CHUNG & VANDER DOELEN
ENGINEERING LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
287 & 291 Woolwich Street
Waterloo, Ontario

SUBMITTED TO:
Mr. Johnny Xu
c/o Brutto Consulting
999 Edgeley Blvd., Unit 6
Vaughan, Ontario
L4K 5Z4

ATTENTION:
Mr. Claudio Brutto

FILE NO /G20144/January 15, 2021

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File No.: G20144

Mr. Johnny Xu
c/o Brutto Consulting
999 Edgeley Blvd., Unit 6
Vaughan, Ontario
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Attention: Mr. Claudio Brutto, MCIP, RPP

RE: Geotechnical Investigation
Proposed Residential Development
287 & 291 Woolwich Street, Waterloo, Ontario

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

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

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

Yours truly,
CHUNG & VANDER DOELEN ENGINEERING LTD.

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

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1.0 INTRODUCTION

CHUNG & VANDER DOELEN ENGINEERING LTD. (CVD) has been retained by Mr. Johnny Xu to carry out a geotechnical investigation for the proposed residential development to be located at 287 & 291 Woolwich Street in Waterloo, Ontario.

It is understood that the 0.85± ha site is to be developed with townhouses having 28 units. The proposed townhouses will be constructed along the north, south and west property lines. The existing cul-de-sac on Pelham Street will be extended into the southwestern portion of the site. Asphalt paved roadway, sidewalk and parking lots are proposed to be constructed at the middle and northwestern portion of the site. The finished floor elevations of the proposed townhouses have not been established at the time of reporting but are expected to match and coordinate with the adjacent subdivisions and municipal road design at elevation 320.75 m to the west and 317.20 m to the east.

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

- Foundation design recommendations;
- Excavation condition;
- Groundwater control during and after construction;
- Slab-on-grade design;
- Backfilling recommendations;
- Foundation soil classification seismic design per OBC 2012;
- Foundation walls and retaining wall design; and
- Pavement design

Infiltration rates of the various soil deposits encountered during the investigation will also be provided for potential design of an at-source storm water management feature.

2.0 FIELD WORK

In order to investigate the subsurface conditions at the site, six (6) boreholes were advanced to depths of 5.0 m below ground surface on December 1, 2020. The borehole locations are indicated on the Borehole Location Plan, Drawing No. 1.

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

In addition, three (3) monitoring wells were installed at Boreholes 1, 3 and 5 to determine the depth/elevation of the groundwater table and allow long-term monitoring of the groundwater conditions.

The location and ground surface elevation of the boreholes were surveyed by CVD for the purpose of this report. The ground surface elevations were referenced to a temporary benchmark (TBM) which is shown on Drawing No. 1 and described below:

TBM: Top of catch basin in road edge of southbound lane on Woolwich Street in front of Civic #287, as shown on Drawing No. 1

Elevation: 317.33 m (as per Topographic Survey Plan prepared by Van Harten Surveying Inc. dated August 7, 2019)

3.0 LABORATORY TESTING

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

In addition, four (4) grain size distribution analyses were performed on the major soil deposits to confirm field identification and to provide information on the soil hydraulic conductivity for the design of infiltration galleries.

4.0 EXISTING SITE CONDITIONS

The site is currently occupied by two (2) 2-storey residential dwellings having 1-level of basement. Asphalt paved driveways are located in front of all residences. Small gardens and storage sheds are observed at the middle and western portion of the site. The remainder of the site is grass-covered with occasional mature trees throughout and along the property line.

The ground surface of the site is relatively flat. Ground surface elevations at the borehole locations ranged between 318.23 and 318.84 m.

5.0 SUBSURFACE CONDITIONS

The detailed subsurface conditions encountered in the six (6) boreholes advanced as part of this investigation are shown on the Borehole Log Sheets, Enclosures 1 to 6. The following sections provide descriptions of the major soil deposits encountered in the boreholes.

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

5.1 Topsoil

Topsoil was encountered at ground surface at all six (6) borehole locations with measured thicknesses between 225 and 700 mm.

5.2 Fill

A layer of fill materials was encountered at ground surface at Boreholes 2 to 5 underlying the surficial topsoil and extended to depths between 0.70 and 1.35 m below ground surface. It is noted that fill materials could be deeper adjacent to the existing buildings on site.

The non-cohesive fill materials at four (4) borehole locations were comprised of sand with varying amount of silt and gravel from silty sand with trace to some gravel to sand and gravel with trace to some silt. Trace organics were observed within the fill materials at Borehole 4.

The SPT “N”-values measured within the non-cohesive fill materials ranged from 5 to 19 blows per 300 mm of penetration, indicating a loose to compact compactness condition. The measured moisture contents of the samples collected from the fill ranged between 4 and 11%, thus indicating a damp to moist moisture condition.

5.3 Sand and Gravel to Gravelly Sand

The surficial topsoil at Boreholes 1 and 6 and the fill materials at Boreholes 2 to 5 were underlain by a layer of sand and gravel to gravelly sand deposit which extended to depths between 2.90 and 4.55 m below existing grades. The coarse granular deposits contained trace to some silt. Occasional sandy layers were encountered within the deposits at Boreholes 1 and 4 to 6. Results of three (3) grain size distribution analyses on representative samples selected from Boreholes 1, 5 and 6 are shown graphically on Enclosures 7, 9 and 10.

The SPT “N”-values measured within the coarse granular deposits ranged from 14 to 69 blows per 300 mm of penetration, indicating compact to very dense compactness condition. Natural moisture contents were measured between 3 and 20%, indicating a damp to saturated moisture condition.

5.4 Sand

A sand deposit was encountered at Boreholes 2 and 3 underlying the sand and gravel to gravelly sand deposit. Both boreholes were terminated within the sand deposit at depths of 5.00 m below ground surface. The sand deposit contained trace to some gravel and silt. Clayey silt seams were observed within the sand deposit at the bottom of Borehole 2.

The SPT "N"-value measured within the deposit ranged from 18 to 33 blows per 300 mm of penetration, indicating a compact to dense compactness condition. The natural moisture contents were measured between 17 and 20%, indicating a saturated moisture condition.

5.5 Clayey Silt Till

The coarse granular deposits at Boreholes 1 and 4 to 6 were further underlain by a layer of cohesive deposit comprised of clayey silt till with varying percentage of sand. These four (4) boreholes were terminated within the cohesive deposit at depths of 5.00 m below ground surface, the maximum depths of exploration. Traces of gravel were encountered within the cohesive deposit at Boreholes 1 and 6. Occasional silt seams were observed within the deposit at Borehole 4. Results of one (1) grain size distribution analysis on representative samples selected from Borehole 1 are shown graphically on Enclosure 8.

The SPT "N"-values measured within the clayey silt till ranged from 22 to 39 blows per 300 mm of penetration. The undrained shear strength obtained on the retrieved samples ranged from 145 to greater than 250 kPa. Based on the above test results, the clayey silt till is considered to have a very stiff to hard consistency. The natural moisture contents were measured between 11 and 22%, indicating a moist moisture condition.

5.6 Groundwater

Groundwater conditions were monitored during advancement of the borehole augering and immediately following the withdrawal of the drilling augers at each borehole location.

Water level and saturated cave-in occurred at Boreholes 2 and 4 at depths of 2.30 and 2.45 m below ground surface, respectively. Dry cave-in was measured to depth of 2.15 m below ground surface at Borehole 6.

In addition, three (3) monitoring wells were installed on site as part of a concurrent geotechnical and hydrogeological subsurface investigation.

The table below summarizes the water level readings in the monitoring wells taken on December 15, 2020:

Borehole No.	Existing Ground Elevation (m)	Observed/Measured Water Level Below Existing Ground Surface (m)	Water Level Elevation (m)
1	318.84	2.33	316.51
3	318.77	2.62	316.15
5	318.90	2.67	316.23

Groundwater levels measured in monitoring wells ranged in depths between 2.3± and 2.7± m below existing grades, corresponding to elevations 316.2± and 316.5± m.

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

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 General

It is understood that the 0.85± ha site is to be developed with townhouses having 28 units. The proposed townhouses will be constructed along the north, south and west property lines. The existing cul-de-sac on Pelham Street will be extended into the southwestern portion of the site. Asphalt paved roadway, sidewalk and parking lots are proposed to be constructed at the middle and northwestern portion of the site. The finished floor elevations of the proposed townhouses have not been established at the time of reporting but are expected to match and coordinate with the adjacent subdivisions and municipal road design at elevation 320.75 m to the west and 317.20 m to the east.

In general, the surficial topsoil at Boreholes 2 to 5 were underlain by a layer of fill materials to depths between 0.70 and 1.35 m below ground surface. The surficial topsoil at Boreholes 1 and 6 and the fill materials at Boreholes 2 to 5 were further underlain by compact to very dense native granular deposits which extended to depths between 2.9 and over 5.0 m below ground surface. A very stiff to hard clayey silt till was encountered in boreholes 1 and 4 to 6 below the granular deposits and extended to at least 5.0 m below ground surface.

Groundwater levels measured in monitoring wells ranged in depths between 2.3± and 2.7± m below existing grades, corresponding to elevations 316.2± and 316.5± m. It is noted that the observed groundwater table will fluctuate seasonally and in response to major weather events.

6.2 Footing Foundations

Conventional strip and spread footing foundations can be used to support the proposed townhouses. Footings cast on competent native sand and gravel to gravelly sand deposit can be designed using a Geotechnical Reaction at SLS of 250 kPa and a Factored Geotechnical Resistance at ULS of 400 kPa. The following table summarizes the highest founding level and elevation for the footing at each borehole location:

No.	Existing Ground Elevation (m)	Highest Founding Depth (m)	Highest Founding Elevation (m)
BH 1	318.84	0.84	318.00
BH 2	318.75	1.45	317.30
BH 3	318.77	1.47	317.30
BH 4	318.23	0.83	317.40
BH 5	318.90	0.80	318.10

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.

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

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

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

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

6.3 Site Grading and Engineered Fill Construction

Approved on site cut material (native granular soils) or imported sand and gravel containing less than 8% silt sized particles can be used to construct the engineered fill under controlled and supervised conditions. The moisture content of the soil is required to be within 3% dry of its optimum moisture condition to achieve the specified degree of compaction. The excavated fill materials contain organic matters and, therefore are unlikely to be reused for engineered fill construction.

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

1. All existing topsoil, fill materials, and otherwise deleterious materials are to be excavated/removed to expose the underlying competent native subgrade;

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. The required grades can then be achieved by placing approved site materials or imported sand and gravel in maximum 300 mm thick lifts and compacted to no less that 98% Standard Proctor maximum dry density (SPMDD) to support footing foundations. The degree of compaction can be reduced to 95% for the support of floor slab. The moisture content of the soil requires to be within 3% dry of its optimum moisture condition to achieve the specified degree of compaction;

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

5. 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 cast on approved engineered fill can be designed using a Geotechnical Reaction of 150 kPa at SLS and a Factored Geotechnical Resistance of 250 kPa at ULS.

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.

6.4 Earthquake Considerations

In accordance with The Ontario Building Code 2012 (OBC), the proposed structure should be designed to resist earthquake load and effects as per OBC Subsection 4.1.8. Based on the anticipated condition of the engineered fill materials and the underlying soil condition encountered at the boreholes, the site can be classified as a Site Class C as per OBC Table 4.1.8.4.A (Page B4-24).

6.5 Floor Slab Construction

As per the City of Waterloo, the basement floor slab should be established at an elevation at least 0.6 m above the “high” groundwater table. The design basement floor elevations will be confirmed based on the groundwater level to be measured in Spring 2021.

The floor slab for the proposed townhouses can be constructed as conventional slab-on-grade on the native granular deposits or engineered fill, if required. 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 imported granular fill which should be compacted 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.

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 and one-quarter of the slab thickness.

Care should be taken to ensure that the backfill against foundation walls, interior piers/columns and concrete pits are placed in thin layers and each layer compacted to at least 95% SPMDD. These types of confined areas should be backfilled with imported granular soils such as OPSS Granular B Type I.

6.6 Lateral Earth Pressure

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

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

where:

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

The backfill for the foundation walls and retaining walls should be free-draining granular materials which should have less than 8% silt particles (OPSS Granular "B" Type I). The backfill should be placed in thin layers and compacted to 95% SPMDD. Over-compaction should be avoided. Weeping tiles leading to a frost-free outlet or weep holes should be installed to effect drainage behind the retaining wall.

The sliding resistance of the retaining wall footings should be checked. The unfactored horizontal resistance against sliding between cast-in-place concrete and the various soils can be calculated using a friction coefficient as follows:

Soil	Unit Weight (kN/m^3)	Friction Coefficient
Well-compacted granular backfill	21	0.4
Sand and Gravel to Gravelly Sand	21	0.4

6.7 Open Cut Excavation and Groundwater Control

Excavations are expected to be in the order of 1.5 to 2.5 m deep for footing foundations and site servicing. The excavations will penetrate loose to compact fill materials and compact to very dense native coarse granular deposits and compact sand deposit. These materials are considered to be Type 3 Soils in accordance with the latest Occupational Health and Safety Act.

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

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

For any excavations carried out below the water table, dewatering may be required as the granular soils will become “quick” and lose its integrity to support loads. The groundwater table must be lowered and controlled to at least 600 mm below the excavation level to facilitate the excavation and construction of footing foundations, and foundation walls to be carried out in the dry condition.

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

6.8 Pavement Design

The earth subgrade soil is generally expected to consist of non-cohesive fill materials or native coarse granular deposits. The following flexible pavement structure is recommended for the local residential roadway based on the results of the gradational analyses, assumed CBR values, groundwater table, frost susceptibility of subgrade soils and anticipated traffic volume.

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

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

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

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

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

6.9 Infiltration Rate of Soils

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

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

Based on the results of grain size analyses and our past experience, the hydraulic conductivity and infiltration rate of the native inorganic soil types encountered at the boreholes are estimated and provided in the following table and may be used for storm water management purposes:

MATERIAL	PERMEABILITY (K) (cm/sec)	INFILTRATION RATE (mm/hr)
Sand and Gravel to Gravelly Sand trace to some silt (Enclosures 7, 9 and 10)	1×10^{-1}	200

7.0 CLOSURE

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

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

Yours truly,
CHUNG & VANDER DOELEN ENGINEERING LTD.

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PROVINCE OF ONTARIO

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APPENDIX A

LIMITATIONS OF REPORT

ENCLOSURES