Peto MacCallum Ltd.
CONSULTING ENGINEERS

GEOTECHNICAL INVESTIGATION
ORCHARD COMMUNITY
BIRD BOULEVARD
BURLINGTON, ONTARIO

for

MANTELLA CORPORATION
c/o IBI GROUP

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Peto MacCallum Ltd.
CONSULTING ENGINEERS

March 27, 2019

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Report: 1

Mantella Corporation
c/o Mr. Stephen Albanese, RPP
IBI Group
55 St. Clair Avenue West
7th Floor
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Dear Mr. Albanese

Geotechnical Investigation
Orchard Community
Bird Boulevard
Burlington, Ontario

Peto MacCallum Ltd. (PML) is pleased to present the results of the geotechnical investigation recently completed for this project. Authorization to proceed with this assignment was provided by Mr. Stephen Albanese of IBI Group in an email dated January 21, 2019 and was confirmed with a signed Engineering Services Agreement dated February 4, 2019.

It is understood that plans call for the development of a subdivision comprised of ten lots for detached residential dwellings and 8 lots for townhouses, as well as, an extension of Bird Boulevard in Burlington, Ontario. It is understood that the proposed dwellings will be two storey buildings with one full basement level. The proposed basement levels will range from about elevation 153.5 at the southeast area of the development to about 154.6 at the north end of the development. A geotechnical investigation is required to provide recommendations for the design and construction of the proposed foundations, roads and municipal services. The purpose of the geotechnical investigation was to assess the subsurface soil and ground water conditions at the site and based on the findings, provide geotechnical comments and recommendations for the design and construction of the proposed development.

The scope of work for this study did not include the observation, recording or assessment of the environmental conditions of the soil, bedrock and groundwater on the property. If excess materials requiring off-site disposal are generated during construction, laboratory analysis will be required to determine the chemical properties of the material and evaluate options for off-site disposal.

The subsurface stratigraphy revealed in the boreholes typically comprised fill, over silt till. Upon completion of augering, free water was observed at depths of 3.0 and 5.4 m (elevation 152.3 and 150.4) in Boreholes 1 and 2 with cave to 3.0 m (elevation 152.3) in Borehole 1. As of the last recorded water level by PML on March 18, 2019, the short term water level was observed at 1.0 m (elevation 154.8) in Borehole/Monitoring Well 2.

Based on the findings of this investigation, it is considered feasible to construct the development using standard construction practices provided that adequate foundation and underfloor drainage systems are provided to handle seasonally high ground water levels, in conjunction with raising of site grades, where practical.

It is recommended to install two additional monitoring wells in order to facilitate longer term groundwater level monitoring to more accurately determine the seasonal high ground water levels which may impact site grading and building drainage considerations.

Detailed comments and recommendations concerning the design and construction of the proposed development program are provided in the attached report.

We trust the information presented in the attached report will be sufficient for your present purposes. If you have any questions, please do not hesitate to contact our office.

Sincerely

Peto MacCallum Ltd.



Matthew D. St. Denis, P.Eng.
Associate
Manager, Geotechnical Services

TF/SJ:ld

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1. INTRODUCTION

Peto MacCallum Ltd. (PML) is pleased to present the results of the geotechnical investigation recently completed for this project. Authorization to proceed with this assignment was provided by Mr. Stephen Albanese of IBI Group in an email dated January 21, 2019 and was confirmed with a signed Engineering Services Agreement dated February 4, 2019.

It is understood that plans call for the development of a subdivision comprised of ten lots for detached residential dwellings and eight lots for townhouses, as well as an extension of Bird Boulevard in Burlington, Ontario. It is understood that the proposed dwellings will be two storey buildings with one full basement level. The proposed basement levels will range from about elevation 153.5 at the southeast area of the development to about 154.6 at the north end of the development. A geotechnical investigation is required to provide recommendations for the design and construction of the proposed foundations, roads and municipal services.

The purpose of the geotechnical investigation was to assess the subsurface soil and ground water conditions at the site and based on the findings, provide geotechnical comments and recommendations for the design and construction of the proposed development.

The scope of work for this study did not include the observation, recording or assessment of the environmental conditions of the soil, bedrock and groundwater on the property. If excess materials requiring off-site disposal are generated during construction, laboratory analysis will be required to determine the chemical properties of the material and evaluate options for off-site disposal.

The comments and recommendations provided in this report are based on the site conditions at the time of the investigation and are applicable only to the proposed development as described in the report. Any changes in development, including finished grades and layout will require review by PML to assess the validity of the report and may require modified recommendations, additional investigation and/or analysis.

2. INVESTIGATION PROCEDURES

The field work was carried out on February 22, 2019 and consisted of two boreholes (Boreholes 1 and 2) drilled to 6.2 m termination depths. The borehole locations are shown on Drawing 1, appended.

The borehole locations were selected and established in the field by PML. Ground surface elevations and UTM co-ordinates at the borehole locations were determined by PML. The following temporary benchmark, as shown on a drawing tilted "Grading Plan", Region of Halton File No. 24T-17001, by IBI Group, dated December 7, 2018, was used for vertical reference:

TBM: Top of sanitary manhole at corner of Stonehaven Drive and Orchard Road.
Elevation: 155.03 m (geodetic)

The boreholes were advanced using continuous flight solid stem augers, powered by a track-mounted Diedrich D50 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of PML's engineering staff.

Representative samples of the overburden were recovered at frequent depth intervals using a conventional split-spoon sampler during drilling. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. In situ pocket penetrometer tests were conducted on cohesive soils to determine in situ shear strength.

The ground water conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and the drill rods as the samples were retrieved and when appropriate by measurement of the water level in the open borehole.

A ground water monitoring well was installed in Borehole/Monitoring Well (BH/MW 2) comprising clean 50 mm diameter screened and solid PVC Schedule 40 pipe. The well was installed in the sand to a depth of 6.0 m and screened at the bottom over a length of 1.5 m. The annular space of the borehole around the screen was backfilled with clean filter sand covered by a bentonite seal and well protector set in concrete. The details of the monitoring well construction are shown on

the appended Log of Borehole/Monitoring Well sheet. A well record was provided by the licensed well contractor and it will be kept on file for future reference in accordance with O. Reg. 903/90, as amended.

In accordance with O. Reg. 903/90, as amended, the owner of a well is defined as the owner of the land upon which the well is situated and the well owner should immediately decommission the well if it is not being used or maintained for future use as a well. PML would be pleased to assist in this regard.

Upon completion of drilling, the remaining boreholes were decommissioned in accordance with O. Reg. 903/90, as amended.

The recovered soil samples were returned to our laboratory for detailed visual examination and classification, and routine moisture content determinations.

3. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Log of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, standard penetration test N values, in situ pocket penetrometer shear test shear strengths, ground water observations, details of monitoring well installation, and the results of laboratory moisture content determinations.

Due to the soil sampling procedures and limited sample size, the depth demarcations on the borehole logs must be viewed as transitional zones between layers and cannot be construed as exact geologic boundaries between layers. PML would be pleased to assist in defining geologic boundaries during construction if required.

The subsurface stratigraphy revealed in the boreholes typically comprised fill, over silt till.

3.1 Fill

A 1.4 m thick fill layer was contacted at the ground surface and was penetrated at a depth of 1.4 m (elevation 153.9 and 154.4) in Boreholes 1 and 2. The fill comprised stiff to very stiff clayey silt and was judged to be drier than plastic limit (DTPL) with in situ moisture content determinations typically ranging from 14 to 16%

3.2 Silt Till

Silt till was encountered below the fill at 1.4 m (elevation 153.9 and 154.4) and extended to the termination depth of 6.2 m (elevation 149.1 and 149.6) in Boreholes 1 and 2. The silt till was very stiff/dense to very dense and was judged to be DTPL/moist with moisture content levels typically ranging from 8 to 15%.

3.3 Ground Water Conditions

Upon completion of augering, free water was observed at depths of 3.0 and 5.4 m (elevation 152.3 and 150.4) in Boreholes 1 and 2 with cave to 3.0 m (elevation 152.3) in Borehole 1. As of the last recorded water level by PML on March 18, 2019, the water level was observed at 1.0 m (elevation 154.8) in Borehole/Monitoring Well 2. Observed ground water levels may fluctuate subject to seasonal variations and precipitation patterns.

4. ENGINEERING DISCUSSION AND RECOMMENDATIONS

It is understood that plans call for the development of a subdivision comprised of ten lots for detached residential dwellings and eight lots for townhouses, as well as, an extension of Bird Boulevard in Burlington, Ontario. The proposed basement levels will range from about elevation 153.5 at the southeast area of the development to about 154.6 at the north end of the development. It is noted that the existing surrounding residential buildings are generally comprised of two storey dwellings with one basement level. A geotechnical investigation is required to provide recommendations for the design and construction of the proposed foundations, roads and municipal services.

The subsurface stratigraphy revealed in the boreholes typically comprised fill, over silt till. Upon completion of augering, free water was observed at depths of 3.0 and 5.4 m (elevation 152.3 and 150.4) in Boreholes 1 and 2 with cave to 3.0 m (elevation 152.3) in Borehole 1. As of the last recorded water level by PML on March 18, 2019, the short-term water level was observed at 1.0 m (elevation 154.8) in Borehole/Monitoring Well 2.

4.1 Site Preparation

Based on a review of the grading plan provided by IBI Group, the proposed ground surface elevation of the Bird Boulevard extension will range from about 155.6 at the south end to 155.9 m at the north end. The ground surface elevations of the lots near the proposed buildings are to range from about 155.4 at the southeast area of the development to about 157.0 m at the north end of the development. The proposed basement floor elevations for the buildings vary between 153.5 at the southeast to about 154.6 m at the north.

Based on a review of the topographic site plan by Mackay Mackay & Peters Limited, ground surface elevation on the site previously ranged from about elevation 155 to 162 m. However, portions of the site were excavated in the Fall of 2018 and an updated survey of the site with current grades was not provided at the time of this report.

It is visualized that site preparation will involve minor cut and fill operations.

Preparation of the site should consist of removal of topsoil and fill in all areas of proposed buildings, roadways and other settlement sensitive structures followed by proofrolling the exposed subgrade under geotechnical supervision to expose soft/loose or unstable areas. Any soft/loose or unstable material should be excavated, removed and replaced with well compacted, approved soil with a moisture content adjusted to within 3% of the optimum moisture content. Approved material should be inorganic material which is free of debris and otherwise deleterious materials.

Bulk fill placed to raise the grades should be placed as an engineered fill in uniform 200 to 300 mm thick lifts within 3% of the optimum moisture content. Engineered fill in the building envelopes should be compacted to at least 98% standard Proctor maximum dry density (SPMDD). Compaction to 95% SPMDD should be suitable in other areas. In this regard, trench

backfill would also be considered as engineered fill. In landscaped areas where post construction settlement may not be a concern, compaction to 90% SPMDD may be suitable. Further recommendations regarding placement of engineered fill are presented in Appendix A.

Based on the borehole information, the in situ native silt till and portions of the fill should be suitable for re-use as engineered fill subject to geotechnical review and approval during construction. However, depending on seasonal conditions at the time of construction, some moisture content adjustments may be necessary.

If construction is scheduled during the winter months, temporary gravel roads may have to be constructed to allow access for heavy construction equipment. A crushed limestone material would be suitable for this purpose. The need for temporary gravel construction roads should be determined by site review during construction.

The native soils are considered to be frost susceptible, and should not be used where frost related movements or heave could present a concern.

Full time site observation should be carried out by PML to examine and approve backfill material, to review placement operations, and to verify the specified compaction is achieved.

4.2 Underground Services

4.2.1 Trench Excavation

Open cut trench excavations are anticipated to extend through the fill and into the native silt till.

Subject to adequate ground water control being achieved, excavation through the overburden is expected to be relatively straight forward using conventional equipment. The possibility of debris in the fill and/or cobbles and boulders in the native overburden should not be overlooked.

Provided adequate ground water control is achieved, the in-situ fill soils are classified as Type 3 soil according to the Occupational Health and Safety Act criteria. Therefore, trench sidewalls should be cut at an inclination of 1 horizontal to 1 vertical (1H:1V) from the bottom of the excavation, otherwise adequate shoring must be provided.

It may be necessary to further flatten the trench side slopes if excessively loose/soft conditions or concentrated seepage zones are encountered locally.

Trench side slopes should be continuously examined for evidence of instability, particularly following periods of heavy rain, thawing or when the trench has been left open for extended periods of time. When required, appropriate remedial action must be taken to ensure the continued stability of the trench slope and the safety of workers in the trench.

Steeper slopes may be permitted if an approved trench liner box is utilized to protect workers. It should be noted that a trench liner box may not be utilized as a means of support for adjacent foundations, structures or utilities.

If space is not available for inclined slopes, it will be necessary to use a braced excavation to support the walls of the excavation and maintain the integrity of existing facilities. The magnitude and distribution of the lateral earth pressures acting on a braced excavation wall is dependent upon the support system used, the number of supports, the allowable movements and the construction sequence.

For a braced excavation option, the recommended design earth pressure distribution for multiple and singly braced walls for the conditions which exist at the site are presented on Figures 1 and 2, respectively. Recommendations concerning design and construction of the braced excavation support systems are also presented on the Figures.

Foundations of heavily loaded/settlement sensitive structures and/or utilities located within close proximity to the excavation may require underpinning to preserve the integrity of these structures. Further comments and general recommendations in this regard are presented in Figure 3.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local regulations.

4.2.2 Ground Water Control

Upon completion of augering, free water was observed at depths of 3.0 and 5.4 m (elevation 152.3 and 150.4) in Boreholes 1 and 2 with cave to 3.0 m (elevation 152.3) in Borehole 1. As of the last recorded water level by PML on March 18, 2019, the water level was observed at 1.0 m (elevation 154.8) in Borehole/Monitoring Well 2.

It is understood that the lowest basement elevation will be near 153.5 m. Cognizant that the excavation will be at or below the observed short term monitoring well water levels, ground water seepage into excavations should be anticipated.

Due to the low permeability nature of the in-situ soils, it is anticipated that ground water seepage or surface water entering the excavation will be handled readily by conventional sump pumping techniques. The possibility of encountering concentrated seepage from permeable layers within the native soil and/or localized perched ground water requiring additional pumps, should not be overlooked. It is recommended that test pits should be carried out during the tendering stage of the project in order that prospective contractors may familiarize themselves with soil and ground water conditions. Also, the dewatering requirements should be established by the contractor in the context of a performance specification. The dewatering system should meet a performance specification to maintain and control the ground water below the excavation base.

The design and installation of the dewatering system should be carried out by specialists in this field. Shop drawings should be submitted to PML for review to verify the design is consistent with the subsurface conditions identified during this investigation.

The contractor must ensure the dewatering is fully operational until the construction operations are completed. There must be adequate provisions for backup power for standby conditions to ensure the continuous operation of the dewatering system.

Dewatering at rates greater than 50,000 L/day and less than 400,000 L/day would require a water taking and discharge plan prepared by a Qualified Person (QP) (P.Geo. or P.Eng.) and submitted to the Environmental Activity Sector Registry (EASR) system for approval from provincial authorities. If dewatering rates are to exceed 400,000 litres per day, then under the Ontario Water

Resources Act, the Water Taking and Transfer Regulation 387/04, a Permit-to-Take-Water (PTTW) from the Ministry of Environment, Conservation and Parks (MECP) will be required together with a hydrogeological site assessment (HSA) report in support of the PTTW application. Based on the soil and ground water conditions encountered in Boreholes 1 and 2, it is not expected that construction dewatering rates will be greater than the limit of 50,000 L/day and therefore, subject to trial excavations, it is not expected that EASR registration/PTTW applications will be required.

It should be noted that ground water sampling and testing was not within the scope of work of the geotechnical investigation. Supplemental sampling and testing of the ground water would be required to assess options for discharge and treatment, if required.

Ground water levels will fluctuate subject to seasonal variations and precipitation patterns. If schedules allow, it is recommended that construction be carried out during the dry summer months, when ground water levels are normally lowest to minimize the amount of ground water to be handled.

4.2.3 Bedding Material

It is anticipated the subgrade for underground services will comprise of silt fill and native silt till. The native silt till and portions of the fill are considered suitable for conduit support. However, in localized areas, loose/soft zones of the subgrade, may require subexcavation or compaction prior to the placement of the granular pipe bedding material. A provision for subexcavation and replacement with non-shrinkable fill is recommended, subject to geotechnical review during construction.

The normal 150 mm bedding thickness of granular material as per Ontario Provincial Standards (OPS) and/or local municipal requirements should be satisfactory. The bedding and cover material should be placed in 150 mm lifts compacted to a minimum 95% standard Proctor maximum dry density (SPMDD). Compaction should be provided beneath the pipe haunches to provide uniform support. The cover material should be carried up as backfill for at least 300 mm above the pipe obvert.

4.2.4 Trench Backfill

In general, backfill should comprise inorganic, debris free material having a moisture content within 3% of the optimum value and placed in uniform 200 to 300 mm thick lifts compacted to 95% SPMDD to minimize post construction settlement.

Organic soil, topsoil, deleterious or excessively wet material should not be used as backfill.

It is anticipated that the excavated material will generally consist of silt fill and native silt till.

Re-use of portions of the excavated silt fill and native silt till from above the water table is considered feasible from a geotechnical perspective, depending on the moisture content of the excavated material at time of construction relative to its optimum moisture content. Depending on seasonal conditions, some moisture content adjustments to the backfill materials may be required. The on-site soils are considered to be frost susceptible and unsuitable for use where free draining backfill is required or at locations where frost related movement would present a concern. It is anticipated that portions of the overburden from below the observed water levels will be wetter than its optimum moisture content and will be unsuitable for backfill unless allowed to air dry prior to reuse.

It should be noted that the excavated till material will tend to retain a voided structure when placed as a controlled fill, including foundation and trench backfill. It will be important to ensure that sufficient compaction effort is applied to thoroughly break down all lumps/clods within the backfill soil matrix to achieve a non-voided condition. Significant post construction settlement could otherwise result. Excavated materials intended for backfilling purposes should not be exposed to the elements for prolonged time periods, as they might be rendered unsuitable for re-use.

Should construction be carried out in the winter season, particular attention must be given to ensure that frozen material is not used as backfill.

The trenching and backfilling operations should be carried out in a manner which minimizes the length of trench left open yet accommodates efficient pipe laying and compaction activities.

Full time site observation should be carried out by PML to examine and approve backfill material, to carefully inspect placement operations, and to verify the compaction by in situ density testing using nuclear gauges.

4.3 Buildings

4.3.1 Foundations

Based on a review of the grading plan provided by IBI Group, the proposed basement floor elevations for the buildings vary between 153.5 and 154.6 m. Uncontrolled fill ranging in thickness from around 1.4 m exists on site.

The depth to competent native very stiff/dense to very dense silt till is 1.4 m below existing grade, near elevation 153.9 to 154.4.

Upon completion of augering, free water was observed at depths of 3.0 and 5.4 m (elevation 152.3 and 150.4) in Boreholes 1 and 2 with cave to 3.0 m (elevation 152.3) in Borehole 1. As of the last recorded water level by PML on March 18, 2019, the short term water level was observed at 1.0 m (elevation 154.8) in Borehole/Monitoring Well 2.

Due to the high ground water table, it will be necessary to raise site grades to maintain basement floor levels a minimum of 1.0 m above the seasonal high ground water level, otherwise, adequate perimeter and separate underfloor drains are required.

In order to facilitate longer term ground water level monitoring it is recommended that two additional monitoring wells be installed to more accurately determine the seasonal high ground water level

Foundations at or below elevation 153.9 at Borehole 1 and 154.4 m at Borehole 2 are considered feasible provided that the fill is subexcavated to the level of competent native undisturbed soil as confirmed by a geotechnical inspector from our office. Fill and/or loose soil depths may vary and therefore a provision should be made for deeper footing excavations, as needed to reach native undisturbed soil. Alternatively, footings may be supported on adequately constructed engineered fill placed and compacted in accordance with the recommendations provided in Appendix A.

Strip and spread footings founded on the native undisturbed very stiff/dense to very dense silt till or adequately prepared engineered fill should be proportioned for a factored net bearing resistance at Ultimate Limit State (ULS) of 150 kPa and bearing pressure at Serviceability Limit State (SLS) of 100 kPa. Foundations should not be placed partially on native undisturbed soils and partially on engineered fill, otherwise differential settlements and structural cracking may occur. Further, the thickness of engineered fill zones should be uniform below buildings being supported. It is recommended that footings and foundation walls for buildings supported on engineered fill be provided with additional reinforcing steel in order to resist possible differential movements; a structural engineer should be consulted in this regard.

In general, where founding levels of adjacent footings vary, the founding elevation between footings should be stepped in maximum 600 mm steps at a maximum inclination of 10 horizontal to 7 vertical (10H:7V). If adequate stepping of the footings is not possible due to site or design limitations, the need for underpinning of the existing foundations should be evaluated

Prior to placement of structural concrete, all foundation excavations should be examined by geotechnical personnel from PML to verify that the founding stratum is in accordance with the assumptions and recommendations of this report. This will be particularly important near existing buildings and underground services, where unsuitable fill material may be present.

All footings subject to frost action should be provided with the normal 1.2 m of soil cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

The overburden soil is prone to disturbance by the weather elements and construction traffic. Accordingly, a 50 mm thick skim slab of lean concrete should be provided over the base of the approved subgrade if structural concrete cannot be provided within 24 hours of approval of the foundation base.

The total settlement of foundations designed in accordance with the foregoing recommendations is not expected to exceed 25 mm. Differential settlement is expected to be less than 75% of this value.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local regulations.

4.3.1.1 Earthquake Considerations

Design provisions for earthquake loading should also be applied. Based on the characteristics of the subsoils encountered in the boreholes at this site and based on a review of nearby publicly available deeper borehole information, the subject property may be classified as Site Class C for footings bearing on native overburden per The Ontario Building Code Act, (2012) Section 4.1.8.4.

4.3.2 Basement Floor Slab

Due to measured ground water level at 1.0 m (elevation 154.8), up to 1.3 m above the basement level, it is recommended that further monitoring be completed to measure seasonal water levels. Consideration should also be given to raising the basement above measured water levels; otherwise adequate perimeter and underfloor drainage systems are required.

Subject to adequate drainage, construction of the floor slab as a conventional slab-on-grade on the native overburden is considered to be feasible.

Preparation of the floor slab subgrade should include stripping of the fill with topsoil inclusions, and otherwise deleterious material followed by proofrolling of the exposed subgrade with a heavy roller to ensure uniform adequate support. Subject to proofrolling, it is envisaged that most of the existing non-organic very stiff fill may remain in place below lightly loaded slab-on-grade floors provided that it is uniformly compacted in place under geotechnical supervision to a minimum of 95% Standard Proctor maximum dry density. Excessively loose/soft or compressible materials revealed during the proofrolling operations should be subexcavated and replaced with well compacted approved material. Where existing in place fill soils are proposed to be left in place below slab-on-grade floors, it is recommended that additional test pits be carried out at the time of construction in order to check for the presence of compressible organic material or deleterious fill at depth.

Fill placed under the floor slab to achieve finished subgrade levels or as foundation excavation backfill should comprise approved inorganic material having a moisture content within 3% of the optimum value, placed in maximum 200 mm thick lifts, and compacted to at least 95% of standard Proctor maximum dry density (SPMDD).

A minimum 150 mm thick layer of well compacted free draining Granular A type material meeting OPSS 1010 specifications should be provided directly beneath the slab-on-grade. A polyethylene vapour barrier should be placed under the slab if a moisture sensitive finish is to be placed on the floor.

Cognizant of the measured water levels, it is recommended that an underfloor drainage system be installed at this site. Underfloor drains consisting of 100 mm diameter tile surrounded by 150 mm of clear stone wrapped in approved geotextile should be installed at a spacing of 7.5 m. The invert of the tiles should be at least 200 mm below the underside of the floor slab. The underfloor drains should be installed separately from the perimeter drainage system and connected by positive drainage to a frost-free sump or outlet. Sump pumps should include emergency back-up pump capacity with backup power supply.

Exterior grades should be maintained at least 150 mm below the ground floor level and sloped to promote drainage away from the building.

Refer to Figure 4, appended, for general recommendations regarding drainage and backfill requirements for basements walls and floor slab construction.

4.4 Subsurface Walls

Provided free-draining granular backfill or prefabricated drainage systems are employed and hydrostatic pressure is not allowed to develop, the lateral earth pressure, p, acting on the subsurface walls should be computed using the following equation, assuming a triangular pressure distribution:

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

where K = lateral earth pressure coefficient
= 0.5 for wall restrained at both top and bottom
γ = unit weight of free-draining granular material
= 21.0 kN/m^3
h = depth below final grade (m)
q = surcharge load (kPa), if present

The in situ soil would not be classified as “free draining”, but may be re-used as exterior foundation backfill if a drainage board product is installed as per Ontario Building Code requirements.

The perforated drainage pipe should be surrounded by a properly designed graded granular filter or wrapped with approved geotextile to prevent migration of fines into the system. The perforated drainage pipe should be placed on a positive grade and lead to a frost-free sump or outlet.

The backfill adjacent to the subsurface walls should be carefully compacted to at least 95% of standard Proctor maximum dry density (SPMDD) in areas beneath entranceways and sidewalks and to at least 90% SPMDD beneath landscaped areas. The backfill should be compacted using light equipment to minimize potential damage to the wall.

It is imperative that the excavation is of sufficient width to enable operation of suitable compaction equipment; use of a hoepac is not suitable for this application.

4.5 Pavement Structure

The subgrade for pavement construction is anticipated to consist of silt fill and engineered structural fill. Based on typical traffic patterns for local residential streets, the estimated strength and frost susceptibility of the anticipated subgrade and assuming adequate drainage, the following pavement structure is specified in the City of Burlington Standards:

Pavement Component	Thickness (mm)
Surface Course (HL3)	40
Binder Course (HL8)	60
Granular A Base Course	150
Granular B Subbase Course	300

The City of Burlington standards should be used as a minimum for pavement structure construction.

The granular materials should conform to the OPS specifications and should be placed in maximum 200 mm thick lifts and compacted to 100% of SPMDD. The asphalt should be placed in accordance with current OPS specifications and compacted to at least 92% of the material's maximum relative density. Reference is made to OPS Specification 310 revised November, 2010.

Preparation of the subgrade for pavement construction should involve stripping obvious deleterious materials followed by proofrolling of the subgrade with a heavy roller. Excessively wet or deleterious material revealed by the proofrolling operations should be subexcavated and replaced. The subgrade surface should be compacted to at least 95% SPMDD.

The pavement design considers that construction will be carried out during the drier time of the year and that the subgrade is stable, as determined by proofrolling operations. If the subgrade should become excessively wet or rutted during construction activities, additional subbase material may be required. The need for additional subbase is best determined during construction.

For the pavement to function properly, provision must be made for water to drain out of, and not collect in, the granular courses. In this regard, the pavement subgrade should be sloped to promote drainage towards catch basins and manholes. The excavation around catch basins and manholes should be backfilled with free-draining granular material to minimize differential movements between the pavement and structures due to frost action. The manholes/catch basins should be provided with perforated stub drains to permit drainage of the backfill.

Site review should be carried out by PML personnel to examine and approve subgrade, backfill/granular materials, to carefully inspect placement operations and verify the compaction (granular and asphalt) by in situ testing using nuclear gauges.

We trust the information presented in this report is sufficient for your present purposes. If you have any questions, please do not hesitate to contact our office.

Sincerely
Peto MacCallum Ltd.

Timothy Feather, B.Eng., EIT
Project Supervisor



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Senior Engineer
Regional Manager, Geotechnical Services

TF/SJ:ld

LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

Standard Penetration Resistance N: - The number of blows required to advance a standard split spoon sampler 0.3 m into the subsoil. Driven by means of a 63.5 kg hammer falling freely a distance of 0.76 m.

Dynamic Penetration Resistance: - The number of blows required to advance a 51 mm, 60 degree cone, fitted to the end of drill rods, 0.3 m into the subsoil. The driving energy being 475 J per blow.

DESCRIPTION OF SOIL

The consistency of cohesive soils and the relative density or denseness of cohesionless soils are described in the following terms:

CONSISTENCY	N (blows/0.3 m)	c (kPa)	DENSENESS	N (blows/0.3 m)
Very Soft	0 - 2	0 - 12	Very Loose	0 - 4
Soft	2 - 4	12 - 25	Loose	4 - 10
Firm	4 - 8	25 - 50	Compact	10 - 30
Stiff	8 - 15	50 - 100	Dense	30 - 50
Very Stiff	15 - 30	100 - 200	Very Dense	> 50
Hard	> 30	> 200		

WTPL	Wetter Than Plastic Limit
APL	About Plastic Limit
DTPL	Drier Than Plastic Limit

TYPE OF SAMPLE

SS	Split Spoon	TW	Thinwall Open
WS	Washed Sample	TP	Thinwall Piston
SB	Scraper Bucket Sample	OS	Oesterberg Sample
AS	Auger Sample	FS	Foil Sample
CS	Chunk Sample	RC	Rock Core
ST	Slotted Tube Sample		

PH	Sample Advanced Hydraulically
PM	Sample Advanced Manually

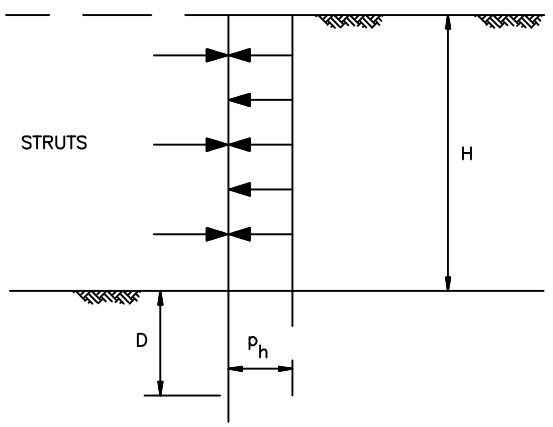
SOIL TESTS

Qu	Unconfined Compression	LV	Laboratory Vane
Q	Undrained Triaxial	FV	Field Vane
Qcu	Consolidated Undrained Triaxial	C	Consolidation
Qd	Drained Triaxial		

NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.
2. Stability of base of excavation must be confirmed when bracing system design, excavation geometry and surcharge loads are established. If groundwater table is well above base of excavation and/or artesian conditions exist, local lowering of the groundwater level will be necessary to prevent bottom heave/piping of the base of the excavation.
3. Earth pressure diagram is applicable to maximum depth of cut of 12m (40 ft.).
4. Structural components of bracing system should be confirmed adequate for each level of excavation.
5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
6. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system are not included in earth pressure diagram.
7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
8. If settlement sensitive structures are located near the excavation, special measures should be undertaken to control settlements. A condition survey should be conducted prior to construction and apppropriate monitoring (surface and insitu) carried out during construction.
9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, earth pressure diagram must be reviewed, and remedial works may be required.
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
11. Bracing system should be regularly examined for signs of distress.
12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

EARTH PRESSURE DIAGRAM



p_h = design lateral earth pressure
= $0.65K\gamma H$

K = lateral earth pressure coefficient

γ = unit weight of soil

H = depth of excavation

D = depth of embedment of soldier piles (if used).

RECOMMENDED DESIGN PARAMETERS

γ = 19.0 kN/m^3

K = 0.30 (movement of retained soil acceptable)
0.50 (movement of adjacent structures/facilities unacceptable)

LATERAL EARTH PRESSURE DISTRIBUTION

MULTI–BRACED CUTS IN COHESIONLESS SOILS

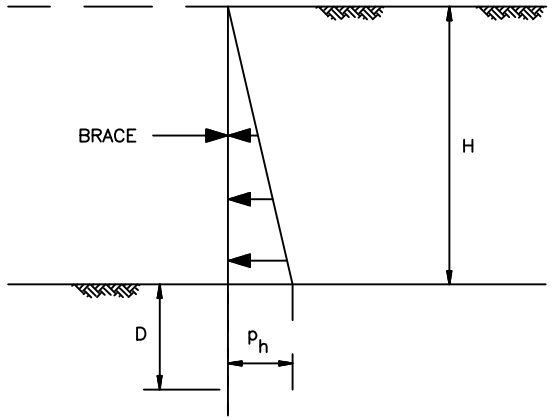
Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN:	TF	DATE	SCALE	JOB NO.	FIGURE NO.
CHECKED:	MDS	MAR. 2019	N.T.S.	19HF001	1
APPROVED:	MDS				

NOTES

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RECOMMENDED DESIGN PARAMETERS

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0.50 (movement of adjacent structures/facilities unacceptable)

LATERAL EARTH PRESSURE DISTRIBUTION

SINGLY–BRACED CUTS IN COHESIONLESS SOILS

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS

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NOTES

1. The need to underpin existing footings/utilities is dependent upon soil type, proximity of the existing facility to the face of the excavation, loads imposed on the foundation and permissible movements.

   ZONE A:
   Foundations of relatively heavy and/or settlement sensitive structures/ utilities located in Zone A generally require underpinning.

   ZONE B:
   Foundations of structures located within Zone B generally do not require underpinning. Consideration should be given to underpinning of settlement sensitive utilities or heavy foundation units located in this zone.

   ZONE C:

   Utilities and foundations located within Zone C do not normally require underpinning.

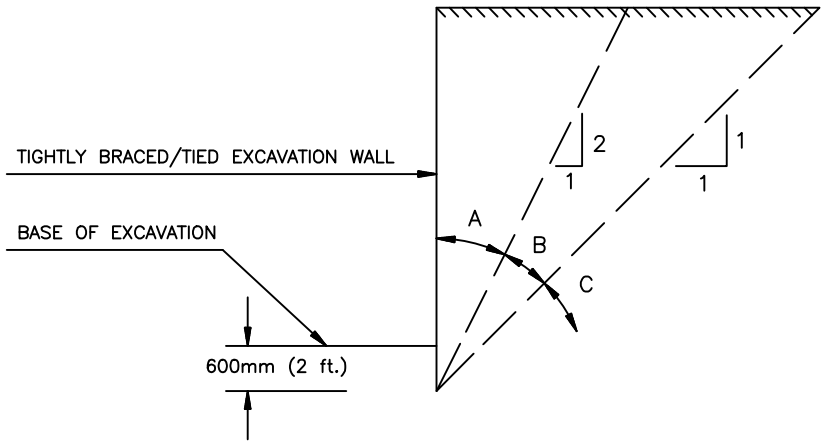
   Underpinning of foundations located in Zones A and B should extend at least into Zone C.

2. As an alternative to underpinning, it may be possible to control movement of existing utilities and foundations by supporting the face of the excavation with bracing/tiebacks or a rigid (caisson) wall. Horizontal and vertical earth pressures imposed on the excavation wall by non–underpinned foundations must be considered in the design of the support system.

3. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction to monitor any movement which may occur.

4. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.

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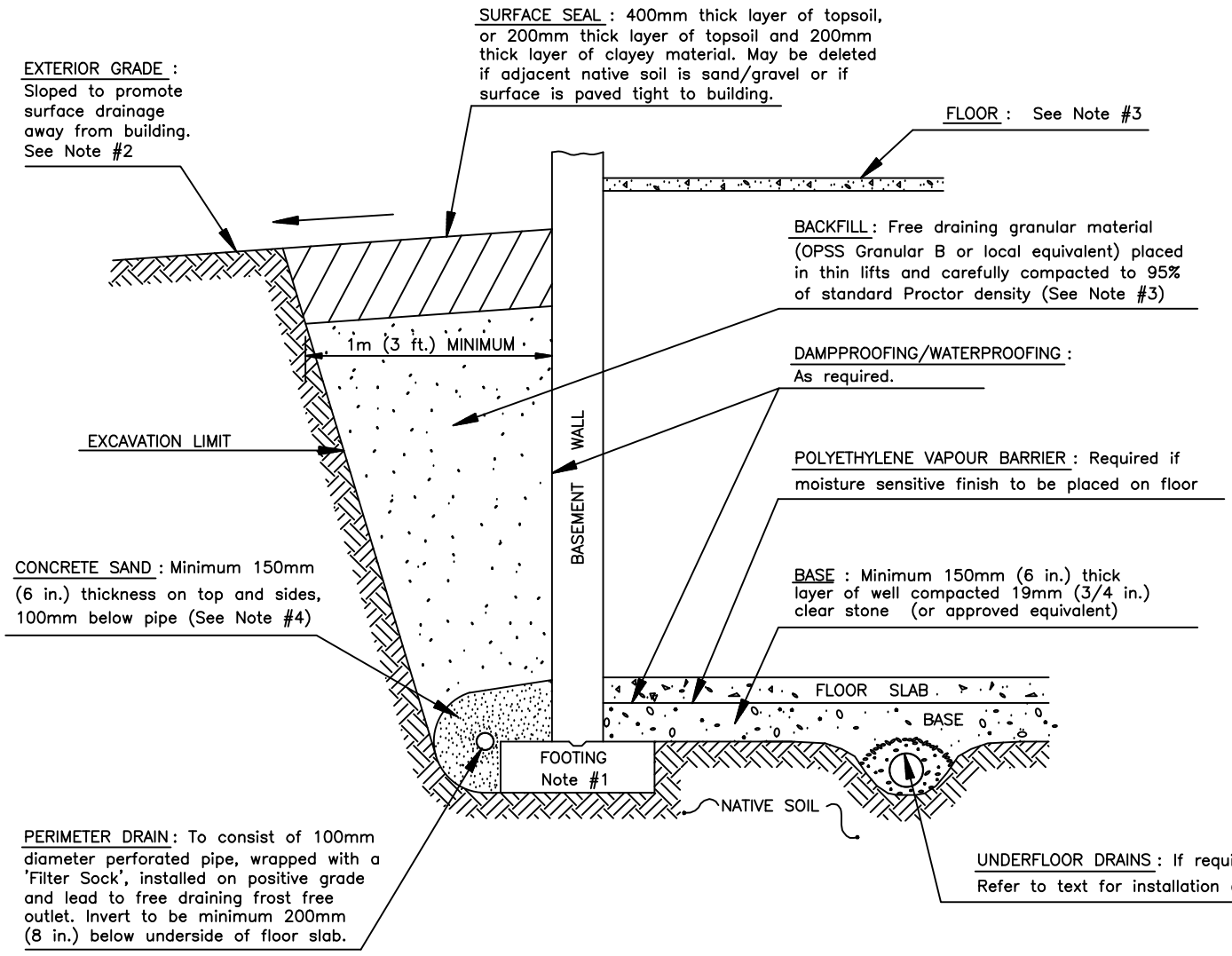


STANDARD DRAWING

GENERAL RECOMMENDATIONS REGARDING UNDERPINNING OF FOUNDATIONS/UTILITIES LOCATED CLOSE TO EXCAVATION

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NOTES

1. Footing may be constructed by placement of structural concrete neat against natural soil. Drain to be installed in a similar manner immediately above footing maintaining 200mm (8 in.) distance between top of drain and underside of floor slab.
2. Exterior grade to be minimum 300mm (12 in.) below interior floor slab, or other means established to prevent entry of surface water into building through building openings.
3. Basement wall to be supported by floor system or interior bracing prior to commencement of backfill placement. Heavy construction equipment should not be permited within a distance from the foundation wall equivalent to half the wall height. Overcompaction of backfill to be avoided as excessive lateral earth pressure may result. Approved drainage board product may be used in lieu of free draining granular.
4. Concrete sand may be replaced with clear stone, if wrapped in approved geotextile.
5. Refer to text for details regarding founding levels, competent bearing material and construction details specific to particular site.

STANDARD DRAWING

GENERAL RECOMMENDATIONS REGARDING DRAINAGE AND BACKFILL REQUIREMENTS FOR BASEMENT WALLS AND FLOOR SLABS

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CONSULTING ENGINEERS

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LOG OF BOREHOLE NO. 1

17T 596843E 4807001N

PROJECT Orchard Community — **PML REF.** 19HF001

LOCATION Bird Boulevard, Burlington, Ontario — **BORING DATE** February 22, 2019 — **ENGINEER** M. D. St. Denis

BORING METHOD Continuous Flight Solid Stem Augers — **TECHNICIAN** T. Feather

DEPTH (metres)	ELEV	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES
		SURFACE ELEVATION 155.31				
0.0		FILL: Stiff to very stiff, brown clayey silt fill, some sand, trace gravel, DTPL; with occasional rootlets, shale fragments		1	SS	10
1.0				2	SS	21
1.4	153.9	SILT TILL: Dense to very dense, brown silt till, some sand, trace to some clay, trace gravel, moist; with occasional shale fragments, a layer of black sandy silt		3	SS	39
2.0				4	SS	90/250 mm
3.0				5	SS	50/150 mm
3.9	151.4	becoming grey				
4.0				6	SS	50/150 mm
5.0						
6.0				7	SS	50/125 mm
6.2	149.1	BOREHOLE TERMINATED AT 6.2 m				
7.0						
8.0						
9.0						
10.0						
11.0						
12.0						
13.0						
14.0						
15.0						

ELEVATION SCALE: 155, 154, 153, 152, 151, 150

SHEAR STRENGTH (kPa)
+ FIELD VANE △ TORVANE ○ Qu
▲ POCKET PENETROMETER ○ Q
50 100 150 200
DYNAMIC CONE PENETRATION ×
STANDARD PENETRATION TEST ●
20 40 60 80

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%)
10 20 30 40

UNIT WEIGHT kN/m³

GROUND WATER OBSERVATIONS AND REMARKS

GRAIN SIZE DISTRIBUTION (%)
GR SA SI&CL

Upon completion of augering, free water and cave to 3.0 m

NOTES

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CONSULTING ENGINEERS

LOG OF BOREHOLE/MONITORING WELL NO. 2

17T 596875E 4806956N

PROJECT Orchard Community — ***PML REF.*** 19HF001

LOCATION Bird Boulevard, Burlington, Ontario — ***BORING DATE*** February 22, 2019 — ***ENGINEER*** M. D. St. Denis

BORING METHOD Continuous Flight Solid Stem Augers — ***TECHNICIAN*** T. Feather

DEPTH (metres) / ELEV	DESCRIPTION	NUMBER	TYPE	"N" VALUES
	SURFACE ELEVATION 155.77			
0.0	FILL: Stiff to very stiff, brown clayey silt fill, trace sand and gravel, DTPL; with occasional rootlets, topsoil inclusions	1	SS	13
1.0		2	SS	18
1.4 / 154.4	SILT TILL: Very stiff, brown clayey silt till, trace sand and gravel, DTPL; with occasional shale fragments	3	SS	22
2.0 / 2.1 / 153.7	becoming dense, sandy, some clay, moist	4	SS	46
2.9 / 152.9	becoming very dense, some sand	5	SS	56
4.0 / 151.8	becoming grey	6	SS	85/175 mm
6.2 / 149.6	BOREHOLE TERMINATED AT 6.2 m	7	SS	50/150 mm

ELEVATION SCALE: 155, 154, 153, 152, 151, 150

SHEAR STRENGTH (kPa): + FIELD VANE △ TORVANE ○ Qu ▲ POCKET PENETROMETER ○ Q — 50 100 150 200

DYNAMIC CONE PENETRATION × STANDARD PENETRATION TEST ● — 20 40 60 80

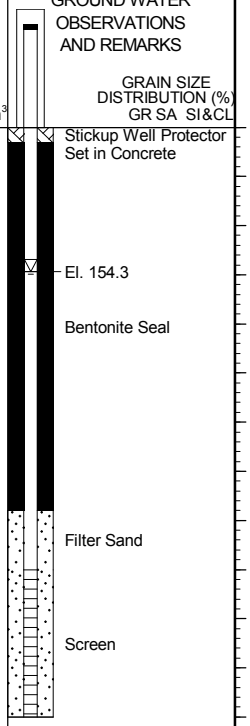
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) — 10 20 30 40

UNIT WEIGHT kN/m³

GROUND WATER OBSERVATIONS AND REMARKS

GRAIN SIZE DISTRIBUTION (%) GR SA SI&CL



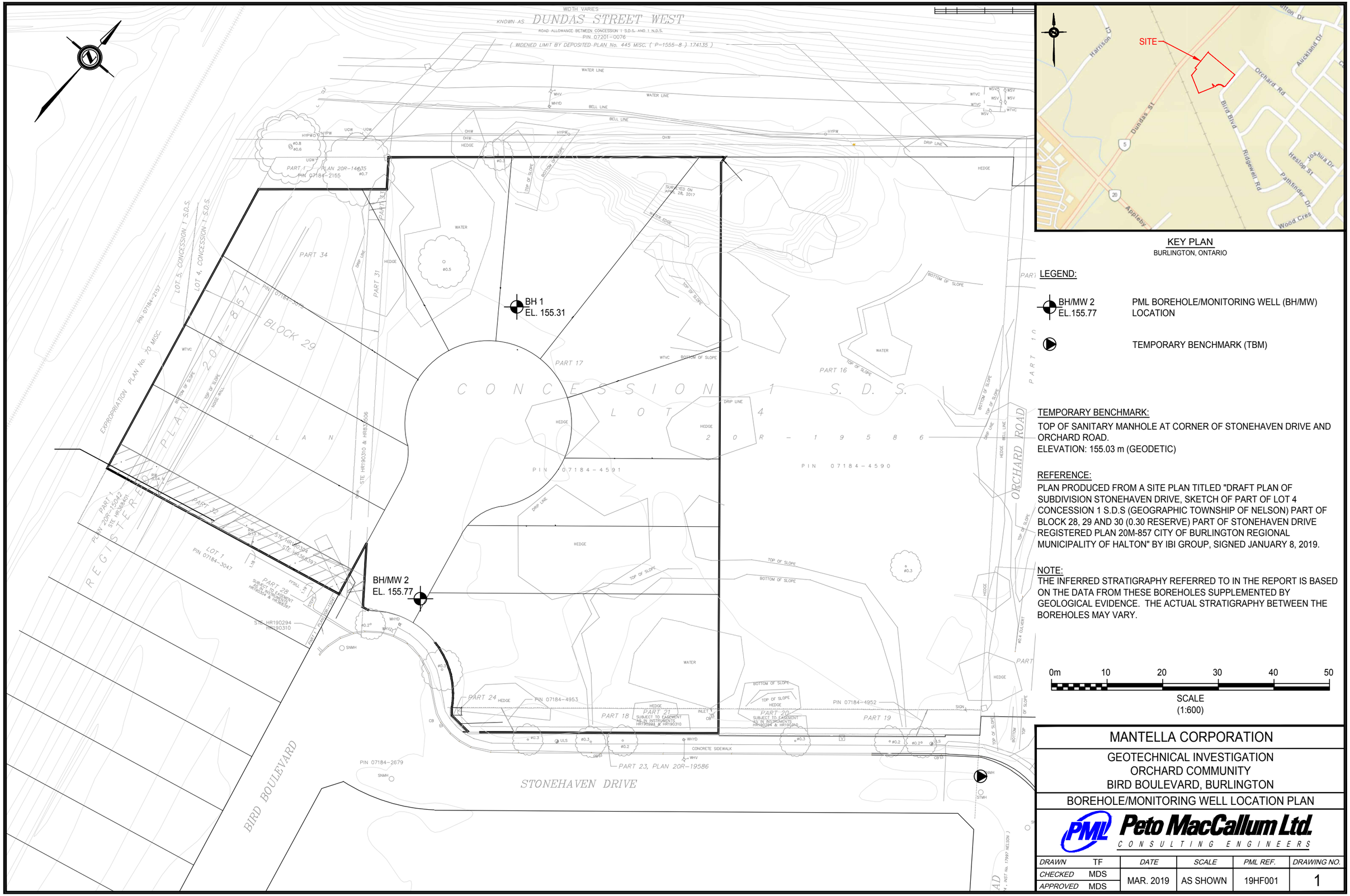
Upon completion of augering, free water to 5.4 m, no cave

Water Level Readings:

Date	Depth	Elev.
2019-03-05	1.5	154.3
2019-03-13	1.3	154.5
2019-03-18	1.0	154.8

NOTES

▽ HYDROSTATIC GROUND WATER LEVEL



KEY PLAN
BURLINGTON, ONTARIO

LEGEND:

BH/MW 2 EL.155.77	PML BOREHOLE/MONITORING WELL (BH/MW) LOCATION
	TEMPORARY BENCHMARK (TBM)

TEMPORARY BENCHMARK:

TOP OF SANITARY MANHOLE AT CORNER OF STONEHAVEN DRIVE AND ORCHARD ROAD.
ELEVATION: 155.03 m (GEODETIC)

REFERENCE:

PLAN PRODUCED FROM A SITE PLAN TITLED "DRAFT PLAN OF SUBDIVISION STONEHAVEN DRIVE, SKETCH OF PART OF LOT 4 CONCESSION 1 S.D.S (GEOGRAPHIC TOWNSHIP OF NELSON) PART OF BLOCK 28, 29 AND 30 (0.30 RESERVE) PART OF STONEHAVEN DRIVE REGISTERED PLAN 20M-857 CITY OF BURLINGTON REGIONAL MUNICIPALITY OF HALTON" BY IBI GROUP, SIGNED JANUARY 8, 2019.

NOTE:

THE INFERRED STRATIGRAPHY REFERRED TO IN THE REPORT IS BASED ON THE DATA FROM THESE BOREHOLES SUPPLEMENTED BY GEOLOGICAL EVIDENCE. THE ACTUAL STRATIGRAPHY BETWEEN THE BOREHOLES MAY VARY.

0m 10 20 30 40 50
SCALE
(1:600)

MANTELLA CORPORATION

GEOTECHNICAL INVESTIGATION
ORCHARD COMMUNITY
BIRD BOULEVARD, BURLINGTON

BOREHOLE/MONITORING WELL LOCATION PLAN

Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN	TF	DATE	SCALE	PML REF.	DRAWING NO.
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Appendix A

Engineered Fill

ENGINEERED FILL

The information presented in this appendix is intended for general guidance only. Site specific conditions and prevailing weather may require modification of compaction standards, backfill type or procedures. Each site must be discussed, and procedures agreed with Peto MacCallum Ltd. prior to the start of the earthworks and must be subject to ongoing review during construction. This appendix is not intended to apply to embankments. Steeply sloping ravine residential lots require special consideration.

For fill to be classified as engineered fill suitable for supporting structural loads, a number of conditions must be satisfied, including but not necessarily limited to the following:

1. Purpose

The site specific purpose of the engineered fill must be recognized. In advance of construction, all parties should discuss the project and its requirements and agree on an appropriate set of standards and procedures.

2. Minimum Extent

The engineered fill envelope must extend beyond the footprint of the structure to be supported. The minimum extent of the envelope should be defined from a geotechnical perspective by:

- at founding level, extend a minimum 1.0 m beyond the outer edge of the foundations, greater if adequate layout has not yet been completed as noted below; and
- extend downward and outward at a slope no greater than 45° to meet the subgrade

All fill within the envelope established above must meet the requirements of engineered fill in order to support the structure safely. Other considerations such as survey control, or construction methods may require an envelope that is larger, as noted in the following sections.

Once the minimum envelope has been established, structures must not be moved or extended without consultation with Peto MacCallum Ltd. Similarly, Peto MacCallum Ltd. should be consulted prior to any excavation within the minimum envelope.

3. Survey Control

Accurate survey control is essential to the success of an engineered fill project. The boundaries of the engineered fill must be laid out by a surveyor in consultation with engineering staff from Peto MacCallum Ltd. Careful consideration of the maximum building envelope is required.

During construction it is necessary to have a qualified surveyor provide total station control on the three dimensional extent of filling.

4. Subsurface Preparation

Prior to placement of fill, the subgrade must be prepared to the satisfaction of Peto MacCallum Ltd. All deleterious material must be removed and in some cases, excavation of native mineral soils may be required.

Particular attention must be paid to wet subgrades and possible additional measures required to achieve sufficient compaction. Where fill is placed against a slope, benching may be necessary and natural drainage paths must not be blocked.

5. Suitable Fill Materials

All material to be used as fill must be approved by Peto MacCallum Ltd. Such approval will be influenced by many factors and must be site and project specific. External fill sources must be sampled, tested and approved prior to material being hauled to site.

6. Test Section

In advance of the start of construction of the engineered fill pad, the Contractor should conduct a test section. The compaction criterion will be assessed in consultation with Peto MacCallum Ltd. for the various fill material types using different lift thicknesses and number of passes for the compaction equipment proposed by the Contractor.

Additional test sections may be required throughout the course of the project to reflect changes in fill sources, natural moisture content of the material and weather conditions.

The Contractor should be particularly aware of changes in the moisture content of fill material. Site review by Peto MacCallum Ltd. is required to ensure the desired lift thickness is maintained and that each lift is systematically compacted, tested and approved before a subsequent lift is commenced.

7. Inspection and Testing

Uniform, thorough compaction is crucial to the performance of the engineered fill and the supported structure. Hence, all subgrade preparation, filling and compacting must be carried out under the full time inspection by Peto MacCallum Ltd.

All founding surfaces for all buildings and residential dwellings or any part thereof (including but not limited to footings and floor slabs) on structural fill or native soils must be inspected and approved by PML engineering personnel prior to placement of the base/subbase granular material and/or concrete. The purpose of the inspection is to ensure the subgrade soils are capable of supporting the building/house foundation and floor slab loads and to confirm the building/house envelope does not extend beyond the limits of any structural fill pads.

8. Protection of Fill

Fill is generally more susceptible to the effects of weather than natural soil. Fill placed and approved to the level at which structural support is required must be protected from excessive wetting, drying, erosion or freezing. Where adequate protection has not been provided, it may be necessary to provide deeper footings or to strip and recompact some of the fill.

9. Construction Delay Time Considerations

The integrity of the fill pad can deteriorate due to the harsh effects of our Canadian weather. Hence, particular care must be taken if the fill pad is constructed over a long time period.

It is necessary therefore, that all fill sources are tested to ensure the material compactability prior to the soil arriving at site. When there has been a lengthy delay between construction periods of the fill pad, it is necessary to conduct subgrade proof rolling, test pits or boreholes to verify the adequacy of the exposed subgrade to accept new fill material.

When the fill pad will be constructed over a lengthy period of time, a field survey should be completed at the end of each construction season to verify the areal extent and the level at which the compacted fill has been brought up to, tested and approved.

In the following spring, subexcavation may be necessary if the fill pad has been softened attributable to ponded surface water or freeze/thaw cycles.

A new survey is required at the beginning of the next construction season to verify that random dumping and/or spreading of fill has not been carried out at the site.

10. Approved Fill Pad Surveillance

It should be appreciated that once the fill pad has been brought to final grade and documented by field survey, there must be ongoing surveillance to ensure that the integrity of the fill pad is not threatened.

Grading operations adjacent to fill pads can often take place several months or years after completion of the fill pad.

It is imperative that all site management and supervision staff, the staff of Contractors and earthwork operators be fully aware of the boundaries of all approved engineered fill pads.

Excavation into an approved engineered fill pad should never be contemplated without the full knowledge, approval and documentation by the geotechnical consultant.

If the fill pad is knowingly built several years in advance of ultimate construction, the areal limits of the fill pad should be substantially overbuilt laterally to allow for changes in possible structure location and elevation and other earthwork operations and competing interests on the site. The overbuilt distance required is project and/or site specified.

Iron bars should be placed at the corner/intermediate points of the fill pad as a permanent record of the approved limits of the work for record keeping purposes.

11. Unusual Working Conditions

Construction of fill pads may at times take place at night and/or during periods of freezing weather conditions because of the requirements of the project schedule. It should be appreciated therefore, that both situations present more difficult working conditions. The Owner, Contractor, Design Consultant and Geotechnical Engineer must be willing to work together to revise site construction procedures, enhance field testing and surveillance, and incorporate design modifications as necessary to suit site conditions.

When working at night there must be sufficient artificial light to properly illuminate the fill pad and borrow areas.

Placement of material to form an engineered fill pad during winter and freezing temperatures has its own special conditions that must be addressed. It is imperative that each day prior to placement of new fill, the exposed subgrade must be inspected and any overnight snow or frozen material removed. Particular attention should be given to the borrow source inspection to ensure only nonfrozen fill is brought to the site.

The Contractor must continually assess the work program and have the necessary spreading and compacting equipment to ensure that densification of the fill material takes place in a minimum amount of time. Changes may be required to the spreading methods, lift thickness, and compaction techniques to ensure the desired compaction is achieved uniformly throughout each fill lift.

The Contractor should adequately protect the subgrade at the end of each shift to minimize frost penetration overnight. Since water cannot be added to the fill material to facilitate compaction, it is imperative that densification of the fill be achieved by additional compaction effort and an appropriate reduced lift thickness. Once the fill pad has been completed, it must be properly protected from freezing temperatures and ponding of water during the spring thaw period.

If the pad is unusually thick or if the fill thickness varies dramatically across the width or length of the fill pad, Peto MacCallum Ltd. should be consulted for additional recommendations. In this case, alternative special provisions may be recommended, such as providing a surcharge preload for a limited time or increase the degree of compaction of the fill.