erosion Allowance, Stable Slope Allowance, and the Erosion Access Allowance.

Since the site is not close to any watercourse and the existing slope is considered stable based on the analysis, the required setback will only consist of the Erosion Access Allowance which is 6 m (Appendix G). It is noted that a 10 m buffer land has been considered during the planning stage between the toe of the slope and the proposed development. Therefore, no additional setback due to Erosion Access Allowance is required for the proposed development.

5.8 Handling of Excess Soil

Excess soil may be generated and removed off-site during the construction activities associated with the proposed site works. The management of excess soil is now governed by O.Reg. 406/19, MECP document entitled "On-Site and Excess Soil Management Regulation". In accordance with the regulation, the Project Leader is responsible for the handling, storage, reuse, transportation, and removal of all soil. To support off-site removal of excess soil, the following is required:

- Planning Documentation
  - Assessment of Past Use
  - Sampling and Analysis Plan
  - Excess Soil Characterization Report
  - Excess Soil Destination Report
- Tracking
- Registry
- Record Keeping

CVD can provide further assistance on this matter as the project develops.

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.

LICENSED PROFESSIONAL ENGINEER
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100528326
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PROVINCE OF ONTARIO

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Nandou Zhao, M.Eng., P.Eng.
Geotechnical Engineer

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

BOREHOLE AND SLOPE SECTION LOCATION PLAN

KEY PLAN **SOURCE:** Google Earth

LEGEND

Borehole Location

Base Drawing Ref.: Flood Risk Topographic Mapping, dated 1985.
Elev. Ref.: Ground surface elevation at the borehole locations obtained with a Leica CC80 Field Tablet and an ICON GPS 70T (Tilt Compensated Network RTK Rover) Global Navigation Satellite System (GNSS) Receiver.

BOREHOLE AND SLOPE SECTION LOCATION PLAN

Proposed Residential Development

384 George Street
Port Stanley, Ontario

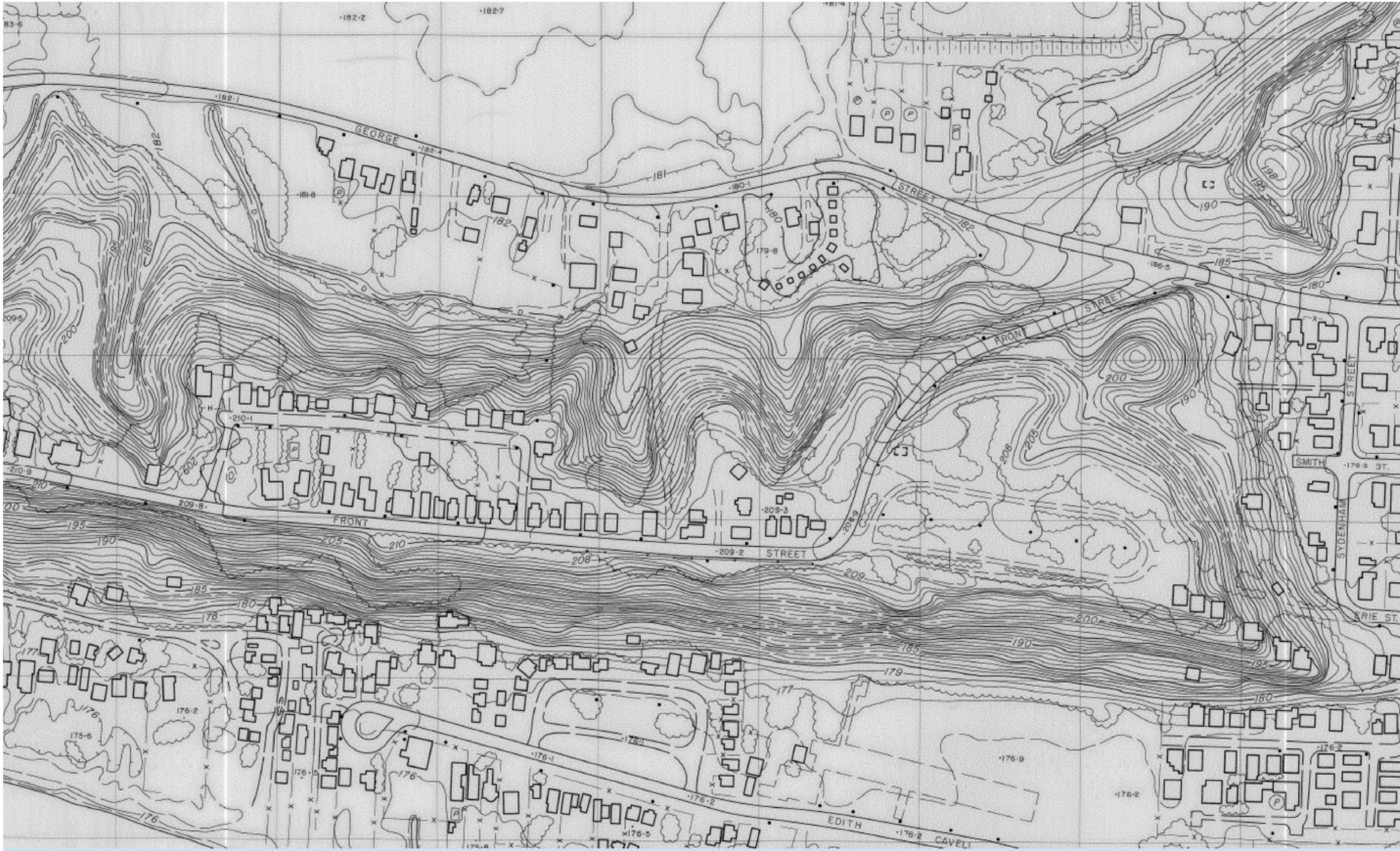
CHUNG & VANDER DOELEN
ENGINEERING LTD.

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

Drawn By: NZ	Date: October 2022	File No.: G22383
Checked By: EYC	Scale: 1:2000	Drawing No.: 1

APPENDIX C

FLOOD RISK TOPOGRAPHIC MAPPING (DATED 1985)
OBTAINED FROM
KETTLE CREEK CONSERVATION AUTHORITY


GEORGE
STREET
FRONT
STREET
FRONT
STREET
SMITH
ST.
SYDENHAM
STREET
ERIE ST.
EDITH
CAVELL

APPENDIX D

SELECTED SITE PHOTOGRAPHS

Central Portion of the Existing Slope

Eastern Portion of the Existing Slope (based on true north direction)

Western Portion of the Existing Slope (based on true north direction)

APPENDIX E

SLOPE STABILITY RATING CHART

TABLE 4.2 - SLOPE STABILITY RATING CHART

Site Location:	384 George Street, Port Stanley	File No.	G22383
Property Owner:	Mr. Matt Morningstar	Inspection Date:	February 8, 2022
Inspected By:	Nandou Zhao	Weather:	-4 degrees C, Mainly Sunny

1. SLOPE INCLINATION

	degrees	horiz. : vert.		
a)	18 or less	3 : 1 or flatter	0	○
b)	18 - 26	2 : 1 to more than 3 : 1	6	◉
c)	more than 26	steeper than 2 : 1	16	○

2. SOIL STRATIGRAPHY

a)	Shale, Limestone, Granite (Bedrock)	0	○
b)	Sand, Gravel	6	○
c)	Glacial Till	9	◉
d)	Clay, Silt	12	○
e)	Fill	16	○
f)	Leda Clay	24	○

3. SEEPAGE FROM SLOPE FACE

a)	None or Near bottom only	0	◉
b)	Near mid-slope only	6	○
c)	Near crest only or, From several levels	12	○

4. SLOPE HEIGHT

a)	2 m or less	0	○
b)	2.1 to 5 m	2	○
c)	5.1 to 10 m	4	○
d)	more than 10 m	8	◉

5. VEGETATION COVER ON SLOPE FACE

a)	Well vegetated; heavy shrubs or forested with mature trees	0	◉
b)	Light vegetation; Mostly grass, weeds, occasional trees, shrubs	4	○
c)	No vegetation, bare	8	○

6. TABLE LAND DRAINAGE

a)	Table land flat, no apparent drainage over slope	0	◉
b)	Minor drainage over slope, no active erosion	2	○
c)	Drainage over slope, active erosion, gullies	4	○

7. PROXIMITY OF WATERCOURSE TO SLOPE TOE

a)	15 metres or more from slope toe	0	◉
b)	Less than 15 metres from slope toe	6	○

8. PREVIOUS LANDSLIDE ACTIVITY

a)	No	0	◉
b)	Yes	6	○

SLOPE INSTABILITY RATING VALUES INVESTIGATION RATING SUMMARY **TOTAL** 23

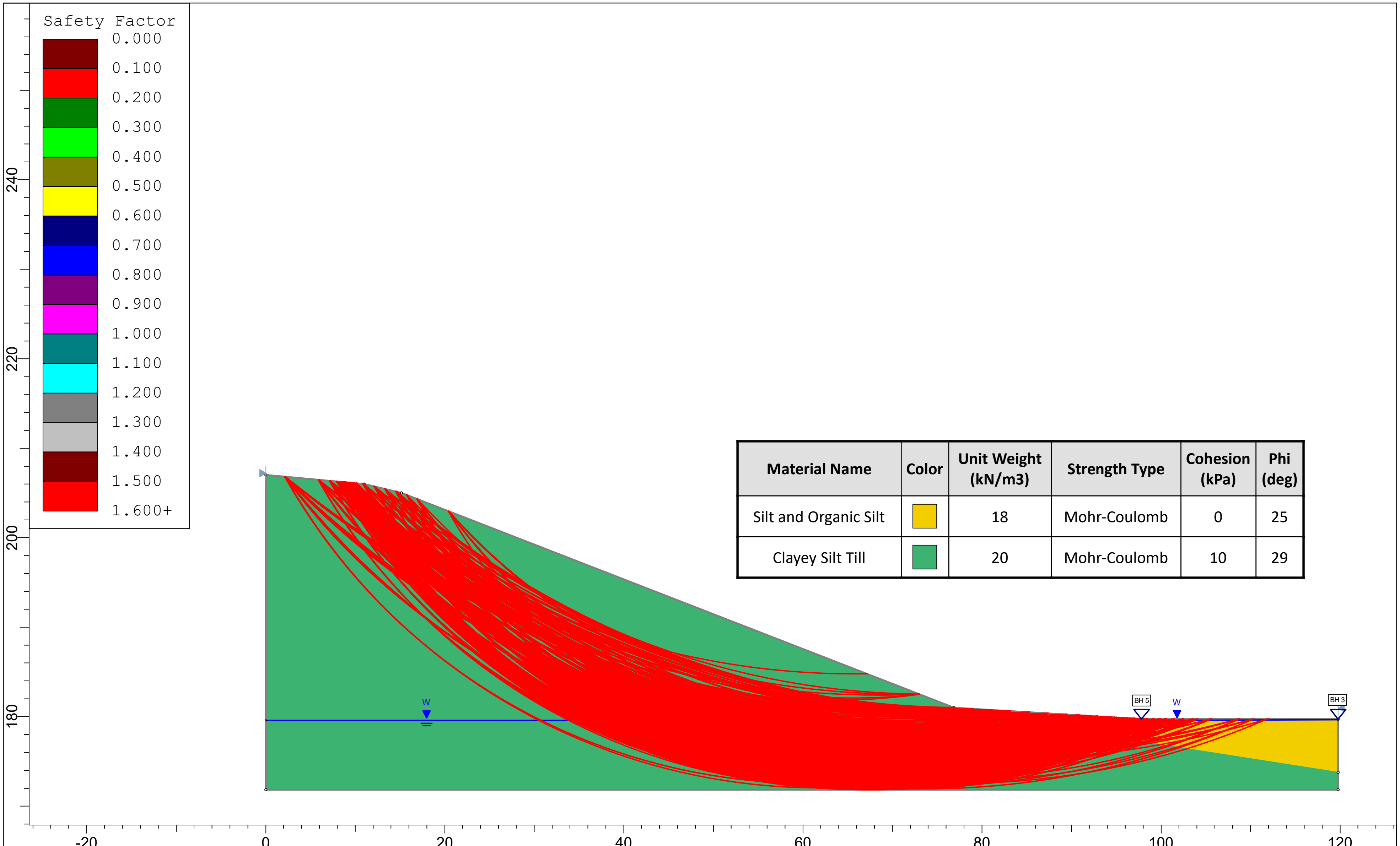
SUMMARY OF RATING VALUES AND RESULTING INVESTIGATION REQUIREMENTS

1.	Low potential	< 24	Site inspection only, confirmation, report letter.
2.	Slight potential	25-35	Site inspection and surveying, preliminary study, detailed report.
3.	Moderate potential	> 35	Boreholes, piezometers, lab tests, surveying, detailed report.

NOTES:
a) Choose only one from each category; compare total rating value with above requirements.
b)If there is a water body (stream, creek, river, pond, bay, lake) at the slope toe; the potential for toe erosion and undercutting should be evaluated in detail and, protection provided if required.

APPENDIX F

SLOPE STABILITY ANALYSIS RESULTS

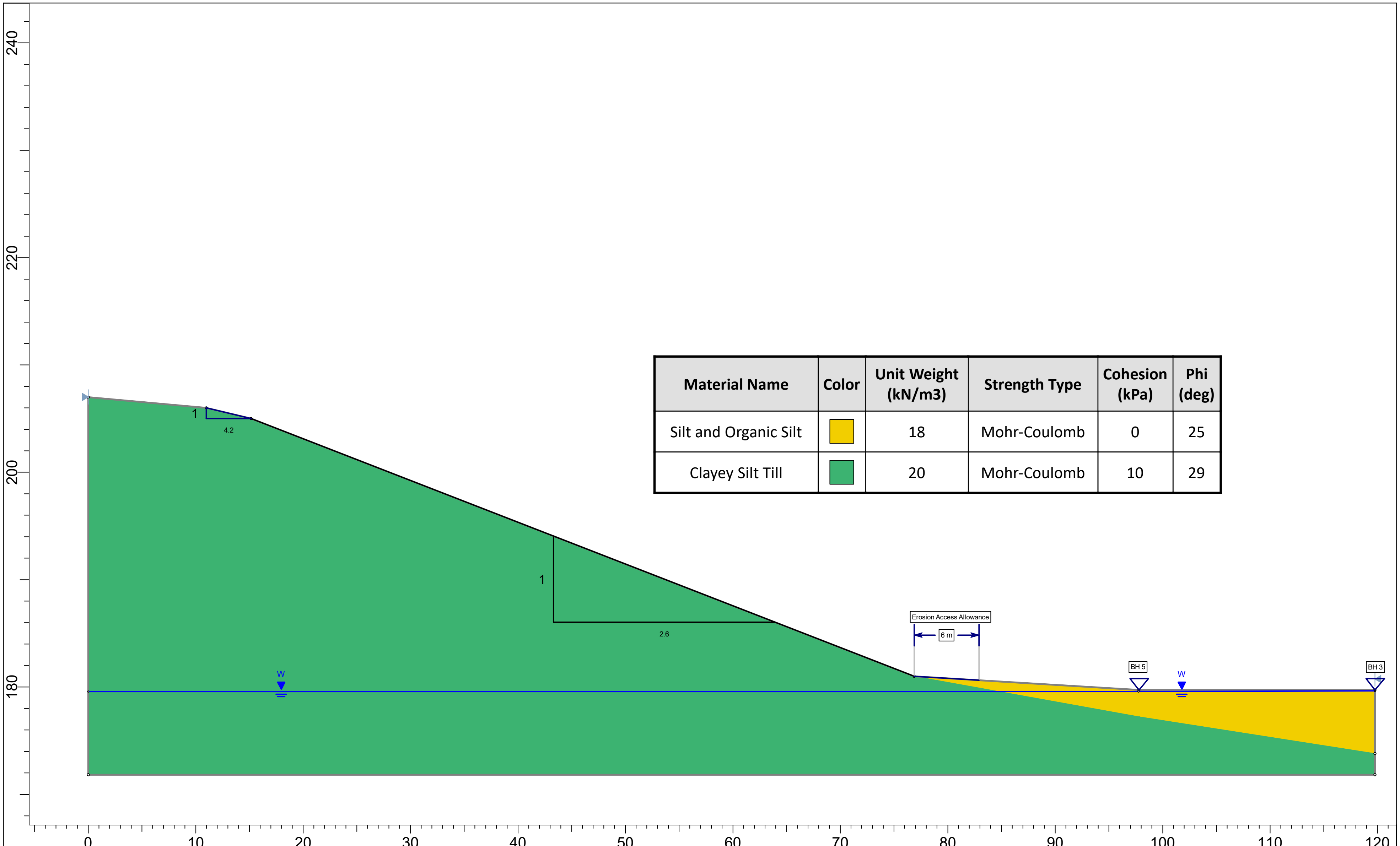


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Silt and Organic Silt		18	Mohr-Coulomb	0	25
Clayey Silt Till		20	Mohr-Coulomb	10	29

rocscience

SLIDEINTERPRET 8.032

Project	G22383 - Slope Stability Assessment
Analysis Description	
Drawn By	NZ
Scale	1:600
Company	CVD
Date	October 28, 2022
File Name	Figure No. 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Silt and Organic Silt		18	Mohr-Coulomb	0	25
Clayey Silt Till		20	Mohr-Coulomb	10	29

rocscience

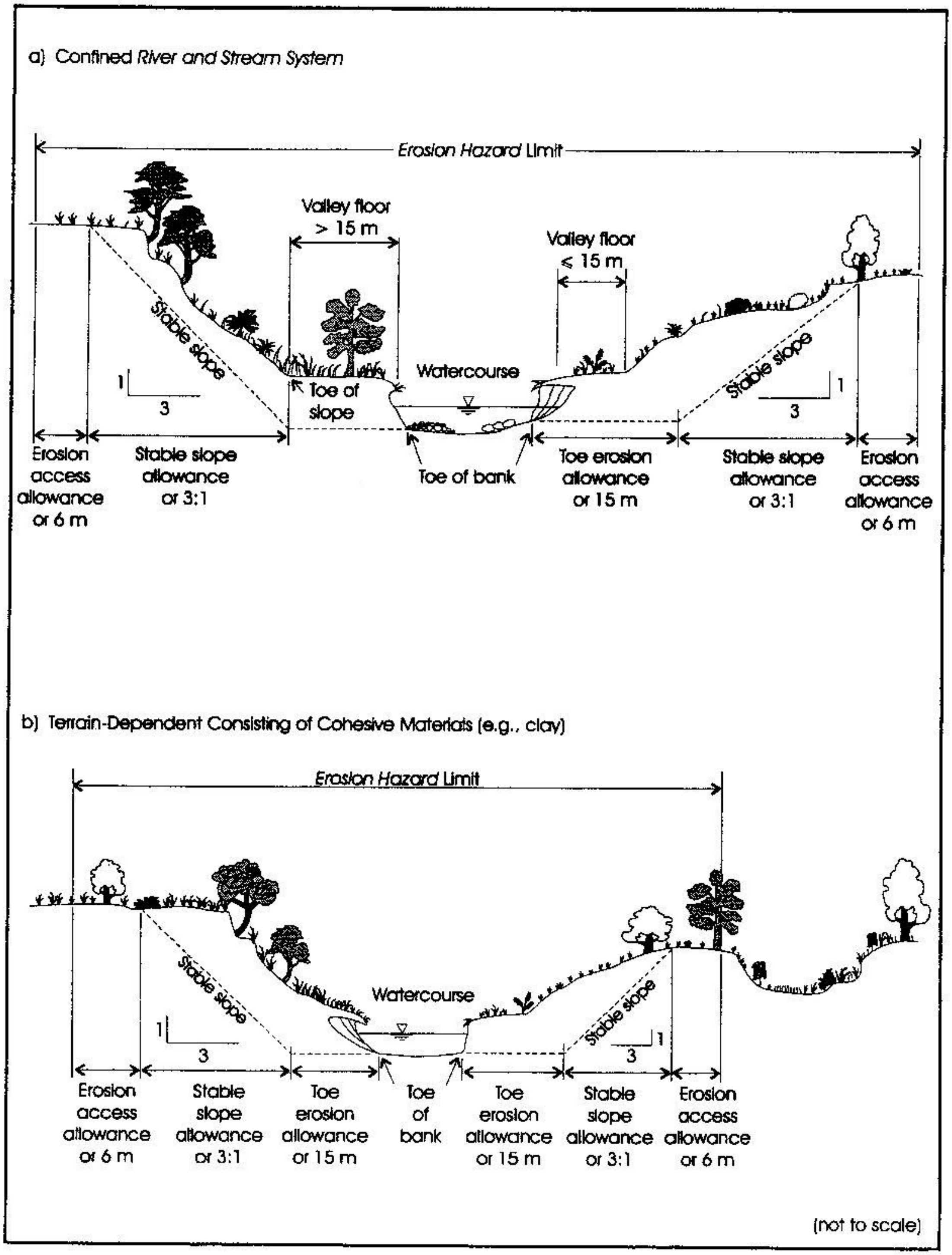
SLIDEINTERPRET 8.032

Project	G22383 - Slope Stability Assessment		
Analysis Description			
Drawn By NZ	*Scale* 1:500	*Company*	CVD
Date	October 28, 2022	*File Name*	Figure No. 2

APPENDIX G

EROSION HAZARD LIMIT

Figure 19: ***Erosion Hazard Limit*****: Confined and Terrain-Dependent (Cohesive)** ***River and Stream Systems***



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Kitchener, Ontario, N2H 5E1

Erosion Hazard Limit 384 George Street Port Stanley, Ontario		Date:	October 2022
		Scale:	NTS
File No.:	G22383	Appendix:	G

ENCLOSURES

FILE No: G22383

BOREHOLE No. 1

Client: **Caliber Contracting**

Project: **Proposed Residential Development**

Location: **384 George Street, Port Stanley, Ontario**

EQUIPMENT DATA

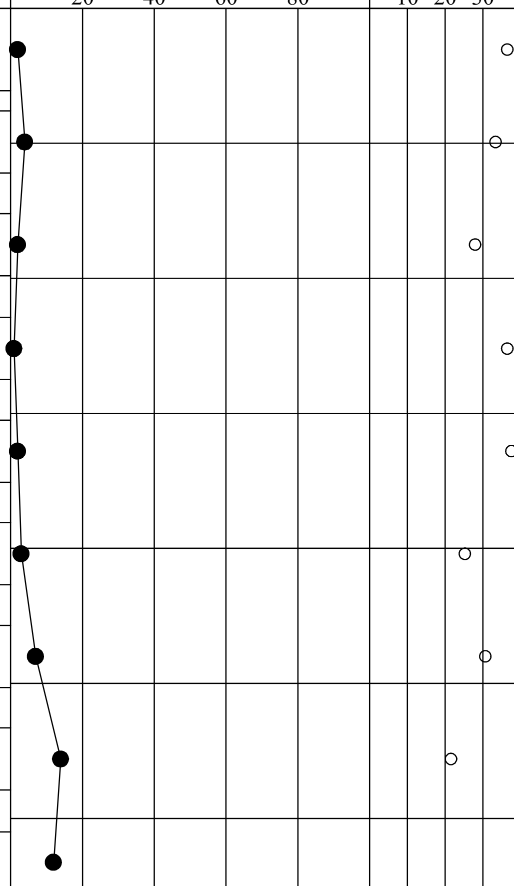
Machine: **Diedrich D50T**
Method: **Solid Stem Auger**
Size: **152 mm O.D.**
Date: **Feb 22 - 22** TO **Feb 22 - 22**

Ground Elevation: **179.67 m**

ELEV./ DEPTH (m)	DESCRIPTION	SAMPLE ID	TYPE	N-VALUE	WELL DATA	REMARKS
179.47 / 0.20	200 mm TOPSOIL					top 180 mm frozen
	very loose to compact dark brown to grey SILT trace sand, trace clay trace organics contains rootlets saturated ------ clayey layer ------	1	SS	2	▽	water level at 0.3 m bgs upon withdrawal of drilling augers
		2	SS	4		
		3	SS	2		
		4	SS	1		
		5	SS	2		
		6	SS	3		
		7	SS	7		
174.07 / 5.60	compact, brown SAND AND GRAVEL trace silt saturated	8	SS	14		
		9	SS	12		borehole open upon withdrawal of drilling augers
173.12 / 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



PROJECT MANAGER: **EYC**

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Enclosure No.: 2
Sheet 1 of 1

FILE No: G22383 — BOREHOLE No. 2

Client:	**Caliber Contracting**
Project:	**Proposed Residential Development**
Location:	**384 George Street, Port Stanley, Ontario**

EQUIPMENT DATA

Machine:	**Diedrich D50T**
Method:	**Solid Stem Auger**
Size:	**152 mm O.D.**
Date:	**Feb 22 - 22** TO **Feb 22 - 22**

SOIL LITHOLOGY

Ground Elevation: **179.84 m**

ELEV./ DEPTH (m)	DESCRIPTION	REMARKS
	350 mm GRANULAR BASE	top 150 mm frozen; water level at 0.18 m bgs upon withdrawal of drilling augers
179.49 / 0.35	very loose to loose brown to grey SILT trace sand, trace clay trace organics contains rootlets and wood fragments saturated ------ clayey layer ------	
172.84 / 7.00	very stiff, grey CLAYEY SILT TILL trace gravel, trace sand moist	borehole open upon withdrawal of drilling augers
171.74 / 8.10	End of Borehole Start of DCPT	Dynamic Cone Penetration testing (DCPT) conducted between 8.25 and 9.15 m depth
170.69 / 9.15	End of DCPT	

SAMPLE

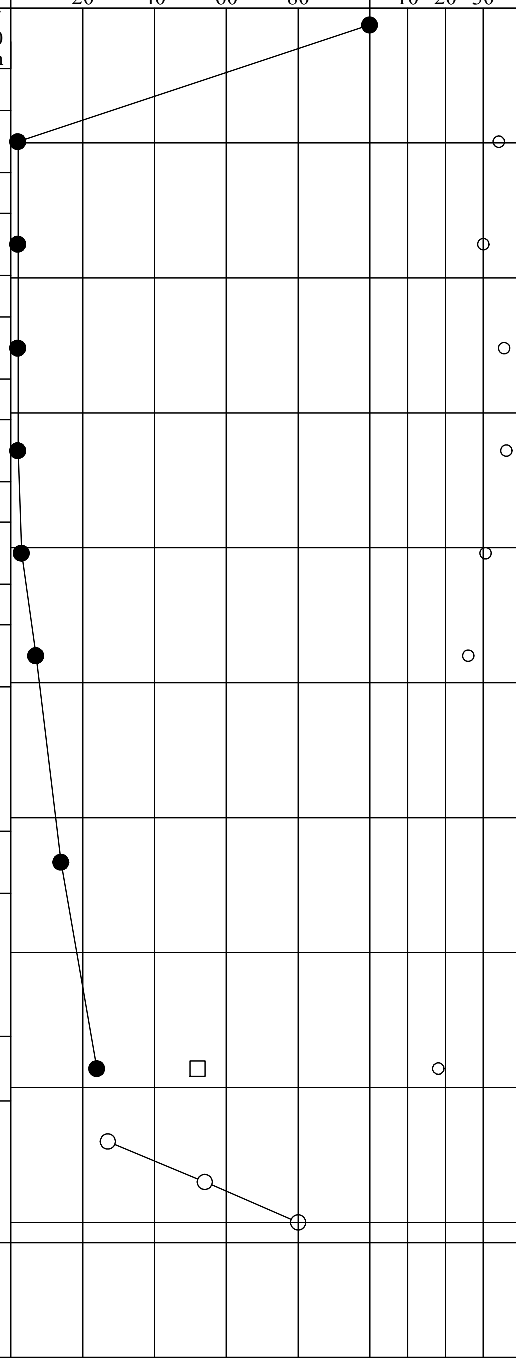
SAMPLE ID	TYPE	N-VALUE
1	SS	50/ 100 mm
2	SS	2
3	SS	2
4	SS	2
5	SS	2
6	SS	3
7	SS	7
8	SS	14
9	SS	24

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

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, 9.0, 9.5



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Enclosure No.: 3
Sheet 1 of 1

FILE No: G22383 — BOREHOLE No. 3

Client: **Caliber Contracting**
Project: **Proposed Residential Development**
Location: **384 George Street, Port Stanley, Ontario**

EQUIPMENT DATA
Machine: **Diedrich D50T**
Method: **Solid Stem Auger**
Size: **152 mm O.D.**
Date: **Feb 22 - 22** TO **Feb 22 - 22**

SOIL LITHOLOGY

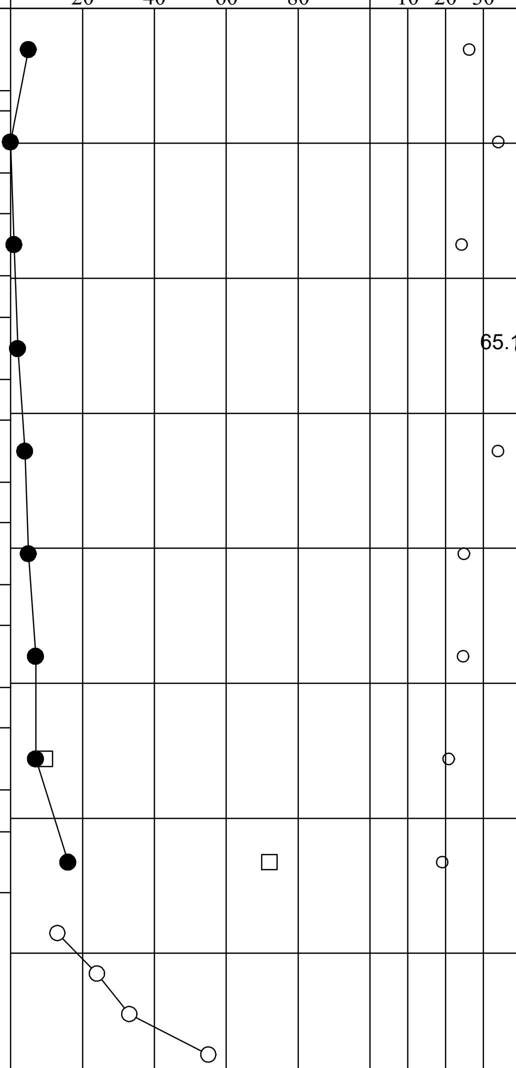
Ground Elevation: **179.74 m**

ELEV./DEPTH (m)	DESCRIPTION
179.56 / 0.18	175 mm TOPSOIL
173.79 / 5.95	very soft to firm; dark brown to grey; SILT and ORGANIC SILT; trace to some sand, trace clay; occ. sandy layers; occ. clayey seams/layers; contains rootlets, wood and shell fragments; saturated; ------; clayey layer
173.19 / 6.55	very stiff, grey; CLAYEY SILT TILL; trace grave, trace sand; moist
171.84 / 7.90	End of Borehole; Start of DCPT
	End of DCPT

SAMPLE

SAMPLE ID	TYPE	N-VALUE
1	SS	5
2	SS	0
3	SS	1
4	SS	2
5	SS	4
6	SS	5
7	SS	7
8	SS	7
9	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

DEPTH (m)	REMARKS
0.0	water level at 0.1 m bgs upon withdrawal of drilling augers; top 250 mm frozen
6.1	borehole open upon withdrawal of drilling augers
6.7	Dynamic Cone Penetration testing (DCPT) conducted between 6.7 and 7.9 m bgs

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CVD BOREHOLE (2017) G22383 384 GEORGE STREET, PORT STANLEY.GPJ CVD_ENG.GDT 22-10-31

Enclosure No.: 4
Sheet 1 of 1

FILE No: G22383 — BOREHOLE No. 4

Client: **Caliber Contracting**

Project: **Proposed Residential Development**

Location: **384 George Street, Port Stanley, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Solid Stem Auger**
Size: **152 mm O.D.**
Date: **Feb 22 - 22** TO **Feb 22 - 22**

SOIL LITHOLOGY

Ground Elevation: **179.91 m**

ELEV./DEPTH (m)	DESCRIPTION
	400 mm GRANULAR BASE
179.51 0.40	very loose, dark brown to blackish SANDY SILT trace organics saturated
178.56 1.35	soft to stiff brown to grey laminated SILT and CLAYEY SILT trace to some sand trace to some organics occ. peat layers/pockets contains rootlets and wood fragments saturated
173.36 6.55	End of Borehole Start of DCPT
172.01 7.90	End of DCPT

SAMPLE

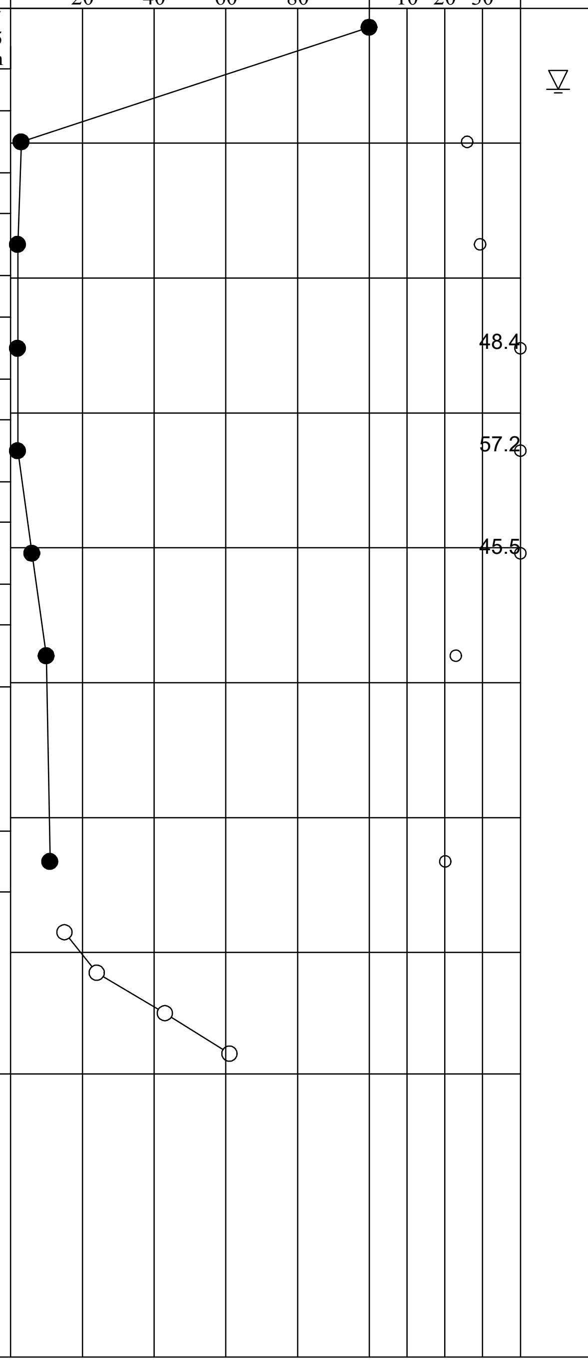
SAMPLE ID	TYPE	N-VALUE
1	SS	50/125 mm
2	SS	3
3	SS	2
4	SS	2
5	SS	2
6	SS	6
7	SS	10
8	SS	11

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



REMARKS

- top 280 mm frozen
- water level at 0.6 m bgs upon withdrawal of drilling augers
- borehole open upon withdrawal of drilling augers
- Dynamic Cone Penetration testing (DCPT) conducted between 6.7 and 7.9 m depth

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FILE No: G22383

BOREHOLE No. 5

Client: **Caliber Contracting**

Project: **Proposed Residential Development**

Location: **384 George Street, Port Stanley, Ontario**

EQUIPMENT DATA

Machine: **Diedrich D50T**
Method: **Solid Stem Auger**
Size: **152 mm O.D.**
Date: **Feb 22 - 22** TO **Feb 22 - 22**

SOIL LITHOLOGY | SAMPLE | SHEAR STRENGTH (kPa) | WATER CONTENT (%) | WELL DATA | DEPTH (m) | REMARKS

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

Ground Elevation: **179.73 m**

ELEV./ DEPTH (m)	DESCRIPTION	SAMPLE ID	TYPE	N-VALUE	REMARKS
179.58 / 0.15	150 mm TOPSOIL				water level at 0.15 m bgs upon withdrawal of drilling augers
	very soft to firm brown to grey SILT and ORGANIC SILT some sand, trace clay trace organics occ. sand seams occ. clayey layers saturated	1	SS	1	
		2	SS	4	
		3	SS	1	
177.58 / 2.15	stiff to hard brown to grey CLAYEY SILT TILL trace gravel, trace sand occ. silt layers moist	4	SS	11	
		5	SS	16	
		6	SS	15	
		7	SS	17	
		8	SS	30	borehole open upon withdrawal of drilling augers
173.18 / 6.55	End of Borehole				

Water content value noted at sample 1: 49

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, 9.0, 9.5

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FILE No: G22383

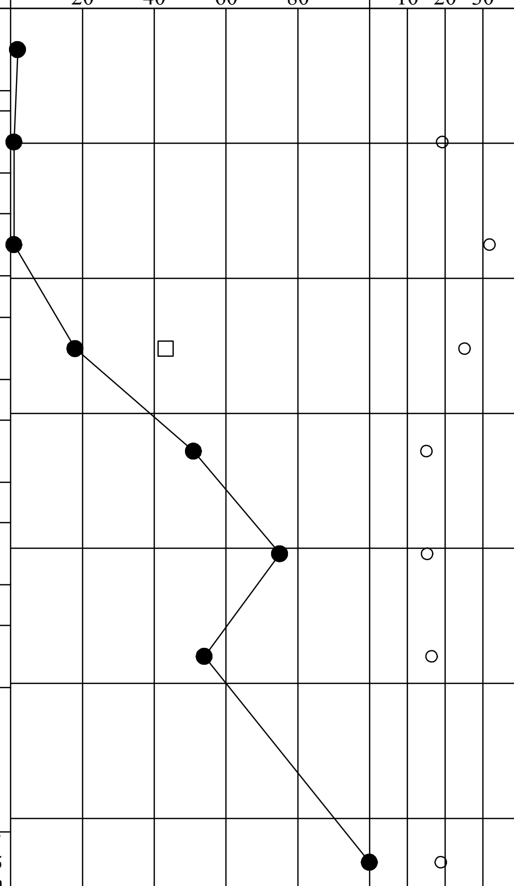
BOREHOLE No. 6

Client: **Caliber Contracting**
Project: **Proposed Residential Development**
Location: **384 George Street, Port Stanley, Ontario**

EQUIPMENT DATA
Machine: **Diedrich D50T**
Method: **Solid Stem Auger**
Size: **152 mm O.D.**
Date: **Feb 22 - 22** TO **Feb 22 - 22**

Ground Elevation: **180.09 m**

ELEV./ DEPTH (m)	DESCRIPTION	SAMPLE ID	TYPE	N-VALUE	WATER CONTENT W (%)	REMARKS
179.99 / 0.10	100 mm TOPSOIL					top 125 mm frozen; water level at 0.3 m bgs upon withdrawal of drilling augers
177.94 / 2.15 (base)	very loose brown to greyish SILT and ORGANIC SILT trace sand, trace clay saturated	1	SS	2		
		2	SS	1	~20	
		3	SS	1	~31	
177.19 / 2.90 (base)	very stiff to hard, brown CLAYEY SILT TILL trace sand occ. silt seams moist	4	SS	18	~26	
173.54 / 6.55 (base)	very dense, brown SILT some sand, trace clay occ. clayey seams (till-like) moist ------ wet	5	SS	51	~15	
		6	SS	75	~15	
		7	SS	54	~16	
		9	SS	50/ 125 mm	~20	borehole open upon withdrawal of drilling augers
6.55	End of Borehole					



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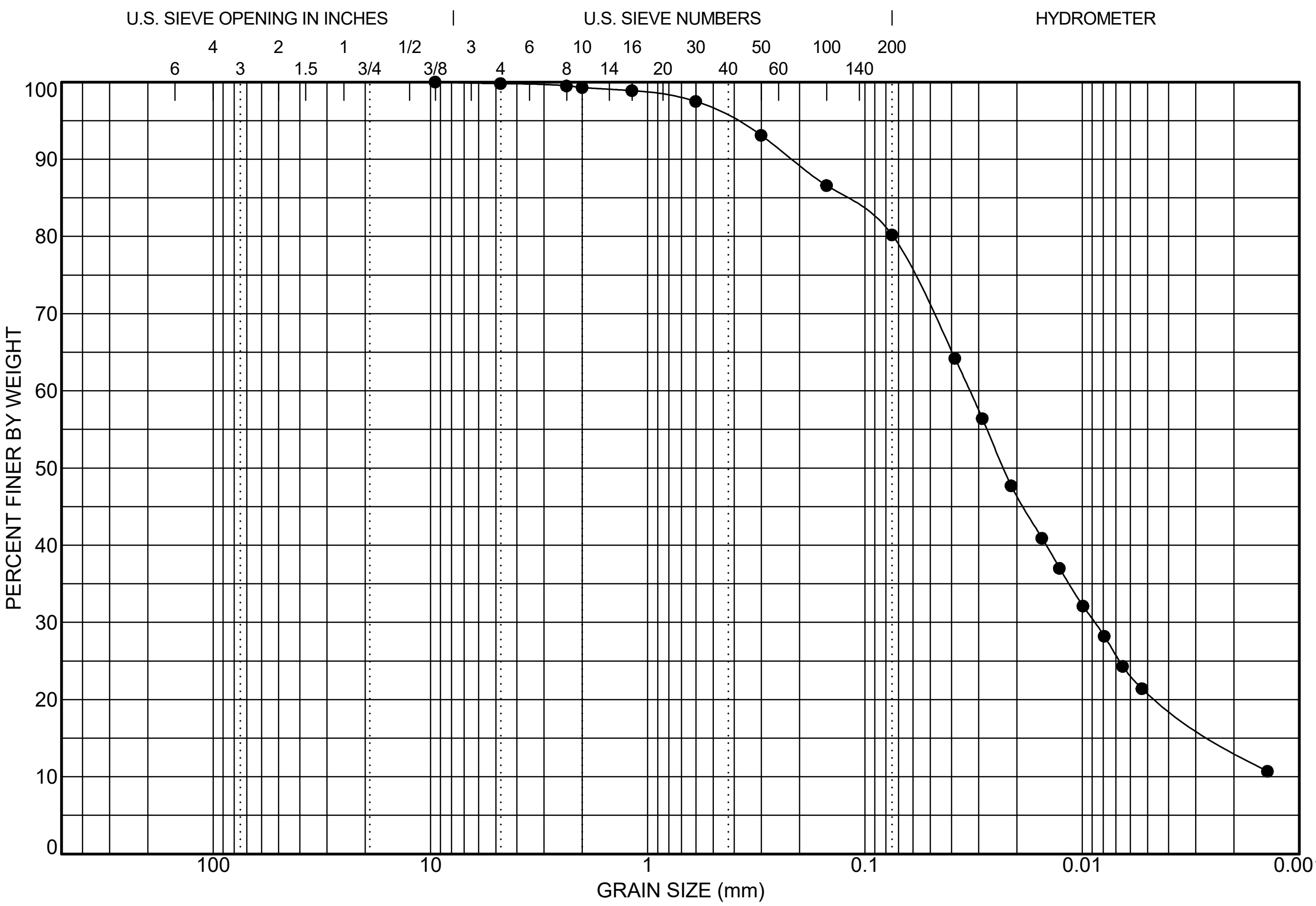

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

LL	PL	PI	Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
					9.5	0.033	0.009		0.2	19.6	80.2	

Date: Mar. 23 - 2022
Client: Caliber Contracting
Contractor:
Source:
Sampled From: BH 5 - SA 3; 1.50 to 1.95 m depth
Sample No.: 5-3
Date Sampled: Feb. 22 - 2022
Sampled By: BC
Lab No.: 188
Date Tested: Mar. 22 - 2021
Type of Material: Silt, some sand, trace clay, trace organics

Sieve Size (mm)	Percent Passing	No Specifications

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e-mail: info@cvdengineering.com

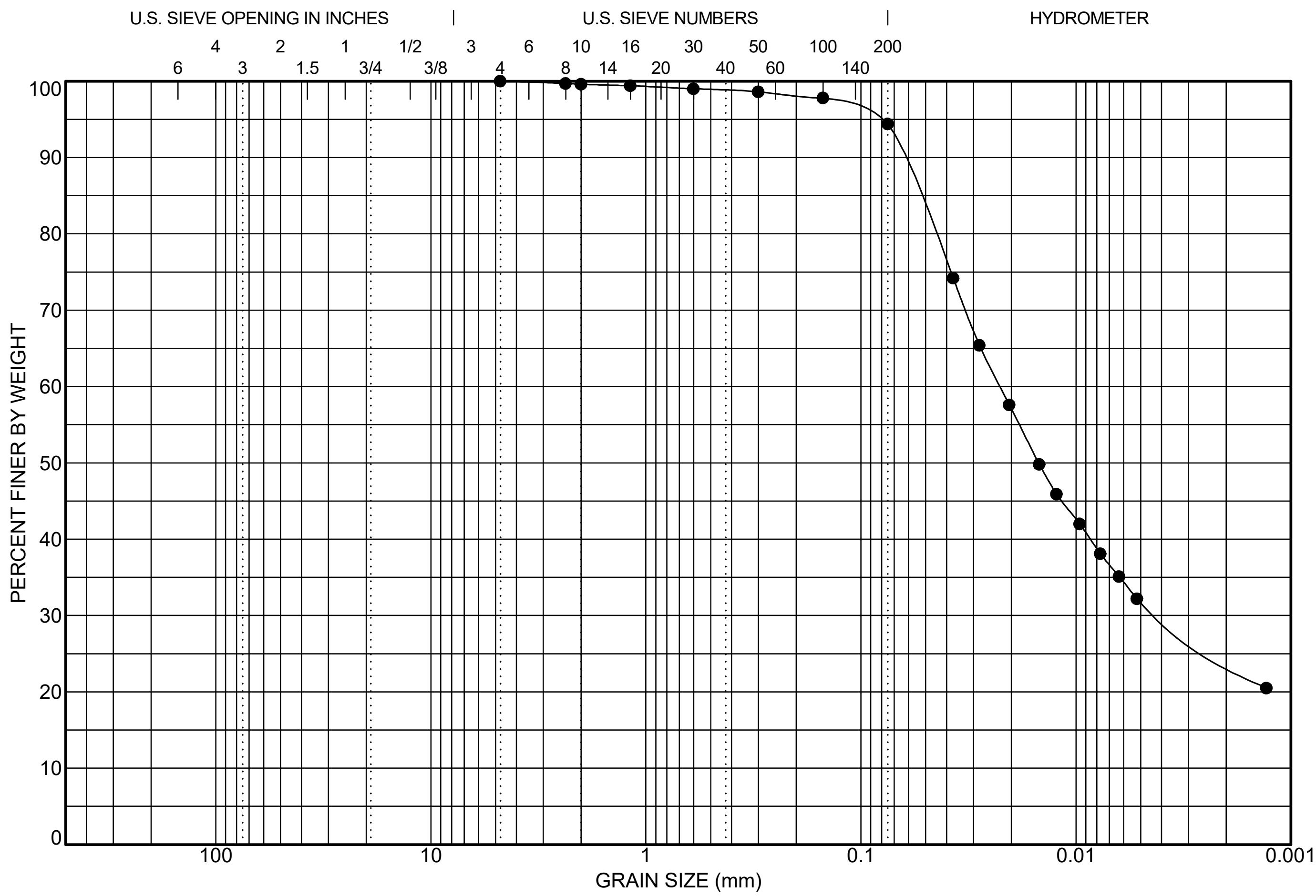
GRAIN SIZE DISTRIBUTION

Project: Proposed Residential Development

Location: 384 George Street, Port Stanley, Ontario

File No.: G22383

Enclosure No.: 7



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	0.023	0.004		0.0	5.6	94.4	

Date: Mar. 23 - 2022
Client: Caliber Contracting
Contractor:
Source:
Sampled From: BH 6 - SA 4; 2.30 to 2.75 m depth
Sample No.: 6-4
Date Sampled: Feb. 22 - 2022
Sampled By: BC
Lab No.: 189
Date Tested: Mar. 22 - 2021
Type of Material: Clayey Silt Till, trace sand

Sieve Size (mm)	Percent Passing	No Specifications

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Fax: 519-742-7739
e-mail: info@cvdengineering.com

GRAIN SIZE DISTRIBUTION

Project: Proposed Residential Development

Location: 384 George Street, Port Stanley, Ontario

File No.: G22383

Enclosure No.: 8

KEY PLAN **SOURCE:** Google Earth

LEGEND

Borehole Location

Elev. Ref.: Ground surface elevation at the borehole locations obtained with a Leica CC80 Field Tablet and an ICON GPS 70T (Tilt Compensated Network RTK Rover) Global Navigation Satellite System (GNSS) Receiver.

BOREHOLE LOCATION PLAN

Proposed Residential Development

384 George Street
Port Stanley, Ontario

CHUNG & VANDER DOELEN ENGINEERING LTD.

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

Drawn By: NZ	Date: March 2022	File No.: G22383
Checked By: EYC	Scale: N.T.S.	Drawing No.: 2