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REPORT ON

**HYDROGEOLOGICAL INVESTIGATION  
AND TERRAIN EVALUATION  
PROPOSED RESIDENTIAL SUBDIVISION  
819 BURRITTS RAPIDS ROAD  
MERRICKVILLE-WOLFORD  
ONTARIO**

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Submitted to:

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## **1.0 INTRODUCTION**

Kollaard Associates Inc. was retained by 2873633 Ontario Inc. of Merrickville, Ontario to undertake a hydrogeological investigation and terrain evaluation for a site located on Burritts Rapids Road. The site is located within Part of Gore Concession in Wolford, within Merrickville-Wolford, Ontario (See Key Plan, Figure 1).

The site consists of an area of approximately 22.7 hectares (56 acres) located on the west side of Burritts Rapids Road on the Rideau River within the village boundary of Merrickville-Wolford, Ontario. It is proposed to subdivide the site into some 29, average 0.65 hectare lots (minimum 0.4 hectare) for single family dwelling construction purposes. The proposed dwellings will be serviced by private septic systems and wells. The subject site is currently mostly treed with the exception of the building envelopes of the existing house and detached garage. The west and north sides of the subject property border the Rideau River and there is an unevaluated wetland within the south portion of the property.

The site is bordered on the west and north by the Rideau River, on the south by Burritts Rapids Road, and on the east by farmland and forest. The existing dwellings in the area are serviced by private septic systems and wells.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by glaciomarine and alluvial deposits of silt, clay, sand and gravel. The bedrock geology map indicates that the bedrock underlying the site consists of dolostone and sandstone of the Beekmantown Group (Attachment A).

## **2.0 FIELD PROCEDURES**

The objectives of this study were:

- to determine the shallow subsurface soil and groundwater conditions relative to the design and impact of Class IV sewage disposal systems; and



- to investigate the potential quantity and quality of groundwater available from drilled wells for domestic supply

## 2.1 Terrain Evaluation

The field work for the terrain evaluation was carried out on March 2, 2023, during which time a total of ten test pits (numbered TP1 to TP10, inclusive) were put down across the site. The approximate locations of the test pits are provided in Site Plan, Figure 2 and the soil descriptions and groundwater conditions are provided in the Record of Test Pits.

The test pits were put down throughout the site. The test pits were advanced to depths of approximately 3.1 to 3.6 metres below the existing ground surface using a track mounted excavator. A member of our engineering staff recorded the soils types, depths to strata changes, and groundwater conditions at each test pit location. Groundwater conditions in the test pits were noted at the time of drilling. Soil samples were obtained from test pits TP4 and TP9 for laboratory grain size distribution analysis of representative analyses for each soil type encountered at the site. All particle analysis results are provided as Attachment C.

To obtain representative samples of the upper groundwater at the site for background testing of nitrogen species, four monitoring wells were installed using hand augering equipment. The augerholes were constructed on March 28 and 29, 2023. The augerholes were identified as AH1, AH2, AH5, AH10. The monitoring wells were installed adjacent to TP1, TP2, TP5 and TP10, but distant enough to avoid being placed in disturbed soil areas, generally about 5 to 10 metres away from the test pit locations. The annular space created around the screened interval and for about 30 centimetres above the screen at each augerhole was filled with silica sand and the remainder of the annular space to ground surface was filled with bentonite pellets. The water levels in the standpipes were measured on April 18, 2023, and water samples were obtained from the standpipes for testing of background nitrogen levels. The parameters that were included were tested for nitrogen species including nitrites, nitrates, Total Kjeldahl Nitrogen (TKN) and ammonia. The results are provided in Attachment D.

### Monitoring Well Sampling Procedure

The sampling procedure was carried out using sampling protocols and methods described in *“Association of Professional Geoscientists of Ontario Guidance for Environmental Site Assessments*



*under 153/04 (as amended), April 2011*". On April 18, 2023, the static water levels were measured in each of the standpipes. The standpipes were subsequently purged of approximately three well volumes, and allowed to recover between purges, prior to water samples being obtained and tested for nitrogen species, including nitrites, nitrates, TKN and ammonia. As no drilling fluids were used during monitoring well construction, the purging of three well volumes was considered to be sufficient to obtain groundwater samples that were representative of the groundwater in the shallow aquifer. The standing water in each monitoring well was purged using a mechanical displacement pump.

## **2.2 Groundwater Supply Investigation**

During the original investigation, to determine the quantity and quality of groundwater available for domestic water supply, four test wells, numbered TW1, TW2, TW3, and TW4 were pump tested and sampled. The approximate locations of the test wells are shown on the attached Site Plan, Figure 2. Air Rock Drilling Company Limited of Richmond, Ontario, drilled three new water supply wells on the subject property for the purpose of this hydrogeological investigation on February 28, March 1, and March 2, 2022. The other well (TW4) is an existing water supply well located at the single family dwelling. A second well that is also located onsite and is also used for the existing single family dwelling (OW1) was used as an observation well during the pumping tests. No well record could be matched for either TW4 or OW1. The locations of the test wells and other area well records are provided herein as Well Locations, Figure 2. The well records for the TW1, TW2 and TW3 and the Certificates of Compliance for the recently constructed test wells are provided herein as Attachment B.

The water well records for the test wells supplied by the well driller indicate that nominal 15 centimetre inside diameter steel casings were installed through the overburden and were set well into the bedrock and grouted in place. The wells were drilled to final depths using a 15 centimetre diameter bit and completed as an open hole in the bedrock. TW1, TW2, and TW3 were drilled into the bedrock to final depths of 33.5, 42.7, and 25 metres, respectively, below the existing ground surface. All three test wells were cased with casing total lengths of between 12.2 metres (40 feet) and 13.1 metres (43 feet) below ground surface. Each test well was cased 1.8 metres into the bedrock. TW4 is an existing well that services the existing onsite dwelling. Despite research, a matching well record could not be located for either onsite wells. It is understood that the well was constructed in about 1987 (based on the age of the dwelling). The well has a 6 inch diameter



casing with wellhead above grade and the well depth was measured to be 24.6 metres. Based on that information, the well is located in the same aquifer as the other wells.

Pumping tests were conducted on TW3, TW1, TW2, and TW4 on June 16, June 17, June 27 and June 28, 2022, respectively. A second pumping test was carried out at TW3 on December 10, 2024. The testing consisted of 6 hour duration constant discharge rate pumping tests. During the pumping tests, water level measurements were made on a regular basis to monitor the drawdown of the water level in the wells in response to pumping. After the pumping period, the pump was shut off and the recovery of the water level in the test well was monitored. During the pumping tests, water levels at adjacent test wells were monitored, using pressure transducers, to determine the potential interference effects between the wells.

Groundwater samples were collected from the test wells at about hour 3 and at hour 6 of the pumping tests to characterize groundwater quality. The groundwater samples from the test wells were collected and prepared/preserved in the field using appropriate techniques and submitted to Eurofins Environmental Laboratory in Ottawa, Ontario for the chemical, physical and bacteriological analyses listed in the Ministry of the Environment (MECP) guideline entitled Procedure D-5-5, Technical Guideline for Private Wells: Water Supply Assessment, August 1996 in addition to select heavy metals. The temperature, pH, turbidity, total dissolved solids (conductivity), and residual chlorine levels of the groundwater were measured at periodic intervals during the pumping tests.

The groundwater samples from the four test wells were collected and prepared/preserved in the field using appropriate techniques and submitted to Eurofins Environmental Laboratory in Ottawa, Ontario for the chemical, physical and bacteriological analyses listed in the Ministry of the Environment (MOE) guideline entitled "Technical Guideline for Water Supply Assessment for Subdivision Development on Individual Private Wells," dated July 1992.



### **3.0 TERRAIN EVALUATION**

#### **3.1 Soil and Groundwater Conditions**

This section provides a summarized account of the subsurface soil and groundwater conditions on the subject property based on the information obtained at the test pit locations. Details of the subsurface conditions at the test pit locations are presented in the attached Record of Test Pits. It is noted that in some cases the stratigraphic boundaries within the overburden represent a transition between soil types rather than an exact plane of geologic change. Subsurface conditions differing somewhat from those reported can be expected to exist at the site.

The soil stratigraphy is characterized as follows:

##### **3.1.1 Topsoil**

All ten test pits encountered 0.30 to 0.60 metres of native topsoil, which consisted of a combination of sand, silt, organic material (humus).

##### **3.1.2 Silty Clay**

Native deposits of red brown silty clay were encountered at all test pits. With depth, silty clay transitions to grey brown at depths of between 0.6 and ~1.2 metres below ground surface. One sample of the soil was submitted for hydrometer analysis (Section 3.1.4). The silty clay is considered to be of low permeability with ~54% clay sized particles and ~44% silt sized particles. The representative charts in the MMAH *Supplementary Guidelines to the 1997 OBC* were used to correlate the soil type to hydraulic conductivity estimates of the soil. The saturated hydraulic conductivity of the soil is between  $1.0 \times 10^{-6}$  and  $1.0 \times 10^{-7}$  cm/s. This is considered to be low permeability.

##### **3.1.3 Glacial Till**

At TP9, the soil consisted of red brown silty sand, some gravel, cobbles, boulders, trace clay (glacial till). The glacial till becomes grey brown at about 1.2 metres, then grey at a depth of about 2.3 metres below ground surface. Till is characterized as a combination of sand, gravel, silt, clay,



cobbles and boulders. The till sample obtained for hydrometer analysis consists of a coarse grained soil with some ~48% of particles passing the #200 sieve. The estimated hydraulic conductivity of the till is  $1.3 \times 10^{-5}$  cm/s (Section 3.1.4). As the soil is coarse grained with less than 10% clay and ~39% silt content, the soil is considered to be of medium permeability. Typically clay content of greater than 30% to 40% would be low permeability/practically impervious.

All of the above noted test pits were terminated in the silty clay (TP1 to TP8, and TP10) or glacial till (TP9) at depths ranging between 3.1 to 3.6 metres. Kollaard Associates Inc. characterizes the receiving aquifer at the site as the red to grey silty clay at nine of the test pit locations. The surficial soil is fine to medium sand based on textural indicators for grain size. Two representative samples were obtained from TP4 and TP9 from depths of 1.6 and 2.5 metres and submitted to a lab for particle size analyses. The resulting sieve analyses are provided as Attachment C and discussed below.

### 3.1.4 Hydrometer Analyses

Representative surficial samples were obtained from TP4 and TP9 from depths of 1.6 to 2.5 metres and submitted to a lab for hydrometer analyses to characterize the receiving aquifer at the site.

The results of hydrometer testing (ASTM D422 and D2216) on two samples of soil (TP4 – 1.6 – 2.5m, TP9 – 1.6 – 2.5m) indicate the samples have the following:

Sample	Depth (metres)	% Gravel	% Sand	% Silt	% Clay
TP4	1.6 – 2.5 m	0.0	1.6	44.4	54.0
TP9	1.6 – 2.5 m	10.6	41.6	38.8	9.0

The results of the laboratory testing are located in Attachment C.

The hydraulic conductivity was estimated for the glacial till sample using the particle size analyses, as follows:

$$k = 0.35 (D_{15})^2$$

Where k = hydraulic conductivity, in cm/s

$D_{15}$  = the particle diameter where 15% of soil is passing, in mm



Sample	D <sub>15</sub> (mm)	K (cm/s)
TP9 (Glacial Till)	0.006	1.3 x 10 <sup>-5</sup>

The samples obtained from TP4 could not be assessed using the above noted procedure as the D<sub>15</sub> particle size approximation is only applicable to coarse grained soils (i.e. where at least 50% of the soil particles are larger than #200 sieve (0.075 mm)). As a result, the hydraulic conductivity was estimated by comparing the sieve analysis sample to representative charts in the MMAH *Supplementary Guidelines to the 1997 OBC*. The soil can be characterized as an inorganic clay. The hydraulic conductivity of the soil is estimated to be between 1.0 x 10<sup>-6</sup> and 1.0 x 10<sup>-7</sup> cm/s. This is considered to be very low permeability/practically impervious.

Based on the above noted information, the hydraulic conductivity of the soil is expected to range between ~1.3 x 10<sup>-5</sup> cm/s to 1.0 x 10<sup>-7</sup> cm/s for till to silty clay soils. The soil can be characterized as silty clay or inorganic clay. This is considered to be a soil of very low permeability.

The surficial soils at the site are of low to very low permeability. Based on the soils information, the site is not considered to be hydrogeologically sensitive as there is a sufficient thickness of soils that are not highly permeable overlying the bedrock.

### 3.2 Shallow Groundwater Sampling

To obtain representative samples of the upper groundwater at the site for background testing of nitrogen species, four monitoring wells were installed using the *ASTM Standard D5092-04(2010) Standard Practice for Design and Installation of Groundwater Monitoring Wells*. The testing includes nitrogen species nitrates, nitrites, Total Kjeldahl Nitrogen and ammonia. For details on construction and purging procedures see Section 2.1. For monitoring well locations, see Site Plan, Figure 2.

The following table summarizes the laboratory results for nitrogen measured at the shallow monitoring wells installed adjacent to four test pits.



Table 3.2.1

Analyte (mg/L)	AH1	AH2	AH5	AH10
Sample Date (yy-mm-dd)	23-04-18	23-04-18	23-04-18	23-04-18
N-NO <sub>2</sub>	<0.10	<0.10	<0.10	<0.10
N-NO <sub>3</sub>	<0.10	<0.10	<0.10	<0.10
N-NH <sub>3</sub>	0.031	<0.020	<0.02	0.043
Total Kjeldahl Nitrogen (TKN)	0.178	0.196	<0.10	0.244
Total Nitrogen = NO <sub>2</sub> +NO <sub>3</sub> +TKN	0.209	0.196	<0.1	0.287

All of the shallow monitoring wells on the site were sampled on April 18, 2023. The laboratory results are provided as Attachment D.

Based on that testing, the background nitrate levels in all four test wells indicate minimal levels (>0.3 mg/L) of nitrates.

### 3.2.1 Shallow Groundwater Water Elevations

Groundwater monitoring wells were installed using hand augering equipment at four locations, identified as AH1, AH2, AH5 and AH10. The groundwater elevations were subsequently measured on April 18, 2023 and on February 20, 2024, as follows.

Monitoring Well	Ground Surface Elevations (masl)	Groundwater Elevations (masl)	
		Apr. 18, 2023	Feb. 20, 2024
AH1	95.33	95.19	94.75
AH2	95.89	95.82	95.22
AH5	91.41	90.77	89.71
AH10	98.08	97.23	96.06

The interpreted groundwater flow directions in the ground water aquifer (sewage effluent receiving aquifer) are expected to be to the northwest towards the Rideau River as shown on the attached Figure 6, Groundwater Flow (shallow).

Water levels in the water supply wells at the site were measured on June 16 or 17, 2022 as follows.



Test Well	Top of Casing Elevations (masl)	Ground Surface Elevations (masl)	Groundwater Elevations (masl)
			Jun 28, 2022
TW1	98.25	97.33	94.03
TW2	95.85	94.89	94.03
TW3	100.65	99.72	94.14
TW4	96.80	96.38	89.68

The deeper bedrock water aquifer groundwater flows are expected to be to the northwest, as shown on the attached Figure 7, Groundwater Flow (deep). It is noted that TW4 had groundwater elevations that were lower than the other wells, possibly due to it being in use since ~1990. This well is servicing the existing dwelling and is of older construction. The water elevation in that well was not used to estimate groundwater flow direction.

### 3.3 Land and Water Use Conflicts

There is not expected to be any soil or groundwater impact from the previous uses of the site. A Phase I Environmental Site Assessment report dated December 6, 2024 was carried out as part of the proposed development. No environmental issues were identified that could impact soil or groundwater on the subject property from current and historical site uses of the subject property and for the Phase I Study Area of 250 metres surrounding the site. The following summarizes former uses of the subject property and the current and historical uses of properties within the site vicinity which have been evaluated in terms of the potential for groundwater interference and/or soil and groundwater contamination on the subject property.

- The site is currently contains a single family dwelling. It is understood this is also the historical site usage.
- Review of air photographs indicates the site was used as pasture fields historically (pre 1946 to ~1996) and has been allowed to become overgrown since that time. In 2014, the site is mainly treed. No previous developed use was identified other than the current single family dwelling and garage.
- The site is mainly tree covered, within the vicinity of the site no potential for groundwater contamination exists.
- No evidence of nitrate impacts were identified in the receiving aquifer (section 3.1)



Based on the current development near the site, there are no concerns with respect to the quality of groundwater supply at the site from the onsite and offsite land uses and the historical use of the site and surrounding properties.

A review of Permit to Take Water Mapping around the site indicates that there are no major water taking activities within 1 kilometre of the area. The Rideau Correctional and Treatment Centre has two PTTW for groundwater listed some 1.29 and 1.3 km north. This listing is incorrect. The actual location is in Burritts Rapids and is greater than 2 km east. Also, the facility is permanently closed.

A review of the Pits and Quarries database indicates no pits or quarries within at least 1 kilometre of the site. The closest is a licensed quarry some 2 kilometres to the east. There is no corresponding PTTW for this property.

### **3.4 Class IV Septic Sewage Disposal Systems**

This section discusses the implications of the site-specific terrain conditions in terms of the feasibility of installing Class IV sewage disposal systems within the proposed subdivision.

#### **3.4.1 Septic System Envelopes**

The septic system envelope area (septic envelope) represents the area on a lot set aside for the construction of the leaching bed and is for the leaching bed only and does not include that area required for the septic tank or the isolation/separation distances required by the Ontario Building Code. The deposit or disposal of any materials or the placement of any structure or the operation of any equipment, other than material, structures or equipment required for the construction of the sewage system within or upon the septic envelope is prohibited.

The size of the septic envelopes are a function of the percolation time of the native soil in the vicinity of the septic envelope and/or the fill used for construction of a septic bed and the daily effluent loading to the septic bed. The native silty clay soil at the site is of low permeability, with an approximate percolation rate of  $\leq 50$  min/cm and a loading rate of  $Q/6$  (where  $Q$  is the daily sewage design flow using Table 8.7.4.1 A).



As a conservative approach to determining the expected largest septic system envelope required to service a single family dwelling at this site, a septic system envelope size was calculated assuming a fully raised bed using a percolation rate of 8 minutes per centimetre for the imported sand required and a daily sewage flow of 2,800 litres. A design flow of 2,800 litres per day is suitable for a four bedroom dwelling with 278 square metres of finished area and 23 fixture units. The following formulae were used to calculate the size of the septic envelope:

The larger of

$$A = \frac{Q}{6} \quad \text{OR} \quad A = \frac{1.6QT}{200} \text{ plus mantle (15 m x 15 m)}$$

4:1 Leaching Bed Side Slopes

Where  $Q$  = daily sewage flow for the proposed dwelling (i.e., 2,800 litres per day)

$T$  = percolation rate of imported fill material

The size of the septic envelopes, based on the conservative approach described above, is approximately 467 square metres. In view of the minimum proposed lot sizes of about 4,000 square metres, and average lot sizes of about 6,477 square metres, sufficient area exists at each of the proposed lots for the construction of a conventional septic system that meets the requirements of the Ontario Building Code. The Lot Development Plan shows the sewage systems with this approximate area and fit on lots using the above noted criteria for most lots (21 of the proposed 29 lots). On the remaining seven lots, there are waterfront setbacks (Lots 1 and 2) or limited frontage (Lots 10, 17, 18, 19 and 20). The sewage system areas are based on the use of Level IV treatment systems and area beds in order to fit the sewage systems in the front yards and be down gradient of wells on these lots. These exceptions will be indicated in the Conclusion and Recommendations Sections of the report.

Prior to establishing the actual septic envelope (leaching bed) location on any particular lot, several test holes should be excavated to determine the consistency/variability of the overburden in the vicinity of the proposed septic envelope and percolation rate tests should be carried out to determine the actual envelope area and whether imported mantles are required.

Other site-specific considerations with respect to the locations of the septic envelopes (leaching beds) on the proposed lots are as follows:



- assuming that shallow groundwater flow within the upper overburden is from topographically higher areas to topographically lower areas, the septic envelopes should be situated in the topographically lower areas with the wells on the topographically higher areas
- the separation distances between septic envelopes and properly constructed drilled and cased wells should be at least twice the grade raise plus 15 metres for partially to fully raised beds as required by the Ontario Building Code

### **3.4.2 Leaching Bed Design Considerations**

The design of leaching beds is a combination of a number of interrelated factors including effluent discharge volume, properties of the soil materials in the leaching bed, length of distribution lines and the subsurface conditions. The construction of individual septic disposal systems on the proposed lots should be carried out in accordance with the specifications set out in the Ontario Building Code.

The design must ensure that the bottom of the absorption trenches is at least 0.9 metres above bedrock or soils that are unsuitable for treatment of septic effluent (those with excessively low permeability), and at least 0.9 metres above the seasonally high groundwater table.

Based on the soil and groundwater conditions at the site, fully raised septic system leaching beds are likely to be used. The actual leaching bed type appropriate for each lot will depend on the individual lot specific soil and groundwater conditions.

Any fully raised leaching beds should be constructed of imported sand having a percolation time of between 6 and 8 minutes per centimetre with less than 5 percent passing the #200 (0.074 mm) sieve. It is recommended that gradation analyses be carried out on any potential sand fill prior to leaching bed construction in order to verify that the percolation time of the fill material is acceptable or that the material is certified as meeting this specification by a licensed pit.



### **3.5 Groundwater Impact Assessment**

#### **3.5.1 Criteria**

The Ministry of the Environment (MOE) Procedure D-5-4 provides guidelines for evaluating "the ability of the lands identified by and restricted to the development document, to treat sewage effluent to meet acceptable limits". The guideline requires that the representative background nitrate levels in the receiving groundwater be determined. Where background levels are greater than 10 milligrams per litre, the ministry indicates development of the site should not be supported unless it can be demonstrated that existing levels of nitrates are the results of historical agricultural practices on the site. In addition, the guideline requires demonstration that the site is not obviously hydrogeologically sensitive such as karstic areas, areas of fractured bedrock exposed at the surface, areas of thin soil cover or areas of highly permeable soils.

The guideline indicates that the assessment involves a three step process.

Step 1 regards lot size considerations. Where the lot size for each private residence within the development is an average of one hectare or larger and no lot is smaller than 0.8 hectares, and provided the site is not hydrogeologically sensitive, the risk that impact limits may be exceeded by individual systems is considered acceptable.

Step 2 is in regards to septic system isolation considerations. Developments are considered low risk when it can be demonstrated that sewage effluent is hydrogeologically isolated from existing or potential supply aquifers. For this case the most probable groundwater receiver for sewage is to be defined through information obtained through a test pit or test hole program, and the most probable lower hydraulic or physical boundary of the groundwater receiving sewage effluent is to be defined. The guideline indicates hydrogeology information concerning lands up to 500 metres beyond the actual development boundary may be required. When it can be demonstrated that the sewage will not enter supply aquifers the lot density of the proposed development is determined based on the space required to install a suitable septic system at each lot in accordance with the Ontario Building Code.

Step 3 is in regards to contaminant attenuation considerations. For this case, it is required to assess the risk that the on-site sewage systems within the proposed development will cause a



concentration of nitrate in groundwater above 10 milligrams per litre at the down gradient boundary of the site.

### **3.5.2 Site Conditions Evaluation**

In order to evaluate the background water quality conditions in the receiving aquifer, four shallow monitoring wells were installed at the site and tested for nitrogen species. The construction details are provided in Section 2.1, the Records of Boreholes are appended to the report and groundwater levels are reported in Section 3.2.1. Background nitrogen concentrations from the shallow groundwater receiving effluent were reported (Section 3.2) and the original laboratory testing results are provided as Attachment D. The Site Plan, Figure 2, shows the locations of the monitoring wells.

The results of nitrogen testing (see Table IX) indicate that four of the four monitoring well locations, nitrate levels were <0.1 mg/l at all four monitoring wells. The background nitrates at the site are considered to be acceptable for development purposes.

The site is not obviously hydrogeologically sensitive as no karstic areas, areas of fractured bedrock exposed at the surface or areas of highly permeable soils are indicated to be present at the site. Two soils samples were submitted for hydrometer analysis, one of the silty clay and one of the underlying glacial till at the site. The surficial soils consist mainly of silty clay with low sand content (1 to 3%) which has an average estimated hydraulic conductivity of between  $1.0 \times 10^{-6}$  and  $1.0 \times 10^{-7}$  cm/s (Section 3.1). The soil can be characterized as silty clay. This is considered to be a soil of very low permeability.

The surficial and underlying soils at the site are of low permeability and the overburden thickness at the site is at least 10 metres or more in thickness. Based on the soils information, the site is not considered to be hydrogeologically sensitive.

The minimum lot size proposed for the development is about 0.40 hectares. Accordingly, the above noted "Step 1" does not apply to this site. Hydrogeological isolation between the receiving and water supply aquifers was not evaluated for this site. Thus, "Step 3" was addressed for this site.



### 3.5.3 Step 3 Assessment

The most probable groundwater receiver for sewage effluent is the red to grey silty clay that was encountered below the topsoil at all of the test pit locations. To obtain a general indication as to the potential impact of septic effluent on the properties adjoining the proposed development, a nitrate dilution model was used. A daily effluent loading of 1,000 litres per day per septic system was assumed and the expected impact of septic systems at this site was determined by considering the attenuation of nitrate in the effluent from an assumed 40 milligrams per litre (mg/l) ( $\text{NO}_3$  as N) after the septic system treatment to the property boundary by dilution as a result of the infiltration of meteoric water only. The following provides the basis whereby the infiltration reduction factors for the site were chosen for the dilution calculations.

Topographic, soil and land cover infiltration factors were selected from *Table 2* of the MOE *Hydrological Technical Information Requirements for Land Development Applications*. The following is a discussion of each of the infiltration reduction factors chosen for the site.

A soil infiltration factor of 0.10 for tight and impervious clay is appropriate for the septic effluent dilution calculations, based on the permeability of the soils encountered across the site. Given the continuous nature of the silty clay overburden at the site, with between 90 and 95 % silt/clay content in the hydrometer analyses, and nine of the ten test pit logs describe the surficial soil as silty clay, a terrain map was not considered to be required to delineate the terrain distribution across the property.

The site is characterized by rolling terrain with highest elevations within the southeast portion of the site sloping to the northwest. The development areas of the site consist mostly of the rolling terrain and the steeper slopes are typically within development setbacks to the Rideau River. The site is considered to be rolling with a slope infiltration factor of 0.20. Using *Table 10* of the Thornthwaite and Mather *Instructions and Tables for Computing Potential Evapotranspiration and the Water Balance*, a soil water holding capacity of 100 millimetres was provided for the clay loam overburden at the site, which is considered to be representative of the top soil layer for the post development conditions.

The type of land cover observed at the site at the time of site visits and by use of satellite imagery consists mostly of treed forest, although there are several undeveloped grass fields across the site.

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It is expected that the post-development conditions at the site will consist mainly of grassy areas with a few trees and shrubs as well as untouched treed areas within development setbacks to the wetland waterfront areas, where existing trees and shrubs will be retained. The land cover infiltration factor of 0.12 was selected, which corresponds to cultivated lands. This is a conservative prediction as it does not account for the mature trees which may be retained or the landscape trees and shrubs that will likely be cultivated on properties post development.

In order to determine water surplus estimates for the site area, a water surplus model was obtained using a site specific water budget provided by Environment Canada (EC). The closest weather station with sufficient data was for Kemptville and some data gaps from other stations, including Drummond Centre and Appleton. The water balance model output the average yearly moisture surplus value, based on monthly moisture surplus averages for the period from 1998 to 2022. The expected moisture surplus or net potential infiltration for the site area was estimated at 377 millimetres. The water balance model output data, provided by EC, are included in Attachment E.

Hard Surfaced Areas post-development were calculated as follows. The total roadway area for the site is 10,820 square metres. The house/garage footprints are assigned to be 326 square metres and driveways of about 200 m<sup>2</sup>, per lot. Additional hard landscaping areas based on potential for pools, other hard landscaping, pathways, etc. were estimated to be up to an additional 174 square metres per lot. The total hard surfaced areas using these values for post-development conditions are 31,120 m<sup>2</sup>.

The results of the sewage dilution calculations indicate that the expected concentration of nitrate at the site boundary due to the proposed 29 sewage systems is about 9.7 milligrams per litre (Attachment E). This is within the Ministry of the Environment acceptable nitrate impact limit of 10 milligrams per litre.

Based on the impact assessment, the development of the site on private sewage disposal systems is not expected to have an adverse impact on groundwater resources in the site area.



## 4.0 GROUNDWATER SUPPLY INVESTIGATION

### 4.1 Supply Aquifer

As mentioned above, a bedrock geology map for the site area indicates that dolostone and sandstone of the March Formation of the Beekmantown Group underlie the site. The surrounding area of the site consists of the Oxford Formation of the Beekmantown Group. According to the Mississippi-Rideau Watershed Characterization Report, the March Formation consists of shallow marine carbonate and clastic sediments consisting of interbedded quartz sandstone and dolostone. The Oxford Formation consists of finer grained sediments, primarily of dolostone that is weathered and existing conformably above the last sandstone layer of the March Formation. The colour of the bedrock varies with rock type and level of weathering, where weathered rock in the March Formation appears as grey sandstone and sandy dolomite alternating with blue-grey dolomite and weathered bedrock of either type appears a dark reddish brown. The colour of unweathered bedrock of the Oxford Formation ranges from dark grey to brownish-grey and greenish-grey while weathered units test to appear lighter grey or reddish-brown. The regional fault mapping from that report indicates that a regional fault occurs some 300 metres north of the site. The MOE well records for the test wells indicate the bedrock encountered was limestone for all three test wells for which well records were available. A review of the MOE water well records for area wells, Attachment B, indicates the following information. Area wells are between 14 and 55 metres in depth, with average well depths of 28 metres, all encountering limestone or dolomite. Where reported, wells were tested at pumping rates of between 11 and 227 L/minute, with average rates of 64 L/minute. Soils overlying bedrock are described as clay or any combination of clay, stones, sand, loam with overburden thickness of between 0 and 14 metres, average of 5 metres of overburden in the area.

For the onsite wells, TW1 encountered water fractures during drilling in the bedrock at depths of some 30 to 32 metres. TW2 encountered a fracture at a depth of about 41 metres, TW3 encountered a fracture at a depth of about 23 metres, and TW4 encountered a fracture at a depth of about 27 metres. Geological cross-sections of the site were prepared using soils and bedrock information from the test well borehole logs and test pit logs. The geological cross sections are provided as Figures 4 and 5.



Based on the available information, the water supply aquifer is characterized as being confined, as the piezometric surface is situated above the top of the aquifer (bedrock elevation). The stratigraphy indicates confining soil conditions in the silty clay and glacial till overburden, which can be characterized as aquitard overlying the bedrock at the site.

## 4.2 Water Quality

The results of the chemical, physical and bacteriological analyses of water samples obtained from the test wells are provided as Attachment F and field water quality data is provided in Table I. A summary of the laboratory test results is attached as Tables II and III. The water quality as determined from the results of the analyses is favourable. The water meets all the Ontario Drinking Water Standards (ODWS) health and aesthetic parameters tested for at the test wells except for the following:

- Hardness at all of the wells
- Iron at TW2 and TW3
- Turbidity at TW2 and TW3 (lab based)
- Organic Nitrogen at TW1, TW2 and TW3
- Sodium above medical advisory limit at TW2 and TW4

### Hardness

The water samples from all of the test wells are considered hard by water treatment standards. Water with hardness above 80 to 100 milligrams per litre as  $\text{CaCO}_3$  is often softened for domestic use. The hardness at the test wells ranges from 265 to 400 milligrams per litre. Water softening by conventional sodium ion exchange will reduce hardness and scaling on fixtures. However, it may also introduce relatively high concentrations of sodium into the drinking water, which may contribute a significant percentage to the daily sodium intake for a consumer on a sodium restricted diet. Where ion exchange water softeners are used, a separate unsoftened water supply could be used for drinking and culinary purposes.

### Iron

Iron was measured at levels between 0.42 and 0.53 mg/l at TW2 and TW3, compared to the aesthetic objective of 0.3 mg/l. Excessive iron levels may cause brown or black discoloration of



laundry and fixtures, affect the taste and colour of water, and iron precipitation in pipes and hot water tank can also promote the growth of iron bacteria. Iron can be effectively removed using conventional ion exchange water softeners. However, depending on the form that iron is in (reduced or oxidized) as well as the concentration and other factors, iron filters may be more effective in removing iron from the water supply.

### Turbidity

Turbidity in drinking water obtained from a groundwater source is considered acceptable provided that it is less than the aesthetic objective of 5 NTU at the point of consumption. In this case, the field readings for turbidity indicate all four test wells measured field turbidity at less than 5 NTU and the elevated levels at the laboratory are due to sample handling and storage for wells where iron was present above its AO.

The lab based turbidity measurements for TW2 and TW3 were above 5 NTU. At TW2, the lab results were 5.8 and 11.4 NTU for the three and six hour samples, respectively, compared to field readings of 2.8 and 2.6 NTU, respectively. TW3 had lab based readings of 6.5 and 5.2 NTU, for the three and six hours, compared to field readings of 1.75 and 1.80 NTU, respectively. Both TW2 and TW3 also had the presence of iron above its AO, which is considered to have caused elevated turbidity in the lab result due to water storage and transportation. Iron and manganese are known to precipitate with exposure to air and temperature changes. TW1 and TW4 both had lab based turbidity below 5 NTU. TW1 and TW4 where iron was less than the AO, the field turbidity in those wells were less than 1 NTU by the end of the test. Field readings for turbidity at TW2 and TW3 indicated that the turbidity was generally declining during the test. The results are indicative of inorganic based turbidity, considering the presence of iron. It is considered that with treatment to reduce iron, the turbidity in the treated water will be less than 5 NTU.

### Organic Nitrogen

Organic nitrogen has an Operational Guideline (OG) of 0.15 mg/l for drinking water. Organic nitrogen levels at TW1 and TW3 ranged from 0.16 to 0.27 mg/l, compared to the operational guideline (OG) of 0.15 mg/l. Organic nitrogen levels at TW2 were 0.16 and 0.03 mg/l after three and six hours, respectively. The OG applies typically to drinking water that is treated using chlorination as organic matter can form trihalomethanes causing taste and odour problems in the treated water. OG typically apply to municipal water treatment where raw water is generally from surface water sources where organic content of the water is high. Organic nitrogen is naturally occurring in



groundwater sourced from sedimentary bedrock. As groundwater is typically not treated using chlorine, the potential presence of organic nitrogen in some wells does not require treatment.

### Sodium

The sodium levels at test wells TW2 and TW4 were between 20 and 34 mg/l, above the 20 milligrams per litre advisory level, whereby the local Medical Officer of Health should be notified when the sodium concentration exceeds 20 mg/L so that this information may be communicated to local physicians for their use with patients on sodium restricted diets. The sodium levels were well within the aesthetic objective of 200 mg/l. TW1 and TW3 had sodium levels less than 20 mg/L.

### **4.3 Water Quantity**

The drawdown and recovery data and plots for TW1, TW2, TW3, and TW4 are provided as Attachments H, I, J, and K respectively. The drawdown and recovery data provided were measured with reference to the top of the well casing at each test well location.

The pumping test data for the test wells were analyzed using the method of Cooper and Jacob (1946). Although the assumptions on which these equations are based are not strictly met, this method provides a reasonable estimate of the aquifer transmissivity.

Transmissivity was calculated using the following relationship:

$$T = \frac{2.3Q}{4\pi ds}$$

where Q is the pump rate, m<sup>3</sup>/day

ds is the change in drawdown over one time log cycle, m

T is the transmissivity, m<sup>2</sup>/day

The analysis of the data obtained from each test well during the pumping tests is summarized in the attached Table IV. The water levels in observation wells were monitored during the pumping tests at TW1, TW2, TW3, and TW4, and the data and charts are provided as Attachment G.



The following sections discuss the results of the analysis of the data obtained during the pumping tests with respect to test well yields.

#### **4.3.1 Test Well TW1**

The six hour duration pumping test was carried out at a discharge rate of 72 litres per minute. The static water level prior to testing was about 4.22 metres below the top of the well casing and the water level after six hours of pumping was about 4.57 metres below the top of the well casing for a total drawdown at the end of pumping of 0.35 metres. The available drawdown in the well is about 20 metres based on a recommended pump depth of 24.5 metres. The specific capacity of the well at this pumping rate is approximately 296 cubic metres per day per metre of drawdown.

Based on the pumping test drawdown data, the transmissivity of the aquifer is estimated to be 43 m<sup>2</sup>/day. Based on the recovery data the aquifer transmissivity is estimated to be 19 m<sup>2</sup>/day. The average transmissivity of the bedrock aquifer in the area of TW1 is estimated to be 31 m<sup>2</sup>/day. At the end of pumping, 95 percent recovery of the total drawdown in the static water level created during pumping occurred in about 18 hours.

Based on the data obtained during the pumping test, it can be concluded that the well is capable of sustaining a short term yield of at least 72 litres per minute and that during the course of the six hour pumping period less than 2 percent of the available drawdown in the test well was utilized.

#### **4.3.2 Test Well TW2**

The six hour duration pumping test was carried out at a discharge rate of 47 litres per minute. The static water level prior to testing was about 1.70 metres below the top of the well casing and the water level after six hours of pumping was about 8.66 metres below the top of the well casing for a total drawdown at the end of pumping of about 7.0 metres. The available drawdown in the well is about 28 metres based on a recommended pump depth of 30 metres. The specific capacity of the well at this pumping rate is approximately 9.7 cubic metres per day per metre of drawdown.

Based on the pumping test drawdown data, the transmissivity of the aquifer is estimated to be 41 m<sup>2</sup>/day. Based on the pumping test recovery data, the transmissivity of the aquifer is estimated to be 62 m<sup>2</sup>/day. The average transmissivity of the bedrock aquifer in the vicinity of TW2 is calculated

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to be about 52 m<sup>2</sup>/day. At the end of pumping, 95 percent recovery of the total drawdown in the static water level created during pumping occurred within about 14 minutes.

Based on the data obtained during the pumping test, it can be concluded that the well is capable of sustaining a short term yield of at least 47 litres per minute and that during the course of the six hour pumping period about 23 percent of the available drawdown in the test well was utilized.

#### **4.3.3 Test Well TW3**

Due to the poor recovery of TW3 during the first pumping test, a second pumping test was carried out December 2024 at a lower rate. The results of both pumping tests are included for discussion purposes. However, the well yield will be based on the second test that was done at a lower rate and was considered to have fully recovered at the lower rate.

##### TEST 1 – June 16, 2022

The six hour duration pumping test was carried out at a discharge rate of 67 litres per minute. The static water level prior to testing was about 6.51 metres below the top of the well casing and the water level after six hours of pumping was about 7.30 metres below the top of the well casing for a total drawdown at the end of pumping of 0.79 metres. At the end of pumping, 80 percent recovery of the total drawdown in the static water level created during pumping occurred after 63 hours.

##### TEST 2 – December 10, 2024

The six hour duration pumping test was carried out at a discharge rate of 33 litres per minute. The static water level prior to testing was about 7.50 metres below the top of the well casing and the water level after six hours of pumping was about 7.81 metres below the top of the well casing for a total drawdown at the end of pumping of about 0.31 metres. The available drawdown in the well is about 18 metres based on a recommended pump depth of 21 metres. The specific capacity of the well at this pumping rate is approximately 157 cubic metres per day per metre of drawdown.

Based on the pumping test drawdown data, the transmissivity of the aquifer is estimated to be 55 m<sup>2</sup>/day. Based on the pumping test recovery data, the transmissivity of the aquifer is estimated to be 16 m<sup>2</sup>/day. The average transmissivity of the bedrock aquifer in the vicinity of TW3 is calculated to be about 36 m<sup>2</sup>/day. At the end of pumping, 90 percent recovery of the total drawdown in the



static water level created during pumping occurred within about 14 hours. The static water level recovered some 100% of the drawdown after 31 hours.

Based on the data obtained during the pumping test, it can be concluded that the well is capable of sustaining a short term yield of at least 33 litres per minute and that during the course of the six hour pumping period less than 2 percent of the available drawdown in the test well was utilized.

#### **4.3.4 Test Well TW4**

The six hour duration pumping test was carried out at a discharge rate of 32 litres per minute. The static water level prior to testing was about 6.36 metres below the top of the well casing and the water level after six hours of pumping was about 15.58 metres below the top of the well casing for a total drawdown at the end of pumping of about 9.2 metres. The available drawdown in the well is about 18.8 metres based on a recommended pump depth of 24.4 metres. The specific capacity of the well at this pumping rate is approximately 5.0 cubic metres per day per metre of drawdown.

Based on the pumping test drawdown data, the transmissivity of the aquifer is estimated to be 8.0 m<sup>2</sup>/day. Based on the pumping test recovery data, the transmissivity of the aquifer is estimated to be 3.0 m<sup>2</sup>/day. The average transmissivity of the bedrock aquifer in the vicinity of TW4 is calculated to be about 5 m<sup>2</sup>/day. At the end of pumping, 95 percent recovery of the total drawdown in the static water level created during pumping occurred within about 2 hours and 20 minutes.

Based on the data obtained during the pumping test, it can be concluded that the well is capable of sustaining a short term yield of at least 32 litres per minute and that during the course of the six hour pumping period about 49 percent of the available drawdown in the test well was utilized.

#### **4.3.5 Interference Effects**

During pumping of the test wells, observation well drawdown was observed in each of the other test wells to measure interference effects. The data was obtained from pressure transducer loggers and confirmed with manual measurements. The observation well drawdown charts and data are provided as Attachment G. Using the observation well data, storativity was estimated using a Cooper-Jacob formula (Table VII). The mutual interference effects were calculated for a centrally



located well (Lot 10) and the well interference at the property boundary was calculated for the down gradient property line along the north side of the site.

In order to estimate the maximum interference between future wells at the site, calculations were carried out to predict the cumulative thirty year drawdown due to the proposed 29 domestic wells at a central well in the proposed subdivision. The cumulative drawdown at the test wells was calculated for a thirty year pumping rate of 1100 litres per day which allows for four persons per household. The following formula was used for the calculation:

$$s = \frac{2.3Q}{4\pi T} \log\left(\frac{2.25Tt}{r^2 S}\right)$$

where Q = 30 year pumping rate, 1,100 L/day

T = average transmissivity, 33 m<sup>2</sup>/day

t = duration, 30 years

S = storativity, 7 x 10<sup>-5</sup>

s = expected drawdown due to each of the other 28 wells

r = distance between the observation well and the pumped well, m

The results of the calculations indicate the thirty year drawdown at a centrally located well due to the interference from the other 28 wells in the subdivision is about 0.94 metres. The cumulative well interference at the property boundary due to the 29 proposed wells was also estimated to determine the impact of the proposed development on water supply outside of the site. The expected thirty year drawdown at the site boundary was found to be about 0.96 metres.

Attachment B contains MOE Well Records of surrounding existing wells that were available for review. The indicated depths of the existing wells range from about 18.3 to 49.4 metres and accordingly are within an aquifer similar to the test wells which range in depth from about 25 to 43 metres. Based on the estimated thirty year drawdown noted above, the expected drawdown is minimal. All wells for which the MOE Well Records were obtained have sufficient available drawdown such that the slight drop in water level that may occur should have no significant impact on water supply at or adjacent to the proposed subdivision. This provides reasonable assurance of adequate water supply in the proposed subdivision as well as at the existing wells.



#### TW1 observation well interference

During the pumping of TW1, pressure transducer logging was carried out at TW2 and TW4, located about 250 and 500 metres, respectively, from TW1. Total drawdown observed at TW2 and TW4 was about 0.25 metres and 0.50 metres, respectively. As with the monitoring during the test on TW3, TW4 was in use by the existing residence during the test and therefore experienced significant interference.

#### TW2 observation well interference

During the pumping of TW2, pressure transducer logging was carried out at TW1 and the Observation Well (OW1) at the existing residence, located about 250 and 650 metres, respectively, from TW2. Total drawdown observed at TW1 and the Observation Well was about 0.20 metres and 0.33 metres, respectively.

#### TW3 observation well interference

During the pumping of TW3, pressure transducer logging was carried out at TW4, located about 375 metres from TW3. Total drawdown observed at TW4 was about 1.45 metres, however, this well was also in use by the residence on the property and therefore was experiencing significant interference throughout the test.

#### TW4 observation well interference

During the pumping of TW4, pressure transducer logging was carried out at TW1, TW2, TW3, and the Observation Well (OW1) at the existing residence, located about 500, 660, 375, and 18 metres, respectively, from TW4. Total drawdown observed was 0.00 metres at TW1, 0.00 metres at TW2, 0.00 at TW3, and 4.74 metres at the Observation Well at the existing residence.

Based on the interference observations, it is considered that the level of interference is acceptable and will not cause unacceptable drawdown in adjacent future wells at the property. It is noted that TW4 is the well that provides water for domestic uses in the single family dwelling onsite. OW1 and TW4 are existing onsite wells that service the single family dwelling at the site. No well records could be located for these wells. TW4 is of similar depth as the other test wells and is considered to be in the same aquifer. Drawdown in OW1 indicates that it is also within the same aquifer. However, as both of these wells were in use throughout the pumping tests, neither was used to determine storativity due to interference effects during pumping.



#### 4.4 Groundwater Flow Directions

The groundwater flow directions in the receiving and water supply aquifers were determined based on the results of a topographic survey of the site and using the static water levels measured at the standpipes (overburden receiving aquifer) and test wells (bedrock water supply aquifer).

##### 4.4.1 Receiving Aquifer

The static water level elevations at AH1, AH2, AH5 and AH10 were 95.26, 95.82, 90.77, and 94.23 metres geodetic, respectively, measured on Apr 18, 2023 and Feb. 20, 2024. Based on that data, the receiving aquifer flow direction is indicated to be west across the site towards the Rideau River (see Figure 6), generally following the topographic slope at the site.

Monitoring Well	Ground Surface Elevations (masl)	Groundwater Elevations (masl)	
		Apr. 18, 2023	Feb. 20, 2024
AH1	95.33	95.19	94.75
AH2	95.89	95.82	95.22
AH5	91.41	90.77	89.71
AH10	98.08	97.23	96.06

Lateral gradient between AH2 and AH5

$$i = dh / L = (95.82 - 90.77) / 371.3 = 0.0136$$

$$V = K * i$$

$$V = (1.0 \times 10^{-6} \text{ cm/s}) * 0.0136$$

$$V = 1 \times 10^{-8} \text{ cm/s}$$

$$V = 1 \times 10^{-10} \text{ m/day (using data from Apr 18, 2023)}$$

Lateral gradient between AH1 and AH10

$$i = dh / L = (97.23 - 95.19) / 406.3 = 0.005$$

$$V = K * i$$

$$V = (1.0 \times 10^{-6} \text{ cm/s}) * 0.005$$

$$V = 5 \times 10^{-9} \text{ cm/s}$$

$$V = 5 \times 10^{-11} \text{ m/day (using data from Apr 18, 2023)}$$



Based on the gradients and the permeability of the silty clay (calculated in Section 3.1), lateral gradients at the site are very slow. Conceptually the site is located in a discharge area and the shallow groundwater likely discharges to the Rideau River. The groundwater flow direction is expected to be to northwest across the site towards the Rideau River.

### Vertical Gradients

Within the receiving aquifer (in this case, arguably aquitard) the vertical gradients could not be established between the standpipes in the surficial clay groundwater and the deeper overburden soils at the site. That would require deeper standpipes to compare elevations. As such vertical gradients have not been established.

#### **4.4.2 Water Supply Aquifer**

Test Well	Top of Casing Elevations (masl)	Ground Surface Elevations (masl)	Groundwater Elevations (masl)
			June 17/18, 2022
TW1	98.25	97.33	94.03
TW2	95.85	94.89	94.03
TW3	100.65	99.72	94.14
TW4	96.80	96.38	89.68

Lateral gradients were not established in the bedrock wells. Based on the data from the above noted Table, the supply aquifer groundwater flow direction is indicated to be northwest across the site (see Figure 7).

#### **4.5 Development Impacts and Neighbouring Land Uses**

Description of existing and historical land uses at and near the site was previously noted (Section 3.3 Land and Water Use Conflicts). There are no concerns with regards to existing and historical area uses to impact the site.



The proposed lot sizes of the development and the nitrate attenuation calculations indicate that the development will not impair adjacent groundwater users. The groundwater flow directions and gradients in the receiving groundwater (aquitard) indicate that groundwater flow is very slow due to the low permeability of the receiving soils at the site. There are no down gradient water users between the proposed development and the adjacent Rideau River. Based on the findings of this study, there is very low risk of impact to other groundwater users from the migrating sewage effluent at the site.

The potential for well interference and long term sustainability of the water supply at the site indicates that long term drawdown due to the proposed developed properties is less than 1.0 metre. The predicted drawdown is not significant with regards to the available drawdown in area wells, such that the proposed development is not expected to cause any significant reduction in groundwater available to the existing well users in the area.

Based on the above noted information, the development impacts, including availability of groundwater resources for drinking water, groundwater impact from proposed sewage systems, are within acceptable limits.

#### **4.5.1 Impact to Surface Water Quality and Quantity**

There are two surface water bodies at the site. The Rideau River exists on the west and north property boundaries and an unevaluated wetland located within the southwest portion of the property. The receiving aquifer at the site flows to the northwest. There are development setbacks and minimum frontage requirements along the Rideau River to ensure that impacts to surface water are within acceptable limits. Similarly, the lots that are adjacent to the unevaluated wetland area also have development setbacks. In both cases, the development setbacks provide the following:

- A vegetated buffer exists between the lots and the adjacent water of a 30 metre no touch boundary. Waterfront lots on the Rideau River, all have an additional 10 metre buffer (total 40 metres) to maintain the landscape character and ecological functions along the shoreline. The vegetative buffer preserves existing trees and shrubs and acts as a nutrient and sediment trap to ensure that the surface water is protected from runoff, erosion, sediments;
- The soils at the site are of low to medium permeability consisting of clay overlying glacial till and are relatively thick (greater than 9 metres), indicating that sewage carrying elevated nutrients including nitrates and phosphorus will migrate very slowly. The slow migration



provides increased chances for other processes to occur to retard the migration of these nutrients towards the adjacent surface water. The vegetated buffer provides aeration through the roots in the unsaturated soil zone, which increases microbial activity and encourages nitrogen fixation, nutrient uptake which reduces nutrient load to the groundwater. The silt and clay content within the clay soils and the underlying till often have higher organic content than sandy soils, which also promotes phosphorus retention and sorption. Depending on calcareous content of till and clay soils, mineral deposition also reduces phosphorus migration;

- The majority of the sewage systems are proposed in the front yards with the mantles oriented to outlet towards the roadside ditches rather than towards the surface water;
- Stormwater Management consists of enhanced treatment including roadside ditches to reduce suspended solids, stormwater storage and sand filters to reduce suspended matter, encourage infiltration and retention of sediments;
- Best Management Practices recommended in the Stormwater Management Report include the following:
  - Construction works are to be timed in order to reduce the length of time between the beginning of construction and the establishment of vegetative cover.
  - Keep sediment and erosion control measures in place and maintained during and following construction until vegetation is established.
  - Do not disturb vegetated areas outside of the development foot print.
  - Use appropriate equipment for the development to reduce the duration of the development.
  - Work should be timed to avoid the wet seasons of the year.
  - Roof runoff should be discharged onto the ground surface and directed to the grass surfaced swales.
  - Winter snow removal, together with salting and sanding should be completed in accordance with an established plan and best management practices to reduce the amount of sand and salt required.

Based on the above noted site conditions, mitigative measures and BMPs including Stormwater Management, it is considered that the potential for surface water impacts is limited.

In terms of affecting water quantity of the surface water, the soils at the site are of relatively low permeability such that the slight change in recharge due to the increased hard surface areas at the site are not anticipated to significantly affect water levels in the adjacent wetland. Stormwater management ensures that post development runoff rates do not predevelopment runoff rates. Stormwater discharge is detained and released at a controlled rate by outlet structures at two stormwater facilities, which discharge to the wetland and to the Rideau River, respectively.



#### **4.6 Well Construction Methodology**

Future wells drilled on the site should be constructed with a minimum length of casing to ensure that casing is set at least 1.8 metres (6 feet) into the sound bedrock. The typical casing length based on the test wells at the site are 12.2 metres to 13.1 metres in length measured from the ground surface. The steel casing placed in the auger holes should be pressure grouted or displacement grouted into place. The material used to seal the annular space could consist of a combination of a cement grout and a commercially available bentonite grout product or a either of these products. Cement grout mixtures should be allowed to set for a minimum two day period for normal cement or twelve hours for a high early strength cement prior to advancing the well further into bedrock. If a bentonite grout product is used, drilling need only be suspended for a few hours depending on the product used. Bentonite grout has the additional advantage of remaining flexible when set and therefore will not crack or shrink thereby ensuring as well as possible that surface water or shallow groundwater will not migrate along the annular space and into the well bore.

Once the casing has been sealed, the well should be advanced uncased in the bedrock until a water supply of sufficient quantity and quality is encountered.

The completed well should then be developed to maximize the yield. The well casings should be completed at least 40 centimetres above the highest point on the finished ground surface within three metres radially from the well after surface drainage is directed away from the well. The casing should be fitted with a pitless adapter to facilitate below ground plumbing and electrical connections. Surface grading should be completed to direct surface water away from the well in order to ensure that water will not collect or pond in the vicinity of the well.

#### **4.7 Post Development Monitoring Program**

The results of this investigation indicate acceptable existing and expected impact on the groundwater quality at this site due to existing neighbouring land uses and the proposed development. The existing nitrate impacts on the receiving aquifer at the site are acceptable for development. The local hydrogeological conditions and existing water quantity and quality all indicate that the impact of the proposed development will not significantly impact the overall groundwater quality and quantity at the site. Septic effluent dilution calculations, which by

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experience are known to be a conservative estimate of actual impact, indicate predicted sewage system impacts at the site are within MECP requirements. There are twenty-nine residential lots proposed for development. Accordingly, a groundwater monitoring program is not considered necessary for this site.

## **5.0 CONCLUSIONS**

This terrain evaluation and groundwater supply investigation was conducted to assess the suitability of a site located at 819 Burritts Rapids Road within the village of Merrickville-Wolford, Ontario, for a proposed residential subdivision development using private wells and individual onsite sewage disposal. The site, consisting of an area of approximately 22.7 hectares (56 acres), is to be subdivided into some 29, average 0.65 hectare (minimum 0.4 hectare) lots for single family dwelling construction purposes.

Based on the results of this investigation, the following summary and conclusions are provided:

- 1) Water well records for wells drilled within 1 kilometre of the site indicate that well yields are in excess of the required minimum well yield of 18.75 L/min for a four bedroom house. Existing wells are drilled wells between 14 and 55 metres in depth, with average well depths of 28 metres, all encountering limestone or dolomite. Where reported, wells were tested at pumping rates of between 11 and 227 L/minute, with average rates of 64 L/minute. The majority of area wells encountered greater than 2 metres of overburden, generally consisting of clay or till (described as clay/sand/stones/loam).
  
- 2) Four water supply wells (including one existing well) were completed in the limestone bedrock formation consisting of Oxford Formation). Six hour constant rate pumping tests were carried out at rates of between 32 and 72 L/min for the four test wells causing drawdown of between 0.30 and 9.2 metres. All of the wells had sufficient recovery at the tested rates. Based on the pumping tests, well yields at the site are variable, ranging from 32 to 72 L/minute. The well yields at all four test wells were adequate to supply the minimum well yields of 18.75 L/minute flow rate for 4 bedroom dwellings. It is considered that future wells constructed to similar depths will provide sufficient well yields for single family dwellings.



- 3) The pumping tests identified acceptable well interference between the respective test wells on the subject site. Test wells were mostly fairly distant from each other. For wells that are some 200 to 300 metres apart, maximum drawdown of 0.2 metres was measured in an observation well. A maximum drawdown of 4.7 metres occurred between the two existing wells at the existing single family dwelling which are 36 metres apart. The observation well was in use during the test, so the results indicate two operating wells at close spacing. Both wells are of older construction. Most future wells will be spaced at least 50 metres apart, based on the lot sizes and layouts. A cumulative drawdown prediction for all operating wells indicates that mutual well interference of 0.94 metres can be expected over the long term (30 years) for wells using water at predicted rates of 1,100 L/day. This drawdown is not considered a concern, as it represents about 5% of the available drawdown in the test wells.
- 4) There is a sufficient groundwater of acceptable drinking water quality in the bedrock aquifer system at this site as it meets all the ODWS concentrations for all health related chemical, physical and bacteriological parameters tested except for the following:
- a. Hardness levels of between about 268 to 400 milligrams per litre are anticipated. The recommended water treatment consists of ion exchange water softeners and maintaining a separate unsoftened water supply for drinking and culinary purposes.
  - b. Iron was measured above the aesthetic objective at TW2 and TW3. Iron can be effectively removed using conventional ion exchange water softeners. However, depending on the form that iron is in (reduced or oxidized) as well as the concentration and other factors, iron filters may be more effective in removing iron from the water supply. Some wells may not encounter iron above the aesthetic objective and may not require treatment.
  - c. Turbidity was less than 5 NTU, the aesthetic objective at all test wells after the pumping tests were completed, indicating that turbidity is within the aesthetic limit for all wells at the point of consumption. Laboratory results indicate that turbidity developed in the water samples and exceeded the aesthetic objective for water



samples from TW2 and TW3 which is caused by elevated iron levels. Treatment to reduce iron will be effective in reducing turbidity for water that is stored.

- d. Organic nitrogen levels may be slightly elevated at 0.17 to 0.20 mg/l, in some future wells, compared to the operational guideline (OG) of 0.15 mg/l. Organic nitrogen has an OG due to its interaction with chlorine used in water treatment causing taste problems in the treated water. As groundwater is typically not treated using chlorine, the potential presence of organic nitrogen in some wells does not require treatment.
  - e. Sodium levels at TW2 and TW3 were between 20 and 33 mg/l, above the 20 milligrams per litre advisory level, whereby the local Medical Officer of Health should be notified when the sodium concentration exceeds 20 mg/L so that this information may be communicated to local physicians for their use with patients on sodium restricted diets. The sodium levels were well within the aesthetic objective of 200 mg/l. It is recommended that if water softeners are used to treat hardness and TDS levels, that an untreated drinking water tap is installed in the kitchen to ensure that excessive sodium levels in treated water are not consumed.
- 5) TW4 is of older well construction and services an existing dwelling on the subject lands. There were no surficial indicators or differing water quality in this well that would indicate any impacts due to it being on the developed part of the property with a sewage system. The levels of DOC, chlorides, sodium, turbidity, nitrates, and bacteria all indicate no changing water quality in this older well over time. Nitrate levels were 0.2 mg/L which is well within a background water quality and of no concern.
- 6) The test pits and augerholes put down on the subject property indicate that soils consist of a silty clay deposit that is at least 3 metres in thickness over the majority of the site, with one test pit encountering till below the clay at about 0.8 metres. The drilled wells indicate that the overburden consists of silt and till of at least 9 metres in thickness. The permeability of the two soil types indicate soils that are of medium to low permeability. The groundwater table was relatively high when measured during Spring conditions and lower in late Winter. This is consistent with poorly drained soils. The site is not considered to be hydrogeologically



sensitive. Class IV sewage disposal systems with fully raised leaching beds will likely be used at this site as well as Level IV treatment systems with area beds depending on lot specific soil and groundwater conditions and building and lot configurations.

- 7) The proposed lot sizes vary between 0.40 and 1.8 hectares, with average lot sizes of 0.65 hectare lots. The theoretical nitrate calculations indicate that the down gradient nitrate concentrations for the development could be 9.7 mg/L as nitrogen. The background conditions at the site indicate that the overburden has very little nitrate and is currently not impacted. Due to development setbacks and in order for sewage systems to be down gradient of wells, some 7 of the 29 lots are proposed to be equipped with Level IV treatment units and reduced sewage leaching beds in order to adequately service the lots. The sewage impact calculations were predicted assuming nitrate loadings of 40 mg/L for all lots.

## **6.0 RECOMMENDATIONS**

The following are recommendations are made based on the study findings.

- 1) Future wells drilled on this property shall be constructed in accordance with Ontario Well Regulation 903 with casing of sufficient length to extend at least 1.8 metres (6 feet) into the sound bedrock. Based on the onsite wells, casing lengths of 12.2 to 13.1 metres measured from ground surface will be typical. The entire annulus of the casing shall be grouted using a pressure injection method. Well casings should be extended a minimum of 0.4 metres above the final finished ground surface at the well with grading completed around the well to ensure surface water drainage is directly away from the wellhead for at least 3 metres radially. Wells should be located at least 15 metres plus twice the grade raise from fully raised septic fields, and up gradient of sewage systems.
- 2) Water wells should be drilled to depths of about 30 metres and up to 45 metres, based on the test wells that were recently drilled onsite. Water fractures are expected to occur at depths of some 30 to 41 metres, with corresponding well yields of at least 32 and up to or possibly exceeding 72 L/minute in some wells. Some wells may require surging at the time of construction to increase yield or develop water fractures that are encountered during construction. The well yields are considered sufficient to provide for single family dwelling domestic uses. If additional water demand is needed for landscaping uses or for secondary dwelling units, future property owners may need to do further well assessment at the time of



or prior to well construction to ensure an adequate water supply is present. Additional recommendations for secondary dwelling units are provided herein (See Bullet 10)

- 3) The recently constructed Test Wells TW1, TW2 and TW3 are located on Lots 11, 4 and 24, respectively. All of the wells could be used to service future dwellings on these lots and do not need to be decommissioned. These wells can be used as future water supply wells provided that the following is verified and/or carried out:
- wells shall be minimally three metres from the property lines in order to ensure that positive drainage is occurring away from the well head;
  - wells shall meet or exceed the minimum separation distances to sewage systems and sewage tanks indicated by the Ontario Building Code;
  - wells should be protected from damage during construction;
  - If required, wells may need additional casing height to ensure that top of casings are at least 0.4 metres above finished ground surface.
  - TW3 shall be located at least 3 metres from the proposed swale on that property.

If any of the existing wells are to be abandoned for any reason, they should be appropriately decommissioned by a licensed well contractor and the Certificate of Abandonment should be provided to the Merrickville-Wolford Building Department prior to any building permit being issued for these Lots.

- 4) The water quality from the test wells has the following aesthetic exceedances and corresponding optional treatment recommendations:
- Elevated hardness levels are anticipated in all future wells. If desired, the appropriate recommended water treatment consists of ion exchange water softeners. Where water softeners are used, sodium levels in treated water may be elevated, and a separate untreated tap can be maintained for drinking and culinary purposes as well as untreated taps for outdoor taps.
  - The sodium levels in some future wells may be present above the medical advisory limit of 20 mg/L, whereby the local Medical Officer of Health should be notified so that this information may be communicated to local physicians for their use with patients on sodium restricted diets.
  - Iron may be present at some future wells, which can typically be reduced through water softeners that are properly dosed for iron reduction. Alternatively, iron filters may be used to reduce iron in treated water.



- 5) Future well owners are advised to obtain water samples and have the bacteriological water analyses performed through the Leeds, Grenville and Lanark Health Unit prior to first use of the water and twice annually thereafter (typically in the Spring and Fall). Homeowners should be aware of their obligations with regards to well maintenance and water and energy conservation. Homeowners are referred to the MECP publication *Water Supply Wells – Requirements and Best Management Practices* manual, April 2015. Additional information and links on water conservation measures are offered at the *wellaware.ca* website link <<https://greencommunitiescanada.org/programs/well-aware/>>
  
- 6) Future sewage systems on the property are expected to be fully raised conventional Class IV sewage systems with minimum 15 metre mantles. All sewage systems should be constructed of imported sand having a percolation time of between 6 and 8 minutes per centimetre with less than 5 percent passing the #200 (0.074 mm) sieve. It is recommended that gradation analyses be carried out on any potential sand fill prior to leaching bed construction in order to verify that the percolation time of the fill material is acceptable.
  
- 7) The lots are sufficiently sized to accommodate a sewage design flow of 2,800 litres per day which is suitable for a four bedroom dwelling with 278 square metres of finished area and 23 fixture units. In order to maintain proper drainage and to maximize separation distances between proposed sewage systems and wells and development setbacks to the adjacent surface water (wetland and river), the general layout shown on the Lot Development Plan shall be maintained on each proposed lot. This means that sewage systems are generally to be kept in the front yards on most lots with mantles having an outlet to front yard roadside ditches. The two exceptions (Lots 2 and 22) have back yard sewage systems. Any proposed alteration to the lot layout (i.e. changing location to back yard) would require a qualified hydrogeologist to review and approve the proposed lot grading and drainage plan. At a minimum, the hydrogeologist must ensure that any such change does not encroach upon any other lots (whether already developed or undeveloped) from following the approved Lot Development Plan or affect the development setbacks that are protective of the natural habitat and surface water. The subsequent report and Lot Grading and Drainage Plan should be approved by the Merrickville-Wolford Township building department prior to building permit issuance.
  
- 8) The Lot Development Plan shows fully raised conventional Class IV sewage systems with mantles based on sewage design flows of 2,800 L/day on most lots (21 of the proposed 29



lots). There are seven lots consisting of Lots 1, 2, 10, 17, 18, 19 and 20 that either have development constraints due to limited frontage on corner lots or to maximize separation distance from adjacent wetland or slopes. These exceptions are to ensure that each property can be serviced adequately with sewage systems in the front yards, down gradient of wells on these lots and maximize separation distances between sewage systems and adjacent surface water. These lots are proposed to have current OBC Level IV treatment units and alternative leaching beds (filter beds, Type A Beds, Type B Beds or shallow buried trenches) or other BMEC/BNQ approved sewage systems. Future owners of these lots should be made aware of this by appropriate means, such as being included in the title or deed that these additional constraints apply to the development of these parcels. A useful reference for home owners is the following resource entitled *Septic Smart! Understanding your Septic System* which is available at the Rideau Valley Conservation Authority website or using the following link <<https://www.ontario.ca/files/2022-10/omafra-septic-smart-understanding-home-wastewater-system-en-2022-10-14.pdf>>

- 9) Existing on-site monitoring wells, including the boreholes AH1, AH2, AH5, and AH10 should be properly abandoned in accordance with Ontario Well Regulation 903. A record of well abandonment should be produced for each well, prior to development approval.
  
- 10) It is understood that Merrickville-Wolford Township allows secondary dwelling units on all residential properties. Based on this assessment, the well yields at the site are variable, ranging from ~32 to 72 L/min. These yields are sufficient to provide for larger single family dwellings and possibly secondary dwellings. If additional water demand is needed for secondary dwelling units, future property owners are responsible to do further well assessment at the time of well construction to ensure an adequate water supply is present. Any well that extends beyond the assessed depths of about 45 metres could encounter differing water quality and quantity and should have additional water quantity and quality testing as per MECP D-5-5 and ensure that interference with adjacent well owners is minimized. A Hydrogeologist with related experience (such as a P. Geo or P. Eng.) should be engaged for this purpose. Likewise, this assessment was carried out assuming sewage design flows based on a total sewage demand of 2,800 L/day for a standard four bedroom dwelling. If the combined sewage demand of the primary and secondary dwelling units exceeds this, the following should be considered:
  - For lots that are not on waterfront properties, the sewage servicing for the additional dwelling unit or for the entire sewage system if a secondary dwelling unit is part of the original building plans should consist of an advanced nitrogen removal system

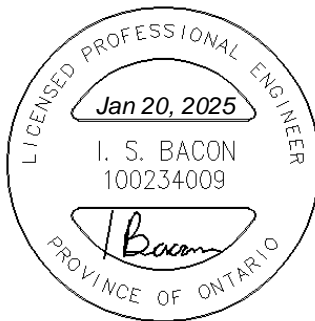


capable of reducing nitrogen to levels of at least 50% reduction. These systems must be approved through the OBC and CAN/BNQ third party testing verified; OR

- A lot specific terrain study to determine whether the additional water demand and sewage load can be supported with consideration of MECP D-5-4 and the potential for down gradient well users.

Regards,

Kollaard Associates Inc.



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