Wellhead Protection Area (WHPA) and Tier 2 Water Budget Study Lansdowne, Ontario

Revision: 3 (Final)

Prepared for:

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EXECUTIVE SUMMARY

Geofirma Engineering Ltd. (Geofirma, formerly Intera) was retained by the Cataraqui Region Conservation Authority (CRCA) to complete a Wellhead Protection Area (WHPA) study for the Village of Lansdowne municipal groundwater supply and a Tier 2 Water Budget / Stress Assessment (WB) study for the subwatershed that contains the Village of Lansdowne. This work was completed in accordance with the most recent guidance from MOE, "*Technical Rules: Assessment Report, Clean Water Act, 2006, dated November 16, 2009*". This report builds on three previous studies including preliminary WHPA study completed by Malroz Engineering Ltd. (Malroz, 2004 & 2009), the Conceptual Water Budget (CWB) study completed by CRCA (CRCA, 2009b) and the Tier 1 Water Budget and Stress Assessment (Tier 1 WB) study completed by XCG Consultants (XCG, 2009).

Lansdowne is a small village located approximately 50 km east of Kingston and situated on the Frontenac Arch of the Canadian Shield. Lansdowne has a population of approximately 600 people and is serviced by two municipal groundwater wells and a sewage treatment pond. During the course of this study, treatment was installed on the municipal water supply to address recent bacteriological contamination of one of the municipal wells. The land use around Lansdowne is mostly agriculture and woodlands.

Additional data was collected as part of field work conducted during September 2009. Field work activities included: [1] groundwater levels (snapshot and continuous), [2] groundwater and surface water sample and chemical analysis, [3] 6-8 hour pumping tests on two bedrock monitoring wells, and [4] straddle packer testing (14 tests) in one monitoring well.

A conceptual model was developed and a numerical model was developed to simulate the three-dimensional distribution of hydraulic head in the study area, using MODFLOW. WHPA Zones A, B, C, and D were delineated using reverse particle tracking and the MODPATH routine in accordance with the Technical Rules. The Lansdowne WHPA is centred on the municipal wells, encompasses the majority of the Village and extends to the sewage ponds. For the purpose of this study, the municipal wells were considered “potentially GUDI”, and WHPA Zones E and F were delineated in accordance with the Technical Rules.

Groundwater aquifer vulnerability was determined within the delineated WHPA using the Intrinsic Susceptibility Index (ISI) method. Vulnerability scoring was completed in accordance with the Technical Rules and the entire Lansdowne WHPA was classified with high aquifer vulnerability to surface contamination.

A long-term average monthly Tier 2 water budget was developed for the Lansdowne subwatershed, and a groundwater stress assessment was completed in accordance with the Technical Rules. Following the MOE Technical Rules, the Lansdowne municipal water supply system is categorized as having a low level of groundwater stress and therefore a Tier 3 Water Budget study is not required.

Conclusions and recommendations were made addressing the apparent vulnerability of the water supply to surface contamination and uncertainty in long-term trends in groundwater level within the aquifer.

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1 INTRODUCTION

Geofirma Engineering Ltd. (Geofirma, formerly Intera) was retained by the Cataraqui Region Conservation Authority (CRCA) to complete a Wellhead Protection Area (WHPA) study for the Village of Lansdowne municipal groundwater supply and a Tier 2 Water Budget / Stress Assessment (WB) study for the subwatershed that contains the Village of Lansdowne. This study was completed in accordance with our proposal dated May 14, 2009 and the Terms of Reference for the study (CRCA, 2009a).

This study has received funding from the Government of Ontario. Such support does not indicate endorsement by the Government of the contents of this material. The methodology used in this study followed that provided in the most recent guidance from MOE, including: [1] "*Technical Rules: Assessment Report, Clean Water Act, 2006, dated November 16, 2009*" (MOE, 2009), herein referred to as the *Technical Rules*, [2] "*The WB Guidance: Water Budget and Water Quantity Risk Assessment, dated March 2007*" (MOE, 2007), herein referred to as the *Water Budget Guidance*, and [3] "*Assessment Report: Draft Guidance Module 3, Groundwater Vulnerability Analysis, dated October 2006*" (MOE, 2006), herein referred to as the *Vulnerability Guidance*.

This report builds on three previous studies including a preliminary WHPA study completed by Malroz Engineering Ltd. (Malroz, 2004 & 2009), the Conceptual Water Budget (CWB) study completed by CRCA (CRCA, 2009b) and the Tier 1 Water Budget and Stress Assessment (Tier 1 WB) study completed by XCG Consultants (XCG, 2009).

1.1 Purpose

Through the Ontario *Clean Water Act*, the Ontario Ministry of Environment (MOE) is requiring that the CRCA provide source water protection for all municipal water supplies within its watershed. The purpose of this study is to fulfill requirements of the CRCA, under the Ontario *Clean Water Act*, to better understand the surface area that contributes water to the municipal drinking water system for the Village of Lansdowne, Ontario. The results of this study will be incorporated into the CRCA Assessment Report, pertaining to the entire Cataraqui Source Protection Area (CSPA), which will ultimately provide the basis to develop the Source Protection Plan (SPP) for the entire CSPA.

1.2 Scope of Work

The scope of work for this study, as outlined in Section 4.1 of the Terms of Reference (CRCA, 2009a), included:

- A limited GUDI assessment based on historical water quality results, and other readily available information;
- Delineation of a WHPA around the municipal wells. This WHPA is defined in the *Technical Rules* (MOE, 2009) as WHPA Zones A, B, C, D, E and F;
- Assessment of groundwater vulnerability (i.e. vulnerability scoring) in each of the WHPA zones in accordance with the Vulnerability Guidance (MOE, 2006);
- Completion of a Tier 2 WB for the subwatershed in which the Lansdowne municipal wells are located ("the Lansdowne subwatershed"), in accordance with the Technical Rules (MOE, 2009) and following methodology from the Water Budget Guidance (MOE, 2007); and

- Evaluation of groundwater quantity stress for the Lansdowne subwatershed, in accordance with the Technical Rules (MOE, 2009) and following methodology from the Water Budget Guidance (MOE, 2007).

1.3 Report Organization

This report is organized into ten sections and eight appendices:

- Section 1 outlines the purpose and scope of work for the study;
- Section 2 provides background information and a summary of previous studies;
- Section 3 describes the regional geology setting and hydrogeologic conceptual model;
- Section 4 summarizes the field work completed as part of this study, and describes the method and results of the limited GUDI assessment;
- Section 5 provides the methodology and results for the WHPA delineation and groundwater vulnerability assessment;
- Section 6 summarizes the Tier 2 Water Budget results;
- Section 7 provides the methodology and results for the groundwater quantity stress assessment;
- Section 8 summarizes conclusions and recommendations;
- Section 9 discussed limitations with respect to the use of this report;
- Section 10 contains a list of references used in this report;

Appendix A contains a list of acronyms and abbreviations used in this report and Appendix B contains all of the figures. Appendix C provides the details of the groundwater chemistry analysis and Appendix D contains a copy of all laboratory results and laboratory QA/QC report. Appendix E contains copies of well logs for the previously installed monitoring wells, Appendix F summarizes the pumping test, Appendix G includes the report prepared by the Department of Civil Engineering at Queen's University summarizing some straddle packer testing at MW5, and Appendix H includes the Peer Review Record.

2 BACKGROUND

2.1 Study Area

The Village of Lansdowne is located within the Township of Leeds and the Thousand Islands in Eastern Ontario, approximately 50 km east of Kingston and 3 km north of King's Highway 401 (Figure 2-1). Lansdowne has a population of approximately 600 residents, mainly serviced by municipal water and sewer. Lansdowne is situated on a local topographic high along the divide between two watersheds (Gananoque River subwatershed above Marble Rock and the St. Lawrence River subwatershed), as shown in Figure 2-1.

The study area for the numerical groundwater flow modelling (used for both WHPA delineation and for Tier 2 WB/Stress assessment) encompassed a portion of both of the above-mentioned subwatersheds in order to provide appropriate model boundary conditions and to encompass the entire area where groundwater contributes to the municipal wells. As shown in Figure 2-2, the WHPA study area is approximately 60.7 km^2 and spans from Highway 401 in the south to Dulcemaine in the north and from Ebenezer in the west to Waterton in the east.

The study area for the Tier 2 WB and Water Quantity Stress Assessment is entirely encompassed within the WHPA model domain and is delineated by a subset of the La Rue Creek subwatershed. The Tier 2 WB study area is approximately 14.6 km^2 and is approximately 2 to 3 km wide and 6 to 7 km long spanning from Lansdowne in the west to Waterton in the east. This is the same subwatershed boundary used in the Tier 1 WB study and is shown in Figure 2-1 and Figure 2-2.

2.2 Lansdowne Municipal Water Supply and Sewage Treatment

The municipal water supply for the Village of Lansdowne comprises two groundwater wells (Well 1 and Well 2), drilled and equipped with submersible pumps in 1975, that draw water from the local bedrock aquifer. The wells are usually operated in alternating sequences, however due to recent bacteriological detections in Well 2 (MOE, 2008) only Well 1 is currently being operated. The municipal drinking water is treated with sodium hypochloride prior to being pumped into an elevated storage tank (~500 m^3 operating capacity) located approximately 150 m south of Well 1.

The Lansdowne municipal wells (Wells 1 and 2) are approximately 50 m deep and draw water from both the granitic gneiss bedrock aquifer (Precambrian basement) and the overlying sandstone bedrock aquifer (Nepean Formation). Both aquifers are fracture dominated and are commonly used for domestic water supply in the surrounding area. The pump intakes are situated approximately 30 m below ground surface (mBGS) (Malroz, 2004b) which is approximately 15 to 18 m below the static water level in the well, and approximately 20 m above the bottom of the borehole.

Each well is housed in a brick building on the northern limit of the Village, separated by approximately 186 m. Well #1 is located at UTM coordinates of 4917618 m Northing and 418675 m Easting and Well #2 is located at 4917845 m Northing and 418707 m Easting relative to NAD 83 datum. The water well records were not available for review, however well completion details were reported by Ian D. Wilson Associates Ltd. (1974). Both wells were drilled in 1974 by RH Cassleman under the supervision of Wilson Associates, who report the wells to be constructed with a 20 cm diameter steel surface casing set approximately 3.6 m deep (1.5 m into bedrock).

During a site inspection on 3 September, 2009, Geofirma staff noted cascading water in both municipal wells. This observation is consistent with a borehole video inspection log at Well # 2, completed in 2002 by International Water Supply (IWS, 2002 and Malroz, 2003), which indicated cascading water at approximately 6 mBGS. Although Malroz (2004) concluded that the well is not under the direct influence of surface water (GUDI), the shallow casing depth, the cascading, and the recent bacteriological contamination (MOE, 2008) may suggest otherwise. At a meeting of the Technical Advisory Group for this study on 22 December, 2010 it was confirmed that treatment equivalent to chemically assisted filtration (suitable for surface water) has been installed on the municipal supply (see also OCWA comments in Appendix H).

A sewage collection and treatment system was installed concurrently with the municipal water supply. The two sewage treatment lagoons (Figure 2-3) incorporate an impermeable clay liner and recent groundwater sampling from nearby monitoring wells has not shown any evidence of bacteriological contamination in the Precambrian bedrock (Malroz, 2008). The possibility of the lagoons acting as a zone of enhanced groundwater recharge is considered in delineation of WHPAs in this study. Both the water supply and sewage systems are owned by the Township of Leeds and the Thousand Islands and operated by the Ontario Clean Water Agency's (OCWA) Chesterville, Ontario office.

2.3 Previous Studies

2.3.1 Preliminary Lansdowne WHPA Studies

Malroz initially completed a preliminary delineation of WHPA zones for the Village of Lansdowne using available published data (Malroz, 2004). During this study a numerical groundwater flow model was developed using the computer program MODFLOW (McDonald and Harbaugh, 1988) based on the Malroz preliminary conceptual model and assumptions. Horizontal time-of-travel (TOT) capture zones were created (50-day, 2-year, 5-year, 10-year, and 25-year) for each of the municipal wells using reverse particle tracking (MODPATH).

This WHPA study was updated after a series of five monitoring wells (MW1/2, MW3/4, and MW5) were installed by Malroz (2008) to depths of 24.4, 10.1, 34.0, 2.6, and 38.5 mBGS, respectively (see Figure 2-3). Monitoring wells MW1, MW3, and MW5 were completed in the combined sandstone and Precambrian bedrock aquifers used by the municipal wells, while MW2 and MW4 were completed in the shallow overburden; all but MW5 were completed adjacent to the sewage lagoons. Each of the newly installed monitoring wells was sampled and analysed for bacterial contamination. None of the bedrock monitoring intervals showed detection of bacteria while the two overburden monitoring intervals each had minor detections (2 counts) of fecal coliforms that were attributed to adjacent farming practices. Based on the additional data the WHPAs (2-yr, 5-yr and 25-yr TOT) were updated.

Important assumptions made during the Preliminary WHPA studies included:

- Groundwater recharge, $R(_{\text{Precambrian}})$ = 50 mm/yr; $R(_{\text{Sandstone}})$ = 250 mm/yr;
- Hydraulic conductivity is isotropic, $K(_{\text{Lower Precambrian}}) = 1\text{x}10^{-5}$ m/s; $K(_{\text{Upper Precambrian}}) = K\ (_{\text{Sandstone}}) = 1\text{x}10^{-4}$ m/s
- Porosity = 5% uniform for all bedrock units
- Minimum model grid spacing = 20 m near municipal wells
- Only 1 model layer, total model thickness = 20 m

2.3.2 Conceptual Water Budget

A Conceptual Water Budget (WB) was prepared by CRCA staff for the entire Cataraqui SPA using available data with respect to average annual conditions. The Conceptual WB study (CRCA, 2009b) provided the background knowledge, data gathering and data gap identification necessary for future assessment (Tier 1 and Tier 2 WB) of water quantity stress levels within the individual subwatersheds. Datasets compiled and reviewed during this work included climate station data, streamflow records, summary of water takings, and a general understanding of how water (groundwater and surface water) moves through the watershed. Although the Conceptual WB was prepared specifically for the CSPA, the process was completed in conjunction with neighbouring Source Protection Regions (SPR) in Eastern Ontario (Mississippi-Rideau SPR and Quinte SPR) to ensure consistency of data use and interpretation. An important component of the Conceptual WB process involved Peer Review by a group of climatology, hydrology and hydrogeology experts.

2.3.3 Tier 1 Water Budget and Stress Assessment (Tier 1 WB)

All SPAs are required to complete a Tier 1 WB study on smaller subwatersheds within the SPA, delineated by the Source Protection Team. The Tier 1 WB process built on the knowledge gained and largely uses available data collected and analysed as part of the Conceptual WB study. The Tier 1 WB and Stress Assessment study is intended to provide a simple estimation of hydrologic stress within each subwatershed in order to screen out "unstressed" areas from requirements of the more detailed Tier 2 and 3 WB processes, thereby reserving efforts and resources for the areas that either are stressed or have a reasonable chance of experiencing water stress.

The Tier 1 WB study for CSPA is summarized by XCG (2009). XCG completed water budget calculations and assessed the level of stress associated with 21 subwatersheds within the CSPA. They concluded that the Lansdowne subwatershed had a stress level of approximately 14% on the average annual basis under future pumping conditions, which categorized it as "moderate" stress. This was one of 16 subwatersheds (10 surface water and 5 groundwater) that were categorized during this study as having a "moderate" or "significant" level of stress associated with water quantity. Only subwatersheds categorized during the Tier 1 WB as having a "moderate" or "significant" level of stress associated with water quantity AND contain a municipal water supply are advanced for further study as part of the Tier 2 or 3 Water Budget process (MOE, 2009). As such, the Lansdowne subwatershed was one of 2 subwatersheds that were advanced to a Tier 2 water budget study (Lansdowne – groundwater, and Sydenham - surface water).

Important assumptions made during the Tier 1 WB study for the Lansdowne subwatershed included:

- Groundwater recharge = 50 mm/yr, evenly divided between all 12 months
- Net lateral groundwater flow into subwatershed is negligible (i.e. $G_{NET} = 0$)
- Groundwater storage is not considered (i.e. $\Delta S = 0$)
- Municipal Well consumptive factor = 1.0 (i.e. 100% of water that is pumped is not returned to the groundwater system within the subwatershed)

Similar to the Conceptual WB study, peer review by the same group of experts and neighbouring SPAs was part of the Tier 1 study.

3 REGIONAL HYDROGEOLOGIC SETTING AND CONCEPTUAL MODEL

The hydrogeologic setting near Lansdowne has been described in earlier reports (Malroz, 2004; Malroz, 2008) and therefore only a brief overview is presented in this report. The Watershed Characterization report (CRCA, 2008) and earlier WB reports (CRCA, 2009b; XCG, 2009) provide a more detailed discussion of regional hydrogeologic setting for the entire CSPA.

3.1 Physiography and Land Use

Chapman and Putnam (1984) have categorized the physiography of Southern Ontario into 55 separate and distinct regions. The Lansdowne study area is situated in the Leeds Knobs and Flats physiographic region (part of the Frontenac Arch), which is characterized by a rocky landscape with relatively thin overburden where Precambrian bedrock knobs are surrounded by clay flats. The nearby “Thousand Islands” are formed from the same Precambrian bedrock knobs.

The current land use around Lansdowne is primarily agriculture (mostly where sufficient overburden exists) and woodlands (more prominent where bedrock knobs are exposed). Agriculture land uses comprise both cropland and pasture. Land uses immediately adjacent to the municipal wells include residential and commercial to the south, agricultural land to the north and west, and a municipal fair ground to the north and east. In addition, County Rd. 3 is approximately 200 m east of the municipal wells and a horse barn is located approximately 200 m north of Well # 2.

3.2 Topography

The Village of Lansdowne is situated on a local topographic high, with an elevation of approximately 110 to 115 mASL. Topographic relief within the study area is relatively high with a total range of elevation from approximately 190 mASL on an isolated bedrock knob in the north, down to approximately 75 mASL near the south (Figure 3-1).

3.3 Geology

Overburden in the Lansdowne study is shown in Figure 3-2. Inspection of Figure 3-2 indicates that the surficial geology types making up at least 98 percent of the model domain (are in order of prevalence): fine-textured glaciolacustrine deposits (i.e., clay, unit 8), bedrock or bedrock drift deposits (units 1 to 4), till/diamicton (unit 5), and organic deposits (unit 20). The mapping and the water well records in the MOE Water Well Information System (WWIS) indicate that the overburden is relatively thin to non-existent (i.e. less than 3 m). The only significant thickness of overburden in the study area exists within the Village, south of Eden Grove Road, where the water wells indicate that up to 9 m of fine grained to coarse-grained (possibly till) overburden exists.

Figure 3-3 shows the generalized distribution of bedrock stratigraphy throughout the Lansdowne study area. Generally, the bedrock geology comprises Precambrian-aged igneous and metamorphic rocks of the Canadian Shield overlain by an isolated pocket of erosion-resistant Palaeozoic-aged sedimentary rocks in and to the north of the Village of Lansdowne. All shallow bedrock appears to be significantly weathered as seen by video logs and hydraulic testing. A single fault is mapped south of Lansdowne in the Precambrian bedrock. Little is known about its influence on local groundwater flow and therefore it was not considered within the context of the conceptual model for this study.

Lansdowne is situated on the Frontenac Arch, a prominent ridge of Precambrian bedrock that remained after the last glaciers retreated and historically separated the sea water in the Ottawa Embayment (Mississippi-Rideau and Raisin-SN SPR) from that in the Lower St. Lawrence Lowlands (CSPA). The Precambrian Shield exists throughout the entire CSPA and it outcrops over the majority of the Lansdowne study area. The geology of Precambrian bedrock within the Lansdowne area is a mixture of rock types including felsic plutonic rocks and clastic metasedimentary rocks (Wilson, 1946).

The isolated sedimentary bedrock primarily consists of Cambrian-aged sandstones (Nepean Formation) with minor dolostones and shales belonging to the Potsdam Group (Cambrian period of the Palaeozoic Era). Malroz (2004) describes this Palaeozoic outlier as an erosional escarpment feature due to the observation of a weathered shaley layer below the more competent sandstone cap. This escarpment feature is reported in MOE well records to be approximately 13 to 15 m thick at the Lansdowne municipal wells and is non-existent approximately 1 km east of the Village near the municipal sewage lagoons.

3.4 Groundwater / Surface Water Interaction

Groundwater typically discharges in low lying areas, as evidenced by wetlands or surface water bodies. Regional (deeper) groundwater flow direction is interpreted from historical water levels to be south (Dillon Consulting, 2001; Malroz Engineering, 2008), eventually discharging into the St. Lawrence River and Lake Ontario. Local (shallow) groundwater is interpreted to flow radially away from the Lansdowne escarpment and discharging into nearby streams towards the west (Gananoque River subwatershed) and the south and east (St. Lawrence River subwatershed). For the modelling conducted for WHPA delineation in this study, only the local, shallow groundwater flow is considered.

No natural surface water features were noted within 600 m of the municipal wells, however information provided by local residents indicated that several residents to the south east of the Village have experienced artesian groundwater conditions in the past. The location of these conditions are shown in Figure 2-3 and are consistent with groundwater flowing away from the high elevation escarpment towards the lower lying areas in the south-east.

Groundwater recharge for use in regional-scale numerical groundwater flow models is difficult to estimate without stream flow records, and/or instrumentation of basins. Lacking such information in this study, a uniform, relatively low recharge rate is assumed for the entire study area. This is considered a conservative assumption. In one calculation case a variable recharge rate is assumed, with the recharge distribution inferred from the method of MOEE (1995) for estimating recharge distribution from topography, soil type and land cover (as described in CRCA (2009b) Section 2.1.4).

3.5 Hydrostratigraphic Units

The geology and hydrogeologic conditions support the conceptualization of 4 hydrostratigraphic units:

- clay and till *Overburden Aquitard*;
- weathered *Palaeozoic Sandstone Aquifer*;
- weathered *Precambrian Aquifer*; and,
- unweathered *Precambrian Aquitard*.

Based on the mapping (predominantly clay, bedrock, or bedrock drift complex, see Section 3.3) and on the water well records, a nominal 2 metre thickness of overburden is assumed to exist outside the vicinity of the Village. Close to the Village, the overburden thickness is assumed to increase to up to 9 m, and is assumed to be accurately represented via interpolation of overburden thicknesses from the water well logs. The overburden unit is assumed to have a porosity of 30% and a hydraulic conductivity of $1x10^{-7}$ m/s, which is consistent with the conceptualization of this unit as an aquitard (i.e., a unit within which groundwater flow is vertical) or as non-existent. (Having a relatively low hydraulic conductivity in a thin uppermost hydrostratigraphic unit could potentially overestimate the backwards vertical travel time at the water table. However, vertical travel time at the water table has zero influence on WHPA delineation, which is based on projection of horizontal particle traces,) It is further noted that the course-grained material south of Eden Grove Road was not included in the model because inspection of the water well records indicated it to be present in small disconnected pockets which would not be significant conduits for horizontal groundwater flow, and which would have little influence on WHPA delineation.

The weathered Palaeozoic sandstone aquifer is conceptualized as an isolated, fracture-dominated semi-confined aquifer with anisotropic hydraulic conductivity. The hydraulic conductivity of this rock is conceptualized as having moderate horizontal hydraulic conductivity, due to the presence of fractures, and lower vertical hydraulic conductivity, which is best estimated through model calibration.

The upper 30 m of Precambrian bedrock is also conceptualized to be a highly weathered and fracture dominated aquifer. The hydraulic conductivity is considered to be moderate, isotropic, and sufficient to support domestic supply in the area. Deeper Precambrian bedrock, below a thickness of 30 m, is conceptualized as an aquitard due to the assumption of fracture spacing increasing (i.e. fracture frequency decreasing).

In the context of WHPA delineation using a calibrated flow model and backwards particle tracking, effective porosity is the ratio of simulated groundwater flux to plume front velocity. Effective porosities for bedrock for use in delineation of WHPAs are difficult to estimate, and there have been a wide range used in studies across the province, which are typically between one and ten percent. The approach adopted for this study is to use a low, conservative value, leading to larger WHPAs. To that end, the effective porosity of all bedrock units was assumed to be 1%.

4 FIELD WORK METHODOLOGY AND DATA ANALYSIS

Although the primary focus of this work was to complete a WHPA study and a Tier 2 WB study for the Lansdowne municipal supply, a significant effort was placed on obtaining additional field data to better understand the groundwater flow system near the Lansdowne municipal wells. Field work activities were completed in September 2009 and included the following:

- Domestic Well Survey - survey of nearby wells to improve accuracy of location;
- Water Level Survey - collecting static water levels in private and municipal wells;
  - snapshot measurements using electronic water level tape;
  - continuous measurements using data loggers;
- Groundwater and Surface Water Sampling and Chemical Analysis – sampling select wells and one surface water locations in an effort to characterize the source of municipal groundwater;
- Hydraulic Testing of Municipal Water Supply Aquifer
  - Pumping tests at MW-3 and MW-5 with a duration of 6-8 hours; and,
  - Straddle packer testing on one of the monitoring wells to differentiate differences in hydraulic properties between the Precambrian bedrock and the overlying sandstone.
- Wellhead Inspection and Limited GUDI Assessment

Each task is described in greater detail below.

4.1 Domestic Well Survey

Geofirma completed a field survey of nearby domestic and municipal wells to identify wells that are no longer being used (i.e. requiring proper abandonment) and to collect accurate UTM coordinates to improve their location accuracy compared to the MOE wells database information. All ground surface and UTM coordinate measurements were collected using a hand-held GPS unit (Garmin Etrex Vistas). Malroz (2004a) previously completed a validation of 289 domestic well coordinates (northing and easting) and concluded that the locations of 220 wells were properly reported and 69 required updating. As such, the majority of domestic wells within Lansdowne and the study area were already identified, therefore Geofirma was only able to identify and confirm UTM coordinates for 10 additional locations as summarized in Table 4-1. The locations of these measurements are shown in Figure 2-3.

Of the 10 locations, four were monitoring wells (MW1, MW2, MW3 and MW5) installed by Malroz (2008), six were domestic wells where an MOE well ID number was identified and one was a domestic well with an unknown MOE well ID.

4.2 Water Level Survey

Geofirma manually collected water level data at 8 locations using an electronic water level tape during September 2009 and mapped the location of artesian conditions at one residence, as reported by the owner. Water levels were measured relative to the top of PVC riser using an electronic water level tape that was cleaned and decontaminated between each location. These water elevation measurements provided high quality data to assist with the numerical modeling calibration process, and are listed in Table 4-1, expressed as both m below top of casing (mBTOC) and m above sea level (mASL).

Table 4.1 Summary of Measured Well Coordinates and Groundwater Elevations

Well ID	Easting (UTM)	Northing (UTM)	Water Level (mBTOC)	Groundwater Elevation (mASL)
MW1	419286.0	4917737.0	1.8	100.07
MW2	419287.0	4917737.0	1.85	99.95
MW3	419328.0	4918047.0	5.59	100.33
MW5	419003.0	4917895.0	7.74	105.30
Unknown	418074.2	4919739.8	0.5	90.63
3609783	416049.3	4918589.0	0.1	94.34
3607584	416063.7	4918306.0	0.33	95.41
3616773	417287.0	4917336.2	0.07	95.39
3605104	419443.6	4917489.1	artesian	artesian
3604860	418818.0	4917703.0	no access	no access

In addition, water level dataloggers (SolinstTM Levelogger Gold) were installed in five wells (Lansdowne municipal Well 1, MW1, MW2, MW3, and MW5) during September 2009 to continuously monitor groundwater elevations over the period of 7 to 10 days and observe daily fluctuations due to municipal well pumping or other influences. All loggers were set to record a pressure measurement every 5 minutes. Figure 4-1 shows the water level fluctuations for each of the wells with Leveloggers and Table 4-2 summarizes the well locations, total water level change and average daily water level change for each location.

As shown in Table 4-2, not every monitoring well has the same change in water level over time. Municipal Well 1 and MW3 appear to decrease in water level over the entire period while MW1, MW2 and MW5 appear to increase. The degree of water level change is influenced by many factors such as the open interval of a borehole, the hydraulic properties of the formations being monitored, pumping from nearby wells, and local recharge.

Table 4.2 Summary of Continuous Head Measurements in Select Wells

Well ID	Formation Monitored	Total WL change (m)	Total # days	Average Daily change
Well 1	SS + PC	-6 cm	10.3	1.35 m each pump cycle -0.6 cm per day
MW1	PC	20 cm	10.3	1.8 cm
MW2	OB	2 cm	10.3	0.2 cm
MW3	PC	-9 cm	7.5	-1.2 cm
MW5	SS + PC	26 cm	9	2.3 cm

**Note: PC = Precambrian, SS = sandstone, OB = overburden.*

4.3 Groundwater and Surface Water Sampling and Chemical Analyses

A total of seven water samples were collected within the Lansdowne study area to characterize groundwater chemistry in the sandstone and Precambrian aquifers. The locations of each sample are shown in Figure 2-3, and are summarized in Table 4-3. Water sampling was conducted between September 16 and October 9, 2010 as outlined below:

September 16, 2009

- two municipal wells (MUN1 and MUN2) – samples collected after minimal purging since the pumps are continuously cycling These samples were collected from the same sampling port used by OCWA during routine sampling activities;
- two monitoring wells (MW3 and MW5) – samples collected after pumping test using a submersible pump (Grundfos Rediflow2);
- residential well (MOE ID#3604741) – sample collected after running tap outside of house for approximately 15 minutes;

September 30, 2009

- One surface water sample (SS1) – sample collected as a grab sample from a ditch to the south of the treatment ponds;

October 9, 2009

- Sandstone interval in monitoring well (MW5) – sample collected during straddle packer testing of sandstone interval using submersible pump.

Table 4.3 Summary of Water Sampling Locations

Sample Location	Bedrock Interval	Notes
Municipal Well 1 (MUN1)	Sandstone and Precambrian	OCWA sample location
Municipal Well 2 (MUN2)	Sandstone and Precambrian	OCWA sample location
Monitoring Well (MW3)	Precambrian	Grundfos pump
Monitoring Well (MW5)	Sandstone and Precambrian	Grundfos pump
Residential Well (MOE ID#3604741)	Sandstone	Tap outside house
Surface Water (SS1)	Creek / wetland	south of treatment ponds
Straddle Packer Testing (sandstone)	Sandstone interval	During straddle packer testing at MW5

Samples were collected directly into laboratory-supplied sampling containers. All samples were stored and shipped in coolers with ice packs. Samples were submitted to three different laboratories under standard chain of custody procedures and in accordance with Geofirma QA/QC procedures, including:

- Paracel Laboratories Ltd. (a CALA-certified analytical laboratory, based in Ottawa, Ontario) - analyzed for metals, microbiological and general chemistry parameters;

- University of Ottawa (UofO) - analyzed groundwater samples for oxygen-18 (^{18}O) and deuterium (^{2}H) isotope ratios as well as for dissolved inorganic carbon and dissolved organic carbon concentrations;
- University of Waterloo (UofW) - analyzed groundwater for tritium (^{3}H) concentrations.

A summary of analytical results are given in Appendix C. Copies of the Paracel laboratory reports are presented in Appendix D.

Figure C-1 shows that groundwater from the municipal wells has a mixed chemical composition that is intermediate between groundwater in the sandstone aquifer and groundwater in the Precambrian aquifer. This is consistent with the conceptual model that the municipal wells obtain water from both aquifers.

The residential well sample was collected from an outside tap which was identified to be located upstream of any treatment equipment, however the chemistry of groundwater from the domestic well was most likely affected by a water softener (based on low Ca and Mg concentrations and high Na concentrations) and therefore cannot be treated as a representative source of groundwater in the Lansdowne watershed.

Figure C-2 shows that all isotopic water samples fall along the meteoric water line. The surface water sample (SS1), collected from a shallow ditch/creek south of the treatment ponds, has the most depleted composition. This brings the validity of this location to represent surface water in question, as surface water is typically enriched in heavy isotopes relative to groundwater due to evaporation (Clark and Fritz, 1997). The municipal well sample (MUN1) has a composition intermediate between the surface water sample and Precambrian groundwater (Well 3). MUN2 and MW5 are isotopically enriched compared with other samples. The results for the sandstone sample collected during straddle packer testing were not usable for stable isotopes due to technical problems with the equipment at the laboratory.

The interpretations that can be drawn from the water chemistry are limited due to the small sample size and the lack of stable isotope data for groundwater from the sandstone aquifer. The municipal groundwater supply appears to have contributions from both sandstone and Precambrian aquifers, and could potentially be influenced by surface water sources. Additional data may provide further understanding of the Lansdowne subwatershed groundwater chemistry and the source of bacteriological contamination in Lansdowne Well #2.

4.4 Hydraulic Testing of Municipal Water Supply Aquifer

4.4.1 6-8 hour Pumping Tests at MW3 and MW5

Single well pumping tests were completed at each of MW3 and MW5 between September 15-17, 2009 using a submersible Grundfos™ Rediflo 2 submersible pump that pumped approximately 30 L/min. MW3 is completed with an open bedrock interval between 6.9 and 34 mBGS in Precambrian bedrock. MW5 is completed with an open bedrock interval between 6.6 and 38.5 mBGS intersecting both the sedimentary sandstone bedrock (Nepean Formation) and the Precambrian bedrock. Well logs for MW3 and MW5 are presented in Appendix E.

A Solinst Levelogger was placed above the pump to automatically record water level measurements throughout the test and manual water level measurements were also collected using an electronic water level tape to ensure Levelogger accuracy. The atmospheric pressure fluctuations were recorded with a Solinst Barologger Gold pressure transducer (Model 3001) throughout the duration of the pumping test. These fluctuations were used to compensate all water level measurements for barometric effects. Leveloggers were set to collect water level data during the pumping test at 1 second intervals (MW3) and 5 second intervals (MW5).

Drawdown data was analysed using the Theis equation, assuming steady-state, radial flow. Transmissivity and hydraulic conductivity values were estimated for each well: T = 69 m^2/day and K = 2.6 × 10^{-5} m/s (MW-5) and T = 12 m^2/day and K = 4.5 × 10^{-6} m/s (MW-3). Appendix F presents the analysed field data from these pumping tests, as well as the matched Theis curve.

4.4.2 Straddle Packer Testing at MW5

Straddle packer testing was completed at MW5 on October 8 and 9, 2010 to provide a better understanding of the hydraulic properties for the sandstone and Precambrian aquifers. MW5 was selected for straddle packer testing because it was the only monitoring well identified that was completed similar to the Lansdowne municipal wells; specifically, it is an open borehole that intersects both the Palaeozoic sandstone layer and the underlying Precambrian bedrock. The Department of Civil Engineering at Queen’s University, under contract with and under supervision of Geofirma, completed a total of 21 slug tests at MW5 using straddle packer testing The testing interval for straddle packers used two different packer spacings (1.44 and 1.51 m) and slug volumes ranged between 5 and 10 L. Hydraulic head was monitored and recorded using a pressure transducer (100 psi range transducer manufactured by DRUK) and a datalogger. A copy of the field report for straddle packer testing, which presents a detailed summary of this work, intervals tested and hydraulic properties analysis, is included in Appendix G.

Prior to straddle packer testing and during an inspection of MW5 on September 15 it was noted that there was a 4-inch steel protective casing, presumably completed only 1-2 m into ground. Inside the 4-inch protective casing there was a 2-inch PVC casing (open at bottom and assumed to be installed to a depth of 6.6 mBGS) that was cemented inside of a 3-inch steel casing (assumed to be installed to a depth of ~ 6 mBGS). This completion did not allow the straddle packer testing equipment to be lowered into the borehole. Upon contacting the consultant who supervised construction of the monitoring well (Malroz Engineering Ltd) and the drilling contractor who installed the monitoring well (GET Drilling) and looking at the MOE well record for this monitoring well (MOE Well #A037933) there was no indication (or reasoning) for the installation of a 2-inch PVC casing. Therefore, Geofirma contracted with Canadian Environmental Drilling Ltd. to use a 2 7/8-inch rotary bit to drill out the 2-inch PVC casing and leave the 3-inch casing “clean”. This work was successfully completed on October 6, 2009.

After this borehole cleaning work and prior to lowering the straddle packer tools down the borehole, a submersible borehole camera was inserted into the borehole on October 8, 2009. The purpose of this work was to verify that the reaming of the borehole had adequately removed the PVC pipe and grout down to the inner surface of the 3-inch diameter casing and that the borehole was of adequate quality for hydraulic testing. Although some remnants of concrete were noted on the walls of the casing, the borehole walls were in fair condition. The borehole camera observations are also summarised in table format in Appendix G.

Slug tests were analysed using the Hvorslev method (see Fetter, 2001). The time-hydraulic head curves were corrected for the case when the head had not quite stabilized prior to the injection of the slug using an extrapolation of the shut in data as shown in Appendix G. This straddle packer testing provided estimates of transmissivity and hydraulic conductivity values for the Precambrian (T = 0.06 m^2/day and K = 2.4 × 10^{-8} m/s) and Palaeozoic sandstone (T = 0.46 m^2/day and K = 1.7 × 10^{-6} m/s) bedrock units separately to assist with WHPA model calibration.

During straddle packer testing a static head measurement was obtained from each packer interval prior to injecting the slug of water. It is noteworthy that none of the packer-isolated intervals during straddle packer testing showed a hydraulic head value that was high enough to maintain the open borehole hydraulic head of approximately 7 mBGS. This indicates that the source of this higher hydraulic head is either a fracture located within the small section (~1.15 m) of un-tested borehole between the bottom of the steel casing (7.09 mBGS) and the top of the upper-most packer interval (8.23 mBGS), or a fracture located within the cased section of borehole and the water is short circuiting into the borehole via a poor casing seal. There were no fractures noted above the upper-most packer interval during the borehole camera logging or in the core logs produced during drilling.

4.4.3 Summary of Hydraulic Testing Properties

Table 4-4 summarizes the transmissivity and hydraulic conductivity of the Lansdowne municipal water supply aquifer based on five different datasets. Each dataset was analysed using the methodology that was best suited for the data provided as discussed below. In general, the T and K values for the interface between sandstone and Precambrian aquifers appear to have higher values compared to each individual bedrock unit.

Table 4.4 Summary of Hydraulic Testing Properties (T and K) for Lansdowne Municipal Supply Aquifer

Data Source	Method	T (m^2/day)	K (m/s)
Sandstone			
MW5 straddle packer tests	Hvorslev (slug tests)	0.5	1.7 × 10^{-7}
MOE database	Thiem	15.5	1.6 × 10^{-5}
	average	*8.0*	*8.0 × 10^{-6}*
Precambrian and Sandstone			
5-6 h pumping test MW5	Theis	69	2.6 × 10^{-5}
MW5 straddle packer tests	Hvorslev (slug tests)	0.4	1.6 × 10^{-7}
Municipal Well 1 pumping cycle	Theis	518.8	1.7 × 10^{-4}
Historical Pumping Tests (Ian D. Wilson, 1974 and M.S., Thompson, 1991)	Theis	276.9	8.5 × 10^{-5}
	average	*216*	*7.0 × 10^{-5}*
Precambrian			
5-6 h pumping test MW3	Theis	12	4.5 × 10^{-6}
MW5 straddle packer tests	Hvorslev (slug tests)	0.0	8.3 × 10^{-9}
MOE database	Thiem	5.6	2.7 × 10^{-6}
	average	*5.9*	*2.0 × 10^{-6}*

4.4.3.1 MW3 and MW5 pumping test data

MW3 and MW5 pumping test data were analysed using the Theis curve matching methodology, as summarized in Section 4.4.1, because the Theis curves provided the best curve matching fit to the data. MW3 is completed entirely in Precambrian bedrock while MW5 is completed over the sandstone-Precambrian interface.

4.4.3.2 MW5 straddle packer slug test data

MW5 straddle packer slug test data were analysed using the Hvorslev methodology, as summarized in Section 4.4.2, because the tested intervals are assumed to be confined and incompressible. MW5 straddle packer testing data provided 4 tests in sandstone, 17 tests in Precambrian and 1 test that crossed the interface between the sandstone and Precambrian bedrock aquifers. In all cases the straddle packer hydraulic testing data shows values for T and K that are approximately 2 to 3 orders of magnitude lower compares to other reported values. The reason for these discrepancies is unknown.

4.4.3.3 Municipal Well 1 typical pumping cycle

Municipal Well 1 cycles on and off two times per day and the recovery data (collected every 5 minutes using a datalogger as described in Section 4.2) from five consecutive cycles between September 15 and September 18, 2009 were analysed using the Theis curve matching methodology because it provided the best curve matching fit to the data.

4.4.3.4 MOE wells database

Estimates of overburden and bedrock hydraulic conductivity (K) were determined from the Theim equation applied to date from the MOE water well records as follows:

$$K = \frac{Q}{\Delta H 2\pi L}\ln(R/r)$$

where,

K	=	estimated hydraulic conductivity (m/s)
Q	=	assumed steady-state pumping rate (m^3/s)
H	=	assumed steady-state drawdown in well determined from static water level and the last recorded water level under pumping conditions (m)
L	=	length of open well subject to pumping test (m)
R	=	radius of influence of pumping test, (assumed to be 50 m)
r	=	radius of well (m)

The hydraulic conductivity (K) estimates calculated using the Thiem equation are only expected to be accurate within approximately one order of magnitude due to the assumptions stated above and potential errors in input parameters such as the assumed radius of influence. These calculations were completed for a total of 59 MOE well records with reported completions in sandstone and 856 with reported completions in Precambrian bedrock.

4.4.3.5 Historical pumping test data

Ian D. Wilson Associates Ltd. (1974) reported hydraulic parameter data for a variety of pumping tests including an 8 hour duration test on test well TW 2-74 (approximate location of municipal Well 2) with no observation wells, a 17 hour duration test on municipal TW 2-74 with useful data from one observation well (TW 4-74), and a 40 hour duration test on TW 4-74 (approximate location of municipal Well 1) with useful data from two observation wells (TW 2-74 and a community rink well). In addition, M.S. Thompson & Associates Ltd. (1991) reported data for a 20 hour duration pumping test on TW 2-74 with no observation wells. All pumping tests were completed in boreholes that intersected both sandstone and Precambrian bedrock. All pumping test data was analysed using Theis methodology for both early and late-time water level responses were reported.

4.5 Limited GUDI Assessment

MOE (2001) defines Groundwater Under Direct Influence of Surface Water (GUDI) as "groundwater having incomplete/undependable subsurface filtration of surface water and infiltrating precipitation". As discussed in Section 2.2, although Malroz (2004) concluded that the Lansdowne municipal wells are not GUDI, there have been sufficient field observations (cascading water, shallow well casing, and recent bacteriological contamination) that suggest the potential for GUDI status according to:

- Section 2(2) line 6 of Ontario Regulation 170/03 (Drinking Water Systems) – a drinking water system that exhibits evidence of contamination by surface water;
- MOE GUDI Terms of Reference (MOE, 2001) – wells regularly contain Total Coliforms and/or periodically contain E. coli.;
- MOE GUDI Terms of Reference (MOE, 2001) – bedrock wells situated within 500 of surface water and the source of groundwater for a well is within 15 m of ground surface.

This issue becomes more complicated because during the field inspection there was no evidence of surface water at ground surface in the immediate vicinity of the municipal wells, and the source of surface contamination may be due to the cascading water, the origin of which is unknown (i.e. could be described as "interflow" or ponding water or perched water table). As part of this study, there was no surface water body observed within 500 m of the Lansdowne municipal wells, however historically surface water ponding near the municipal wells has been reported. Therefore the issue of cascading water and recent bacteriological contamination may be more accurately described as a transport pathway caused by an improper well seal.

As noted in Section 2.2, treatment equivalent to chemically assisted filtration was installed and commissioned in 2010, meaning that the water supply is effectively treated as if it were surface water and that the issue of bacteriological contamination has been addressed. Notwithstanding this, implementation of improvements to the well seal (i.e. deepening of the casing) would be consistent with a "multi barrier approach" to source water protection, and would improve the overall reliability of the water supply (i.e., it may potentially mitigate future contamination by substances not removed by the recently installed system).

For the purpose of this report the Lansdowne wells are considered "potentially" GUDI and WHPA Zones E and F have been delineated.

5 GROUNDWATER VULNERABILITY ASSESSMENT

5.1 Groundwater Flow Modelling

A numerical model was developed to simulate the three-dimensional distribution of hydraulic head in the study area, using MODFLOW (McDonald and Harbaugh, 1988). The model was developed through the establishment of a finite difference grid, distribution of hydraulic conductivity and distribution of boundary conditions, with adjustments, as necessary, to match the output of the model to observed conditions. The observed conditions, for the purpose of this modelling study included the observed static water levels in the MOE WWIS, in combination with hydraulic heads measured in the monitoring wells by Geofirma staff during September 2009. Four sensitivity “scenarios” were developed from various combinations of boundary conditions and hydraulic conductivities. Geofirma’s in-house pre- and post-processor, mView, was used to develop the MODFLOW input files, and to view the MODFLOW output.

5.1.1 Finite Difference Grid

The finite difference grid divides the model domain into rows (separated by lines of constant northing) and columns (separated by lines of constant easting), and layers (vertically) of grid blocks. The grid was set up such that the rows and columns containing the two production wells were 5 m in size, and were increased in size outwardly from the production wells by a factor of 5% per row or column. The resulting finite difference grid contained 187 rows, and 163 columns of grid blocks.

The model domain was divided into 10 layers of grid blocks, with individual block thicknesses chosen to best match the hydrostratigraphy. The upper 2 model layers were used to simulate an assumed-to-be-homogeneous overburden, and were sized according to the overburden thickness estimated from the WWIS data. As noted in Section 3.5, the upper 2 model layers summed to a total of 2 m thickness everywhere except in the vicinity of the Village, where they summed to up to 9 m thickness. The next three layers down were used to simulate the Palaeozoic sandstone aquifer, where present, or a thin layer of the upper Precambrian aquifer. The next two layers down were used to simulate the upper 30 m of weathered Precambrian bedrock aquifer, while the lowest 3 layers were used to simulate a 120 m thickness of unweathered Precambrian bedrock aquitard.

5.1.2 Boundary Conditions

No-flow boundary conditions were assigned to the exterior of the model domain at inferred flow divides (either at topographic highs, or at topographic lows evidenced by surface water features), or parallel to inferred paths of horizontal flow (see Figure 5-1). The assumption of no flow across all exterior lateral boundaries assumes that there is no regional influence on local flow directions at the wellhead, which is a reasonable assumption given the data in Figure 5-1.. Following application of the no-flow boundary conditions, as shown in Figure 5-2a, each layer of the finite difference grid contained 28,301 active grid blocks.

Groundwater recharge was simulated using constant flux boundary conditions, assigned in the uppermost grid block layer. In three of the four simulations, a uniform recharge rate was assigned over the entire model domain. The rate of groundwater recharge in these cases ranged from 40 to 80 mm/year. In a fourth simulation, a distribution of high (87 mm/year), medium (58 mm/year) and low recharge (38 mm/year) was assigned based on the method of MOEE (1995) for estimating recharge distribution from topography, soil type and land cover (as described in CRCA (2009b) Section 2.1.4), as shown in Figure 5-2.

To assess the impact of recharge rate from the lagoons to the water table, simulations were performed in which the recharge rate for the lagoon (see Figure 5-2) was set at 0 and 400 mm/year.

Natural groundwater discharge was simulated with drain boundary conditions assigned in all uppermost grid blocks intersected by mapped surface water features (see Figure 5-2). The drain elevation was set equal to the grid block top elevation. Based on McDonald and Harbaugh (1988), the drain conductance was set according to the equation:

$$C = 2K_z \frac{A}{dz}$$

Where C is the vertical conductance, K_z is the vertical hydraulic conductivity, A is the plan view area of the grid block, and dz is the grid block thickness.

Pumping at municipal production wells was simulated using constant-flow boundary conditions. Based on McDonald and Harbaugh (1988), the extraction rates from each grid-block penetrated by the non-cased portion of the well was specified based on the transmissivity of each grid block, as follows:

$$Q_i = Q_T \frac{K_i dz_i}{\sum_{i=1}^{n} K_i dz_i}$$

where:

Q_i = flow rate from layer i [L^3/T]

Q_T = total flow rate from the well [L^3/T]

K_i = hydraulic conductivity of layer i [L/T]

dz_i = thickness of layer i [L]

n = number of grid block layers intersected by the non-cased portion of the well.

It is noted that in the simulations used to assess model calibration (see Section 5.1.4), two different pumping scenarios were considered. For comparison of simulated hydraulic heads to those contained in the WWIS, zero pumping was assumed. This is because the water levels in the area water wells and contained in the WWIS were generally measured prior to the construction of the municipal wells. For comparison of simulated hydraulic heads to those measured in the local monitoring wells, the total present day pumping rate of 71,000 cubic metres per year was split evenly between the two production wells.

5.1.3 Hydraulic Conductivity and Effective Porosity

Hydraulic conductivities were assigned based on: [1] the results of the hydraulic testing (pumping tests and straddle packer testing) conducted as part of this project, [2] typical ranges for clay, sandstone, and weathered and unweathered Precambrian bedrock (Freeze and Cherry, 1979), [3] professional experience with the modelling similar hydrostratigraphic units, and [4] the results of model calibration. As noted in Section 3.5, the overburden was assumed to be homogeneous. A hydraulic conductivity of $1x10^{-7}$ m/s was assigned to both the overburden and un-weathered Precambrian bedrock aquitard, while a variety of values were assigned to the Palaeozoic sandstone aquifer and weathered Precambrian aquifer, as part of the uncertainty analysis (see Section 5.1.4). The distribution of hydraulic conductivity zones is shown in Figures 5-3.

In studies of this type, effective porosity, θ_e, is the ratio of simulated groundwater flux, q, to particle velocity, v, used in backwards particle tracking for delineation of WHPAs. It is intended that, through the transformation, $v = q/\theta_e$, applied to the simulated groundwater flow field prior to particle tracking with MODPATH (Pollock, 1994), particles are tracked through the simulated flow field at the velocity of typical contaminant (i.e., bacteria) or dissolved phase contaminant plume. Therefore, effective porosities of 0.3 and 0.01 were used for overburden and bedrock, respectively. The value of 0.01 for bedrock porosity was chosen as a conservative (low) value, to account for uncertainty.

5.1.4 Model Calibration and Scenario Development

Model calibration is the exercise of adjusting model properties and boundary conditions, within reasonable bounds, so as to best match the model output to the observed conditions. In this study, the “observed conditions” were limited to historic water levels in domestic wells taken from the WWIS, and recent measured water levels in the monitoring wells (September, 2009 field data). The corresponding simulated hydraulic heads were interpolated from the model results, and assigned to the elevation corresponding to the average elevation of “water found” (WWIS data), or at the mid-point of the screened interval (monitoring well data). The match between the observed and simulated hydraulic head was assessed from: [1] plots of simulated versus observed hydraulic head, and [2] calibration statistics: maximum over-prediction, maximum under-prediction, residual mean, absolute residual mean, root mean square (RMS), normalized RMS, and correlation coefficient.

Since steady-state model calibration to hydraulic heads in the absence of groundwater flux information is subject to the problem of non-uniqueness of the solution (Healy, 2010), and since the hydraulic conductivity and recharge rate are subject to considerable uncertainty, a multi-scenario approach was taken in this study. In this approach, different combinations of hydraulic conductivity and recharge rate are found that produce reasonable calibration statistics, and the resultant flow fields are all used in the particle tracking exercise. The weight (or importance) applied to the various sets of particle tracks is adjusted during the process of WHPA delineation according the quality of each calibration.

Table 5-1 shows the different scenarios, or cases, considered in this study. These cases acknowledge that for this particular study, the most important uncertainties are, even with the available hydraulic test information, the horizontal and vertical hydraulic conductivity of the sandstone, the hydraulic conductivity of the upper Precambrian bedrock, and the recharge rate.

Table 5.1 Summary of Model Input Parameters

Case	Recharge Rate (mm/yr)	Hydraulic Conductivity (m/s)		
		Sandstone		Precambrian
		Horizontal	Vertical	Isotropic
Case 1	80	$1x10^{-6}$	$1x10^{-9}$	$1x10^{-5}$
Case 2	40	$1x10^{-6}$	$1x10^{-9}$	$4x10^{-6}$
Case 3	80	$1x10^{-5}$	$1x10^{-6}$	$1x10^{-6}$

Case 4	87,58, 38 (see Figure 5.2 for distribution)	$1x10^{-6}$	$1x10^{-9}$	$7x10^{-6}$

The calibration statistics for the four scenarios (or cases) are summarized in Table 5-2, while a graphical presentation of the four calibrations, being plots of simulated versus observed hydraulic head, are shown in Figure 5-4. In Figure 5-4, the vertical separation between the plotted point and the line of perfect agreement (the 45 degree line) indicates the size of the residual (difference between simulated and observed hydraulic head). The size and location of the residuals around Lansdowne are indicated by the coloured dots on Figure 5-5, which also shows contours of Case 1 simulated hydraulic head, in the uppermost layer of weathered Precambrian bedrock, layer 6.

Table 5.2 Summary of Calibration Statistics

Calibration Statistic	Case 1	Case 2	Case 3	Case 4
Number of Observations	153	153	153	153
Max. Over-prediction (m)	23.9	23.8	27.3	23.7
Max. Under-prediction (m)	10.8	11.0	8.0	11.1
Residual Mean (m)	1.6	1.2	4.8	1.2
Absolute Residual Mean (m)	3.4	3.3	5.4	3.3
Root Mean Square (RMS) (m)	5.0	4.9	6.9	4.9
Normalized RMS (%)	11.5	11.4	16.0	11.4
Correlation Coefficient	0.65	0.65	0.60	0.65

The results presented in Figures 5-4 and 5-5 and Table 5-2 indicate that the Case 1, 2, and 4 calibrations are basically indistinguishable from one another, and are indicative of an acceptable agreement between simulated and observed hydraulic head. It is important to note that while the model appears to over-predict the lowest observed hydraulic heads, these data were likely measured by the driller prior to complete recovery of the well. The hydraulic head measured between 70 and 75 masl east of Blue Mountain Road and south of the easterly extension of Eden Grove Road (see Figure 5-1), with associated "bulls-eye" in the contours of hydraulic head provides an example of an artificially low observed hydraulic head which the model appears to over-predict.

The reasonableness of the model calibration is further supported by the lack of bias in the colour coded Case 1 residuals shown in Figure 5-5 (i.e. the fact that the larger over-predictions (yellow and red dots) are not concentrated in particular areas, and are often found adjacent to the larger under-predictions (light and dark blue dots)). Although small clusters of over-predicted heads (2 to 3 orange/red dots) or under-predicted heads (2 to 3 blue dots) exist locally, the overall calibration is considered sufficient.

The Case 3 calibration is noticeably poorer, indicating the higher overall sandstone permeability and in particular the higher vertical sandstone permeability incorporated into this case to be physically unrealistic in combination with the other modelling parameters. Calibration to the monitoring well data is acceptable, although the model tends to under-predict head (over-predict drawdown) at the monitoring well closest to the municipal wells. The over-prediction of drawdown at the closest monitoring well may imply a higher rate of recharge locally (i.e., within the cone of depression of the pumping wells), or some other local hydrogeological feature. It is considered conservative to omit such local-scale features from the model, in the absence of multiple lines of evidence supporting them.

5.2 Wellhead Protection Area Delineation

5.2.1 WHPA Zone A, B, C, and D Delineation

WHPA Zone A was delineated as a circle of 100 metre radius centred around each of the two municipal production wells. To delineate WHPA Zones B, C, and D it was important to consider that the Lansdowne water supply may be taken in any rate combination from the two existing production wells. To account for this fact, and to account for the uncertainty in the true values of hydraulic conductivity and recharge, time-of-travel based WHPAs were delineated based on particle tracking conducted in twelve flow simulations: each of the four calibration cases with pumping taken entirely from Well #1, taken entirely from Well #2, and taken evenly from Wells #1 and #2. The total flow rate assumed in the WHPA delineation was 85,200 m^3/year, or 20% greater than the average rate pumped from the municipal wells over the last 10 years. A 20% increase over the historic rate accounts for potential population increases (J. L. Richards, 2005), and is conservative.

For the purpose of WHPA delineation, a ring of 20 particles was released in each grid block containing a pumping boundary condition (at each active production well, in layers 3 to 8), with radius 2.5 metres. WHPA Zones B, C, and D were delineated by drawing a smoothed polygon around all of the particle traces from all of the twelve flow simulations truncated to 2 years, 5 years, and 25 years time-of-travel, respectively (see Figure 5-6), weighted according to the calibration. Because of the poorer calibration for Case 3, a lower weighting was applied to this case than to the other cases, which resulted in a slightly smaller WHPA Zone B than would have resulted if all cases were weighted equally. This is apparent from the red particle traces (0-2 yr travel time) which transition to blue particle traces (2-5 yr travel time) just outside the WHPA Zone B line (2 yr tot based WHPA Zone) for Case 3 particle traces in Figure 5-6. (To draw the Zone B WHPA, the difference was split evenly between the outside envelope of the Case 3 particle traces, and the outside envelope of the other three cases combined.)

To assess uncertainty with regards to the recharge rate to the water table from the lagoons, a set of simulations were performed which were identical to the 12 used for WHPA delineation, with recharge rate across the lagoons footprint set to zero, and to 400 mm/year, in contrast to the rates between 38 and 80 mm/year used in the standard suite of simulations (see Figure 5-2). Figure 5-7 shows a close up of the particle traces used to delineate the WHPAs, along with the delineated WHPA in the standard suite of simulations. Also shown are the particle traces from the alternate lagoon recharge simulations. The results show that the length of the particle traces is controlled by the lagoon recharge rate, local to the lagoon, however they also show that only a slight modification to the WHPA Zones C and D would result if the 400 mm/yr lagoon recharge rate was adopted. Given that the WHPA Zones C and D intersect the lagoons as delineated, and given the difficulty in supporting the assumed 400 mm/yr recharge rate at the lagoons, it was decided to exclude the 400 mm/yr lagoon simulations from the WHPA delineation.

The final wellhead protection areas are shown in Figure 5-8. The WHPA is concentric about the municipal production wells, because of the fact that the wells are located on a topographic high, in a system in which groundwater flow is driven by local topographic relief. The WHPA contains much of Lansdowne.

5.2.2 WHPA Zone E and F Delineation

As discussed in Section 4.5, for the purposes of this report the Lansdowne municipal wells are considered to be "potentially GUDI", requiring delineation of WHPA Zones E and F . According to the Technical Rules (MOE, 2009) WHPA Zones E and F are delineated based on the "point of interaction" between the municipal groundwater capture zone and the surface water influencing this groundwater supply. In the case of Lansdowne no surface water body exists and the issue is more accurately described as a transport pathway, therefore for the purpose of delineating a WHPA Zone E and zone F, the "point of interaction" is defined as the municipal well location (i.e. where the municipal well intersects the surface water).

According to the Technical Rules (MOE, 2009), a WHPA Zone E comprises two parts: [1] the area within the surface water body where the time of travel is equal or less than the time that is sufficient to allow the operator of the system to respond to an adverse condition in the quality of the surface water (minimum of 2 hours), and [2] a maximum of 120 m setback onto land if the area within the surface water body abuts land. For the Lansdowne municipal wells where there is no surface water body identified, Part 1 of WHPA Zone E was not delineated and therefore the WHPA Zone E was defined entirely as a 120 radius around each well as shown in Figure 5-8.

Similarly, WHPA Zone F is defined in two parts; [1] as the area within each surface water body through which modeling demonstrates contaminants released during an extreme event may be transported to the intake (i.e. area in surface water beyond Zone E), and [2] maximum 120 m setback onto land. WHPA Zone F was therefore defined identical to WHPA Zone E as indicated in Figure 5-8.

5.3 Groundwater Vulnerability Analysis

5.3.1 Vulnerability Mapping

Groundwater vulnerability mapping was performed within the delineated WHPA Zones A through D using the Intrinsic Susceptibility Index (ISI) method applied to the materials overlying the supply aquifer (as suggested in MOE (2006) for cases where insufficient information about the water table is available to make the more typical calculation of ISI between the ground surface and the water table). The ISI provides a quantitative measure of the degree of protection afforded an aquifer by the overlying geologic material: the higher the index, the greater the degree of protection. The ISI was calculated using the equation:

$$ISI = \sum_{i=1}^{n} K_i B_i$$

where:

K_i = K-Factor for layer i;

B_i = thickness of layer i;

n = number of geological layers overlying the supply aquifer.

To calculate a conservative (i.e., maximum possible) estimate of ISI, the upper surface of the supply aquifer was assumed to coincide with the top of the bedrock, the total thickness of the overburden (as interpolated from the WWIS, and which is less than 3 m across most of the WHPA), the overburden is likely above the water table and, if clay, weathered. Where the overburden is thicker, at the south end of the WHPA, there are coarser grained materials present at depth, which if explicitly accounted for in the ISI calculation (they are not explicitly accounted for in the flow model), would require a K-Factor of 3 or lower. Figure 5-9 shows the overburden thickness and the upper estimate of the ISI, within the delineated WHPA. Based on this calculation, the value of ISI was determined to be less than 30 throughout the entire WHPA.

According to Table 4.1 of Draft Appendix 4 in the Vulnerability Guidance (MOE, 2006), the vulnerability of the aquifer is determined based on the ISI according to Table 5.3 below:

Table 5.3 Groundwater Vulnerability from ISI

ISI	Vulnerability
Less than 30	High
30-80	Medium
Greater than 80	Low

Since the ISI was determined to be less than 30 throughout the WHPA, the entire WHPA was assigned a vulnerability of "High", and adjustment of vulnerability to account for transport pathways was not necessary.

5.3.2 Vulnerability Scoring

Vulnerability scores for WHPA Zones A, B, C, and D were assigned as directed by Table 2(a) of the Technical Rules (MOE, 2009), as shown in Table 5.4.

Table 5.4 Vulnerability Scoring

WHPA Zone	Vulnerability		
	High	Medium	Low
A	10	10	10
B	10	8	6
C	8	6	4
D	6	4	2

Vulnerability scores (V) for WHPA Zones E and F were calculated using the following equation, as provided in Part VIII of the Technical Rules (MOE, 2009):

$$\text{Vulnerability Score (V)} = B \times C$$

where B is the Area Vulnerability Factor with an assigned value between 7 (lowest vulnerability) and 9 (highest vulnerability) for WHPA Zone E, and C is the Source Vulnerability Factor with an assigned value between 0.5 (lowest vulnerability) and 1.0 (highest vulnerability) for WHPA Zone E and Zone F. By definition, B for WHPA Zone E must be greater than B for WHPA Zone F. For the purpose of this study, the area vulnerability factor (B) was conservatively assigned a value of 9 due to the high vulnerability associated with the cascading water and potentially poor well seal (i.e. transport pathway). For the purpose of this study, the source vulnerability factor (C) was conservatively assigned a value of 1.0 due to the transport pathway, and the fact that bacteriological contamination has been reported.

Based on the above, the vulnerability scores for WHPA E and F were both been calculated as 9, which is the highest vulnerability score possible.

The final vulnerability scores for the Lansdowne WHPAs are shown in Figure 5-10. Where multiple vulnerability scores exists for the same area (i.e. WHPA Zones A, B, E and F), the final vulnerability score is defined as the highest vulnerability score (i.e. most conservative). For Lansdowne both the WHPA Zones A and B have a vulnerability score of 10 which is greater than the vulnerability score for WHPA Zone E and F. Therefore, the final vulnerability scores mapped in Figure 5-10 remain as WHPA Zones A, B, C, and D.

5.4 Identification of Transport Pathways

Geofirma completed a field inspection throughout the Village of Lansdowne in an effort to identify possible preferential pathways for surface contamination to reach the underlying aquifer(s). The following preferential pathways were identified near the Lansdowne WHPA: improperly abandoned domestic wells, poor casing seals on municipal wells and municipal service trenches. The majority of domestic wells identified in the MOE WWIS were not visible during the field inspection and therefore the exact location of each well was not measured. In addition, the exact location of municipal service trenches was not available at the time of this report. It is noted that although the “high” vulnerability of the supply aquifer meant that the identified transport pathways could not be used to justify an increase in the vulnerability rating (a “high” rating cannot be adjusted higher), this should not take away from the fact that elimination of transport pathways such as improperly abandoned wells would provide increased protection for the aquifer.

5.5 Uncertainty Analysis

According to Part I.4 of the Technical Rules (MOE, 2009), an analysis of the uncertainty, characterized by “high” or “low”, shall be made in respect of the WHPA delineation and groundwater vulnerability mapping. Uncertainty was assessed based on the available data and on professional judgement of the results of this study in comparison to other studies.

WHPA Zones A, E and F were assigned a low uncertainty rating, reflecting the fact that these zones are fixed relative to the coordinate of the production wells. WHPA Zones D was also assigned a low uncertainty rating. This reflects the fact that the direction of groundwater flow is reasonably well established (by the monitoring wells and water levels available in the WWIS for Lansdowne) and the hydrostratigraphy is reasonably well defined by the available geological mapping and borehole logs. WHPA Zone B and C were assigned a high uncertainty rating. This reflects the fact that they are strongly influenced by bedrock porosity, which is a highly uncertain value.

The vulnerability mapping is assigned a low uncertainty rating, reflecting the fact that the thickness of the clay layer overlying the source aquifer is reasonably well defined by the available borehole logs (WWIS and other borings).

5.6 Data Gaps

Data gaps associated with this WHPA delineation and groundwater vulnerability assessment include:

- Geological (depth of sandstone) and hydrogeological (i.e. water levels) data north of municipal wells – with no high quality targets immediately north of the municipal wells it is hard to judge calibration.
- Accurate recharge estimates near Lansdowne – a very difficult parameter to measure, this value directly influences the size of the WHPA capture zones.
- Role of cascading water with respect to GUDI status of municipal wells – the “potential” GUDI status of these wells requires delineation of WHPA Zones E and F.
- Vertical hydraulic conductivity of sandstone – the only way the numerical groundwater flow model was able to better represent elevated hydraulic heads within the sandstone escarpment was to decrease the vertical hydraulic conductivity by three orders of magnitude.

6 TIER 2 WATER BUDGET

The Tier 2 water budget was completed for the Lansdowne subwatershed and was carried out on a monthly basis using the following equation:

$$P + G_{NET} = Q + ET + \Delta S$$

where:

P	=	precipitation (groundwater source),
G_{NET}	=	net groundwater in (groundwater flowing into subwatershed minus that flowing out),
ET	=	evapotranspiration (groundwater sink),
Q	=	net streamflow out (groundwater sink), and
ΔS	=	change in storage (groundwater source or sink).

6.1 Tier 2 Water Budget Data Sources

6.1.1 Precipitation (P)

The average monthly and annual precipitation (P) data used in the Tier 2 water budget are the average values over the Lansdowne subwatershed during the period 1971-2000 as reported by Natural Resources Canada (McKenney et al., 2006b). This is the same dataset that was used in the Tier 1 WB for many SPAs in Eastern Ontario. These data are derived from Meteorological Service of Canada spatial models and have undergone considerable quality assurance efforts by both Environment Canada and the Great Lakes Forest Research Centre. These data were interpolated between all local climate stations and even attempted to account for “lake effects” as a function of distance away from major water bodies (i.e. Lake Ontario). Therefore the McKenney et al. (2006a) dataset is generally considered to be the best estimate of spatially distributed precipitation for locations that do not measure their own climate data and is the most recent dataset to undergo such a comprehensive analysis.

Precipitation records were analysed from individual climate stations nearby Lansdowne (i.e. Brockville, Kingston) that provided data as far back as 1872, however no one dataset was complete. The precipitation records for individual climate stations show a lesser amount of precipitation before 1970 however the 1971-2000 dataset used in this study is considered to be sufficiently accurate for the purpose of calculating current water budgets and projecting into the near future (i.e. 25 yrs).

6.1.2 Evapotranspiration (ET)

Actual evapotranspiration (AET) values for the study area were computed using the Thornthwaite method (Thornthwaite and Mather, 1957), utilizing the 1971-2000 temperature datasets from McKenney et al. (2006b). Water deficits were estimated using the Canadian Ecodistrict Climate Normals database (AgriCan, 1997). The distribution of Soil Water Holding Capacities (SWHC) for the Lansdowne subwatershed was obtained from CRCA, and were used to calculate water deficits.

6.1.3 Lateral Groundwater Flow (G_{net})

The net amount of groundwater flowing into the Lansdowne subwatershed (G_{net}) was estimated from the groundwater flow model simulations as discussed in Section 5. This value is output from the calibrated groundwater flow model as groundwater flowing into minus groundwater flowing out of the Lansdowne subwatershed boundary. Since the value is essentially zero, zero is assumed in the calculations.

6.1.4 Streamflow (Q)

Lansdowne is located in the St. Lawrence River ungauged watershed, therefore no direct measurements of streamflow are available. Streamflow (Q) in Lansdowne was thus estimated using data from the nearby Lyn Creek subwatershed, which is the recommended gauged watershed to use as a proxy for ungauged subwatersheds in the St. Lawrence River watershed as discussed in the Tier 1 WB study (XCG, 2009).

6.1.5 Storage (ΔS)

The change in storage (ΔS) for the watershed includes all components of water storage (surface water, groundwater, and snowpack). This value was the "unknown" value in the water budget equation and was calculated as the difference between all of the other parameters discussed above. The storage value therefore also includes the long-term mean residual (uncertainty in other water budget parameter values).

6.2 Tier 2 Water Budget

The water budget for the Lansdowne subwatershed is summarized in Table 6.1. The annual sums are given to the nearest millimetre. The true value for the annual ΔS is zero; the non-zero value is the result of rounding and represents the residual uncertainty in all parameters.

Table 6.1 Average Long Term Monthly Water Budget for the Lansdowne Subwatershed

Month	P (mm)	ET (mm)	Q (mm)	G_{net} (mm)	ΔS (mm)
January	78	0	38	0	40
February	61	0	36	0	25
March	70	0	94	0	-24
April	77	32	101	0	-56
May	78	77	29	0	-29
June	76	105	12	0	-41
July	75	105	4	0	-34
August	87	94	3	0	-11
September	100	74	5	0	22
October	85	36	18	0	31
November	94	9	37	0	48
December	87	0	40	0	47
Total Annual	968	533	417	0	19

7 TIER 2 GROUNDWATER QUANTITY STRESS ASSESSMENT

7.1 Methodology

Groundwater quantity stress is to be determined as a calculation of % Water Demand as described in the WB Guidance (MOE, 2007 - Appendix A) and shown in the flowing equation:

$$\%Water\ Demand = 100\frac{Q_{Demand}}{Q_{Supply} - Q_{Reserve}}$$

where,

Q_{Demand} = water demand as calculated in Section 7.4,
Q_{Supply} = water supply as calculated in Section 7.3, and
$Q_{Reserve}$ = water reserve, and is calculated as 10% of the groundwater discharge.

These calculations are to be completed under a variety of scenarios including current pumping, future pumping (20 years, based on growth estimates in the Official Plan) and drought scenarios, and on a monthly and annual time-scale, as described in Section 7.2. The determination of subwatershed stress level for Tier 2 WB studies is further outlined in The Technical Rules (MOE, 2009 - Part III.4) and is summarized in Table 7.1 below.

Table 7.1 Groundwater Quantity Stress Level Determination

Groundwater Quantity Stress Level	Significant	Moderate	Uncertain Moderate	Low
Annual Average	> 25%	> 10%	8 to 10%	0 to 10%
Maximum Monthly	> 50%	> 25%	23 to 25%	0 to 23%

The highest stress level determined for all of the scenarios is the final stress level assigned to the subwatershed area.

7.2 Scope of Work

Stress levels are assigned to a subwatershed based on the output of several scenarios for the Tier 2 Water Budget analysis. These scenarios are specified in Table 1 of the Technical Rules (MOE, 2009) and are reproduced below as Table 7.2; scenarios applicable to this study are shaded.

Table 7.2 Subwatershed Stress Level Scenarios

Scenario	Description	Data Restrictions - Demand	Data Restrictions - Supply and Reserve
A	Existing system, average		Data related to climate and stream flow shall be the historical data set for climate and stream flow.
B	Existing system, future demand	Data related to demand associated with the system within the subwatershed shall be reflective of the future development in the subwatershed.	Data related to climate and stream flow shall be the historical data set for climate and stream flow. Data related to land cover shall be reflective of the future development in the subwatershed.

Scenario	Description	Data Restrictions - Demand	Data Restrictions - Supply and Reserve
C	Planned system demand, operational year	Data related to demand associated with an existing type I, II or III system within the subwatershed shall be reflective of the demand that would exist in the year that the planned system will be operational.	Data related to climate and stream flow shall be the historical data set for climate and stream flow. Data related land cover shall be reflective of the year that the planned system will be operational.
D	Existing system, two year drought		Data related to climate and stream flow shall be reflective of the two year drought period.
E	Existing system, future two year drought	Data related to demand associated with an existing type I, II or III system within the subwatershed shall be reflective of the future development in the subwatershed.	Data related to climate and stream flow shall be reflective of the two year drought period. Data related to land cover shall be reflective of the future development in the subwatershed.
F	Planned system, operational year, two year drought	Data related to demand associated with an existing type I, II or III system within the subwatershed shall be reflective of the demand that would exist in the year that the planned system will be operational.	Data related to climate and stream flow shall be reflective of the two year drought period. Data related to land cover shall be reflective of the year that the planned system will be operational.
G	Existing system, ten year drought		Data related to climate and stream flow shall be reflective of the ten year drought period.
H	Existing system, future ten year drought	Data related to demand associated with an existing type I, II or III system within the subwatershed shall be reflective of the future development in the subwatershed.	Data related to climate and stream flow shall be reflective of the ten year drought period. Data related to land cover shall be reflective of the future development in the subwatershed.
I	Planned system, operational year, ten year drought	Data related to demand associated with an existing type I, II or III system within the subwatershed shall be reflective of the demand that would exist in the year that the planned system will be operational.	Data related to climate and stream flow shall be reflective of the ten year drought period. Data related to land cover shall be reflective of the year that the planned system will be operational.

7.3 Groundwater Transient Model Scenarios and Parameter Estimation

To assist with the groundwater quantity stress assessment, a transient groundwater flow simulation was conducted using the model developed for the WHPA delineation. In this simulation, a monthly variable recharge rate was applied over the entire model domain totalling 60 mm/year (the middle of the range considered in the WHPA cases), with Case 4 hydraulic conductivities (see Section 5.1.4), and overburden and bedrock specific storage values of $1x10^{-3}$ m^{-1} and $5x10^{-5}$ m^{-1}, respectively.

The monthly recharge rates were determined by scaling the annualized amount (60 mm) by the monthly water surplus (P – ET), weighted to exclude the summer months (June, July and August) when no recharge occurs, and to consider the reduced recharge in the winter months by halving the water surplus in December, January and February, and redistributing the excess over the spring months (1/3 to March and 2/3 to April) to account for an increased amount of recharge due to snow melt and spring freshet.

The hydraulic head at the well was monitored during the simulation and is plotted in Figure 7-1. The entire simulation comprised 4 transient model scenarios, including: [1] 10 years of average "normal" conditions as described above, followed by [2] the 2-yr drought scenario (i.e. zero recharge), followed by [3] 10 years of average "normal" conditions to "reset" the model, followed by [4] the 10-yr drought scenario. The 3 m amplitude of the fluctuations in water level at the municipal wells is consistent with observed seasonal fluctuations in water level, providing confidence in the selection of specific storage values.

Average annual values for recharge, change in groundwater storage (ΔS), total groundwater discharge to surface water, and lateral groundwater flow (G_{in}), were obtained from the transient MODFLOW simulation for a typical year (i.e., year 10 of the simulation) by averaging monthly flows determined using the "Zone budget" capability. Average annual rather than monthly values are mandated for use in the stress assessment by the Technical Rules (MOE, 2009).

7.4 Supply and Reserve

7.4.1 Groundwater Supply (Q_{Supply})

For the purpose of the Tier 2 WB percent water demand calculations, groundwater supply (Q_{Supply}) is defined in the Technical Rules as the sum of the estimated groundwater recharge rate and the estimated groundwater flow INTO a subwatershed (i.e. $Q_{Supply} = R + G_{in}$). It is important to note that this groundwater "in" term (G_{IN}) is not equal to the estimate of "net" lateral groundwater flow and therefore does not take into account groundwater flowing out of the subwatershed. Similar to the Tier 1 WB study, and as per the WB Guidance, groundwater storage was not considered within the context of the percent groundwater demand calculations (i.e. $\Delta S = 0$). In addition, the WB Guidance indicates that the monthly values for recharge are to be calculated as the annual estimate divided by 12 months.

The best estimate of annual recharge is 60 mm/yr and was obtained from the calibrated groundwater flow model, as described in Section 7.3.1. Therefore the monthly recharge value for use in the Q_{Supply} becomes 5 mm/month uniformly applied over the entire subwatershed area (~14.6 km^2) which equates to 73,000 m^3/month or 2400 m^3/d (i.e. 60 mm/yr x 14.6 km^2).

Similarly, the best estimate of the total rate of groundwater flowing into the Lansdowne subwatershed

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(G_{in}) was also obtained from the calibrated groundwater flow model. The WB Guidance indicates that lateral groundwater flow should be considered constant throughout the year (i.e. annual average divided by 12). Therefore, G_{in} is estimated to be 18,318 m^3/month which is equivalent to approximately 15 mm/yr (1.25 mm/month) uniformly applied over the entire subwatershed area or approximately 25% compared to estimate average annual recharge.

7.4.2 Groundwater Reserve ($Q_{Reserve}$)

Groundwater reserve ($Q_{Reserve}$) is subtracted from the total groundwater supply term, prior to evaluating the percent groundwater demand, and is intended to "reserve" a small portion of groundwater supply in support of other uses in the subwatershed that are unaccounted for. Several examples of other uses include ecosystem requirements, dilution for sewage treatment plant (STP) discharge, or navigation needs.

The Technical Rules define groundwater reserve as 10% of the existing groundwater discharge (i.e. baseflow). Average annual groundwater discharge obtained from the groundwater flow model is approximately 800,000 m^3/yr (55 mm/yr) which equates to 67,000 m^3/month or 91% of estimated average annual recharge. Therefore average annual groundwater reserve, for the purpose of Tier 2 WB percent groundwater demand, is calculated to be 6,700 m^3/month (i.e. 10% groundwater discharge which equates to approximately 9% of recharge). The groundwater supply and reserve are summarized in Table 7.3.

Table 7.3 Summary of Water Supply (m^3)

	Recharge	Lateral Inflow (G_{in})	Supply (Q_{Supply})	Reserve ($Q_{Reserve}$)
January	73,000	18,300	91,300	6,700
February	73,000	18,300	91,300	6,700
March	73,000	18,300	91,300	6,700
April	73,000	18,300	91,300	6,700
May	73,000	18,300	91,300	6,700
June	73,000	18,300	91,300	6,700
July	73,000	18,300	91,300	6,700
August	73,000	18,300	91,300	6,700
September	73,000	18,300	91,300	6,700
October	73,000	18,300	91,300	6,700
November	73,000	18,300	91,300	6,700
December	73,000	18,300	91,300	6,700
Annual	**876,000**	**219,600**	**1,095,600**	**80,400**

7.5 Groundwater Demand

Groundwater demand within the Lansdowne subwatershed comprises two municipal wells, approximately 78 domestic wells, and agriculture demand. Aside from the two municipal wells, no other MOE Permit to Take Water (PTTW) uses are reported within the study area. Each of these groundwater demands are discussed in the following sections.

7.5.1 Municipal Demand

The municipal water system is the most significant taking of groundwater in the Lansdowne subwatershed, where there are two wells with a total design capacity of 1226 m^3/day. Water use for the municipal wells has been recorded since 1998, and are shown in Figure 7-2. Monthly water use records over the period of 1998-2008 (OCWA, 2009) were used to estimate the municipal demand for groundwater. With the exception of 2001, water use from the Lansdowne municipal system has been relatively consistent, at between 60,000 – 80,000 m^3 per year and consistently alternating between high 60,000 and low 70,000 m^3/year each year over the past five years. Therefore, although the Technical Rules request an estimation of municipal demand for the “Study Year”, defined as the most recent full year of data, to ensure that the most representative pumping rates are being used, this is not the most representative pumping rate for the Lansdowne municipal supply wells. The most recent and representative municipal well pumping rates for Lansdowne is an average annual municipal water pumping rate over the most recent two years of reported data (2007 and 2008). This value, approximately 71,000 m^3/yr, was used in the WHPA modelling for current conditions.

A consumptive use factor was applied to the municipal water takings to determine the municipal demand by accounting for non-consumptive returns to the subwatershed (i.e. sewage treatment plant discharge, septic systems, lawn watering, etc). The WB Guidance (MOE, 2007) recommends the use of a consumptive factor of 0.2 for a municipal water supply unless better data is available.

Wastewater for the Lansdowne municipal system is routed to sewage lagoons which are diverted to creeks, so do not likely recharge the groundwater directly. OCWA estimates the quantity of “unaccounted for” water, which can be defined as the difference between the volume of water treated at municipal wells and the volume of water either measured at domestic water meters or used for system maintenance. OCWA records show an increasing trend of unaccounted for water in the Lansdowne distribution system from approximately 13% in 2003 to approximately 28% in 2008 (OCWA, 2009). One explanation for this discrepancy is that some water is leaking from the system and recharging the groundwater system. The annual average of “unaccounted water” for all available data (2003-1009) is approximately 18%. These factors prompted the use of a more conservative value of 0.82 for a consumptive use factor to the municipal water demand.

7.5.2 Unpermitted Domestic Demand

The unpermitted domestic demand was assumed to be entirely from private residential wells. The MOE database was used to find the total number of domestic wells (78) in the Lansdowne subwatershed that are located outside of the serviced-area for the village of Lansdowne. The service area for Lansdowne was estimated based on information provided by OCWA (2008). Census data is collected approximately every 5 years in Canada, the most recent survey completed in 2006. The most recent census data for the Township of Leeds and the Thousands Islands (StatCan, 2006) was used to estimate an average of 2.6 persons per domestic well. Municipal water records were used to estimate the average annual per capita daily water withdrawal as 288 L/person/day (range between 270-342), based on a serviced population of 675 over the period of 1998-2008. This was done on a monthly basis to account for increased water consumption in the summer. A consumptive use factor of 0.2 was applied to the unpermitted domestic demand, as recommended by The WB Guidance.

7.5.3 Unpermitted Agricultural Demand

There are no agricultural operations in the study area large enough to require a PTTW, therefore non-permitted agricultural demand was estimated using data from de Löe (2002). These data provide estimates of agricultural water demand on a watershed basis derived from farming practices, crop type and size, and livestock type. The de Löe data was scaled to the percentage of area occupied by the Lansdowne subwatershed to determine the agricultural demand for the study area. All agricultural takings are assumed to be from groundwater sources.

Livestock use demands were applied throughout the year, whereas crop use demands were distributed over the summer months only as recommended by The WB Guidance. A consumptive use factor of 0.9 was applied to all agricultural takings.

7.5.4 Estimation of Future Demand

Present and future water demands are summarized in Table 7.4, where future water demand is the projected demand associated with the subwatershed based on forecasted development (Technical Rules).

Table 7.4 Summary of Water Demand (m^3/month)

	Permitted	Non-Permitted		Total Demand	
	Municipal	**Domestic**	**Agricultural**	**Present**	**Future**
Consumptive Use Factor	*0.82*	*0.2*	*0.9*		
January	4,700	400	400	5,500	6,400
February	4,300	400	400	5,100	6,000
March	4,500	400	400	5,300	6,200
April	4,600	400	400	5,400	6,300
May	5,100	400	400	5,900	6,900
June	5,100	400	400	5,900	6,900
July	5,800	400	700	6,900	8,100
August	5,400	400	700	6,500	7,600
September	4,700	400	400	5,500	6,400
October	4,800	400	400	5,600	6,600
November	4,700	400	400	5,500	6,400
December	4,500	400	400	5,300	6,200
Annual	**58,200**	**4,800**	**5,400**	**68,400**	**80,000**

For the Lansdowne subwatershed, future water demand was estimated based on estimated population growth 20 years into the future, which is presumed to be the only factor affecting future municipal water demand. The following data was considered when estimating population growth for Lansdowne:

- The Official Plan for the Township of Leeds and the Thousand Islands (J.L. Richards, 2005) assumes a growth rate of 1% per year for planning purposes;
- The 2006 census data (Statistics Canada, 2006) shows a 4% population increase for the entire Township of Leeds and the Thousand Islands between 2001 and 2006 (i.e. 20% increase in 25 years);
- OCWA records show a stable serviced population of 675 people since 1998 (i.e. 0 % increase in 25 years), and the most recent population estimate for Lansdowne is approximately 590 people;

A 20% increase in municipal water demand was assumed for the purpose of the Tier 2 WB study, consistent with the average population increase within the entire Township of Leeds and the Thousand Islands over the past 5 years. It is also consistent with population growth estimates made in the Official Plan for the Township of Leeds and Grenville (J. L. Richards & Associates, 2005). Although this value seems high based on a relatively constant population in Lansdowne over the past 11 years, some conservatism is necessary due to the relatively low water demand currently on the Lansdowne system. For example, the addition of one large PTTW user or new subdivision could significantly increase the Lansdowne municipal demand. The overall increase in total demand (municipal + PTTW + domestic + agriculture) was determined to be approximately 17%, as shown in Table 7.4.

7.6 Percent Groundwater Demand – Normal Conditions (Scenarios A & B)

Using the average monthly and annual values for Q_{Supply}, $Q_{Reserve}$, and Q_{Demand} and applying the equation from Section 7.1, the percent groundwater demand for the Lansdowne municipal subwatershed under average annual conditions is approximately 7% for present-day demand, with monthly demands ranging from 6.0% (February) to 8.2% (July). The demand marginally increases to approximately 8% under average annual conditions for future (20% increase) in municipal demand, with monthly demands from 7.1% to 9.6%. The results are summarized in Table 7.5

Table 7.5 % Groundwater Demand, Normal Conditions (m^3/month)

	Q_{supply}	$Q_{reserve}$	Q_{demand}		Stress (%)	
			Present	Future	Present	Future
January	91,300	6,700	5,500	6,400	6.5	7.6
February	91,300	6,700	5,100	6,000	6.0	7.1
March	91,300	6,700	5,300	6,200	6.3	7.3
April	91,300	6,700	5,400	6,300	6.4	7.4
May	91,300	6,700	5,900	6,900	7.0	8.2
June	91,300	6,700	5,900	6,900	7.0	8.2
July	91,300	6,700	6,900	8,100	8.2	9.6
August	91,300	6,700	6,500	7,600	7.7	9.0
September	91,300	6,700	5,500	6,400	6.5	7.6
October	91,300	6,700	5,600	6,600	6.6	7.8
November	91,300	6,700	5,500	6,400	6.5	7.6
December	91,300	6,700	5,300	6,200	6.3	7.3
Annual	**1,095,600**	**80,400**	**68,400**	**80,000**	**6.7**	**7.9**

7.7 Percent Groundwater Demand – Drought Conditions (Scenarios D,E,G & H)

Evaluating the level of stress with respect to Drought Scenarios D, E, G and H, according to Section 35 (2) f of the Technical Rules (MOE, 2009), is limited to an assessment of the resulting drawdown in the municipal well and if it affects the operation of the pump. Specifically, a moderate level of groundwater quantity stress is assigned if either of the following conditions occurs:

(i) The groundwater level in the vicinity of the well was not at a level sufficient for the normal operation of the well; or

(ii) The operation of a well pump was terminated because of an insufficient quantity of water being supplied to the well.

As discussed in Section 2.2, the pump operates at a depth of approximately 30 mBGS in both Well 1 and Well 2, which is approximately 15 m below the static hydraulic head.

7.7.1 2-Year Drought - Scenarios D & G

In the 2-year drought scenario, recharge is effectively stopped for two years. This scenario was implemented in years 11 and 12 of the transient simulation, resulting in an average drop in water level at the municipal wells of 8 m (see Figure 7-1). Although this drawdown does not lower the water level below the simulated pump level, there is a reasonable chance that the efficiency of the pump is reduced and therefore the 10-yr drought scenario was assessed.

7.7.2 10-Year Drought – Scenarios E & H

The 10-year drought scenario was based on the lowest 10-year moving average of precipitation on record. Kingston climate data was used, as its record extends as far back as 1873 (Environment Canada, 2009) and the lowest 10-yr moving average annual precipitation was 750.5 mm from 1914-1923. The 10-yr drought was simulated in years 31 to 40 of the transient simulation. An equivalent value of groundwater recharge for the 10-yr drought was determined by normalizing the ratio of recharge to precipitation for the current rate to the 10-drought rate. This resulted in reducing the annual recharge rates to 46.5 mm (60 mm/year in the typical year multiplied by 750.5 mm year, divided by 968 mm/year), resulting in an average drop in water levels at the municipal wells of 1.5 metres. This level of drawdown would not likely impact the operation of the well as described in Section 35 of the Technical Rules (MOE, 2007).

7.8 Uncertainty Analysis

In accordance with the Technical Rules, the uncertainty of the groundwater quantity stress assessment calculations was considered by examining hydrologic parameter selection, consumptive demand assumptions, and consistency of simulated model data with field conditions. Each of these uncertainty categories are discussed in detail below.

7.8.1 Hydrologic Parameters

Assessing the range of each hydrologic parameter used in the numerical groundwater flow model and its impact on the percent groundwater demand calculations provides a thorough assessment of uncertainty in the calculations.

Recharge - The average annual value of 60 mm/yr was used for the percent demand calculations, however recharge was varied between 40 and 80 mm/yr during the model simulations for WHPA delineation. Taking the most conservative value of recharge (i.e. value that will achieve highest groundwater stress), the annual average percent groundwater demand under current pumping rates and normal climate conditions (i.e. no drought) is 9.3% and 10.9% under future pumping rates. This results in a “moderate” stress with respect to future pumping rates. Based on the fact that the % water demand score changed from a “low” category to a “moderate” category simply by varying one hydrologic parameter within a reasonable sensitivity range, the uncertainty associated with estimation of hydrologic parameters is “high”.

Lateral Groundwater Flow – Although not considered in the recalculations above, if recharge is decreased by 1/3 to 40 mm/yr, the volume of water flowing into the Lansdowne subwatershed would be smaller compared to a recharge value of 60 mm/yr. Therefore assuming G_{in} is also reduced by 1/3, the annual average percent groundwater demand under current and future pumping rates becomes 10.4% and 12.2%. This results in a moderate stress with respect to both current and future pumping rates, further emphasizing the high level of uncertainty in estimation of hydrologic parameters.

7.8.2 Consumptive Demand

The consumptive demand factor assumed for the Lansdowne municipal water supply was estimated based on “unaccounted for” water in the municipal distribution system (Section 7.5.1). Although this analysis supports the selection of a consumptive factor, there remains a high level of uncertainty with respect to the true value. For example, the percent of water pumped into the sewage treatment ponds that returns to the groundwater system is unknown. Preliminary analysis of OCWA records indicates that more water is entering the sewage treatment ponds compared to the amount pumped from the municipal wells. One possibility is that groundwater is seeping into the sewage pipes, thereby reducing the amount of water available for Q_{Supply}.

In addition, the Tier 1 WB study assumed that 100% of the groundwater pumped from the Lansdowne wells was removed from the groundwater system due to the fact that the sewage ponds were discharged to a nearby ditch or creek. Applying this conservative estimate with the conservative estimates of recharge and lateral groundwater flow into the subwatershed, the percent groundwater demand under current and future pumping conditions becomes 12.3% and 14.5%. Similarly, this shows the high level of uncertainty in the percent water demand classification.

7.8.3 Consistency with Field Data

The most obvious indication of uncertainty with respect to the percent water demand calculations is the declining water levels measured in the Lansdowne municipal Well 1 between 2001 and 2010 as shown in Figure 7-3. OCWA collects groundwater levels in each municipal well on a monthly basis and these values show a consistent decline of approximately 2 m over the 9 year period. This groundwater level decline is viewed to be in excess of any small increase in municipal pumping rate over the same period.

In a meeting of the Technical Advisory Group for this project on 22 December, 2010, it was indicated by OCWA that the apparent falling water levels may in fact be due to the “drift” of the instrument measuring the water levels, and that levels may not actually be falling at all. When the well pump was removed recently in the treatment system upgrade, the water levels measured were found to be very similar to those seen in the early 1970’s. Accurate water level monitoring in the municipal wells is important, as these various discussions indicate.

7.9 Groundwater Stress Level Determination

Part III.4 of the Technical Rules (MOE, 2009) specifies the classification of a groundwater stress level. Section 35 (reproduced below) is most relevant to the Lansdowne study based on the percent groundwater demand calculations and outlines the rational for assigning a “moderate” or “low” level of stress to a subwatershed within the context of a Tier 2 groundwater stress assessment.

35. For the purposes of rule 23 or 25, a subwatershed shall be assigned a groundwater stress level of significant, moderate or low in accordance with the following,

(1) Significant, if one or more of the following circumstances exist:

(a) During scenario A or B in Table 1 the annual percent water demand for groundwater for the subwatershed would be greater than or equal to 25%.

(b) Where there is a planned type I, II or III system proposed to be located within the subwatershed, during scenario C in Table 1 the annual percent water demand for groundwater for the subwatershed would be greater than or equal to 25%.

(c) During scenario A or B in Table 1 the maximum monthly percent water demand for groundwater for the subwatershed would be greater than or equal to 50%.

(d) Where there is a planned type I, II or III system proposed to be located within the subwatershed, during scenario C in Table 1 the maximum monthly percent water demand for groundwater for the subwatershed would be greater than or equal to 50%.

(2) Moderate, if a stress level was not assigned by subrule (1) and one or more of the following circumstances exist:

(a) During scenario A or B in Table 1 the annual percent water demand for groundwater for the subwatershed would be less than 25% but greater than 10%.
(b) Where there is a planned type I, II or III system proposed to be located within the subwatershed, during scenario C in Table 1 the annual percent water demand for groundwater for the subwatershed would be less than 25% but greater than 10%.
(c) During scenario A or B in Table 1 the maximum monthly percent water demand for groundwater for the subwatershed would be less than 50% but greater than 25%.
(d) Where there is a planned type I, II or III system proposed to be located within the subwatershed, during scenario C in Table 1 the maximum monthly percent water demand for groundwater for the subwatershed would be less than 50% but greater than 25%.
(e) At any time after January 1, 1990, in relation to a type I, II or III system within the subwatershed, one or both of the following circumstances occurred:
(i) the groundwater level in the vicinity of the well was not at a level sufficient for the normal operation of the well; or
(ii) the operation of a well pump was terminated because of an insufficient quantity of water being supplied to the well.
(f) In relation to a type I, II or III system within the subwatershed, one or both of the circumstances described in clause (e) would occur:
(i) during scenarios D or E; and
(ii) during scenarios G or H.
(g) In relation to a planned type I, II or III system proposed to be located within the subwatershed, either of the circumstances described in clause (e) would occur:
(i) during scenarios D, E or F; and
(ii) during of scenarios G, H or I.
(h) All of the following are true:
(i) the result of one or more annual percent water demand calculations made in accordance with subclause (a) or (b) of subrule (2) is between 8% and 10%, inclusive;
(ii) the uncertainty associated with the percent demand calculations required by this rule, when evaluated to be high or low considering the factors set out in rule 36, is high;
(iii) a sensitivity analysis of the data used to prepare the Tier Two Water Budget suggests that the stress level for the subwatershed could be moderate.
(i) All of the following are true:
(i) the result of one or more maximum monthly percent water demand calculations made in accordance with clause (c) or (d) of subrule (2) is between 23% and 25%, inclusive;
(ii) the uncertainty associated with the percent demand calculations required by this rule, when evaluated to be high or low considering the factors set out in rule 36, is high;
(iii) a sensitivity analysis of the data used to prepare the Tier Two Water Budget suggests that the stress level for the subwatershed could be moderate.

(3) Low, if a stress level was not assigned by either subrule (1) or subrule (2).

The stress assessment did not result in any of the circumstances described in subrules (1) and (2), therefore the Lansdowne subwatershed is assigned a low stress level in accordance with Rule 35(3)) of the Technical Rules (MOE, 2009).

7.10 Data Gaps

Data gaps associated with this Tier 2 WB and groundwater quantity stress assessment include:

- The consumptive factor for municipal demand. The actual amount of water lost in the system is relatively unknown and therefore more information is needed on the design of the water supply and sewage systems. This may include inspection or replacement of current water meters in homes or as part of the system, or onto lines flowing into the sewage ponds.
- Accurate recharge estimates near Lansdowne – a very difficult parameter to measure, this value directly influences the amount of water available to Q_{Supply}. The percent demand calculation is very

sensitive to this parameter and is shown to change from a low to a moderate level simply by varying this parameter within its estimated range.

- The lateral groundwater flow term is a calculated estimate based on the groundwater flow model. The flow model is most accurate around the Lansdowne municipal wells due to a higher density of hydrogeologic data, and the certainty is lower further away from the wellhead. Therefore the amount of lateral groundwater flow into the Lansdowne subwatershed is relatively uncertain.
- Accurate measurements of groundwater level in the municipal wells and in elsewhere in the aquifer would allow for a more comprehensive evaluation of groundwater mining over the long term.
- The lack of groundwater calibration water levels on the topographic high west and north of the village.
- An understanding of the accurate groundwater catchment area for the Lansdowne municipal wells. This area will most likely provide a more accurate study area for % groundwater demand compared to the subwatershed boundary.

8 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions are made:

1) WHPA Zones A, B, C, D, E, and F were delineated in accordance with the Technical Rules (MOE, 2009) and the most recent Vulnerability Guidance (MOE, 2006).

2) A long term annual water budget has been developed for the Lansdowne subwatershed.

3) The Lansdowne subwatershed was classified with a low level of groundwater stress in accordance with the Technical Rules (MOE, 2009), although the groundwater stress determination has a high level of uncertainty.

4) Recent installation of treatment equivalent to chemically assisted filtration (suitable for surface water) on the municipal supply has addressed the issue of recent bacteriological contamination.

5) The shallow well casing, the cascading water, and the recent bacteriological contamination may indicate a less-than-ideal well seal. Improvements to the well seal (i.e. deepening of the casing) would be consistent with a "multi barrier approach" to source water protection, and would improve the overall reliability of the water supply.

6) The long-term trend in groundwater levels in the Lansdowne municipal wells is uncertain due to potential instrumentation errors.

Based on the conclusions, the following recommendations are made:

1) To improve the overall reliability of the water supply, consideration should be made to improving the well seal at the municipal wells.

2) To assess long-term trends in groundwater level within the supply aquifer, regular water level monitoring should be carried out in the municipal wells and at other locations within the aquifer (i.e. existing or new monitoring wells). New monitoring wells are recommended in the vicinity of the municipal wells (i.e., north and west of the village). Additional groundwater level information in this area would improve the quality of any subsequent groundwater flow model calibration.

3) Discuss with MOE/MNR the possibility of completing the groundwater stress assessment on a "groundwater-based" catchment.

4) Complete depth discrete hydraulic testing within municipal well when there is a scheduled pump removal for maintenance. Although not required by the Technical Rules (MOE, 2009), additional hydraulic testing is recommended to better estimate the bedrock aquifer parameters and provide an opportunity to collect groundwater samples from isolated intervals in an attempt to better understand the source of the bacteriological contamination in the municipal well. The additional hydraulic data is not expected to alter the WHPA delineation but would provide an opportunity to fine tune the conceptual model and input parameters, and reduce uncertainty.

5) Although not required by the Technical Rules (MOE, 2009), collect more groundwater and surface water samples (including a sample of the "cascading" water in the municipal wells) to

allow a better understanding of the source of municipal groundwater (i.e. which aquifer) and also a better understanding of the bacteriological source.

9 CLOSURE

This report has been prepared for the exclusive use of the Cataraqui Region Conservation Authority (CRCA) using a methodology for conducting source water protection studies that is acceptable within the profession. Data obtained from sampling investigations represent the conditions at the time of sampling and are subject to variability with building activities and weather conditions.

Geofirma Engineering Ltd. (Geofirma) has exercised professional judgment in collecting and analyzing the information and in formulating recommendations based on the results of the study. The mandate at Geofirma is to perform the given tasks within guidelines prescribed by the client and with the quality and due diligence expected within the profession. No other warranty or representation expressed or implied, as to the accuracy of the information or recommendations is included or intended in this report.

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Respectfully submitted,

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