Geotechnical and Hydrogeological Investigation

Proposed Subdivision by Hilden Homes
Picton, Ontario

April 24, 2019
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Proposed Subdivision by Hilden Homes
Picton, Ontario

prepared for

The IBI Group

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The IBI Group– 1 PDF copy
Hilden Homes – 1 PDF copy
Malroz Engineering Inc. – 1 PDF copy

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Respectfully submitted,

MALROZ ENGINEERING INC.

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1.0 INTRODUCTION

Malroz Engineering Inc. (*Malroz*) was retained by Hilden Homes ("the client") to conduct a geotechnical and hydrogeological investigation in support of the construction of a new subdivision in Picton, Ontario (the "Site"). This letter has been prepared to provide results of field investigations conducted in June and July 2018 at the Site. We understand that the proposed subdivision plan is not finalized, but currently consists of residential and commercial properties, as well as open, green parkland. Roads and municipal services, including a stormwater management facility, will also be extended as a part of this development.

The proposed subdivision falls under the authority of Quinte Conservation, whose comments on the draft plan noted that the proposed development required a hydrogeological assessment to evaluate potential impacts on groundwater (quality and quantity) on the subject property and neighbouring water wells. Although the proposed subdivision will be serviced, the purpose of the hydrogeological assessment is to provide a baseline survey of neighbouring private water wells. PEC also indicated that the hydrogeological assessment should assess whether there are concerns regarding high water table and slope stability.

This letter summarizes the results of a geotechnical and hydrogeological investigation completed by *Malroz* in support of the Picton subdivision plan by Hilden Homes. Work for this investigation was completed in accordance with *Malroz*'s approved proposal (ref. 1126-100.00, May 14, 2018). The agreed scope of work consisted of the following:

1. Review of available reports and hydrogeological information, including Ontario Ministry of the Environment and Climate Change (*MOECC*) Water Well Records (WWR), in the vicinity of the Site;
2. Site visits to assess the potential for groundwater seeps, springs, and to review the surface water features in the vicinity of the Site;
3. Advancement of fourteen (14) boreholes at the Site to assess overburden, including eleven (11) borings to 3.7 metres or refusal, and three (3) to a lesser depth or 15 metres or refusal;
4. Installation of five (5) monitoring wells at select borehole locations, screened in bedrock and/or overburden, to observe static groundwater elevations;
5. A hydrogeologic study consisting of Water Well Records review, groundwater elevation evaluation, and groundwater sampling to assess baseline quality;
6. Evaluation of slope stability; and
7. Reporting of field and laboratory results.

Malroz was provided with the following related documents, which provided information for this report:

- Quinte Conservation's April 23, 2010, letter to PEC titled, "Proposed Draft Plan of Subdivision 13-T-10-501 and Zoning Amendment Z18-10, Part 19, Concession 1, South East of Carrying Place, Parts of Lots 1080 & 1081, Plan 24, Ward of Hallowell, Vacant Lands – 12697 Highway No. 33, Owner: David Clegg Holdings Inc., Agent: RFA Planning Consultant Inc."
- Quinte Conservation's February 29, 2016, letter to PEC titled, "Proposed Draft Plan of Subdivision 13-T-16-502, Part 19, Concession 1 South East of Carrying Place, Parts of Lots 1080 & 1081, Plan 24, Ward of Hallowell, Vacant Lands – 12697 Highway No. 33, Owner: David Clegg Holdings Inc., Agent: RFA Planning Consultant Inc."
- The Corporation of the County of Prince Edward's (PEC) March 1, 2017, letter titled, "David Clegg Holdings Limited, 12697 Loyalist Parkway – Draft Plans Circulation, Picton Ward – County File No. 13-T-16-502."
- Picton Secondary Plan Gas Well Map drawing, dated June 2014, IBI Group.

The recommendations and comments contained herein are based on factual information obtained during the investigation and are intended only for the use of project designers and engineers. They have been prepared with the understanding that the design will be carried out in accordance with applicable codes and standards. The General Conditions and Limitations (Section 6.0 of this report) form an integral part of this report.

2.0 METHOD OF INVESTIGATION

2.1 Fieldwork

A total of fourteen (14) boreholes were advanced across the Site, identified as BH-1 through BH-14. A Site Location Plan (Figure 1) and a Borehole Location Plan (Figure 2) are presented in Appendix A. All borings were advanced to practical auger or sampler refusal upon inferred bedrock, at depths ranging from 1.3 to 5.6 m below ground surface (mbgs). Boreholes were completed with a CME-55 drill rig equipped with solid stem continuous flight augers. Drilling was completed under the supervision of geotechnical personnel from *Malroz*. Borings were backfilled with drill cuttings and/or bentonite hole plug. Boring locations (UTM 18, NAD83) are presented in the following table:

Table 1 – Borehole Locations

Boring Location	Easting	Northing
BH-1	329451	4875466
BH-2/MW2	329516	4875574
BH3	329564	4875686
BH4	329516	4875169
BH-5/MW5 BH-5b	329571	4875306
BH6	329582	4875448
BH-7/MW7	329603	4875554
BH8	329633	4875671
BH9	329657	4875218
BH-10/MW10	329682	4875313
BH11	329706	4875441
BH-12/MW12	329714	4875605
BH-13	329859	4875553
BH-14	329846	4875680

Input By: CM
Validated By: DPH

Ground surface elevations at each borehole location were interpolated from the provided topographic survey.

An unused water supply well was also observed during the field investigation, located at coordinates 329692 E 4875535 N. A well tag was not observed for this well. Current water levels in this well were recorded by *Malroz* (Apprendix E).

Soil samples were collected while performing the Standard Penetration Test (SPT) in general accordance with the procedure as described in ASTM D1586. This consisted of freely dropping a 63.5 kg (140 lb.) hammer from a vertical distance of 0.76 m (30 in.), in order to drive a 51 mm (2 in.) outer diameter split-barrel (split spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground a distance of 300 mm (12 in.) was recorded as the SPT 'N' value, which correlates to the relative density of non-cohesive soils and is indicative of the consistency of cohesive soils. Each retrieved sample was placed in a resealable plastic bag, with multiple materials encountered in the same sample being bagged separately.

Upon completion of the drilling, several of the samples retrieved during the field investigation were selected and transported to the geotechnical laboratory for testing (SNC-Lavalin GEM Ontario Inc. in Kingston, ON).

Following the completion of drilling activities, *Malroz* staff visited the site on July 11th, 2018 to measure and record groundwater levels. A qualitative slope stability investigation was also completed by a *Malroz* geotechnical engineer at this time.

Groundwater sampling was conducted on August 9th, 2018. Three well volumes of water were purged from each of the monitoring wells using Waterra inertial pumps. Water was purged into a graduated bucket to measure the total volume of water removed. The intended sampling program included groundwater samples from three monitoring wells (MW5, MW10, MW12), as well as a duplicate for QA/QC. However, no sample was obtained from monitoring well MW10, as it was found dry after 6 L of purging. Samples were sent to Paracel Laboratories Ltd. for analysis.

2.2 Picton Gas Well Field

Further to the initial issuance of this report, Malroz was asked to comment on site observations in relation to presence of gas wells on the Site. During our site walkthroughs for both drilling fieldwork and subsequent hydrogeological work, no signs or presence of gas wells were apparent. If any suspected gas wells are uncovered during construction, work in the immediate area should be discontinued and a licensed well contractor should be retained to appropriately decommission the well, in accordance with applicable legislation.

3.0 SUBSURFACE CONDITIONS

Details of the subsurface conditions encountered are presented in the borehole records presented in Appendix B of this report. We strongly emphasize, however, that the soil types, their sequence, thickness and physical properties may vary between borehole, sample and test locations, both vertically and horizontally. In addition, it should be noted that the information provided is solely for general planning purposes and should not be used for detailed quantity take-offs.

Borehole observations are summarized in the following sections.

3.1 Local Geology / Physiography

The regional overburden consists primarily of Quaternary deposits and till. In the Site area, overburden materials are generally less than 1 m thick due to shallow bedrock (Dillon, 2004).

The bedrock geology of Prince Edward County was mapped by D.M. Carson with the Ontario Geologic Survey (OGS) in 1981. The bedrock around the Site is predominantly limestone, characterized by medium brown and grey, finely crystalized bioclastic limestone, and overlain by bluish-grey, fine to medium crystalline limestone. The contact between both limestones is expected to be located just east of the Site.

The topographic gradient at the Site is remnant of a 30 m high fault scarp, dipping towards the northwest, and belonging to the Picton Fault. The Picton Fault is a normal fault which extends northwards through Picton Bay towards Long Reach, and south towards and past Picton.

The nearest observed body of water is Picton Bay, located approximately 200 m west of the Site. A small tributary creek crosses through the site, however, the creek bed was dry during both site visits in July 2018. Topography is moderately graded by a westward slope throughout the proposed site area.

3.2 Surficial Materials

3.2.1 Topsoil

A surficial covering of topsoil was encountered at all borings, with the exception of BH-10, BH-13 and BH-14. Topsoil thicknesses ranged from 0.1 to 0.3 m, with the exception of BH-8 location, where 1.6 m of topsoil mixed with wood fragments and native soil (silty sand, some clay and gravel) was encountered. It should be noted that in our experience, topsoil thicknesses can vary greatly between boring locations. Contractors should carry out their own investigations in order to confirm topsoil depths for quantity takeoffs.

3.2.2 Wood Mulch

A surficial covering of wood fragments/mulch was encountered at boring BH-13 location, approximately 0.2 m in thickness. Other surficial piles of mulch were located throughout the cleared area of the Site during the field investigation.

3.3 Native Silty Sand

A brown to greyish brown native silty sand, with trace to some clay and trace to some gravel was encountered at borings BH-2, BH-3, BH-5b, and BH-6 through BH-14. This material was visually described as being damp to moist, and was found to be between 0.6 and 1.8 m in thickness, and was found to be interbedded with native silty clay at borings BH-10 and BH-12. The relative density of this material was found to range from very loose to very dense, with SPT 'N' values measured between 3 and 75 blows per 300 mm of penetration. Refusal to the sampler was encountered within this material at samples identified as BH-2/SS2, BH-8/SS4, BH-9/SS3, BH-10/SS4, BH-10/SS5, and BH-13/SS2.

3.4 Native Clayey Sandy Silt

A brown to grey native clayey sandy silt was encountered at borings BH-1, BH-4 and BH-5. This material was visually described as being damp at BH-1 and moist to wet at BH-4 and BH-5. The unit was found to be between 1.2 and 2.1 m in thickness. This silt was found to be generally non-cohesive, with a relative density found to range from very loose to compact with SPT 'N' values measured between 3 and 17 blows per 300 mm of penetration, with the exception of BH-1/SS2 with an SPT 'N' value of 58 blows per 300 mm of penetration. This material was noted to contain significantly more gravel towards the end of boring BH-5 (from 4.4 mbgs to end of borehole at 4.6 mbgs).

3.5 Native Silty Clay

A native silty clay with trace sand was encountered at borings BH-10 through BH-12. A thin, sandier zone of this stratum was also encountered at boring BH-7 (0.1 m in thickness). This material was visually described as being generally moist, and was found to be between 0.4 and 0.8 m in thickness. The consistency of this material was estimated to range from soft to hard, with SPT 'N' values ranging from 4 to 47 blows per 300 mm of penetration.

3.6 Limestone Bedrock

Possible weathered limestone was encountered at borings BH-1 through BH-4, BH-6 through BH-8, and BH-12 through BH-14, prior to encountering auger refusal. This material was inferred at these locations via auger grinding during drilling and observation of drill cuttings, with the exception of boring BH-7, where the material was sampled –shale with interbedded silty clay material was noted here, prior to encountering auger refusal. Bedrock in the immediate area of the Site, according to published geological mapping (Ontario Geological Survey, 1981) is expected to consist of interbedded limestone and shale of the Verulam formation.

4.0 LABORATORY TESTING

4.1 Geotechnical Laboratory

Samples recovered during drilling were transported to *Malroz*'s office in Kingston, Ontario where they were reviewed by a geotechnical engineer. A total of three (3) samples were submitted to SNC-Lavalin GEM Ontario Inc. in Kingston, Ontario for testing in their geotechnical laboratory. Testing consisted of evaluation of sample gradation and plasticity. Two (2) samples identified as BH-4/SS2 and BH-9/SS3 were found to be non-plastic. Geotechnical laboratory reports are presented in Appendix C of this report. A summary of the testing performed and results are presented in the following table:

Table 2 – Summary of Geotechnical Laboratory Testing

Sample ID	Depth (mbgs)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index
BH-4/SS2	0.8-1.3	0	30	49	21	-	-	-
BH-5/SS3	1.5-2.1	4	21	52	23	32.7	15.8	16.9
BH-9/SS3	1.5-1.9	11	37	33	19	-	-	-

Input By: DPH
Validated By: DH

4.2 Analytical Laboratory

Three groundwater samples including one QA/QC duplicate sample, were collected from two shallow wells (MW5 and MW12) and analysed for a suite of parameters commonly used for hydrogeological site assessments. Additional monitoring wells were intended for sampling (MW5 and the supply well), however these were found to be dry or obstructed and no samples were obtained. Since the proposed development will be fully serviced, it is not necessary to evaluate the water chemistry for drinking water parameters. The groundwater results are tabulated in Table 1 (Appendix D) and represent the groundwater chemistry at the Site.

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 Hydrogeological Investigation

Prior to completing the investigation, water well records from about 500 m around the site were obtained from the MECP's water well records online database, and a cursory review of the Quinte Groundwater Study (Dillon, 2004) was completed to assist with an initial understanding of the local and regional hydrogeology. The field component of the hydrogeological investigation included borehole drilling, monitoring well installation, water level monitoring, groundwater sampling and analyses. The hydrogeologic investigation is summarised below.

5.1.1 Regional Hydrogeology

The regional hydrogeology is influenced by Lake Ontario. The predominant aquifer in Prince Edward County is the fractured bedrock, notably the top 10 to 30 m of the Paleozoic sequence. Both vertical and horizontal fractures are present, rendering the aquifer largely unconfined (Dillon, 2004). There is also a significant flow system in the regional overburden, where sand and gravel deposits are deeper due to glacial features (e.g., drumlins, etc). The overburden flow system can act as a recharge zones for the unconfined bedrock aquifer.

Groundwater flow patterns are largely driven by topography and are especially prone to local features, given the irregular land surface around the County. Most of Prince Edward County represents a water table low and drainage point for the Quinte Region, with the exception of the bedrock plateau located about 7 km northeast of Picton, ON, which represents the local water table high and recharge zone (Dillon, 2004).

Groundwater flows laterally through horizontal fractures along the limestone aquifer. Well yield varies but is generally poor and the water quality is hard, with some salt and sulphur impact (Dillon, 2004). The overburden sand and gravel aquifers are present throughout the County. However, these aquifers tend only to serve as bedrock recharge, due to heavy fracturing in the limestone and a lack of lateral continuity in the shallow overburden units.

5.1.2 Local Hydrogeology

There are several Water Well Records available for wells around the site (MOE Open Data catalogue). The data from wells located east of the Site suggest that the Picton fault line is located between Bridge St. and the shoreline of Picton Bay. As a result, the overburden depth varies from 6 m (by the Bay) to 14 m (by Bridge St.). Data from wells located south of the Site (along Union St), record 1.5 to 3 m of clay overburden underlain by limestone. Depth to water in this area is widely variable: from 3 to over 20 m.

On July 11, 2008, *Malroz* collected water level data at monitoring wells installed at the Site in June 2018, in addition to the existing unused water supply well (Appendix E). The depth to water varied between 1.0 to 3.5 m in the overburden wells and approximately 13.5 mbgs in the supply well (bedrock aquifer). Shallow groundwater in the soil overburden is expected to be localized, and encountered in low lying areas of the Site. Based on the measured elevations, groundwater flow direction at the Site was interpreted to be north-westwards towards Picton Bay.

Groundwater analytical results from two shallow wells (MW5 and MW12) are tabulated in Table 1 (Appendix D) and represent the chemistry at the Site.

5.2 Excavation and Temporary Shoring

It is expected that typical residential homes constructed as a part of this subdivision will have one (1) below grade level. Accordingly, excavations of up to approximately 3 to 3.2 m may be expected at this Site, but will depend on final site grading plans.

All excavations and construction of any shoring should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. The OHSA regulations require that if workers must enter an excavation deeper than 1.2 m, the excavation must be suitably sloped and/or braced in accordance with OHSA requirements. OHSA specifies the maximum slope of excavations into four (4) broad soil types, summarized as follows:

Table 3 – OHSA Soil Types

<table>
<tr><th>Soil Type</th><th>Maximum Slope Inclination</th><th>Base of Slope Location</th></tr>
<tr><td>Type 1</td><td rowspan="3">1 horizontal to 1 vertical</td><td rowspan="2">Within 1.2 m of base of excavation</td></tr>
<tr><td>Type 2</td></tr>
<tr><td>Type 3</td><td rowspan="2">From bottom of excavation</td></tr>
<tr><td>Type 4</td><td>3 horizontal to 1 vertical</td></tr>
</table>

At this Site, native soils above the water table could be considered as Type 3 to Type 4, provided they are unaffected by seepage. As mentioned above, low lying areas of the Site may encounter groundwater within overburden soils. Any soils which are affected by seepage should be considered as Type 4. The highest Soil type within any given excavation shall govern the excavation side slopes for the entire excavation.

Excavations into overburden soils should be relatively easy using conventional excavating equipment; however, fill soils commonly contain cobbles, boulders and construction debris. Given the location of the Site, it is possible that zones within the fill could contain large blast rock. The native soils should be expected to contain large particles (i.e. cobbles and boulders).

Stockpiles of excavated materials should be kept away from the edges of open excavations by a distance at least equivalent to the depth of the excavation in order to avoid slope instability. Care should be taken in order to avoid overloading any underground services/structures from any construction stockpiles. It should be noted that this distance is also applicable to the passage of heavy machinery near excavations. This condition should be respected at all times, unless specific studies are conducted for individual cases.

5.3 Dewatering

Dewatering in excavations for proposed typical residential homes in this subdivision is expected to be able to be controlled substantially using conventional construction pumps and filtered sumps. Surface grades should be sloped away from excavations where possible in order to minimize the amount of surface water infiltration. Excavating during dry seasonal periods may minimize dewatering requirements.

5.4 Foundations

Foundations for the proposed residential homes may consist of conventional spread, strip or pad footings constructed on a competent native soil subgrade or limestone bedrock, which has been inspected by a geotechnical engineer (or their designate). Foundation design is expected to be carried out in accordance with Part 9 of the Ontario Building Code (OBC) 2012 – a maximum allowable bearing resistance of 75 kPa may be utilized for footings placed on native soils at this Site, or 300 kPa on weathered rock, in accordance with table A-9.4.4.1 of OBC. Any soft soils encountered shall be removed (i.e. in the vicinity of boring BH-5).

Excavations for sumps or utility trenching should not penetrate a zone extending approximately 10 horizontal to 7 vertical from the bottom outside edge of any new or existing footings.

5.4.1 Frost Protection

A minimum of 1.4 m of soil cover should be provided for foundations in the Picton area in order to prevent detrimental frost action. If adequate soil cover is not available, foundations should be insulated. Insulation details should be in accordance with the product manufacturer's instruction. Contractors should be aware that some insulation details require pour concrete directly over top of insulation materials.

5.5 Slope Stability

A slope stability assessment was carried out based on the conditions observed at the boring locations in addition to the topographic information provided by the Client's surveyor. Stability analyses were carried out using Rocscience Slide 2018, with several methods of analyses used to evaluate the factor of safety: Bishop's, Morgenstern-Price and Janbu methods. The slope conditions and results of the stability analyses are presented in the following sub-sections.

5.5.1 Slope Conditions

A site visit was carried out by a geotechnical engineer from *Malroz* on July 11, 2018. The height of the slope, as obtained from the provided topographical information was approximately 22 m, over a length of roughly 290 m. The slope was covered in light vegetation, with few mature trees, none of which were noted to be significantly slumping. Mulch piles were noted over the slope face (presumably following previous vegetative clearing of the Site). No evidence of running water seepage on the slope surface or near the toe was noted. One (1) cross-section was analysed, as shown in the attached plan and profile sketch in Figure 3 in Appendix A of this report.

5.5.2 Stability

The calculated factor of safety of the existing slope along the analysed section was found to be 2.68, which exceeds the generally accepted value of 1.5 for long-term stability of slopes. Several loading conditions with sustained loads of 50 kPa to 100 kPa were introduced over the slope as a part of this exercise. Factors of safety in excess of 1.5 were still achieved. Accordingly, providing the developed profile is generally of the same or shallower slope as the existing, we would consider this slope stable over the long-term, provided no active erosional conditions are induced. General considerations when developing on slopes include maintenance of as much vegetation and tree cover as possible on the existing slope, ensuring surface water is directed away from or down the slope in a suitable manner, and avoiding significantly surcharging any portion of the slope with any significant quantity of excess soils (however, less of a concern with this particular slope given the shallow overburden conditions).

5.6 Pavement Design

5.6.1 Design Inputs

No traffic data was provided to Malroz for pavement design input for this subdivision. However, given the size and location of the subdivision, an AADT of 500 vehicles with 1% heavy vehicle traffic was selected for new design purposes. No growth factor was considered. Table D-5 of the *MTO Materials Information Report MI-183 –Adaptation and Verification of AASHTO Pavement Design Parameters for Ontario Conditions* (March, 2008) (“MI-183”). Based on the above, the projected number of ESALs over a standard 20 year design period would be 18,300. The required structural number to achieve this design traffic loading would be approximately 59. Traffic data used in the design is summarized in the following table:

Table 4 – Summary of Design Traffic (ESALs)

ADTT	% Direction	LDF	Truck Factor	% Growth	ESALs (20 y)
500	50	100	0.5	0	18,300

While Marshall hotmix asphalt mixes are presented in the design options below, they may be substituted for equivalent Superpave mixes (traffic Category A, for this Site). All mix designs should be reviewed by a qualified engineer or engineering technologist.

5.6.2 Design Recommendations

Malroz carried out the design in accordance with the AASHTO 1993 *Guide for the Design of Pavement Structures*. Input parameters were selected in accordance with MI-183. As this is a new subdivision construction, a single pavement section for new design was evaluated which meets the above noted pavement structural requirements, based on our assumptions regarding future traffic. If traffic projection data is available or projection assumptions should be adjusted, *Malroz* should be contacted to re-evaluate the proposed design section.

Based on the above noted design inputs, a new pavement section consisting of 50 mm of HL3, overlying a base layer of 150 mm of new OPSS Granular ‘A’, and 200 mm of OPSS Granular ‘B’ Type I or II would provide a suitable pavement structural capacity. This design would produce a structural number of approximately 60.

5.6.3 Materials and Construction Considerations

Paving work should be completed in accordance with the requirements of applicable OPSS and municipal standards. All asphalt mix designs should be reviewed prior to the commencement of construction.

HMA used in this project should meet the minimum requirements of OPSS 1150/1151 (depending on whether Marshall or Superpave mixes are utilized). Asphalt cements should be minimum grade of PG 58-28, and meet the requirements of OPSS 1101.

Tack coat should be applied between any vertical surfaces or joints including curbs, abutting and walls, etc., butt and lap joints and at all tie-ins to other existing asphalt. SS-1 emulsified asphalts used for this purpose should meet requirements of OPSS 1101.

Proof rolling of all subgrades should take place prior to placement of granular base materials and should consist of running a loaded tandem or triaxle dump truck over the subgrade under the presence of qualified geotechnical personnel. Any localized identified weak areas should be sub-excavated and replaced with suitable fill soils as directed by the geotechnical personnel.

5.7 Stormwater Management Pond

We understand that one or more stormwater management ponds will be required to be constructed in this subdivision. Based on the generally shallow overburden conditions encountered, with the exception of in the vicinity of boring BH-5, BH-10 and BH-11 (3 m or more of overburden), excavations for these areas may extend into rock, depending on final site grading, required pool elevations etc.

As mentioned above, pond locations and configurations have not been finalized at the time this report was written. If pond geometry requires sideslopes steeper than 3H:1V, the configuration, location, proposed pond materials and construction method should be reviewed by a geotechnical engineer.

If ponds are constructed in overburden, subsequent to completing the required excavations, the exposed surface of the subgrade should be proof rolled with heavy construction equipment, under the supervision of qualified geotechnical personnel. Any identified weak areas should be subexcavated and replaced with suitable fill soils as directed by geotechnical personnel. If pond excavations extend into rock, excavation sidewalls and base should be scaled.

If ponds are constructed in the vicinity of boring BH-5, within predominantly clayey/silty soil, hydraulic conductivity of this material is expected to be less than 10^{-6} cm/s. However, it is recommended that this be confirmed with an in-situ infiltration test – seams of higher permeability may be present within the material (i.e. coarse sand seams, etc.).

If ponds are constructed in bedrock, use of a geosynthetic clay liner ("GCL") may be required. Manufacturers should be consulted in order to determine the most appropriate product for the desired base/side permeability, in addition to construction requirements, however we would recommend that a scrim reinforced product be used.

Upon completion of construction, containment berms and outer slopes should be seeded/planted in order to prevent surficial erosion and promote naturalization. Surface grades around any ponds should be sloped away in order to minimize potential for damage to pond sidewalls from excess surface flows. Additional protection should be provided in inflow/outflow areas, i.e. provision of rip rap/armour stone.

5.8 Engineered Fill

Engineered Fill application may be required on this project to raise subgrade elevations and under pavement areas or in the event of over-excavation for foundations. For any operation to be considered Engineered Fill, the following criteria must be satisfied:

- Materials used as Engineered Fill must be uniform and homogenous. The material should be free of deleterious materials and organics;
- Prior to the placement of Engineered Fill, it must be assessed in a geotechnical laboratory for, at a minimum, gradation and Standard Proctor analyses;
- The material must be within +/- 2% of its optimum moisture content, as determined through laboratory testing;
- Engineered Fill operations must take place under the supervision of a geotechnical engineer or their designate;
- Suitable compaction equipment must be selected for the operation, based on the material to be compacted;
- Materials should be placed in lifts which are suitable for the compaction equipment utilized, but generally not greater than 0.2 m loose lifts;
- Density testing must be taken on each lift of Engineered Fill. Any Engineered Fill which is tested and found to be outside of the specified density range shall be either removed, reworked or retested; and
- Under no circumstances shall frozen material be placed in any Engineered Fill operation.

6.0 GENERAL CONDITIONS AND LIMITATIONS

This report was completed for the specific needs of Hilden Homes and is based on a specific scope of work which is defined in the mutually agreed upon workplan. The scope of work has limitations as described throughout the report and in the notice to reader. Data, tables, charts, and interpretive illustrations presented in this document are instruments of service for this mandate and can only be properly evaluated when reviewed together with the accompanying report. Reference to this report should only be made to the complete signed document.

By issuing this report, *Malroz* is the Geotechnical Engineer of Record for this project. It is recommended that *Malroz* be retained during construction of all foundations, during earthwork operations and paving operations. The intent of this requirement is to verify conditions encountered during construction are consistent with the findings in the report and, that inherent knowledge developed as a part of our study is correctly carried forward to construction phases. We should be retained to review whether our recommendations have been applied appropriately, once drawings and specifications are complete. Without this review, *Malroz* will not be liable for any misunderstanding of our recommendations or their application and adaptation into final designs.

The work performed in this report was carried out in accordance with the terms and conditions made as a part of our proposal and/or contract pursuant to which this report was issued, in a manner consistent with that level of care and skill ordinarily exercised by members of the Geotechnical Engineering profession currently practicing under similar conditions in the same locality. The conclusions presented in the report are based solely upon the scope of services, governed by the time and budgetary considerations to which this work was subject.

The factual data, recommendations and comments in this report pertain to the specific project as described in the report, and are not applicable to any other project or location. If the project is conceptually modified or changes location, or if it is not initiated within twelve months of the date of this report, *Malroz* should be given an opportunity to confirm that the information in this report is still valid and/or applicable.

The comments in this report are intended only for the guidance of project designers and engineers. Contractors bidding on or undertaking the work should rely on their own investigations, as well as their own interpretations of the factual borehole and in-situ test information, and how subsurface conditions may affect their work.

This report must be read as a whole, as sections taken out of context can be misleading. Drafts and working copies, whether or not marked as “draft”, “for discussion purposes” or otherwise, do not necessarily reflect *Malroz*’s final opinion following consideration of all matters which are subject to the study giving rise thereto; they are issued for comment and information purposes only, and are subject to change and should not be relied upon in any way or for any purpose.

It is important to emphasize that a soil investigation is, in fact, a random sampling of a site and the comments included in this report are based solely on the results obtained at the borehole locations only. Soil and groundwater conditions between and beyond the borehole locations may differ both horizontally and vertically from those encountered at the borehole locations and may become apparent during construction, which could not be detected or anticipated at the time of our investigation. Should any conditions at the site be encountered which differ from those found during this investigation, we request that we be notified immediately in order to permit a reassessment of our comment and recommendations. If changed conditions are identified during construction, no matter how minor, the recommendations in this report shall be considered invalid until sufficient review and written assessment of said conditions by *Malroz* has been completed.

7.0 CLOSURE

We trust that this report meets your present requirements. Please do not hesitate to contact us should there be any further questions or comments.

Respectfully Submitted,

Malroz Engineering Inc.,

LICENSED PROFESSIONAL ENGINEER
D. HILL
100160715
June 10, 2019
PROVINCE OF ONTARIO

per: Camille Malcolm, M.Sc., G.I.T.
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encl.: Appendix A – Figures
Appendix B – Record of Boreholes
Appendix C – Geotechnical Laboratory Results
Appendix D – Groundwater Sampling Results
Appendix E – Water Level Measurements

8.0 REFERENCES

Carson, D.M., 1981, Paleozoic Geology of the Belleville-Wellington Area, Southern Ontario, Ontario Geological Survey Preliminary Map (P.2412). 1: 50 000.

Dillon Consulting Ltd., 2004, Quinte Regional Groundwater Study 2004 Final Report (No. 03-1813). Quinte Conservation: Belleville ON.

Appendix A
Figures

Appendix B
Record of Boreholes

Appendix C
Geotechnical Laboratory Results

Appendix D
Groundwater Analytical Results

Appendix E
Water Level Measurements