Tier 3 Water Budget and Local Area Risk Assessment for the Greensville Groundwater Municipal System

Updated Risk Assessment Report

Prepared for:

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Earthfx Incorporated

Earth Science Information Systems

July 3, 2017

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RE: Update to the Tier 3 Water Budget and Local Area Risk Assessment for the Greensville Groundwater Municipal Supply

Dear Mr. Strakowski:

We are pleased to provide our final report of the *Update to the Greensville Tier 3 Water Budget and Local Area Risk Assessment Report*. The report describes the new data made available since the original Greensville Tier 3 model was developed by Earthfx in 2014, changes to the conceptual model to incorporate new geologic and hydrogeologic information, and improvements made to the integrated numerical model developed to simulate the surface water and groundwater systems in the Middle Spencer Creek subwatershed. The report presents the results of the revised Risk Assessment analyses (Scenarios C, D, G, and H), updated water balances, and updated delineation of significant groundwater recharge areas using the updated model.

We trust this work report meets with your satisfaction and if you have any questions, please call. Thank you for the opportunity to be of service.

Yours truly,
Earthfx Incorporated

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J.D.C. KASSENAAR
PROVINCE OF ONTARIO

July 3, 2017
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Table of Contents

List of Tables

List of Figures

List of Appendices

1 Introduction

The Ontario government passed The Clean Water Act in October 2006 to protect drinking water at the source as part of an overall commitment to human health and the environment. Conservation Authorities, in partnership with local municipalities, have been charged with coordinating the Drinking Water Source Protection process, including the provision of technical expertise to determine the best ways to protect the quality and quantity of sources of drinking water within a watershed. This is considered to be the first stage in a multi-barrier approach to ensuring safe drinking water.

An Assessment Report was prepared on behalf of the Halton-Hamilton Source Protection Committees for the Hamilton Region Source Protection Area (Halton-Hamilton Source Protection Staff, 2012). An update of that report (Version 2.7) was released in July of 2015. A critical element of the Assessment Report was the identification of potential risks to municipal water supplies from both a water quantity and water quality perspective. A three-tiered approach was defined under the Clean Water Act for the purpose of screening and assessing the risks to water quantity and municipal water supplies. Tier 1 and Tier 2 screening-level water-budget studies were completed for the subwatersheds in the Hamilton Region Source Protection Area (HRSPA). The Tier 2 Water Budget and Water Quantity Stress Assessment found the Middle Spencer Creek subwatershed to be potentially stressed and it was assigned a moderate stress level. This finding required that a Tier 3 level analysis be conducted for the subwatershed to verify the levels of stress and to identify potential water quantity threats. In 2013, Conservation Halton retained Earthfx to complete the Tier 3 Assessment, which focused on the Greenville municipal well, referred to as FDG01, located in the southern part of the Middle Spencer Creek (location shown on Figure 1.1).

Since the completion of the previous Tier 3 study (Earthfx, 2015), the City of Hamilton has completed an exploration program to expand the current Greensville municipal supply system with the addition of a second supply well, referred to as TW-2-13. Figure 1.2 shows the location of the existing a new Greensville municipal supply well and the vicinity of the municipal wellfield. As the City of Hamilton is nearing the integration of the new supply well into the Greensville municipal system, the City and Source Protection agency initiated an update to the previous Tier 3 work to incorporate the new well, along with new site data that have recently been made available.

1.1 *Study Area*

The study area for the Greensville Tier 3 extended beyond the Middle Spencer Creek subwatershed boundaries and encompassed several neighbouring subwatersheds in Spencer and Grindstone Creek (Figure 1.3). The study area straddles the Niagara Escarpment, a major physiographic feature. Other significant local features, discussed further in this report, include a managed reservoir system (Christie Lake), several wetland complexes, and two large limestone quarries.

1.2 *2013-2015 Tier 3 Assessment*

The previous Tier 3 Water Budget and Local Area Risk Assessment study of the Greensville municipal water supply was initiated in 2013. That study is documented in two reports. The first report, Earthfx (2014), discusses the physical setting of the study area and the development of an integrated numerical model to represent the groundwater and surface water flow systems in the study area. The Tier 3 model development was a complex, data-intensive process. The second report, Earthfx (2015), presented the results of the Tier 3 Water Budget and Risk Assessment. A summary of both phases of the previous Tier 3 study is provided in the following sections.

1.2.1 Previous Tier 3 Data Analysis and Conceptual Models

The study began with a review of the physiography, topography, climate, hydrology, and geology of the study area. A three-dimensional conceptual stratigraphic model, referred to as the "Greensville Tier 3 Conceptual Stratigraphic Model", was developed to describe the overburden and bedrock units in the study area. The conceptual stratigraphic model was created by mapping the tops of each geologic unit and then overlaying them to create a fully three-dimensional representation. Data collected on the hydrogeology of the area was used to transform the conceptual stratigraphic model into the 11-layer hydrostratigraphic model, which represented the sequence of aquifers and aquitards underlying the study area. Hydraulic properties were assigned to the units represented by these surfaces.

Climate data, specifically precipitation, temperature, net solar irradiation, and the hourly rainfall intensity distribution, were compiled from multiple sources and reviewed in the previous Tier 3 study. These data were used to create climate datasets necessary for modelling analyses and included:

1. a model calibration dataset (Water Year ($_{WY}$) 2005-$_{WY}$2011);
2. a long-term (25-year) dataset for model verification; and
3. a processed 10-year time series for drought analyses.

Land use and land cover data were compiled from multiple sources and reviewed. Hydrologic properties (e.g., percent imperviousness and vegetative cover type and density) were assigned to land use categories for use in hydrologic modelling.

Surface water data, including stream network mapping (Figure 1.4), streamflow measurements, dam and reservoir operations, and information on the wetlands and wetland complexes within the study area, were collected and reviewed. Streamflow data were analyzed to estimate the component of groundwater discharge (baseflow). The stream mapping and channel geometry data were used as inputs to the hydrologic models developed for the previous Tier 3 study, while the observed streamflow measurements provided targets for model calibration.

There are two active quarries in the study region, the Dufferin Aggregates Flamboro Quarry and the Lafarge Dundas Quarry, also shown on Figure 1.2. The two operations are adjacent to each other and are located to the north and west of the Greensville municipal wellfield. Dewatering for the quarries has had a significant effect on local groundwater flow system, and discharge from the quarries affects streamflow in Logie's and Spencer creeks. The available data on local geologic conditions, quarry operations, and quarry water management were collected and incorporated in the hydrostratigraphic model and the subsequent Tier 3 modelling analyses.

A key task in the previous Tier 3 study was compiling and assessing surface and groundwater use. While municipal water use is a small component of the overall water demand, the municipal wellfield is in close proximity to large permitted water takers (the aforementioned quarries) and numerous distributed smaller water users. Water use data were compiled from Permit to Take Water (PTTW) information supplemented by self-reported data in the Water Taking Reporting System (WTRS) database, both maintained by the Ministry of Environment and Climate Change (MOECC). The Greensville well, FDG01, serves 36 homes (about 127 people) and is operated under PTTW 2476-9F5KM6. The well is allowed to extract a maximum of 197 cubic metres per day (m^3/d), although actual use is much less on an average daily basis. A maximum reported daily taking, 105 m^3/d, occurred in August 2011. Although the settlement area is expected to grow, the City of Hamilton does not plan to increase the number of connections to the municipal wellfield.

1.2.2 Previous Tier 3 Integrated Model Development

The data collection, site characterization and conceptual model development were used to construct the integrated groundwater/surface water model to represent the complex surface water and groundwater processes in the Tier 3 study area. The integrated model was then applied to assess the performance of the municipal well under a variety of scenarios that considered future increases in water use, land use change, and drought conditions as part of the previous Tier 3 study. The model was based on the U.S. Geological Survey (USGS) GSFLOW code (Markstrom *et al.*, 2008). GSFLOW is a fully-integrated model developed from two widely-recognized USGS submodels: the Precipitation Runoff Modelling System (PRMS) (Leavesly *et al.*, 1986) and the modular groundwater flow model MODFLOW-NWT (Niswonger et al., 2011). The PRMS submodel represents hydrologic processes at land surface (precipitation, interception, runoff, and snowpack dynamics) and the soil-water balance. The MODFLOW submodel accepts recharge from the PRMS submodel, simulates saturated groundwater flow, and computes discharge to streams, wetlands and back to the PRMS soil zone. Additional submodels in GSFLOW are used to simulate water balances in study area lakes, route overland runoff to streams, and route flow through the stream network.

GSFLOW uses a fully-distributed representation of the physical system where the study area is subdivided into small cells, each with unique physical (e.g., layer geometry) and hydrologic properties (e.g., hydraulic conductivity). Different grid cell resolutions were used for the climate, surface hydrology and subsurface groundwater processes based on the location of features of interest, the availability of data, and the level of detail needed. For example, a uniform grid was used for the PRMS submodel to provide an equal level of detail across the study area while the MODFLOW submodel grid was refined in the vicinity of the municipal wellfield to provide increased resolution in that area.

Building the Tier 3 integrated model started with the construction and pre-calibration of a stand-alone PRMS submodel. The submodel simulated distributed surface water processes, including:

1. input of daily spatially-distributed rainfall, snowfall, temperature, and solar radiation;
2. calculation of canopy interception and simulation of snow pack growth and melt processes;
3. calculation of infiltration and routing of runoff from pervious and impervious areas;
4. computation of soil moisture balance terms including interflow, evapotranspiration (ET), and groundwater recharge; and
5. calculation of discharge from the PRMS groundwater reservoir to streams (baseflow).

The PRMS pre-calibration produced a set of reasonable parameter values that were used in the GSFLOW model with only a minimal need for additional re-calibration effort. Adjustment of some PRMS parameters was needed in the final calibration to account for transfer of flows through feedback mechanisms not fully represented in PRMS-only simulations, such as groundwater discharge from the shallow water table to the soil zone.

Similarly, a pre-calibration of the groundwater submodel was undertaken to test the conceptual hydrostratigraphic model and to obtain reasonable values for key model parameters such as hydraulic conductivity of the aquifers and aquitards. These values did not change significantly once the submodel was integrated back into the integrated GSFLOW model. The calibration of the groundwater submodel was evaluated based on the match between simulated groundwater levels and static water level data from the MOECC Water Well Information System, and thereby the ability of the model to represent observed groundwater flow patterns.

Once the PRMS and MODFLOW submodels were reasonably well calibrated, the additional data sets and required changes to the model input were made to construct the integrated GSFLOW model. Model inputs were adjusted to represent the time-dependent operation of the Christie Lake reservoir, quarry water management, and daily water takings. The GSFLOW model was calibrated to the seven-year period from October 2005 to September 2011. The calibration period covers an extreme dry year ($_{WY}$2007) and a number of relatively wet years. This period also contains the largest amount of higher quality data for input and calibration purposes including continuous water level data, actual water takings, streamflows, and hourly climate data. Calibration was an iterative process in which results of successive model runs were used to improve the initial estimates of model parameters (in particular parameters related to aquifer storage). Qualitative checks on the calibration were done by visual comparison of hydrographs of simulated and observed flows at gauges and groundwater levels. Statistical measurements of calibration error were also examined to ensure the overall quality of the calibration.

1.2.3 Previous Tier 3 Risk Assessment Study

Earthfx (2015) documents the application of the integrated model to analyze the water budget of the study area and to conduct the formal risk assessment scenario analyses required as part of the previous Tier 3 project. This phase of the study began with a review of the existing and planned municipal supply system. As was noted, at the time of the previous Tier 3 study, there were no official plans to expand the capacity of the Greensville wellfield either through increased permit capacity or through construction of additional supply wells.

A detailed characterization of the existing municipal well was then conducted to identify any operating constraints relative to water levels in the wells. The average pumped water level for 2007 to 2012 was 229.8 metres above sea level (masl) and the available drawdown (the difference between the average pumped water level and the minimum allowable operational in-well water level) was determined to be 4.5 m. Non-municipal water demand in the study area was also quantified. Aside from the quarry permits, 20 groundwater permitted takings from 35 sources and 4 surface water permitted takings from 6 sources were identified from the 2012 permit database. Two additional water use permits within the study area belong to the Flamboro Quarry and the Lafarge Quarry. Reported takings for the municipal well FDG01 were provided by City of Hamilton staff. WTRS records were available for many of the groundwater and surface water permits and were used directly in the model to simulate daily takings. Where WTRS data were not available, daily takings were estimated from the maximum permitted rates. Consumptive use factors were applied to the takings to estimate the portion of water not returned to the source aquifer. To be conservative, the municipal takings were treated as 100% consumptive.

Projected land use changes were analyzed to quantify the rates of groundwater recharge and the sustainability of the municipal groundwater supply. Existing land use was mapped using SOLRIS (v1.2) data, along with additional data provided by Conservation Halton. Future land use was projected based on the 2012 Rural Hamilton Official Plan. According to the Official Plan, new residential developments to the west of the municipal wellfield will be predominantly single-family, detached dwellings. Minimal infilling of existing settled areas is expected and no increase in commercial areas is planned. Model parameters, such as percent impervious and vegetative type and cover densities were incorporated into the model for areas with future land use change.

The Tier 3 integrated model was run for the assessment period with existing pumping and land use conditions. Model output was used to develop a long-term average water budget for the study area under these conditions. Another simulation was done to establish baseline (pre-development conditions) in which all pumping was turned off. In this baseline scenario, the quarries were allowed to fill to their natural levels in this baseline assessment. Groundwater levels were compared between the baseline and existing conditions and used to define "drawdowns". The 1 m drawdown contour in the weathered bedrock aquifer encompassed an area extending approximately 60 m around the municipal well FDG01 (a total area of 0.01 km^2) and did not intersect the 1 m drawdown contour around the quarry. The 1 m drawdown in the semi-confined Gasport/Goat Island aquifer encompassed a larger area (34.2 km^2) which included the Greensville municipal well. This area was used to define the water quantity wellhead protection area (WHPA-Q1) for the municipal well. The effects of future changes in land use, including new private wells and expansion of the quarries, were simulated with the model and results were compared against existing conditions. Although simulated drawdowns of up to 1 m occurred within the areas of future rural residential development, the proposed land use changes did not have a measureable impact on the Greensville municipal well. The WHPA-Q2 area was therefore defined to be coincident with the WHPA-Q1 area.

Additional transient analyses were conducted to evaluate whether the municipal water supply well was able to pump at the existing pumping rates under drought conditions. Climate from a historic 10-year drought (the 10-year period from 1957 to 1966) was used in these analyses. The analysis was repeated under future land use and pumping conditions. Impacts of the municipal well on other water uses (permitted takings, private wells, cold water streams, and provincially-significant wetlands (PSW)) was also assessed using model results.

Results of the study demonstrated the overall resilience of the Greensville municipal well to water quantity threats related to future land use change, increased competition from additional private residential wells, and the eventual build-out of the quarry operations to the north. Under the transient 10-year drought conditions, the Greensville municipal well was able to meet the water demands on the system, both under existing conditions and with the consideration of future quarry, land use change, and additional private well takings. The resilience of the municipal well can be attributed to three key factors identified through the Tier 3 modelling study: (1) the presence of a groundwater divide between the Greensville well and the quarry operations; (2) the hydraulic separation of the shallow production aquifer from the deeper Goat Island/Gasport aquifers by shale confining beds of the Vinemount member in the vicinity of the well; and (3) the relatively small volumes associated with the municipal water takings. The simulations indicated that future quarry developments create a large drawdown in the aquifers beneath the shallow bedrock aquifer, but the intervening confining layers isolate the municipal well from these effects.

Overall, the previous Tier 3 study presented a unique challenge because of the need to consider both quarry engineering and wellfield-scale issues within a larger watershed framework. The integrated modelling approach proved essential to the representation of competing water use. The previous Tier 3 study significantly improved the understanding of risks to the existing Greensville municipal well, FDG01. The update to the Tier 3 represents an opportunity to re-apply the Tier 3 integrated model to develop the same level of understanding within the expanded Greensville municipal wellfield.

1.3 *Post Tier 3 Studies*

In 2013, Golder Associates Ltd. issued a Hydrogeology and Hydrology Technical Report for the proposed Lafarge Dundas South Quarry Extension (SQE). The report provided a considerable amount of new hydrogeologic data from the field investigation that included the completion of geologic boreholes, installation and monitoring of observation wells, and aquifer testing results. These data were not available at the time of the model construction for the previous Greensville Tier 3 study (Earthfx, 2014), and were, instead, reviewed during the subsequent Risk Assessment phase of the project (Earthfx 2015). In 2015 Earthfx was retained by the City of Hamilton to begin an assessment of the proposed SQE, and information and insight from that preliminary assessment have been incorporated into this study.

1.4 *Tier 3 Update - Project Objectives and Scope*

In 2013, the City of Hamilton initiated a hydrogeological investigation to construct and test a new water supply well to expand the existing Greensville municipal water supply system, and act as a backup well to the existing supply well FDG01 (Stantec, 2013). Three test wells (TW-1-13, TW-2-13, and TW-3-13) were drilled at a site located 150 m northeast of the existing well, at Johnson Tew Park, 20 Medwin Drive, Hamilton.

Preliminary testing results revealed that TW-3-13 did not have sufficient yield and was eliminated from consideration as a backup supply. Both TW-1-13 and TW-2-13 were initially assessed for development as a combined backup supply due to the encouraging well yield testing results from both wells. However, the City of Hamilton has elected to pursue TW-2-13 as the sole backup water supply well for the Greensville municipal system. Further well development and testing of TW-2-13 was conducted in by Lotimer & Associates Inc. in August of 2015 and April of 2016. Following the second round of well enhancement on TW-2-13, SNC Lavalin completed a step drawdown test on the well in June of 2016 to confirm its long-term yield.

The addition of the new backup well TW-2-13 for the Greensville municipal supply system necessitates an update of the Tier 3 analyses. The availability of new data collected in the wellfield vicinity along with the data from the Lafarge SQE provides an opportunity to revisit and update the geologic and hydrogeologic understanding of the Tier 3 study area. Accordingly, the key objectives and tasks of the present study were to:

- Assemble new wellfield and quarry data that were not available for the previous Greensville Tier 3 study, including all new well construction and testing details, water levels, and streamflow data. Enter the new well data into the project database. Analyze step test results and aquifer test results.
- Update the hydrogeologic representation in the vicinity of the wellfield by re-interpolating hydrostratigraphic surfaces and adjusting hydraulic properties, as needed, based on the new City of Hamilton wells and the 15 new SQE geologic boreholes;

- Updated the GSFLOW model and undertake a focused re-calibrate to new data including new groundwater level observations from the municipal wellfield and the SQE dataset;
- Locally refine the model grid in the vicinity of the Dundas Quarry to better represent the effects of quarry operations under current and future build-out conditions;
- Repeat the specific analyses required of a Local Area Water Quantity Risk Assessment study using the updated Tier 3 model. This includes the delineation of the local area, WHPA-Q1, and WHPA-Q2, completing Risk Assessment Scenarios C, D, G, and H, and evaluating the effects of municipal pumping on other water uses.
- Use the updated model results to evaluate tolerance levels, assign water quantity risk levels, and assess the effects of uncertainty on model results.
- Update the water budget for the Middle Spencer Creek subwatershed using the updated Tier 3 model, and undertake the delineation of significant groundwater recharge areas (SGRAs).

At the time this study was undertaken, the Municipal Environmental Assessment (MEA) process for the expanded Greensville municipal water supply system had yet to commence. While only new system that have successfully received MEA approval are typically included under the Tier 3 Risk Assessment framework, the City of Hamilton has made a significant commitment in both time and resources to developing the new water supply, including the completion of several hydrogeologic studies (Stantec, 2014; Lotimer, 2016a, 2016b; SNC Lavalin, 2017, that warrant its consideration in this study. Earthfx has worked closely with the City of Hamilton to ensure that the operational considerations represented in the Tier 3 Risk Assessment are consistent with those reflected in the City's upcoming MEA and PTTW submissions.

1.5 *Tier 3 Assessment Methodology and Project Scope*

The Water Budget and Water Quantity Risk Assessment Guide (referred to herein as the Water Budget Guide) (Ontario Ministry of Natural Resources and Ontario Ministry of the Environment, 2011) lists ten steps for completing a Tier 3 Water Budget and Local Area Risk Assessment. These are discussed below with specific reference to this update study and the organization of this report.

1) **Develop the Tier 3 water budget model:** The surface water and groundwater models should be based on conceptual models representing detailed conditions around the wells. The models should be calibrated to represent typical operating conditions under average climate conditions and drought. As noted earlier, an integrated groundwater/surface water model was developed for the study area. Model development and calibration is described in detail in Earthfx (2014), and specific discussion of the model updates are presented in Section 2 through 4.

2) **Characterize the municipal wells:** The Tier 3 assessment requires a detailed characterization of the municipal wells, specifically identifying the low-water operating constraints. Section 3 of this report provides figures and discussions regarding the expanded Greensville municipal wellfield. In particular, the construction and planned operations of new well TW-2-13 are presented.

3) **Estimate allocated plus planned quantity of water:** Municipal water takings, in terms of existing, committed, and planned pumping rates for municipal wells, should be quantified for the Tier 3 analysis. As noted above, no change is expected in overall water demand on the Greensville municipal supply system. However, the planned operation of the expanded municipal wellfield is expected to meet the demand by cycling between the two wells according to a 6-to-1 day rotation, as discussed in Section 3.2.

4) **Identify and characterize drinking water quantity threats:** Threats to drinking water quantity can include municipal and non-municipal consumptive water demands, as well as reductions to groundwater recharge. Section 3 of this report discusses updates to the estimated water takings in the study area based on new data from the MOECC PTTW and WTRS databases. Land use changes that can affect the rate of recharge are also discussed in Section 3.7.

5) **Characterize projected land use:** Changes in land use due to the expansion of urban or settlement areas can affect recharge rates by increasing imperviousness and changing vegetative cover. An evaluation of the potential impact of projected land use changes on water supply is included in the Tier 3 analysis. Projected land use change was determined in Earthfx (2015) by comparing Official Plans with existing land use. Projected change in recharge was determined through simulations with the GSFLOW model and incorporated reasonable assumptions relating to imperviousness, vegetative type and cover density for projected land use changes. However, no changes in the characterization of future land use were made as part of this Tier 3 update study; assumptions and model recharge inputs are consistent with those of the previous Earthfx (2015) study.

6) **Characterize other water uses:** Other water uses that might be affected by municipal pumping need to be identified as part of the Tier 3 analyses. In addition to non-municipal takings, these water uses include aquatic habitat, provincially significant wetlands, waste assimilation, and recreational water use. Other water uses are discussed in Section 3. Impacts to other uses were re-evaluated with the updated model and results are provided in Section 6.6.

7) **Delineate vulnerable areas:** A specific requirement of the Tier 3 assessment is that the model should represent the "local area" around the municipal wells. Vulnerable areas from a groundwater quantity perspective (i.e., the WHPA-Q1 and WHPA-Q2) were re-drawn using the updated integrated groundwater/surface water model. The WHPA-Q1 is delineated by computing the drawdown cone for the municipal wells with existing plus committed plus planned rates. The WHPA-Q2 expands this zone to include areas where recharge reductions result in a measurable impact to water levels at municipal wells, and the "local area" is identical to the WHPA-Q2. The methodology and results for delineating the local area, WHPA-Q1 and WHPA-Q2 for the Greensville municipal wellfield are provided in Section 6.2.

8) **Evaluate risk scenarios:** A set of risk assessment scenarios have been developed to evaluate the sustainability of the municipal water supply, taking into account the allocated quantity of water, average climate and drought conditions, and projected change in land use. The scenarios were re-evaluated for the Greensville municipal wellfield (including both FDG01 and TW-2-13) using the updated integrated Tier 3 model. The impacts to other water uses were also assessed using these risk assessment scenarios. Results of the analyses are described in Section 6.5.

9) **Assign risk level**: A risk ranking of low, moderate, or significant must be assigned to the well based on the results of the Tier 3 Risk Assessment. An uncertainty level (high/low) is also derived for each risk ranking. The risk levels are presented in Section 6.7.

10) **Identify drinking water quantity threats:** Drinking water quantity threats, such as consumptive uses or reductions in recharge, must be identified at the significant and moderate levels within the WHPA-Q1 and WHPA-Q2 areas. Water quantity threats are discussed in Section 6.8.

1.6 *Figures*

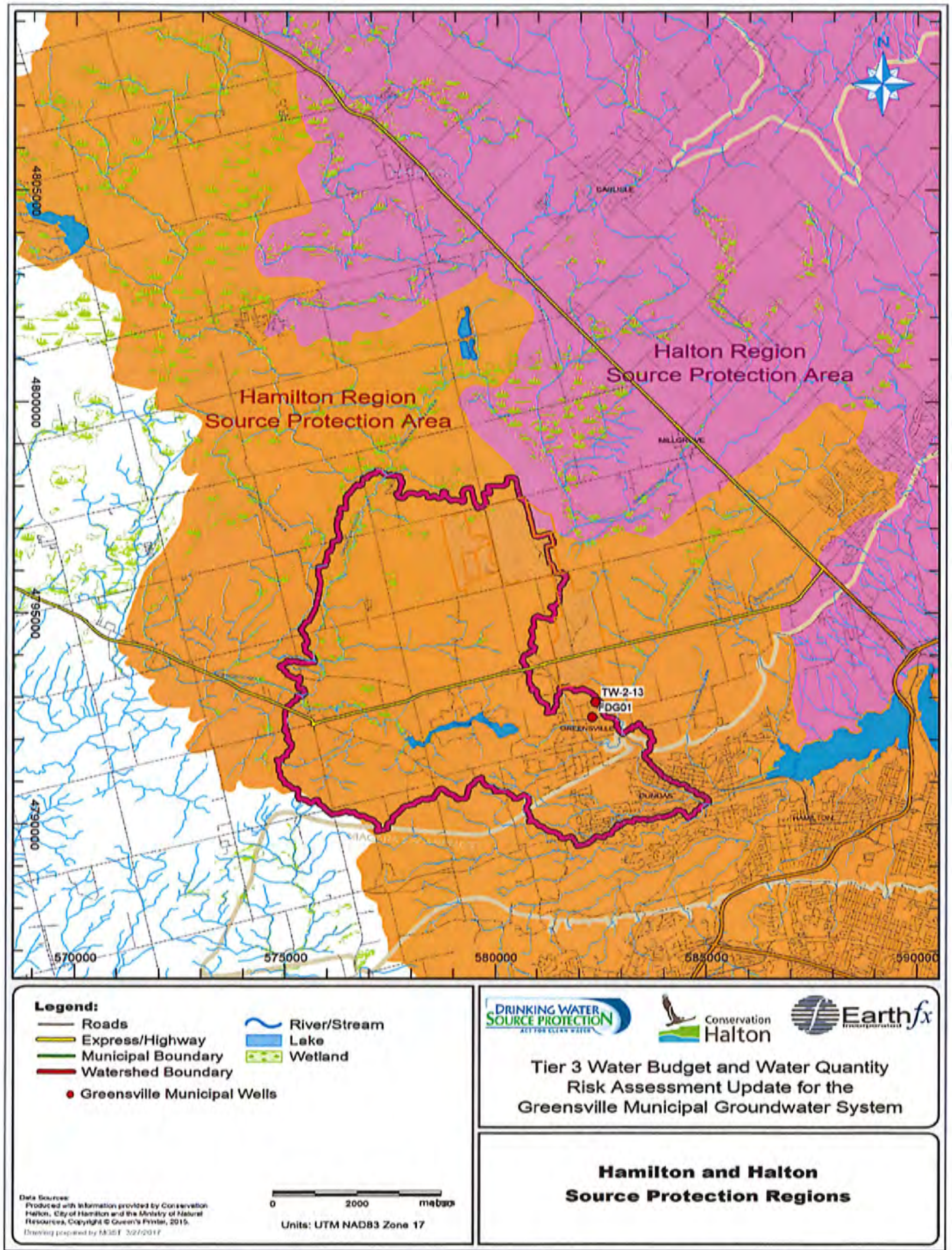


Figure 1.1: Halton and Hamilton Source Protection Areas and location of Greensville wellfield.

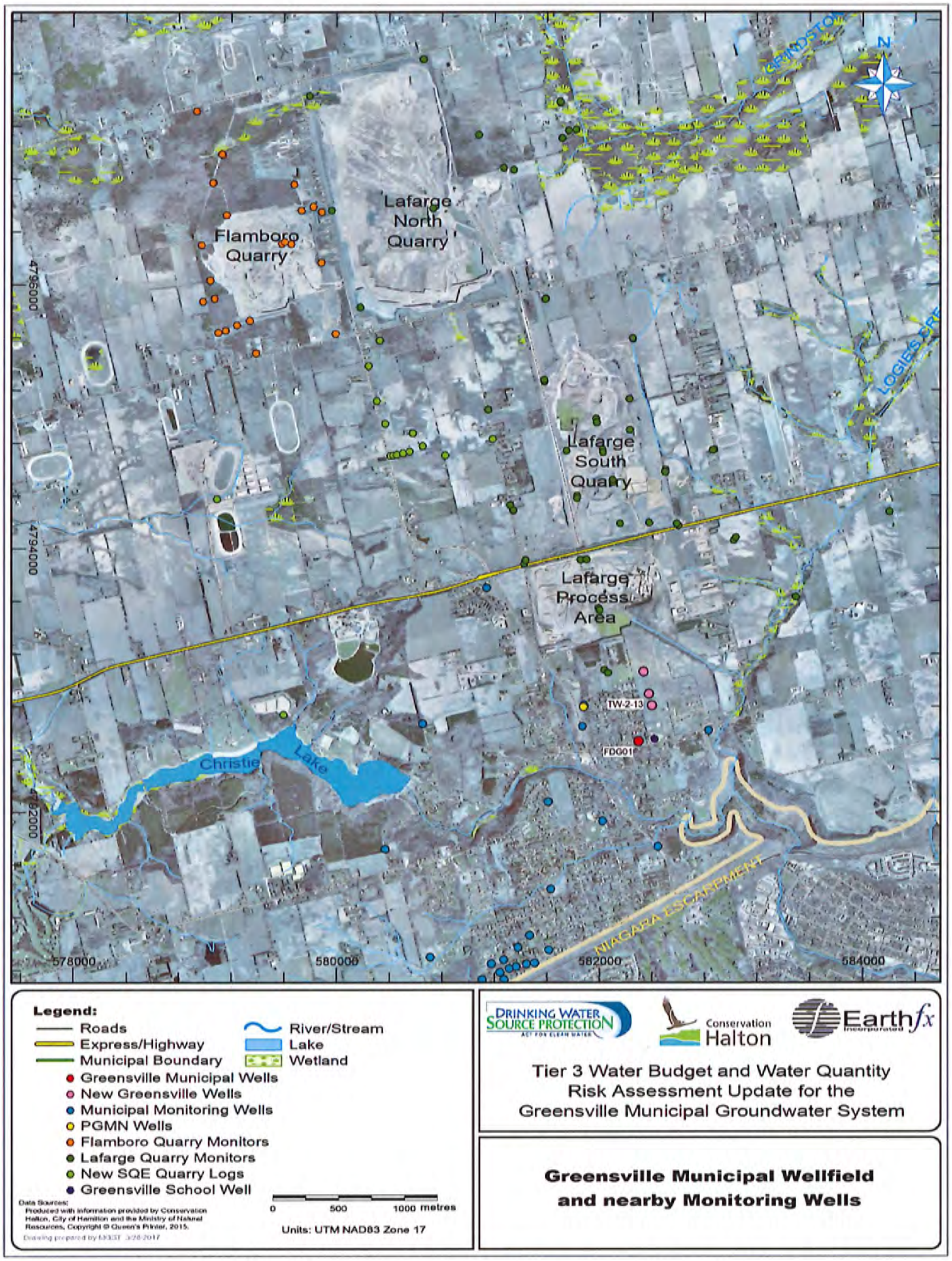


Figure 1.2: Location of the Greensville municipal wells, nearby monitoring wells, and test wells.

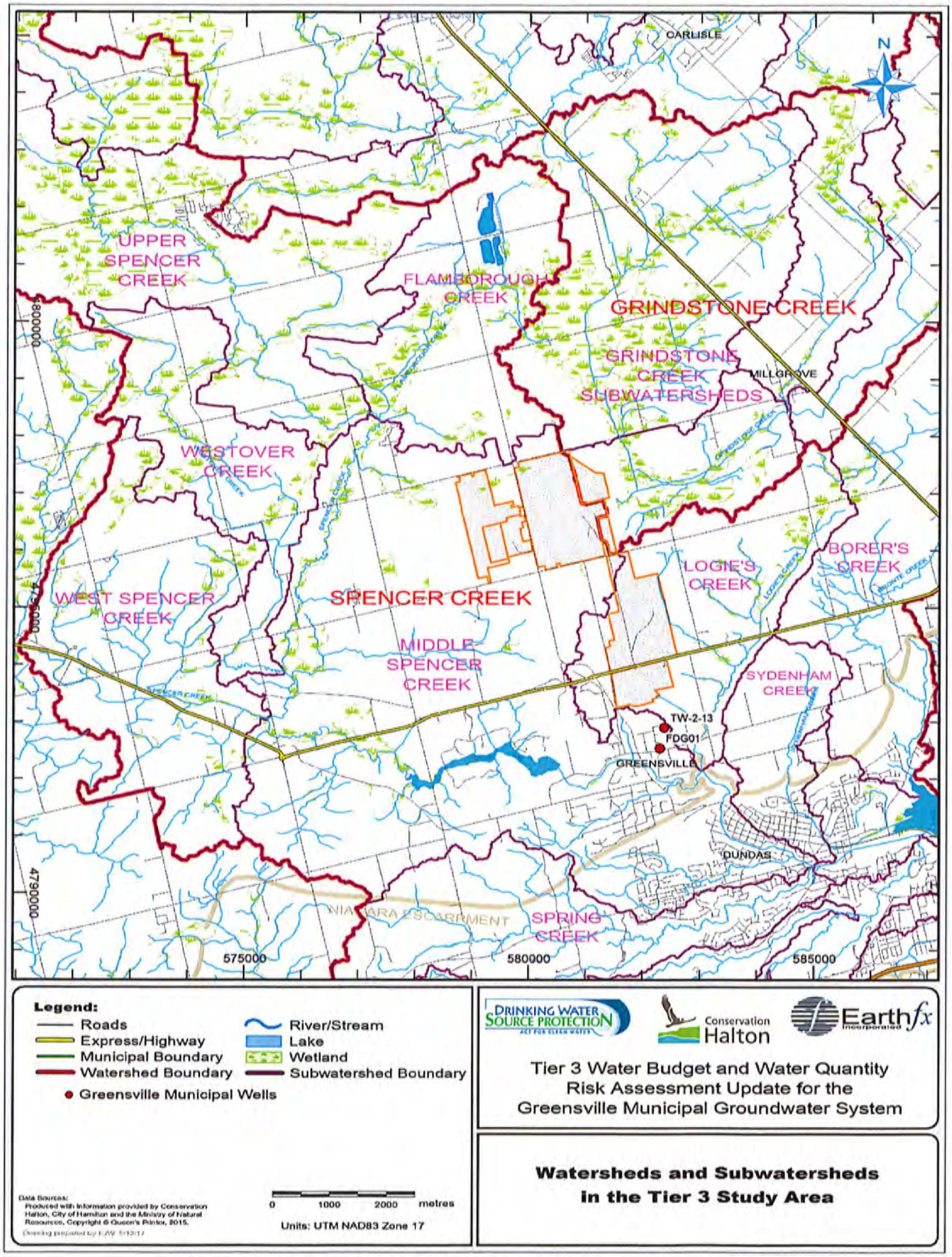


Figure 1.3: Major watersheds and subwatersheds in the study area.

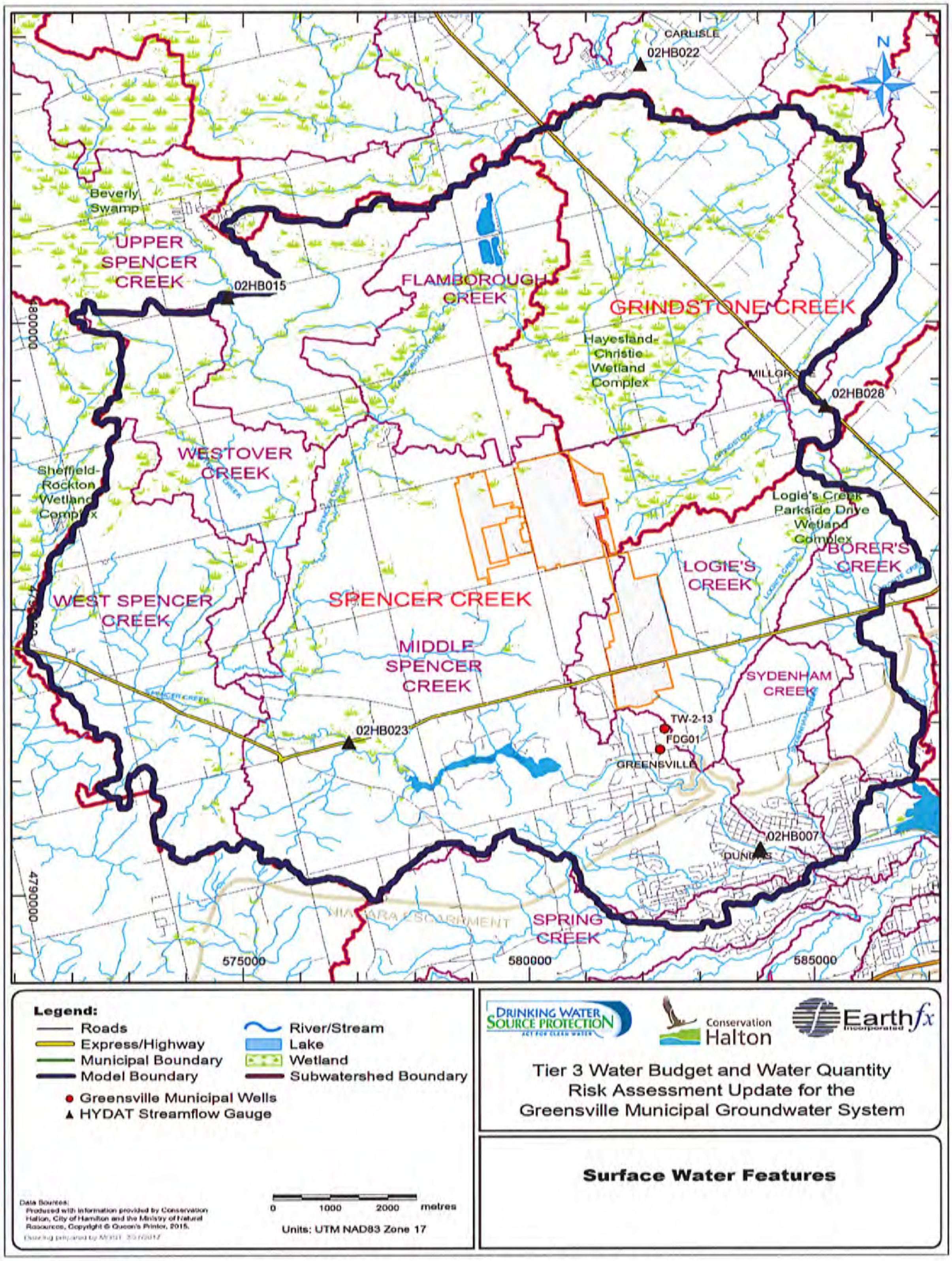


Figure 1.4: Surface water features in the study area.

2 Data Compilation from New Data Sources

Earthfx (2014) and Earthfx (2015) provide information on the hydrologic, geologic, and hydrogeologic settings of the study area and the conceptual geologic and hydrostratigraphic models developed. Figure 2.1 presents a schematic of the conceptual geologic model which is comprised of four Quaternary units overlying multiple layers of clastic and carbonate sedimentary rocks of Middle Silurian to Late Ordovician age. It is recommended that the reader refer to those reports as necessary for this background information. This section focusses on new data collected or made available since the previous Greensville Tier 3 model was developed.

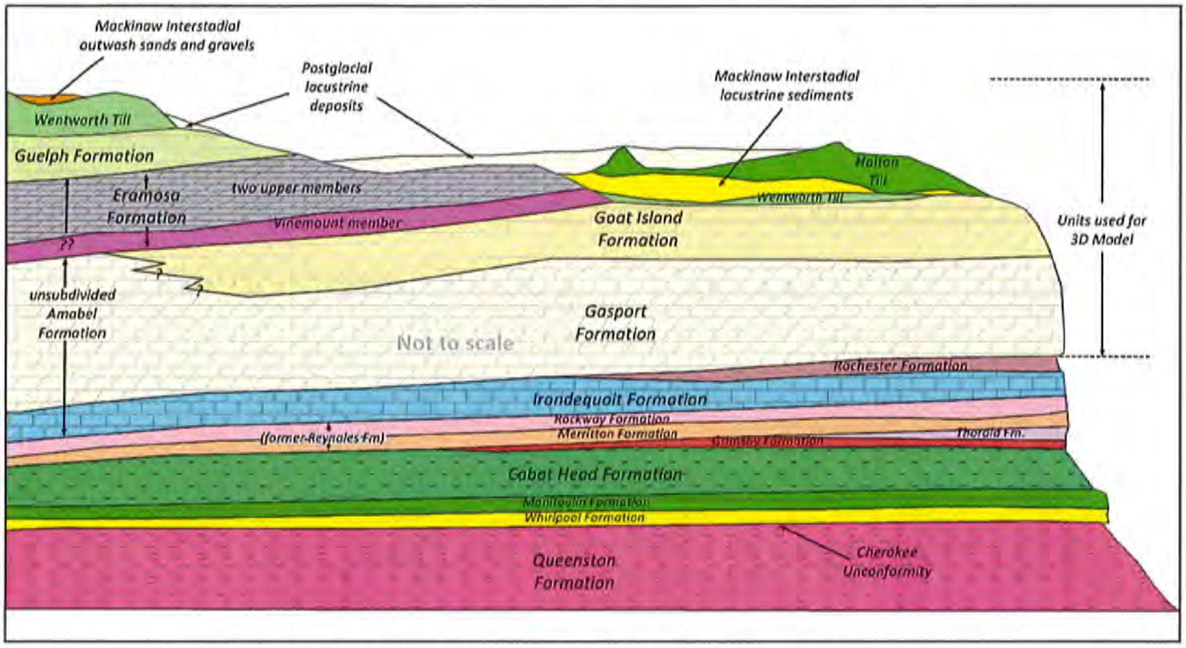


Figure 2.1: Schematic north-south stratigraphic cross section.

As part of this updated Tier 3 analysis, Earthfx reviewed the most recent versions of the MOECC PTTW and WTRS (2013 and 2014) databases to provide a more up-to-date representation of water use across the study area. The updated water demand estimates for municipal and non-municipal permits are presented in detail in Section 3.

2.1 *Hydrogeological Investigations for New Greensville Well TW-2-13*

The City of Hamilton retained Stantec Consulting Limited (Stantec) in 2013 to undertake construction and testing of a new water supply well to act as a backup well to the existing Greensville municipal well. A site located 150 m northeast of the existing well was identified for further exploration, culminating in the installation of three test wells (TW-1-13, TW-2-13, and TW-3-13) in February 2013 to depths ranging from 21.67 to 26.34 metres below ground surface (mbgs). The wells were drilled through overburden consisting of topsoil underlain by brown silty fine sand with trace to some gravel, with less silt at depth (Stantec, 2014). A gravel layer with traces of coarse sand was observed at TW-2-13 from depths of 6.1 m to 7.6 m below ground surface (BGS) and at TW3-13 from depths of 7.6 m to 10.7 m BGS. All three wells were cased into the underlying weathered bedrock contact aquifer. Well locations are shown on Figure 1.2. Geologic logs and geophysical logs for these holes were reviewed and the data were entered into the Greensville Tier 3 model database. Table 2.1 provides a summary of the well construction details for the new and existing municipal supply wells and the test holes from the recent exploration program.

Table 2.1: Summary of municipal supply well and test hole construction details.

Well Name	Easting	Northing	Construction Date	Depth (mbgs)	Ground Surface (masl)	Unit	Open Interval		Static Water Level (mbgs)
							Top (mbgs)	Bottom (mbgs)	
New Greensville Municipal well and test holes									
Greensville Well TW-2-13	582378	4792898	5-Mar-2013	21.3	243.8	Weathered Bedrock	12.5	21.3	10.8
TW-1-13	582335	4793065	5-Mar-2013	22.9	248.9	Weathered Bedrock	13.7	22.9	15.9
TW-3-13	582402	4792811	5-Mar-2013	26.2	245.2	Weathered Bedrock	14.6	26.2	13.4
Existing Greensville Municipal Well									
Greensville Well FDG01	582300	4792531	1-Jul-1972	12.2	234.8	Weathered Bedrock	10.7	12.2	4.6

Notes: Static water level measured at time of well construction

Preliminary testing showed that TW-1-13 and TW-2-13 had potential to serve as backup wells while TW-3-13 had insufficient capacity. A 72-hour aquifer test was conducted at the site in November 2013. Well TW-1-13 started pumping at 50 L/min but rates were reduced to 22 L/min during the test. Well TW-2-13 was able to maintain a constant rate of 44 L/min for the test duration (57 hrs). Stantec (2014) noted a near steady-state drawdown of 3.72 m in TW-2-13 after 56.5 hours of pumping. They estimated a transmissivity of 2.0×10^{-4} m^2/s.

Based on these results, the City of Hamilton elected to pursue TW-2-13 for development as a single municipal backup supply well for the Greensville system. Enhanced well development (chemical rehabilitation) was performed on the well in August of 2015 by Lotimer & Associates (2016). A down-hole video reconnaissance of the well during pumping revealed that the primary water bearing fracture was located between 1.6 m and 2.7 m below the top of bedrock. These observations seem to confirm the well is connected to the same weathered bedrock contact aquifer as the existing well FDG01. Results showed significant improvements, with an increase in well yield from 55 L/min to 90 L/min. Additional well enhancement was undertaken in April of 2016, resulting in further improvements to the well yield based on sustainable step test pumping rates as high as 120 L/min (Lotimer & Associates, 2016).

In 2016, the City of Hamilton retained SNC-Lavalin to determine whether TW-2-13 could serve as a single backup water supply well, and whether the well yield was sufficient to completely meet the system demands on its own. A borehole (BH-101) was drilled 5 m north of TW-2-13 and converted into a monitoring well (MW-101). A step drawdown test was conducted at TW-2-13 in June 2016, from which the estimated transmissivities of the weathered bedrock contact aquifer ranged from 6.3×10^{-3} m^2/s to 2.0×10^{-3} m^2/s. A second 72-hour test was conducted by SNC Lavalin in 2016 (SNC Lavalin, 2017) at a constant pumping rate of 90 L/min. Test results produced transmissivity values for the wellfield ranging from 1.3×10^{-3} to 3.1×10^{-3} m^2/s, with an average value of 2.2×10^{-3} m^2/s.

2.2 *Lafarge South Quarry Extension Data*

Golder Associates Limited has conducted ongoing surface water and groundwater field investigations at the Lafarge Dundas Quarry starting in 2006. New drilling was conducted in 2006 to support a proposed South Quarry Extension (SQE) and included completion of seven rotary core drilled boreholes at locations along the western and southern perimeters of the site. These include the MW06 series cores and core OW06-1. In addition, geophysical logs were obtained for eight air percussion drilled boreholes. Locations for the 15 boreholes are shown in Figure 2.3. Other data collection involved installation of nested groundwater monitoring wells, single-well and multi-well hydraulic testing, and surface water flow and stage monitoring.

2.2.1 SQE Hydrogeologic Data

Well logs and geophysical logs were reviewed and data were entered into the Greensville Tier 3 model database. These boreholes are located in an area with little previous information and provide improved control on the Paleozoic bedrock surfaces. Data from the 15 new geologic borehole records were used in conjunction with other geologic borehole data (Figure 2.4) to update the stratigraphic model layers.

Groundwater level data were available from 11 new monitoring wells nests, comprising a total of 67 intervals. These include monitoring well nests that were installed at the locations of the geologic boreholes and at additional offset holes completed after the primary cored boreholes (Figure 2.3). The four monitoring well nests MW25, MW50, MW100 and MW150 were installed near MW06-3C for use in an aquifer test. Nests ERI A and ERI D were installed as part of a karst investigation. Two potentially insightful offsite nests were completed at Christie Lake (CR1) and Flamboro Downs (FD1); however, for unknown reasons, these wells have only been briefly monitored. Most of the other new monitoring well installations were monitored on a monthly basis by Golder from 2007 to 2011.

The majority of the new monitoring wells were screened within the Guelph (21) and the Eramosa (20) Formations. Eleven monitors were screened across the formation contact. Other monitoring intervals were completed in the deeper aquifer system below the Vinemount aquitard. These include five monitors in the Goat Island, five in the Gasport, and four in the underlying Clinton-Cataract Group aquitard. Monitoring well MW06-4-D-1 was the only new overburden monitoring well.

Figure 2.5 presents an east-west geologic section through the proposed SQE showing the updated stratigraphic model layers. A corresponding north-south section is presented in Figure 2.6. Observed water levels and interpolated average heads within the different formations are also presented. The observed water levels show a downward vertical gradient with levels decreasing from the Guelph Formation to the underlying Eramosa Formation, and with a further decrease below the confining Vinemount aquitard in the Goat Island/Gasport aquifers. Water levels are generally lower in wells closer to the quarry face.

The water level data were interpreted by Earthfx to show evidence of a locally-enhanced hydraulic connection through the Vinemount and Lower Eramosa aquitards in the quarry vicinity. For example, groundwater levels at monitoring nest BH94-2 (Figure 2.7), located at the northwest corner of the North Quarry, show that excavation in the well vicinity in 2007 caused an increased connection between the Goat Island and Lower Eramosa through the confining Vinemount shale aquitard. Water levels and model simulations completed for the original Tier 3 study suggested that the Vinemount aquitard was locally-compromised within the quarry footprints based on observed water level patterns in the Goat Island and Gasport aquifers, and this has been carried forward into the present Tier 3 update.

Monitor information and water level data were reviewed and entered into the Greensville Tier 3 model database. The groundwater levels at the 67 new monitoring wells provided additional calibration targets for the Tier 3 model update, and are included in the discussion of updated model calibration in Section 4.4.

Results of hydraulic testing of the bedrock wells within the SQE were presented in Golder (2013). Data sources included (1) laboratory testing of samples from MW06-4D and MW06-5; (2) 79 packer tests completed in the cored boreholes (MW06-1, MW06-2, MW06-3, and MW06-4); (3) 44 rising head response tests conducted in new wells; and (4) a 72-hour aquifer test conducted at MW06-3C. Results of these tests are summarized in Table 2.2 along with data from earlier (1988 and 1990) aquifer testing. In general, conductive zones in the bedrock units are primarily related to open, near-horizontal fractures along the bedding partings and limited vertical fracturing. Where the beds are mostly intact, they act as vertical barriers to flow between the horizontal fractures.

Hydraulic testing of the Guelph and Upper Eramosa units indicated medium to high hydraulic conductivities for these units, with hydraulic conductivity values generally decreasing with depth approaching the Lower Eramosa member. The shales of the Vinemount member are considered to have very low hydraulic conductivities; however, it was noted that some packer tests straddling this unit had higher than anticipated conductivity values, which have been attributed to fractures at the contact with the underlying and overlying dolostone units.

The aquifer testing results detailed in the Golder (2013) study were used to guide the re-calibration of the updated Tier 3 model. Model values, presented in Table 2.2, compare well with the field values of hydraulic conductivity for the SQE and fall within the ranges presented in the table. In the case of the deeper Goat Island and Gasport aquifer units, hydraulic conductivity values in the model trend toward the upper range for these units and represent the presence of conductive fracture zones that may not have been encountered in the field tests. The selection of these values through the model calibration process is discussed further on.

Table 2.2: Summary of hydraulic conductivity values measured at Lafarge North and South Quarries and the South Quarry Extension.

Formation	North and South Quarry Data (1988, 1990)		SQE Application Data (2013)		Updated Tier 3 Model Hydraulic Conductivity (m/s)
	Arithmetic Mean (m/s)	Geometric Mean (m/s)	Arithmetic Mean (m/s)	Geometric Mean (m/s)	
Guelph	2.8×10^{-6}	6.3×10^{-7}	1.0×10^{-5}	1.1×10^{-6}	8.0×10^{-6}
Upper Eramosa	1.0×10^{-5}	2.4×10^{-6}	1.7×10^{-5}	1.9×10^{-6}	4.5×10^{-5}
Lower Eramosa	2.4×10^{-6}	4.8×10^{-7}	2.0×10^{-6}	3.6×10^{-7}	8.0×10^{-8}
Vinemount	4.4×10^{-6}	1.8×10^{-6}	3.6×10^{-6}	9.2×10^{-7}	4.0×10^{-8}
Goat Island	2.7×10^{-6}	8.1×10^{-7}	1.8×10^{-6}	3.3×10^{-7}	5.0×10^{-6}
Gasport	2.0×10^{-6}	6.7×10^{-8}	1.2×10^{-6}	4.8×10^{-8}	3.0×10^{-6}
Clinton-Cataract Group	5.0×10^{-6}	2.0×10^{-6}	1.9×10^{-6}	4.8×10^{-8}	5.0×10^{-8}

2.2.2 SQE Hydrologic Data

Surface water flow and level monitoring was conducted at eight locations along Spencer Creek, Logie's Creek and Grindstone Creek as part of the Golder (2013) study. Locations of the surface water monitoring stations are shown in Figure 2.8 and a summary of recorded flows is provided in Table 2.3. The data provided in Table 2.3 were measured using a combination of monthly instantaneous measurements (spot flows) and continuous estimates based on water level transducers and developed stage-discharge relationships.

Table 2.3: Summary of surface water monitoring stations from Golder (2013).

Station	Northing (m)	Easting (m)	No. of Measure-ments	Minimum Flow (m^3/s)	Maximum Flow (m^3/s)
SW1B	4794191	579282	36	0.000	0.235
SW2	4794176	577760	46	0.000	0.500
SW3	4792802	579263	45	0.003	0.081
SW4	4792704	579688	43	0.003	0.158
SW5	4792595	579880	45	0.000	0.087
SW6	4792962	583081	49	0.003	0.989
SW7	4797967	581598	51	0.000	2.396
SW8	4798431	584479	42	0.000	2.203

Note: Northing and easting (NAD83) for surface water monitoring locations are approximate.

It should be noted that neither transducer data nor derived continuous flow datasets were made available for use in the Tier 3 update study. Only the instantaneous spot flow data were used for comparison against the predicted streamflows from the updated Tier 3 model. These provided a largely qualitative assessment of the model outputs and are considered to be of limited use for validating or calibrating the Tier 3 model.

Instead, high-quality surface water flow targets, used in developing the original model, were used to provide a validation for the updated model. This includes Water Survey of Canada (WSC) streamflow data from stations 02HB023 (Spencer Creek at Highway 5), and 02HB007 (Spencer Creek at Dundas), along with reported discharges from the Lafarge North and South quarry sumps, Lafarge Processing Pond (via Railway Cut), and the Flamboro Quarry sumps.

2.3 *Update to Model Climate Inputs*

Climate inputs for the integrated model were updated as part of this study so as to extend the simulation period through $_{WY}$2016. Multiple climate datasets were created as part of the original Tier 3 study. The primary calibration dataset spanned $_{WY}$2005-$_{WY}$2011 and was derived from hourly NEXRAD Doppler radar data obtained from the US National Weather Service installation in Buffalo, NY. In addition, the original Tier 3 drought simulations were conducted with data from the MNR Infilled Climate Database for a single station (Hamilton RBG). The NEXRAD dataset was favoured for calibration as this product better captures the spatial distribution of precipitation over the study area. However, this dataset has several drawbacks; the dataset is large and requires post-processing for use as model inputs, daily rainfall volumes must be corrected against ground based precipitation stations (requiring time-consuming processing), and NEXRAD stations do not detect precipitation in the form of snow, therefore the record must be supplemented with snowfall data from other sources. Additionally, as the useful period of record only dates back to 2000 (Earthfx, 2014), long-term and historical drought simulations required the use of a different (and possible inconsistent dataset).

For this Tier 3 update, a single climate dataset spanning $_{WY}$1947 through $_{WY}$2016 was created. This dataset is based on ground stations only and provides a single, consistent climate product for use in the model. As this dataset does not require the heavy pre-processing and snow corrections of NEXRAD data, model inputs can be generated with less effort in the future. As discussed below, this dataset was generated in a manner consistent with the MNR Infilled Climate Database with exception that all daily volumes are interpolated from stations proximal to the study area, and includes stations operated by the City of Hamilton, Hamilton Conservation Authority (HCA), and McMaster University. Hourly intensities are synthetically generated based on hourly precipitation data collected at Hamilton Airport and processed with the method developed by Schroeter *et al.* (2000) discussed below.

2.3.1 Station Selection

Climate data for this Tier 3 update were obtained from the Meteorological Service of Canada (a division of Environment and Climate Change Canada), the City of Hamilton, HCA, and the MESONET network operated by McMaster University. There are several climate stations sited around the study area (Figure 2.9). Data from stations within 20 km of the model boundary were obtained from October 1946 through September 2016 to develop a representative group of stations with which to characterize the subwatershed climate. Locations of the 89 stations used in creating the time-series data are shown by operator in Figure 2.9 and operational status in Figure 2.10 Station information for the selected stations is presented in Table 2.4. The climate stations employed in this update are coincident with the stations employed in the original Tier 3. Earthfx (2014) evaluated the climatic variability within the station pool and found the density sufficient to represent the climate within the study area.

The available period of record for the selected stations is illustrated on Figure 2.11. Over 403,586 daily precipitation observations are available within the station data for $_{WY}$1947 through $_{WY}$2016 inclusive. Figure 2.12 presents the number of stations within the analysis group with complete monthly precipitation records post-1945 as determined by the WMO standard "3 and 5" rule which excludes months with more than 3 consecutive days of missing data or more than 5 days total with missing data (WMO, 1989). The available record within the station group varies significantly over the post-war period. The number of active stations increased through the 1950s and 60s and peaked in the mid-1980s. Due to budgetary constraints, many climate stations have been discontinued in the past decade, with less than 4 stations in the study area providing complete annual precipitation records from 2014 onward. Seasonal monitoring by local authorities significantly increases the available summer precipitation data available in the greater Hamilton area.

(Note: Data used in this analysis were not corrected for differences in synoptic measurement intervals. The climatological day used to derive daily minimum and maximum temperatures and precipitation totals has varied historically. Since July 1st, 1961 principal stations have treated 0600Z the following day as the end of the observation period. At ordinary stations, 0800Z is typically reported; however, 0000Z, 1230Z and 1700Z have also been historically employed. Conclusions related to the long-term climate trend analysis which includes periods pre-1961 may be affected by these differences in daily observation periods.)

2.3.2 Interpolated Gridded Climate Datasets

Daily climate data at the 89 selected climate stations were interpolated to a 500 m by 500 m grid using an inverse-distance-squared weighting technique (Figure 2.2). Inverse-distance-weighting is a computationally efficient way of interpolating the spatial data which assumes the correlation between the data at the nearby stations drops off rapidly with distance. The study area sees both frontal and convective storms in the summer months; frontal systems are a result of air masses with different temperatures and air pressures colliding whereas convective storms are generated by heat and available moisture and generally have a smaller footprint. Small-scale (2–20 km) circulations termed 'severe deep moist convective storms' can dump several months' worth of precipitation on a relatively small area in a matter of hours. A frontal storm would be captured by all the study area climate stations, while a convective storm may not be. For this reason it is preferable to capture data from as many climate stations in the study area as possible. Inverse-distance-weighting preserves more spatial information than some other interpolation schemes such as Thiessen polygons and is therefore preferred for spatially distributed models. Given the dense distribution of climate stations in the study area, and the small scale storms that can control the hydrologic response during the summer months, this interpolation technique was used to derived the basin-averaged normal and ultimately the integrated model inputs.

a)

$$P_{cell} = \frac{\sum \frac{P_{gauge}}{dist^2}}{\sum \frac{1}{dist^2}}$$

$$P_{cell} = \frac{\frac{34}{1^2} + \frac{33}{2^2} + \frac{27}{2.5^2} + \frac{30}{3^2} + \frac{22}{4^2}}{1 + \frac{1}{2^2} + \frac{1}{2.5^2} + \frac{1}{3^2} + \frac{1}{4^2}}$$

$$P_{cell} = 32.381$$

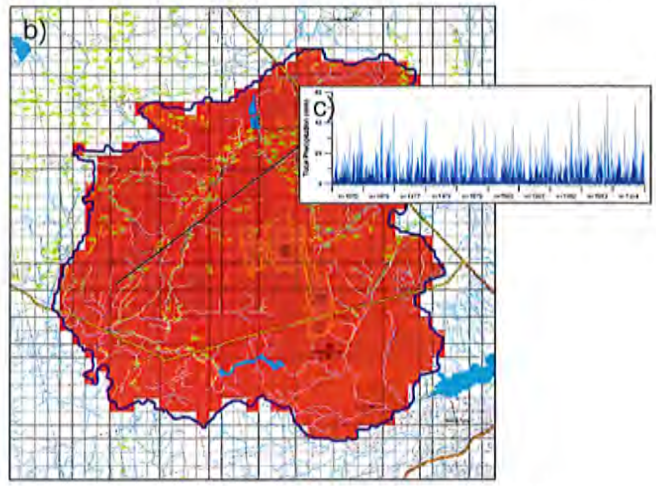


Figure 2.2: a) Inverse distance squared formula and example; b) meteorological interpolation grid with climate stations and subwatershed cells for averaging, c) typical interpolated precipitation hydrograph at a grid point.

A complete daily precipitation (total, rain, and snow) record was generated for the period spanning $_{WY}$1947 through $_{WY}$2016 on a gridded basis with the inverse-distance-squared weighting approach (one complete interpolated 500 m by 500 m grid array for each day). Similarly, a complete daily temperature (minimum, mean, and maximum) record was gridded for the same period spanning.

To assess spatial patterns, average annual precipitation for the period of record was calculated from the daily grids. Average annual precipitation varies from a high of 830 mm to west of the study area to a low of 800 mm in the Dundas Valley (Figure 2.14). Annually averaged daily temperature (Figure 2.15) demonstrates an inverse relationship with elevation, with a 0.6°C difference observed across the watershed roughly correlating to topography.

The daily gridded climate data was averaged over the Tier 3 study area (Figure 2.2b) to generate a daily time series of basin-averaged precipitation (Figure 2.2c) and temperature annual. The following analysis of climate normals and trends was undertaken with this interpolated, basin-averaged time series. Figure 2.16 presents the annual average precipitation observed over the study watershed for a 70-year period showing long-term trends and the number of stations sourced for the interpolation. Estimated average annual precipitation between $_{WY}$1947 and $_{WY}$2016 was 860 mm/year over the study area. Periods of precipitation drought in the observed record occur in the late-1950s to mid-1960s, and the late 1990s with recent extremes observed in $_{WY}$2012 and $_{WY}$2016 nearing historic lows.

Figure 2.17 presents the basin-average annual temperature observed in the study watershed between $_{WY}$1947 and $_{WY}$2016. Figure 2.18 overlays the annual mean temperature with the annual precipitation totals. Some years of reduced precipitation correspond to years with a higher than normal mean temperature, notably $_{WY}$1998, $_{WY}$2012, and $_{WY}$2016. Figure 2.19 presents a breakdown of annual precipitation volumes by rain and snow. Some drought periods clearly correspond to phases of reduced rainfall (e.g. the late 1990s), while some extreme years correspond to average rainfall with lower than average snowfall, such as $_{WY}$2012.

2.3.3 Synthetic Hourly Rainfall Intensities

The Source Water Protection Program Technical Rules (OMOE, 2011) indicate that infiltration is to be simulated on an hourly time step. Accordingly, the Green and Ampt method is applied in this study on an hourly basis. Daily rainfall volumes were applied as determined by the gridded inverse-distance weighting technique discussed above. Hourly intensity curves were synthesized after the method outlined by Schroeter *et al.* (2000) (discussed further in AquaResource (2008)). Hourly precipitation data from Hamilton RBG (Figure 2.20), Hamilton RCG CS (Figure 2.21), and Hamilton A (Figure 2.22) were obtained from Environment Canada for this task. Hourly data from Hamilton A were used to derive the synthetic hourly intensities as this station has the most complete hourly record. This station is also situated above the escarpment and should better represent the storm patterns observed in the Tier 3 study area.

While it would be preferable to apply *observed* (not synthetically infilled) hourly precipitation measurements directly into the model, the density of directly measured hourly stations is significantly less than primary and ordinary climate stations with daily data. There is a high degree of monthly variation during the summer months (Earthfx, 2014) across the study area and increasing the number of available stations improves the spatial quality of the precipitation inputs. The use of significantly more climate stations with daily data may be a preferable modelling strategy over using fewer stations with hourly data from disparate sources. Past projects have demonstrated that synthetic hourly rainfall intensities produce similar runoff volumes and timing as using observed data (Schroeter *et al.*, 2001).

2.3.4 Solar Radiation

Solar radiation observations serve as one of the primary drivers of the evapotranspiration module within the hydrologic submodel. Incoming solar radiation is controlled primarily by the number of possible hours of sunshine per day and the percent cloud cover. Solar radiation data are collected at few stations in Ontario; therefore, data had to be compiled from a variety of sources (Table 2.5). Through linear regression analysis, it was shown (Earthfx, 2010) that the southern Ontario solar radiation stations exhibited good inter-station correlation. The stations with available data are within 100 km of each other; Sucking and Hay (1976) suggest that stations within 250 km should demonstrate good correlation. Accordingly, a continuous dataset for 1956 through 2015 was created by averaging and infilling daily solar radiation information from nine southern Ontario stations. Data provided in sub-daily increments were summed to daily energy gains and converted to langleys per day (one ly/d = 1 cal/cm^2/day or 41.84 kJ/m^2-day), the input units required by the hydrologic model.

The incoming solar radiation dataset was based primarily on the average of measurements from four climate stations maintained by EC between 1956 and 2005. These stations include: 611KBE0 (Egbert CARE); 6142285 (Elora Research Station); 6158350 (Toronto); and 6158740 (Toronto MET Research Station). Unfortunately, the period of record of these four sites does not extend beyond August 31, 2003; therefore the remaining data up to 2015 had to be infilled using measurements from the University of Waterloo, York University, and the University of Toronto Mississauga campus. The properties of the climate stations used to create the composite solar radiation dataset are summarized in Table 2.5. A portion of the available incoming solar radiation record is provided on Figure 2.23. A histogram of monthly observed solar radiation is provided on Figure 2.24.

2.3.4.1 *Infilling Solar Radiation Dataset*

Where direct observations were unavailable, solar radiation was estimated by the Hargreaves and Samani (1982) method which uses daily minimum and maximum temperatures to correct incidental extraterrestrial radiation to match observed local conditions. A complete daily temperature record was created for the watershed for the period spanning $_{WY}$1947 through $_{WY}$2016 discussed above. This dataset was used to generate a complete solar radiation time series. The constants in the Hargreaves and Samani method were calibrated against the observed solar radiation time series (Figure 2.25). A correction factor (KT) of 0.151 was found to best fit the data; this compares well to the value of 0.162 recommended for "interior" regions. Adequate correlation was found between the calculated and observed solar radiation values on a daily basis (r^2 = 0.73) as shown on Figure 2.26a. Although more scatter is present in the daily dataset calculated with the Hargreaves and Samani method, an excellent match was achieved (r^2 = 0.96) on a monthly basis (Figure 2.26b). The estimated data, while not as good as actual field observations, are sufficient for the objectives of this study.

2.3.5 Climate Inputs Summary

Data from a 70-year period spanning $_{WY}$1947 through $_{WY}$2016 were employed to generate inputs for the Tier 3 model. Not all years of data are required for the current Tier 3 risk assessment update, but the complete climate inputs are available for use in future studies. Climate inputs required for the model include daily precipitation, maximum and minimum air temperature, hourly rainfall intensity, and solar radiation. The daily precipitation and temperature data from multiple stations were interpolated over the study area using an inverse-distance weighting method. Hourly intensities were derived from long-term hourly precipitation record processed with the method developed by Schroeter *et al.* (2000). The solar radiation dataset was updated with new data where available; gaps were infilled with the Hargreaves and Samani (1982) method.

2.4 *Tables and Figures*

Table 2.4: Climate stations proximal to the study watershed.

Reference ID	Name	Climate ID	Station Operator	Easting (m)	Northing (m)	Start Date	End Date	Water Years with Data	Average Number of Days with Data per Water Year
1	5th Line	–	MESONET	598,342	4,818,356	Apr 30 2007	May 02 2011	5	123
2	Christie Dam	–	HCA	580,486	4,792,149	Apr 30 2002	Sep 29 2012	11	153
3	Orchard	–	MESONET	581,881	4,788,143	Sep 11 2007	Sep 29 2012	6	131
4	Governor Rd.	–	MESONET	581,599	4,789,245	Sep 09 2007	Sep 29 2012	6	131
5	Greensville Monitoring - Site 1	–	CoH	580,049	4,795,648	Jun 02 2010	Sep 30 2011	2	91
6	Greensville Monitoring - Site 2	–	CoH	576,803	4,798,283	May 28 2010	Sep 30 2011	2	109
7	HRCA Weather St	–	MESONET	588,654	4,808,841	Apr 30 2007	May 02 2011	5	123
8	Hwy 5 near Westover	–	HCA	576,846	4,792,666	Apr 30 2009	Sep 29 2012	4	152
9	Kelso Weather St	–	MESONET	586,316	4,818,708	Apr 30 2007	May 02 2011	5	123
10	Mohawk College	–	HCA	590,544	4,787,776	May 23 2009	Sep 29 2012	4	147
11	Red Hill at Barton	–	HCA	599,539	4,788,270	Jun 10 2009	Sep 29 2012	4	132
12	Regional Rd. 25	–	MESONET	584,680	4,824,279	Apr 30 2007	May 02 2011	5	123
13	Stoney Crk at Jones	–	HCA	606,409	4,786,408	Apr 30 2003	Sep 29 2012	10	138
14	Stoney Crk at Queenston	–	HCA	601,417	4,786,616	Sep 25 2007	Sep 29 2012	6	128
15	Valens Dam	–	HCA	570,198	4,803,748	Sep 25 2008	Sep 29 2012	5	122
16	Westover at Safari	–	HCA	574,734	4,800,446	Jun 01 2001	Sep 29 2012	12	150
17	Workshop at Mineral Springs	–	HCA	580,548	4,787,335	Jul 04 2007	Sep 29 2012	6	142
18	ALBERTON	6150060	ECCC	577,203	4,781,612	Jun 01 1994	Dec 27 2008	16	327
19	ALDERSHOT	6150135	ECCC	591,901	4,796,605	Feb 17 1947	Feb 28 1977	31	352
20	APPS MILL	6140286	ECCC	550,155	4,775,806	Oct 01 1972	Nov 23 1972	1	54
21	BINBROOK	6130740	ECCC	594,913	4,774,431	Feb 09 1967	Mar 31 1970	4	191
22	BLUE CORWHIN	6140Q17	ECCC	571,373	4,820,422	May 14 1974	Mar 31 1976	3	229
23	BRANTFORD	6140941	ECCC	559,643	4,775,882	Jan 01 1945	Jun 30 1963	19	326
24	BRANTFORD AIRPORT	6140942	ECCC	554,376	4,776,301	Dec 12 2014	*Realtime*	3	210
25	BRANTFORD BRANT PARK	6140948	ECCC	556,917	4,777,710	Nov 25 1972	Nov 30 1973	2	186
26	BRANTFORD MOE	6140954	ECCC	562,355	4,775,907	Jun 01 1960	Jan 20 2013	54	339
27	BRANTFORD MORELL	6140951	ECCC	558,272	4,777,722	May 01 1959	Oct 31 1964	7	287
28	BURLINGTON	6151053	ECCC	597,253	4,800,382	Apr 16 1947	Apr 24 1974	28	331

Reference ID	Name	Climate ID	Station Operator	Easting (m)	Northing (m)	Start Date	End Date	Water Years with Data	Average Number of Days with Data per Water Year
29	BURLINGTON DRURY LANE	6151055	ECCC	597,280	4,798,531	Oct 01 1961	Aug 31 1970	9	321
30	BURLINGTON ELIZABETH GDN	6151057	ECCC	602,628	4,802,313	Dec 01 1961	May 31 1977	15	293
31	BURLINGTON FIRE HQ'S	6151059	ECCC	595,903	4,800,363	May 01 1970	Jan 31 1983	14	307
32	BURLINGTON PIERS (AUT)	6151061	ECCC	597,333	4,794,829	Dec 03 1992	*Realtime*	25	327
33	BURLINGTON SKYWAY	6151063	ECCC	597,307	4,796,680	Apr 01 1968	Jun 30 1970	3	203
34	BURLINGTON TS	6151064	ECCC	594,578	4,798,493	Apr 01 1951	Sep 30 1999	49	350
35	CALEDONIA	6131081	ECCC	585,468	4,770,604	Jan 01 1945	Nov 30 1966	23	346
36	CAMBRIDGE GALT MOE	6141095	ECCC	555,395	4,798,059	Mar 01 1947	Feb 28 1994	47	318
37	CAMBRIDGE-STEWART	6141100	ECCC	556,731	4,799,921	Sep 01 1973	Dec 31 2000	29	344
38	CAMPBELLVILLE	6151141	ECCC	586,282	4,813,192	Jun 01 1967	May 31 1968	2	183
39	CHRISTIE CONSERVATION	6151512	ECCC	579,781	4,792,749	Jun 23 1976	Dec 31 1994	20	336
40	CLARKSON	6151548	ECCC	611,804	4,819,122	Dec 01 1949	Sep 30 1967	18	355
41	COPETOWN	6151866	ECCC	571,707	4,788,956	Oct 01 1970	Jun 30 1993	23	361
42	DRUMQUIN	6152114	ECCC	599,626	4,822,633	Sep 01 1978	Jun 30 1980	3	223
43	ERINDALE	6152335	ECCC	609,020	4,824,631	Apr 04 1959	Dec 31 1978	21	343
44	GALT OSMOND	6152661	ECCC	571,589	4,800,062	Jun 01 1965	Mar 31 1978	14	335
45	GRIMSBY ALLAN	6133050	ECCC	613,804	4,780,274	Aug 01 1970	Mar 31 1975	6	284
46	GRIMSBY CHATEAU GAI	6133052	ECCC	611,033	4,783,931	Aug 09 1978	May 31 1981	4	257
47	GUELPH ARBORETUM	6143069	ECCC	563,276	4,822,192	Jul 23 1975	Aug 31 1997	22	339
48	GUELPH OAC	6143083	ECCC	561,964	4,818,478	Jan 01 1945	Nov 30 1973	30	352
49	GUELPH OAC PHYSICS DEPT	614308C	ECCC	559,237	4,822,156	Jun 01 1962	Mar 31 1963	2	138
50	GUELPH TURFGRASS	6143089	ECCC	563,276	4,822,192	Sep 29 2006	*Realtime*	12	287
51	GUELPH TURFGRASS CS	6143090	ECCC	563,276	4,822,192	Oct 01 1995	Jul 31 2004	8	334
52	HAMILTON	6153192	ECCC	589,271	4,791,016	Jan 01 1945	Aug 31 1958	14	354
53	HAMILTON A	6153193	ECCC	586,562	4,780,645	Dec 15 2011	*Realtime*	6	287
54	HAMILTON A	6153194	ECCC	586,632	4,780,432	Nov 06 1959	Dec 14 2011	53	357
55	HAMILTON GAGE PARK	6153238	ECCC	592,026	4,787,350	Sep 01 1953	May 31 1956	4	244
56	HAMILTON MARINE POLICE	6153264	ECCC	591,976	4,791,052	Apr 01 1980	Jul 31 1983	4	297
57	HAMILTON MUNICIPAL LAB	6153290	ECCC	600,119	4,789,316	Feb 01 1967	Feb 28 1994	28	346
58	HAMILTON PSYCH HOSPITAL	6153298	ECCC	589,320	4,787,314	Mar 01 1960	Aug 31 1993	34	354
59	HAMILTON RBG	6153300	ECCC	590,599	4,792,885	Apr 01 1950	Oct 31 1997	49	354

Reference ID	Name	Climate ID	Station Operator	Easting (m)	Northing (m)	Start Date	End Date	Water Years with Data	Average Number of Days with Data per Water Year
60	HAMILTON RBG CS	6153301	ECCC	588,558	4,793,783	Nov 01 1997	*Realtime*	20	345
61	HANNON	6153328	ECCC	594,836	4,779,984	Apr 16 1967	Jul 31 1967	1	107
62	HORNBY	6153540	ECCC	594,241	4,822,555	Jun 07 1947	Feb 28 1959	13	324
63	HORNBY	6153545	ECCC	592,869	4,824,388	Jul 19 1967	Aug 31 1978	12	339
64	HORNBY TRAFALGAR TS	6153552	ECCC	602,347	4,820,823	Jun 01 1982	Oct 31 2001	21	315
65	MIDDLEPORT TS	6155097	ECCC	578,642	4,774,224	Jul 01 1980	Jan 31 2000	21	341
66	MILLERS LAKE	6145160	ECCC	550,032	4,792,464	Jun 01 1964	Aug 31 1971	8	320
67	MILLGROVE	6155183	ECCC	583,792	4,796,499	Mar 19 1951	Apr 30 2006	56	351
68	MILTON KELSO	6155187	ECCC	584,887	4,816,877	Jun 01 1966	Apr 30 1987	22	321
69	MILTON SOUTH	615EAQG	ECCC	594,397	4,811,449	Oct 01 1993	Jul 31 1997	4	326
70	MORRISTON	6145495	ECCC	571,452	4,813,018	Apr 01 1948	Jul 31 1966	19	352
71	MOUNTSBERG	6145516	ECCC	578,193	4,813,093	May 01 1976	Oct 31 1985	11	180
72	MT ALBION CONSERVATION	615E498	ECCC	594,784	4,783,685	Jul 01 1976	Jun 30 1983	8	245
73	OAKVILLE	6155744	ECCC	606,592	4,807,928	Apr 01 1956	Sep 30 1971	16	354
74	OAKVILLE GERARD	6155PD4	ECCC	605,469	4,809,176	Apr 01 1990	Oct 31 2006	18	334
75	OAKVILLE GLEN ABBEY	6155PM4	ECCC	603,836	4,811,588	Oct 01 1983	Apr 30 1987	4	206
76	OAKVILLE SOUTHEAST WPCP	615N745	ECCC	610,518	4,815,398	Jul 01 1970	Dec 31 2001	33	345
77	OAKVILLE STP	6155746	ECCC	606,592	4,807,928	Nov 01 1964	Jan 31 1970	6	308
78	OAKVILLE TWN	6155750	ECCC	605,885	4,818,563	Jan 30 2007	*Realtime*	11	300
79	PALERMO	6156215	ECCC	599,845	4,807,825	Apr 01 1962	Nov 30 1967	7	224
80	PETERS CORNERS	6156470	ECCC	573,000	4,794,523	May 01 1952	Jan 31 1970	13	282
81	PORT CREDIT	6156609	ECCC	611,712	4,824,675	Nov 14 1951	Dec 31 1959	9	316
82	PRESTON WPCP	6146714	ECCC	552,650	4,803,590	Oct 15 1970	Feb 28 1997	27	344
83	REGION OF WATERLOO INT'L AIRPORT	6149388	ECCC	550,360	4,811,964	Oct 03 2002	Apr 17 2010	8	344
84	SPEYSIDE IHD	6157976	ECCC	579,431	4,822,364	Jun 14 1968	Jul 31 1975	8	303
85	TRAFALGAR MARINE	6158860	ECCC	602,403	4,817,121	Apr 01 1959	Mar 31 1971	13	323
86	VALENS	6159127	ECCC	570,200	4,803,749	Sep 01 1968	Apr 30 1994	27	327
87	VAN WAGNERS BEACH	6159133	ECCC	601,472	4,789,336	Aug 01 1958	Jun 30 1959	2	167
88	WESTOVER	615R457	ECCC	574,311	4,798,240	Oct 01 1967	Oct 31 1967	1	31
89	WINONA	6139556	ECCC	611,003	4,785,782	May 01 1977	Sep 30 1977	1	153

Table 2.5: List of solar radiation stations available to compile study-area solar radiation estimates.

Station	Location	Coordinates	Sensor type(s)	Data Interval	Units	Available Period of Record
University of Waterloo Weather Station	North Campus	43°28'25.6" N, 80°33'27.5" W, 334.4 masl	Kipp & Zonen Model: CM11	15 minute	W/m^2	1998-2014
University of Toronto Weather Station*	University of Toronto at Mississauga Meteorological Station (UTMMS)	43° 33' N, 79° 40' W	Kipp & Zonen model CM-5 and Kipp & Zonen CM-11 (from July 2007)	hourly	mv and W/m^2	1999-2015
York University ESSE Meteorological Observation Station (EMOS)	York University (northwest gate)	43.7753N, 79.5100W, 196masl	Kipp & Zonen CNR1 Net Radiometer	5 minute	W/m^2	2006-2015
Environment Canada**	611KBE0 Egbert Care	597434m E, 4898143m N	unknown	daily	MJ/m^2	1988-2003
Environment Canada**	6142285 Elora Research Station	546774m E, 4833164m N	unknown	daily	MJ/m^2	1970-2003
Environment Canada**	6143083 Guelph OAC	562230m E, 4818850m N	unknown	daily	MJ/m^2	1962-1970
Environment Canada**	6158350 Toronto	628988m E, 4836465m N	unknown	daily	MJ/m^2	1956-2000
Environment Canada**	6158740 Toronto Met Res Station	616643m E, 4850681m N	unknown	daily	MJ/m^2	1967-1988
Environment Canada**	6158776 Toronto Scarborough	642575m E, 4842297m N	unknown	daily	MJ/m^2	1959-1973

* mv to W/m^2 conversion factor was 93.63 $W/m^2/mv$ for the CM-5 and 77.276 $W/m^2/mv$ for the CM-11 (Ken Turner, Department of Geography, University of Toronto, Mississauga, pers. comm. 2010).

** All EC stations correlate well (Earthfx, 2010), having correlation coefficients greater that $r^2>0.9$ amongst all pairings with little systematic error.

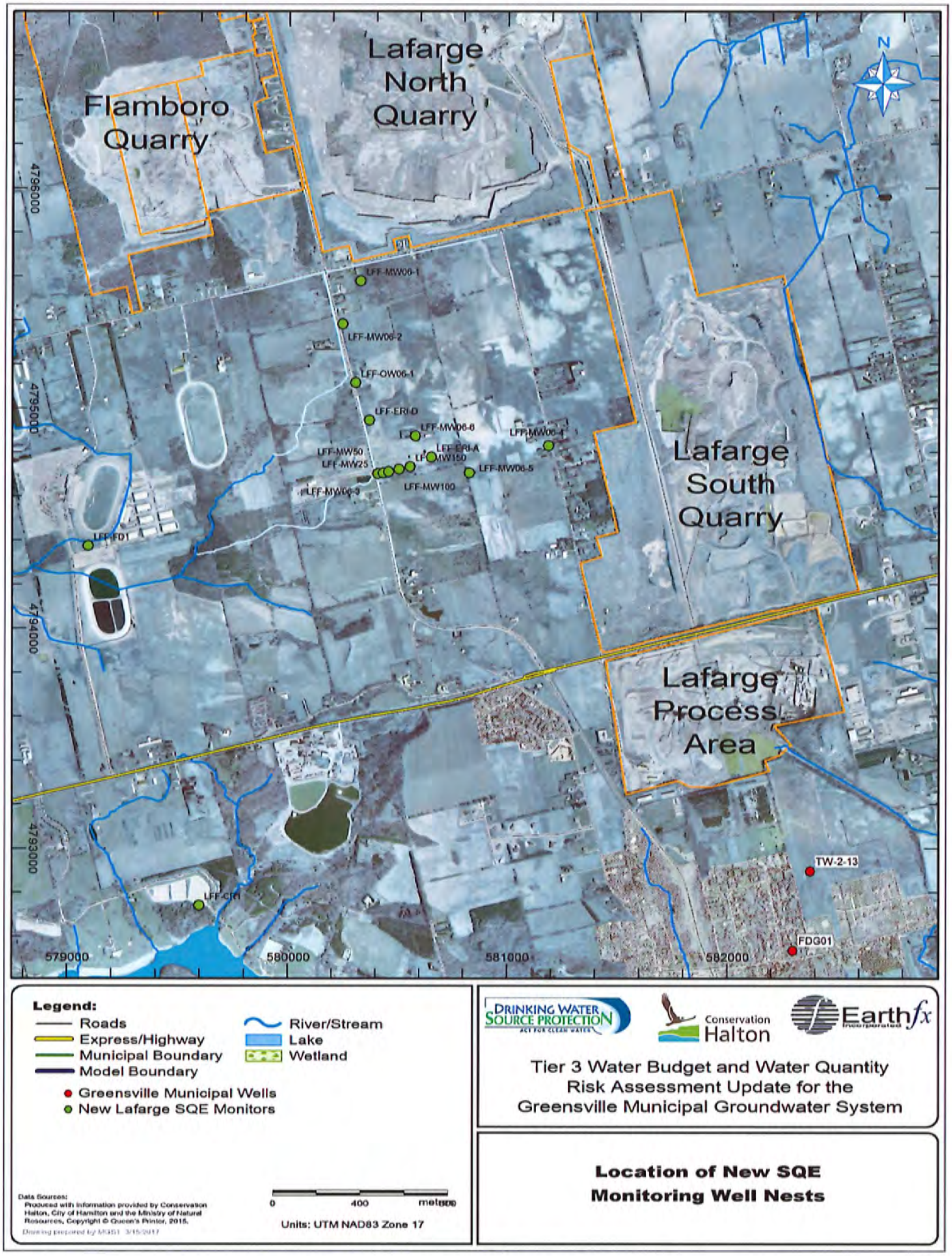


Figure 2.3: Location of new SQE monitoring well nests.

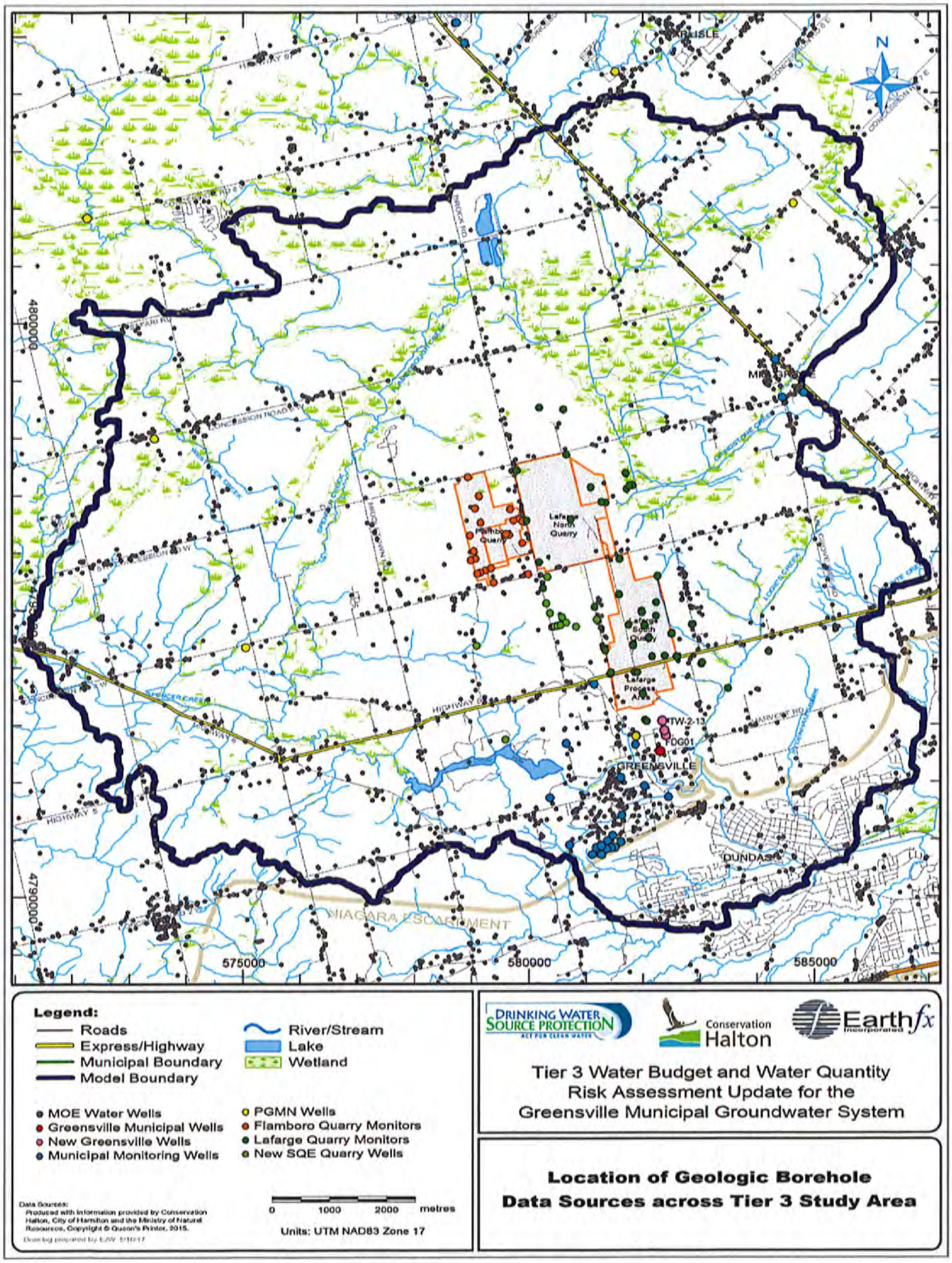


Figure 2.4: Location of geologic boreholes including new SQE wells.

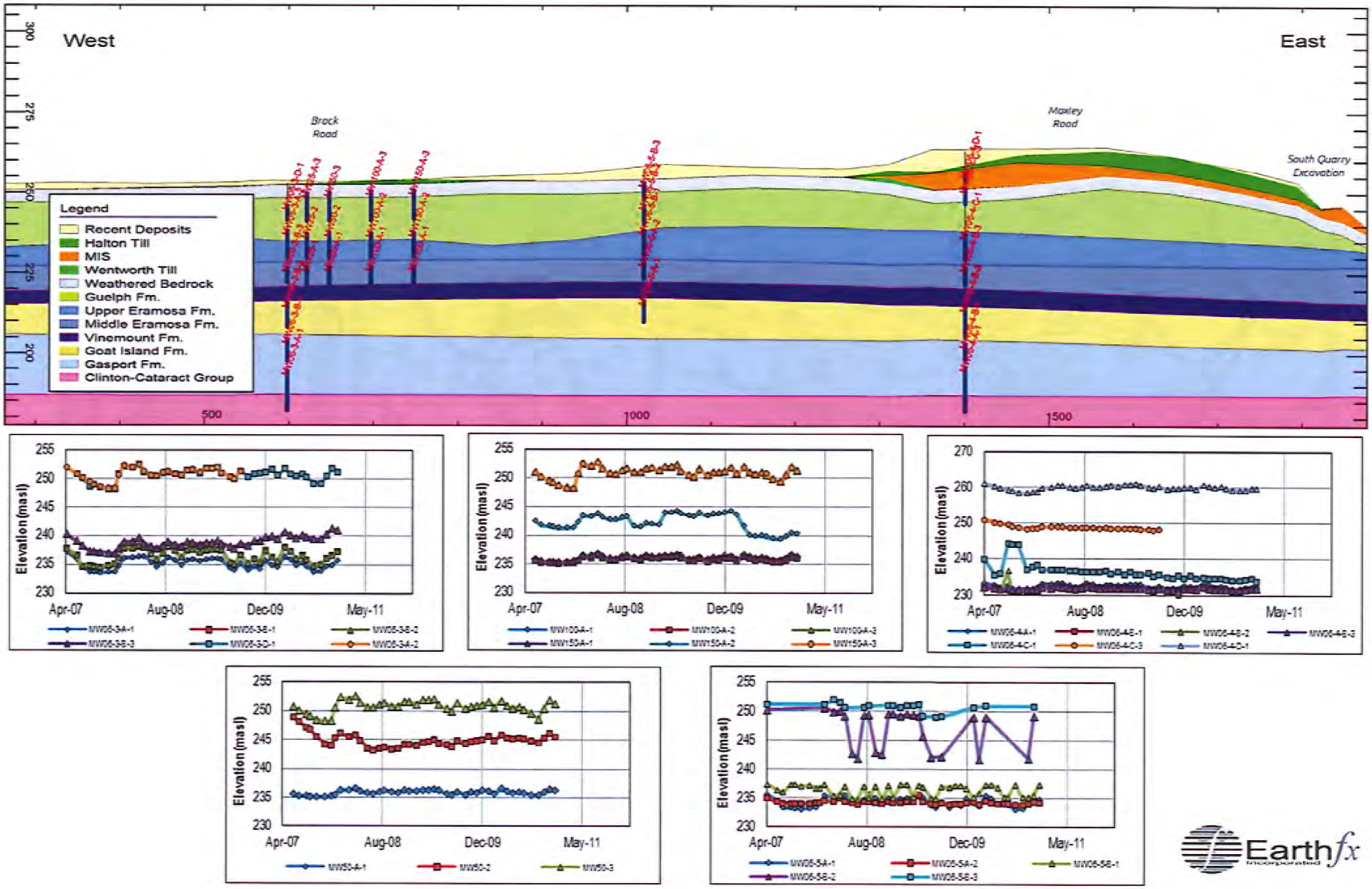


Figure 2.5: East-West section through SQE area with groundwater levels at nested monitoring wells.

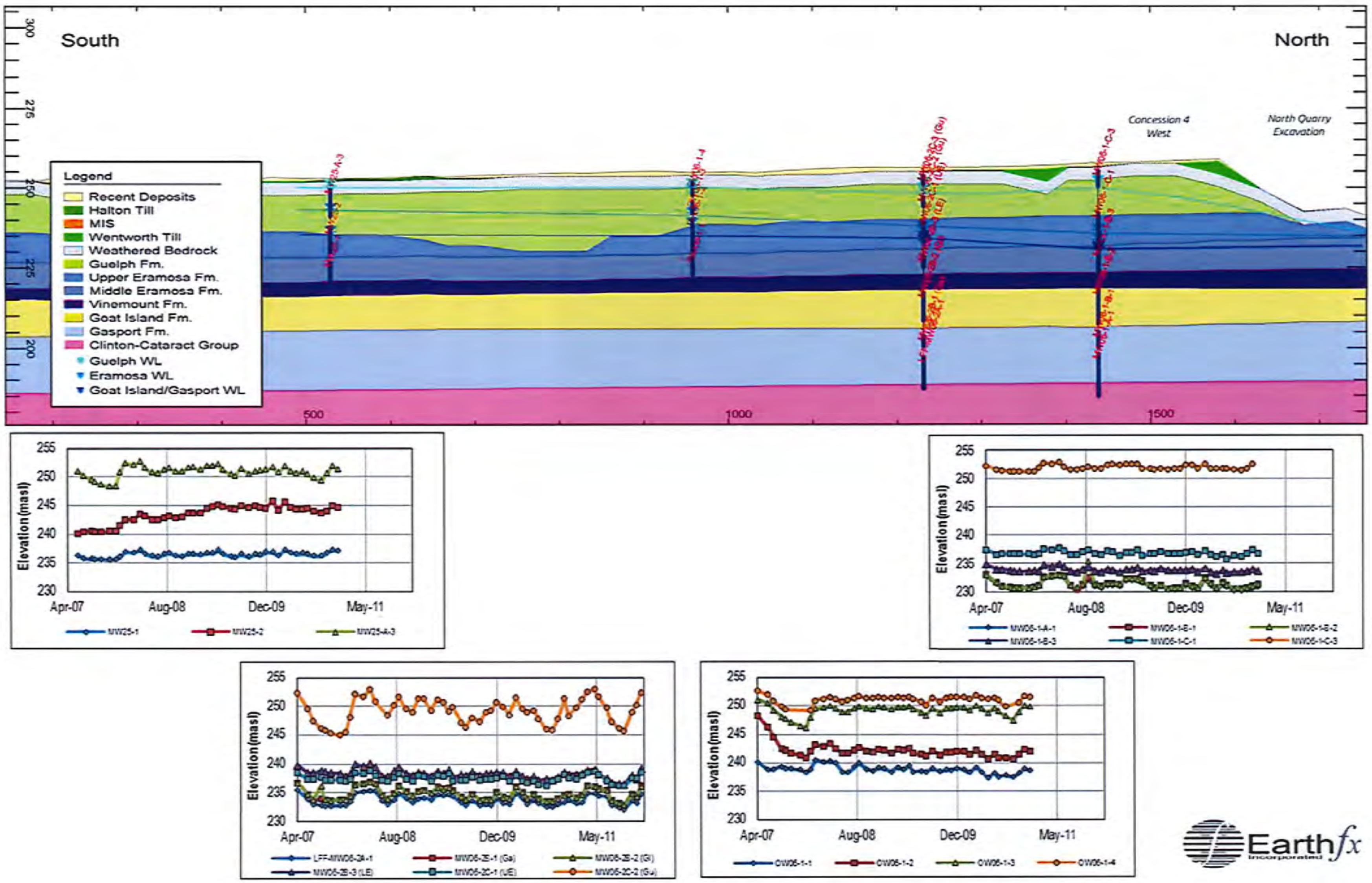
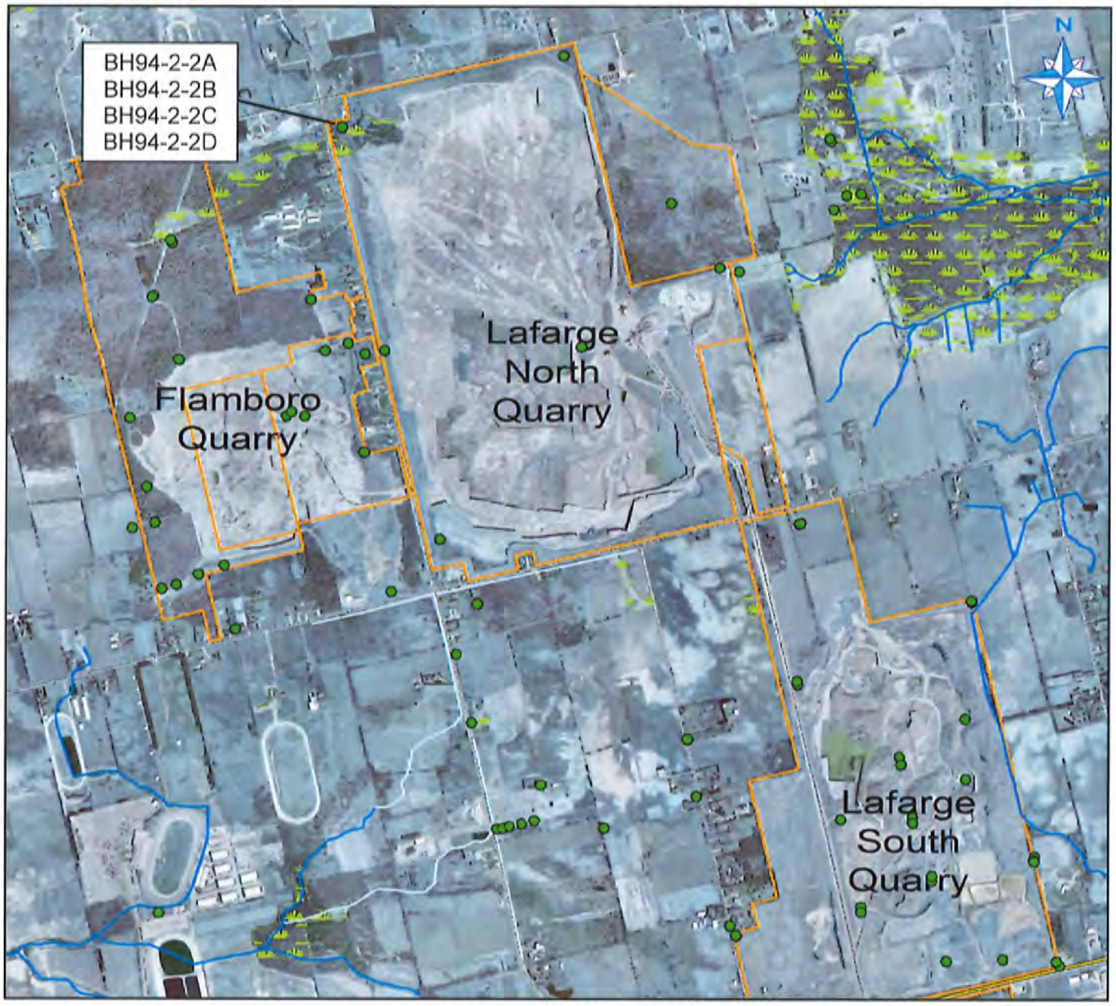
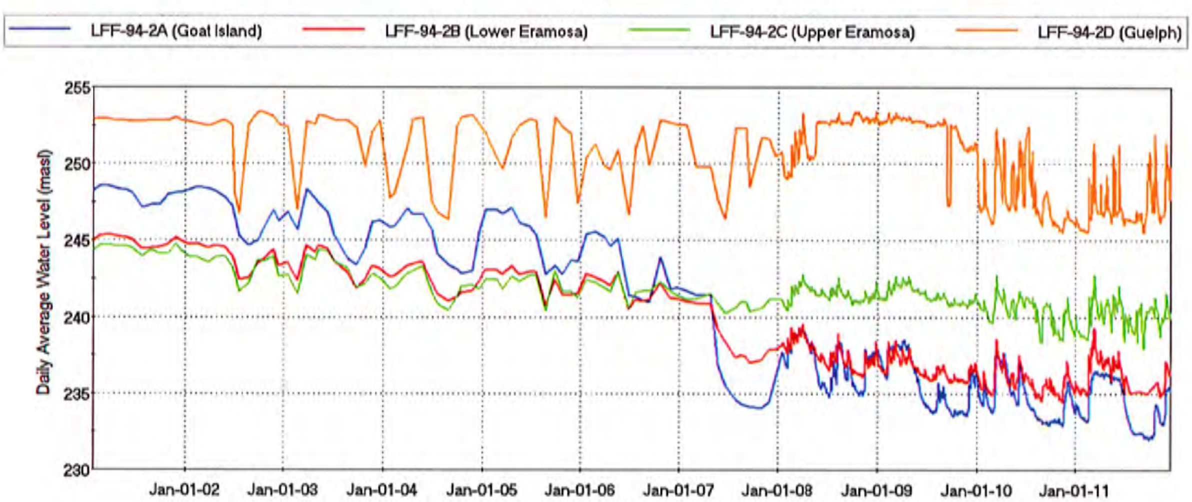


Figure 2.6: North-South section through SQE area with groundwater levels at nested monitoring wells.



(a)



(b)

Figure 2.7: (a) North Quarry monitoring nest BH-94-2; and (b) hydrograph showing reduced confinement of the deep Goat Island aquifer.

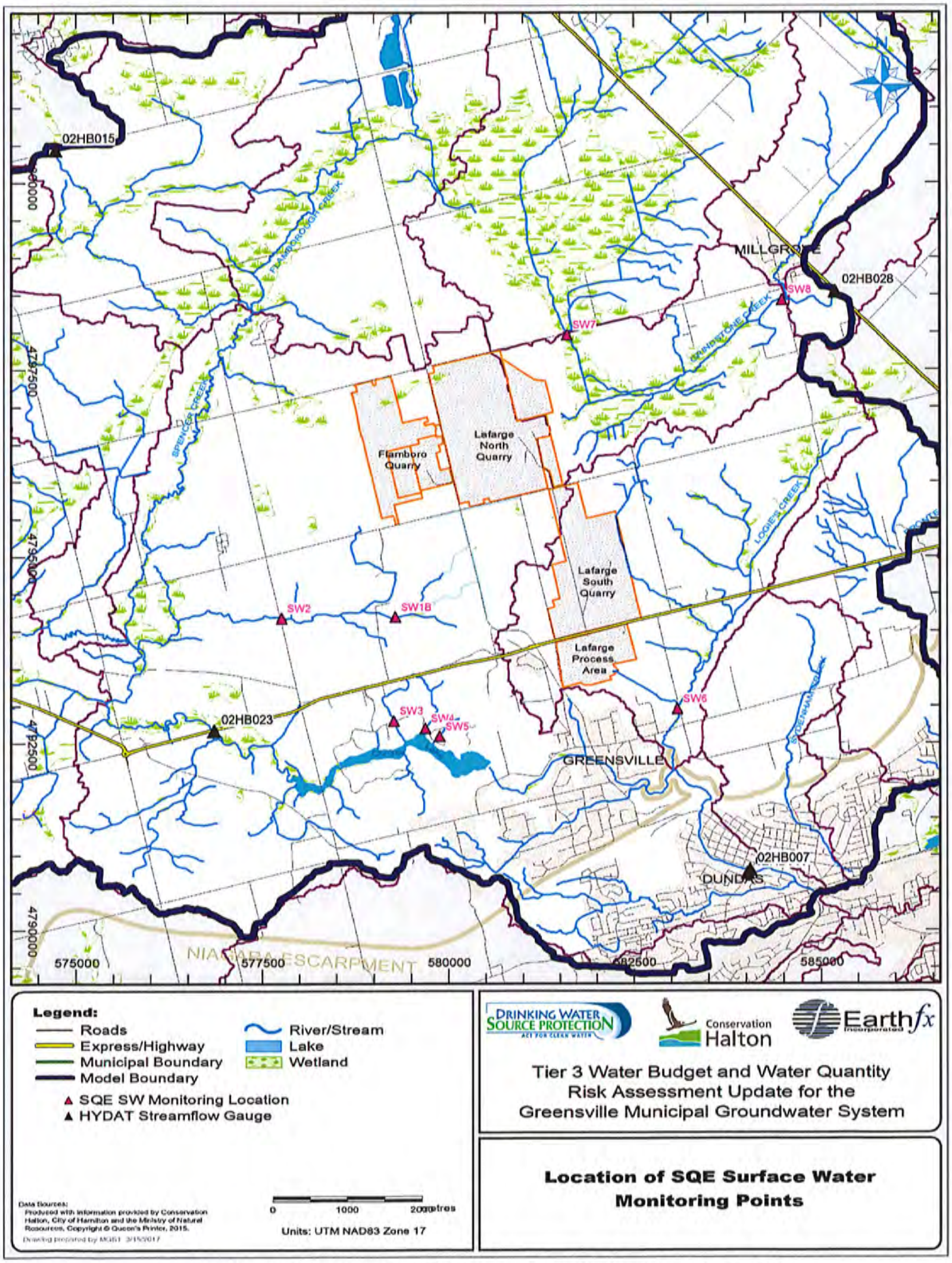


Figure 2.8: Surface water monitoring locations for the SQE application.

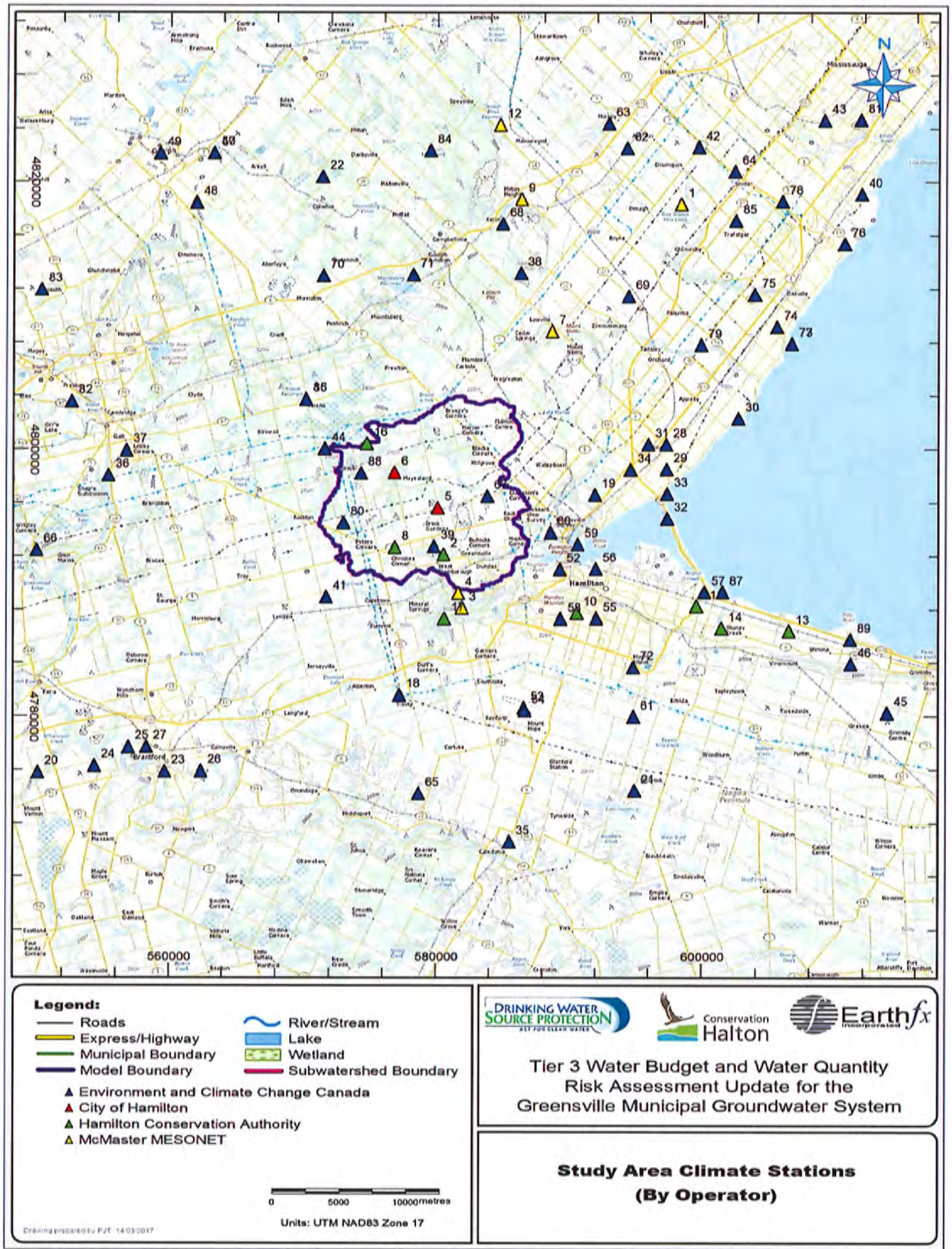


Figure 2.9: Climate stations proximal to the study area with operating authority.

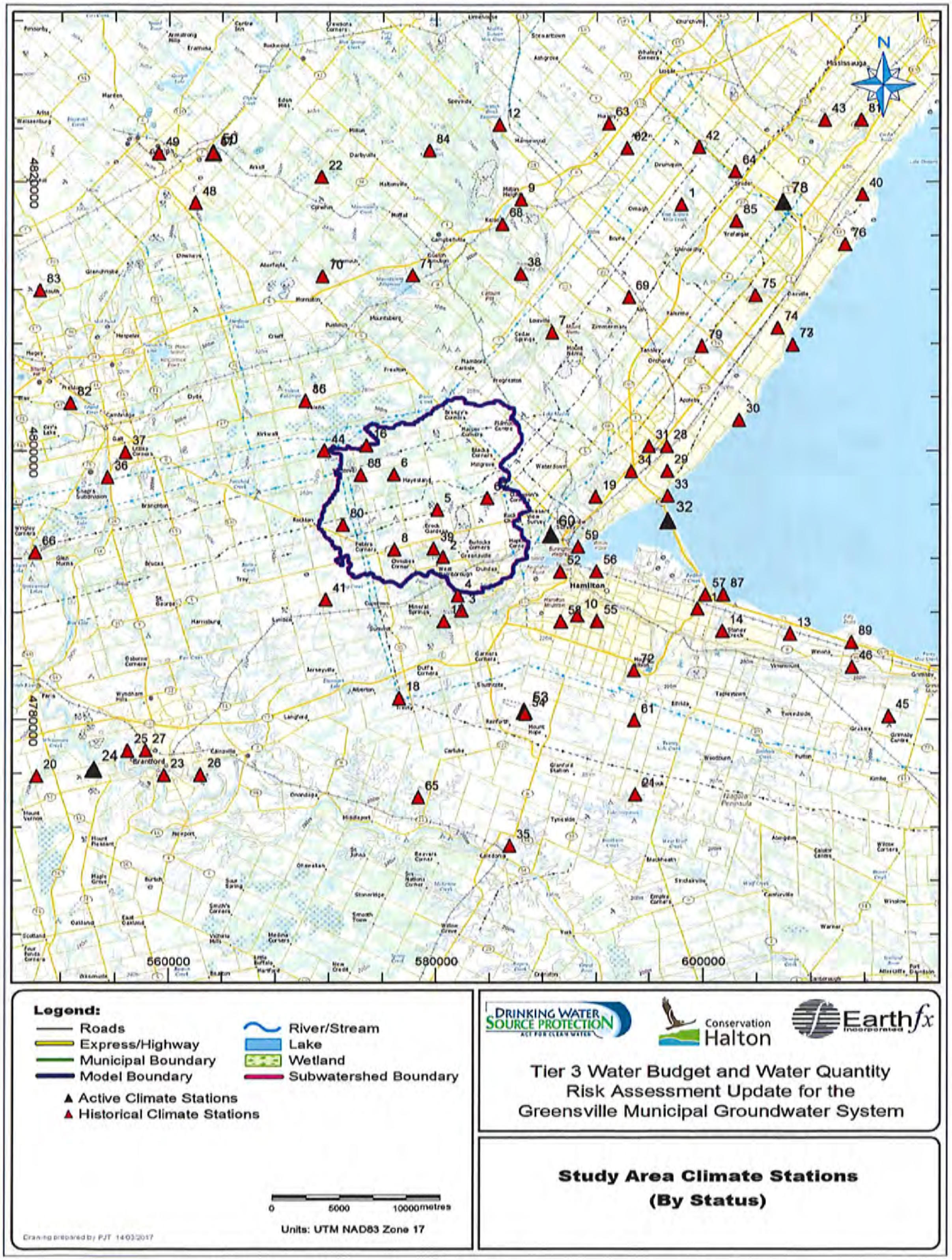


Figure 2.10: Climate stations proximal to the study area with staus.

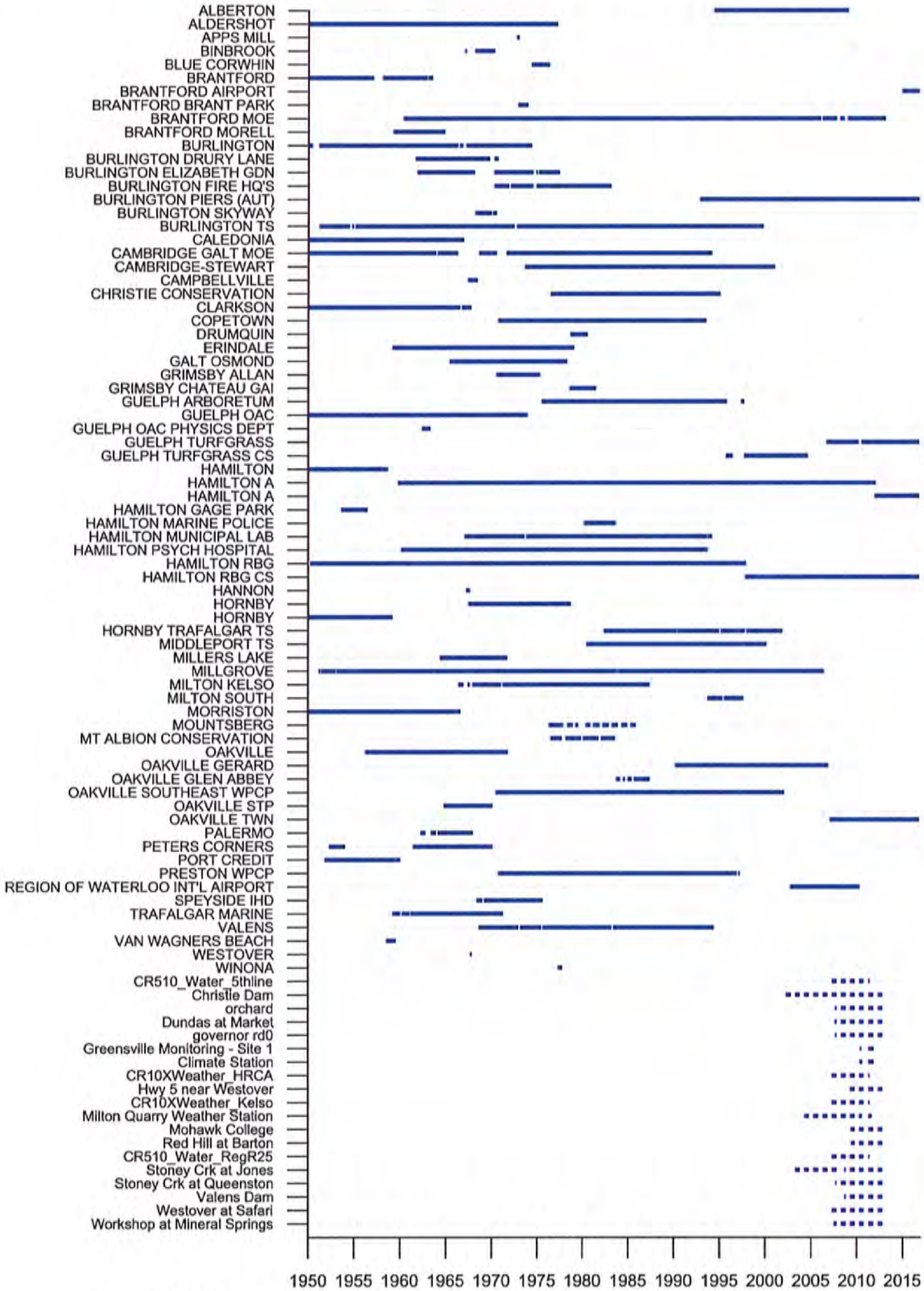


Figure 2.11: Available period of record at climate stations proximal to the study area.

Figure 2.12: Number of climate stations with complete monthly precipitation record (1945 through 2016).

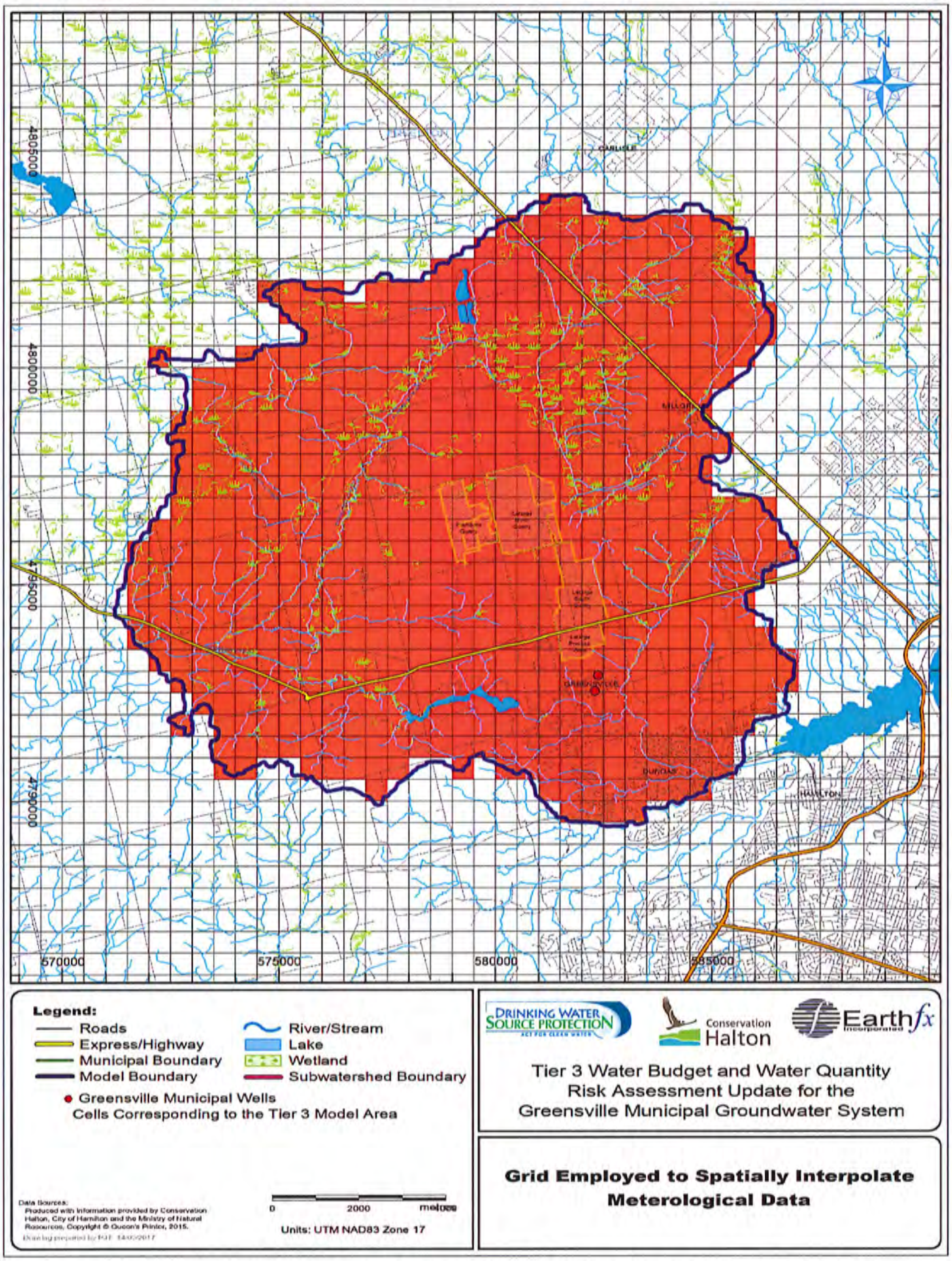


Figure 2.13: Grid employed to spatially interpolate the meteorological data and raster grid employed to derive basin averaged normals.

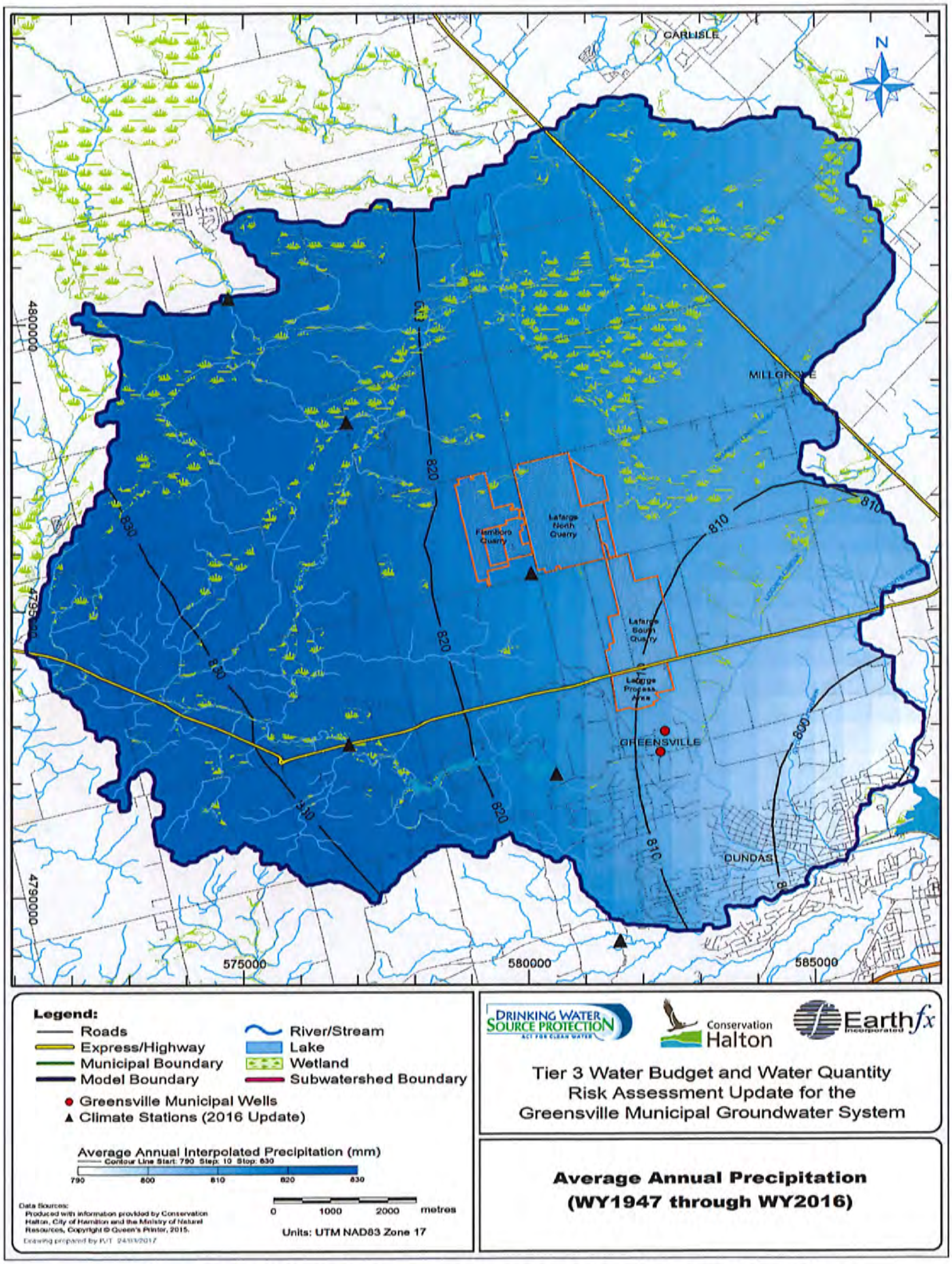


Figure 2.14: Annual average interpolated precipitation ($_{WY}$1947 through $_{WY}$2016).

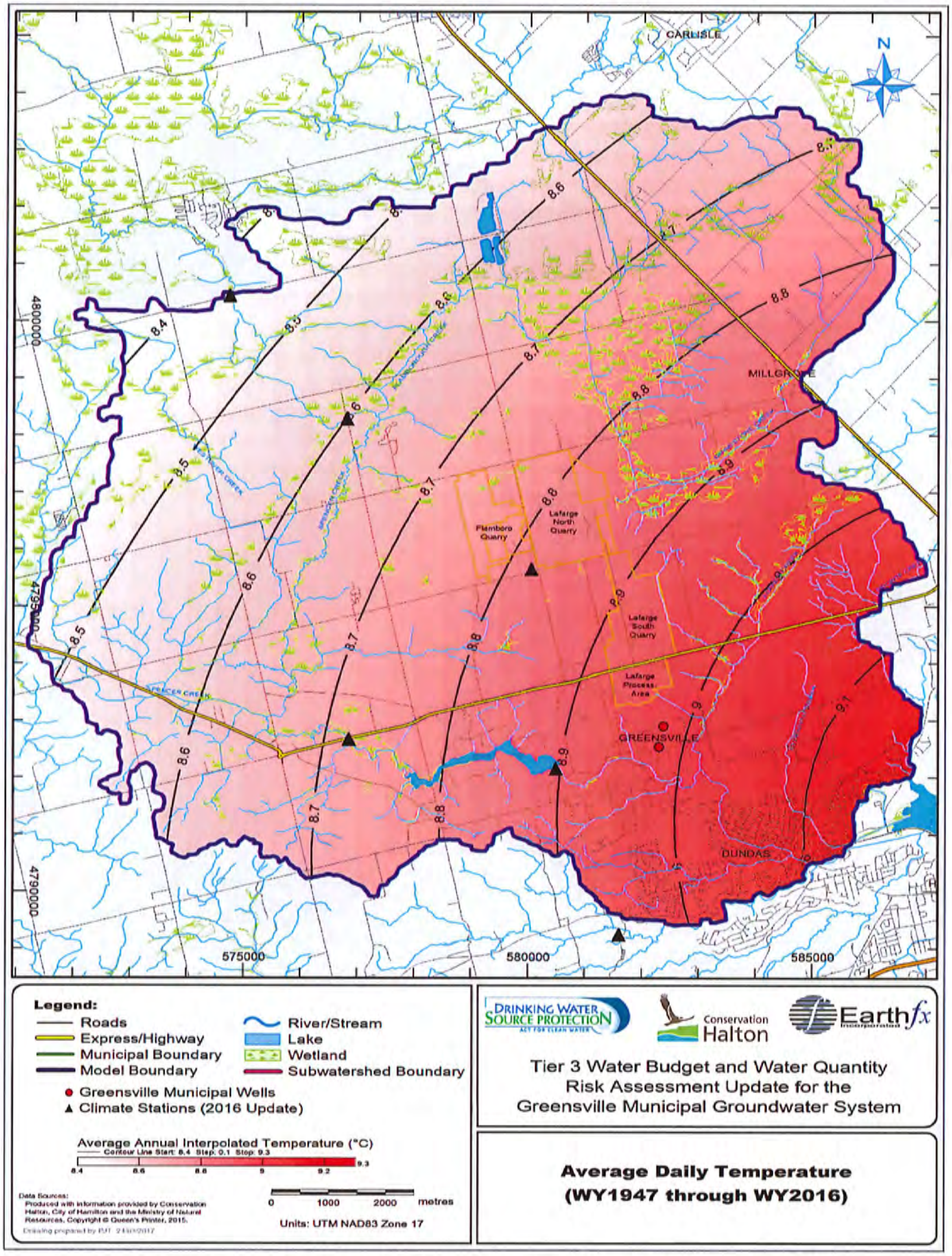


Figure 2.15: Daily average interpolated mean temperature ($_{WY}$1947 through $_{WY}$2016)

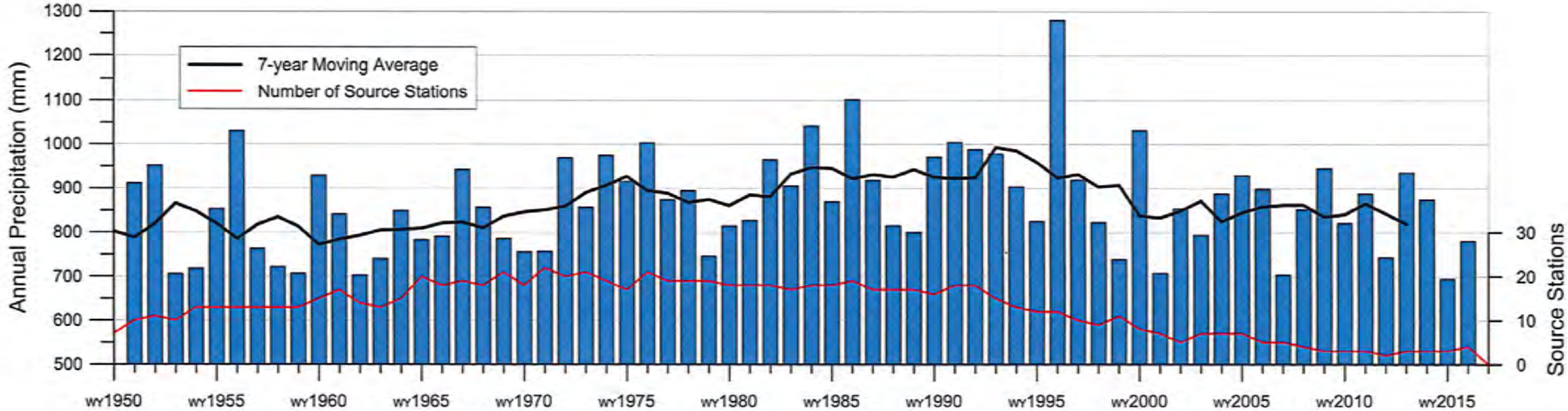


Figure 2.16: Mean annual precipitation (WY1947 through WY2016) averaged over the Tier 3 model area.

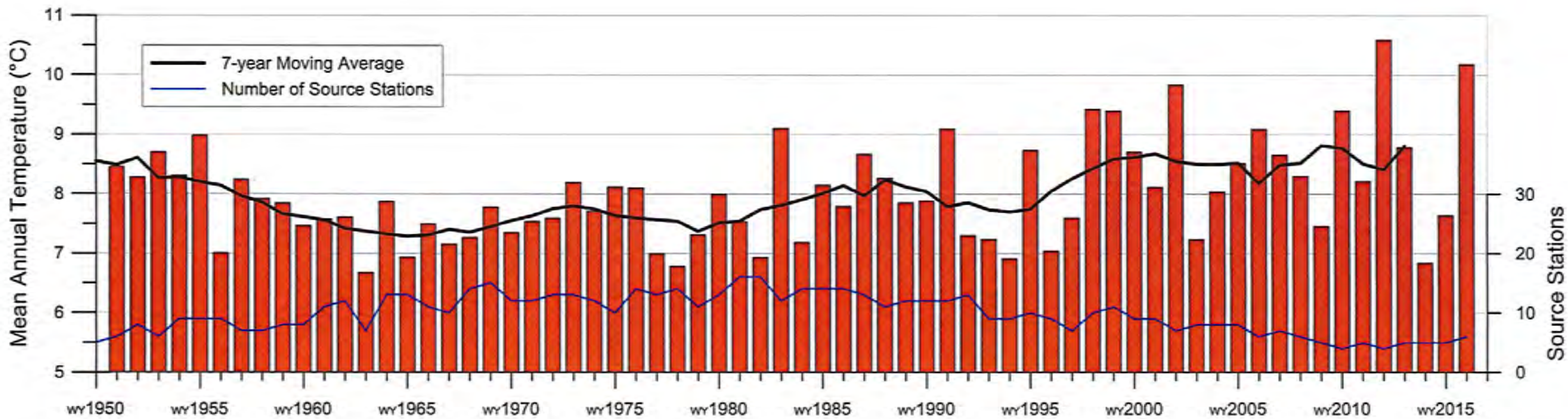


Figure 2.17: Mean annual temperature (WY1947 through WY2016) averaged over the Tier 3 model area.

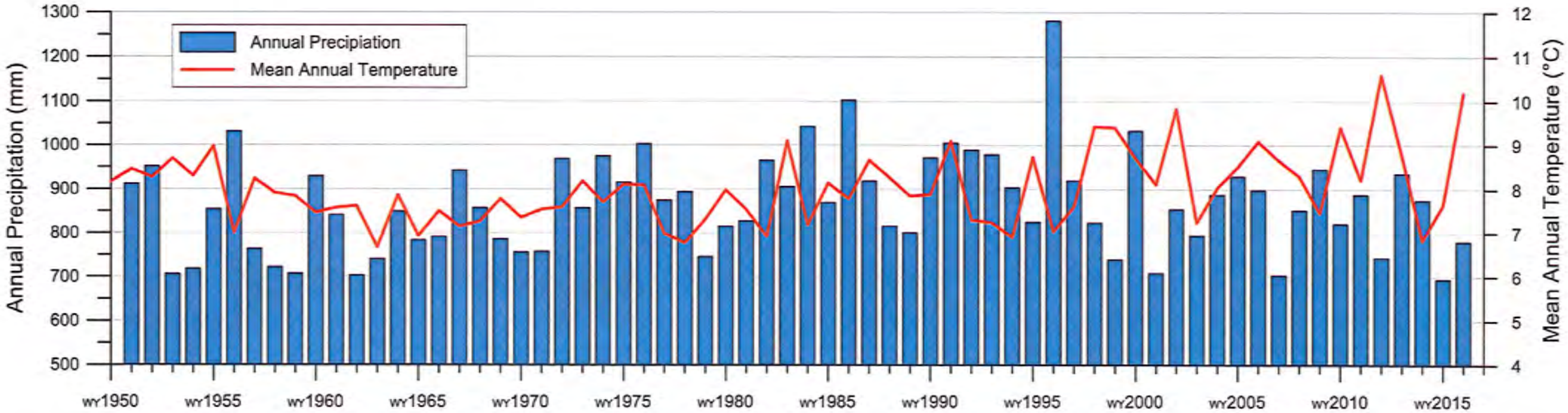


Figure 2.18: Mean annual temperature and precipitation averaged over the Tier 3 model area.

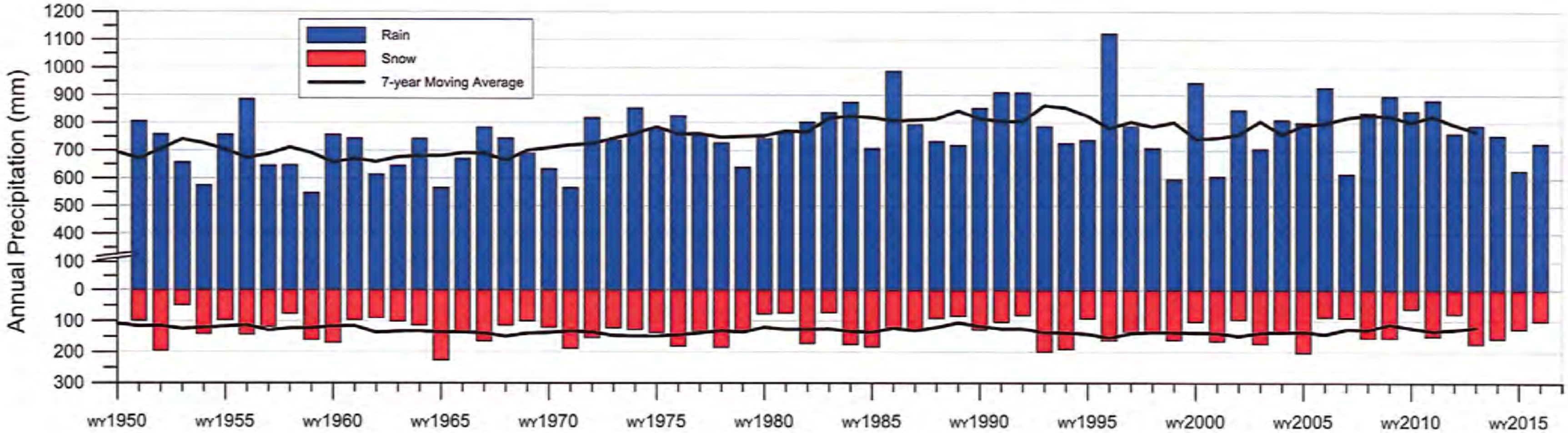


Figure 2.19: Annual summary of precipitation form averaged over the Tier 3 model area.

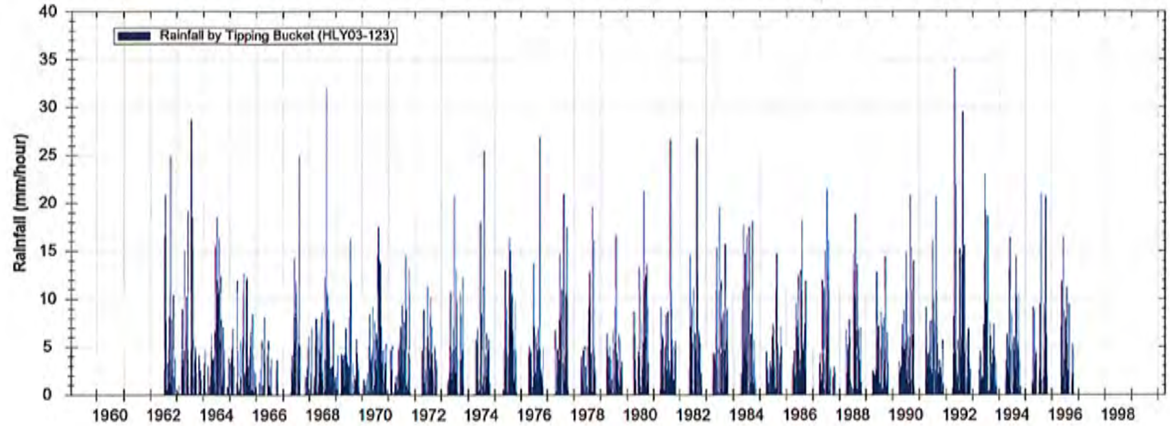


Figure 2.20: Hourly rainfall measured by tipping bucket (HLY03-123) at ECCC Climate Station HAMILTON RGB (6153300).

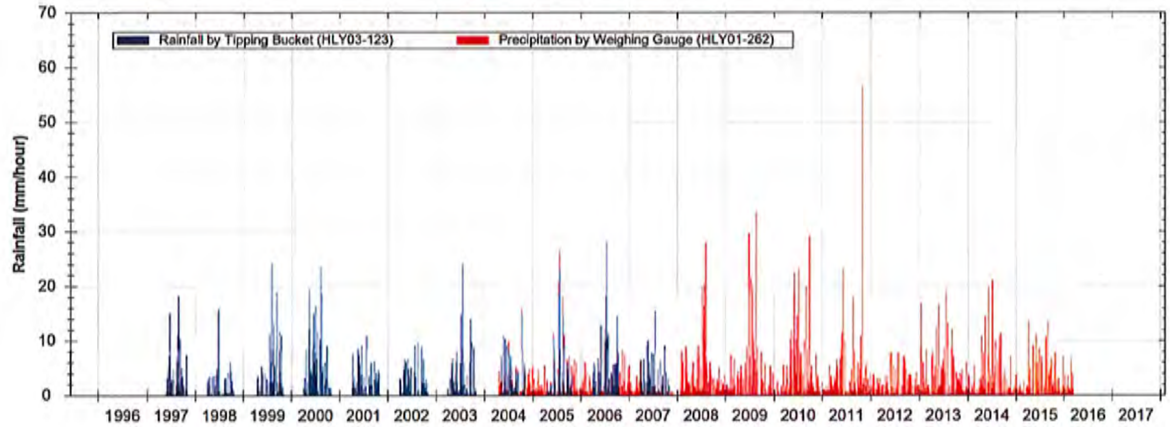


Figure 2.21: Hourly rainfall measured by tipping bucket (HLY03-123) and weighing gauge (HLY01-262) at ECCC Climate Station HAMILTON RGB CS (6153301).

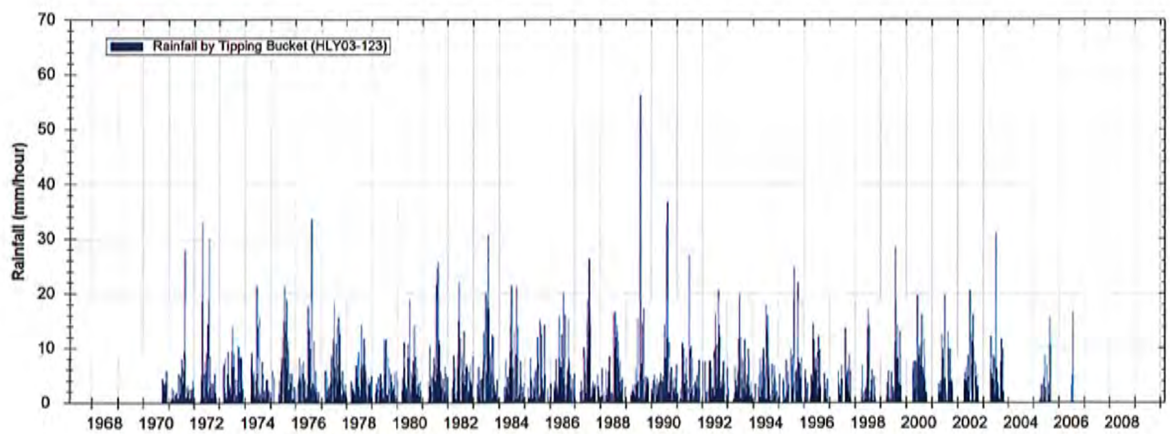


Figure 2.22: Hourly rainfall measured by tipping bucket (HLY03-123) at ECCC Climate Station HAMILTON A (6153194).

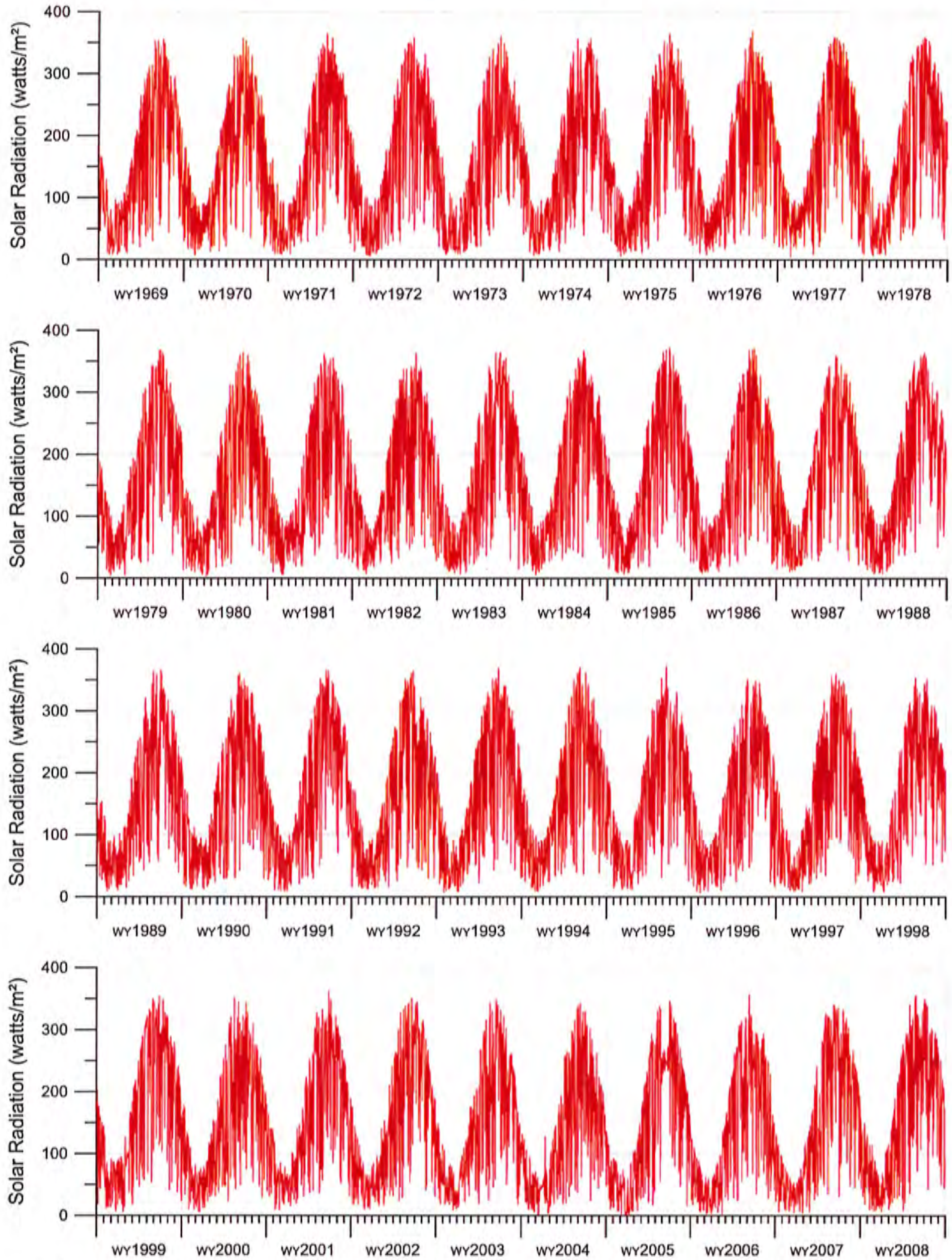


Figure 2.23: Observed daily global incoming radiation data derived from stations in southern Ontario.

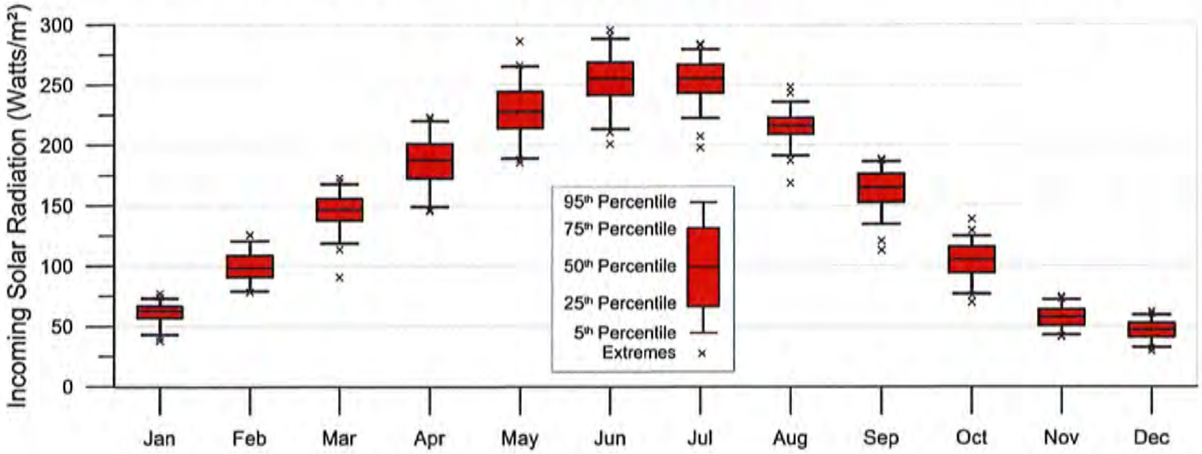


Figure 2.24: Monthly histogram of observed incoming solar radiation.

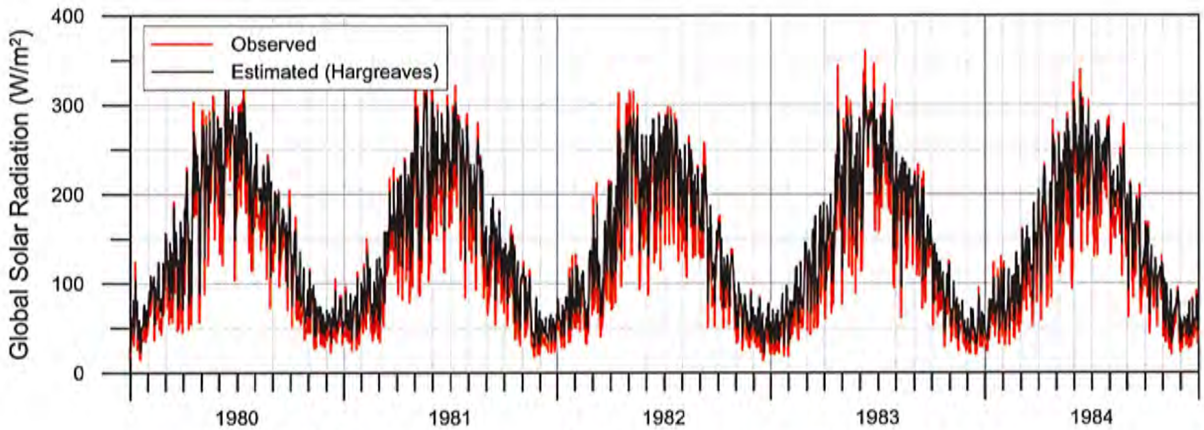


Figure 2.25: Observed versus estimated daily solar radiation time series.

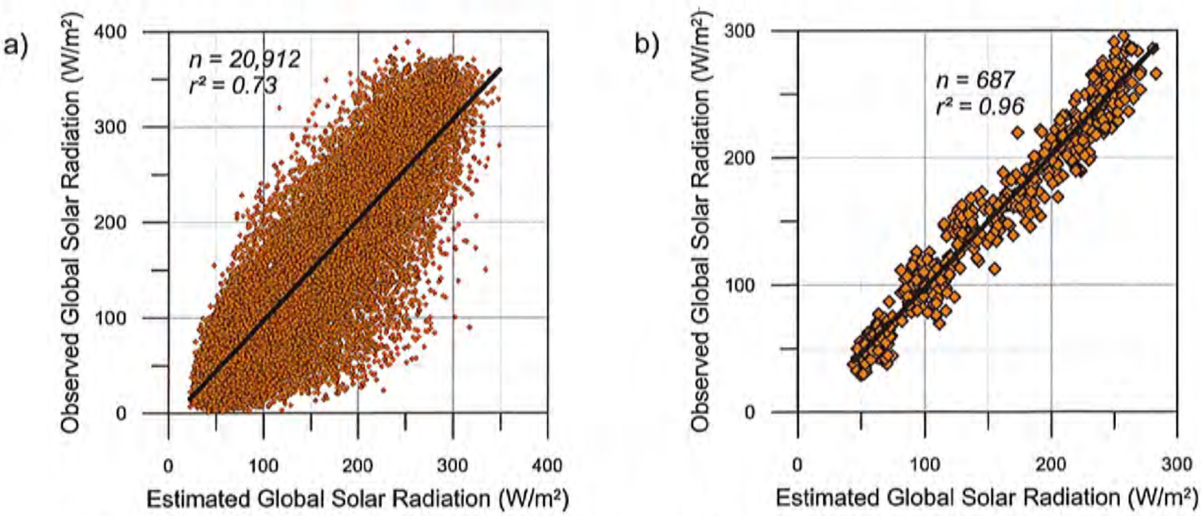


Figure 2.26: Observed versus estimated (Hargreaves and Samani, 1982) (a) daily and (b) monthly average solar radiation.

3 Water Demand and Land Use Change

3.1 *Municipal Water Supply Systems*

The Greensville municipal supply system serves 36 homes (about 127 people) and is operated under PTTW 2476-9F5KM6. Currently, all water is withdrawn from well FDG01 (location shown in Figure 1.1). The well was drilled in 1972 (MOE Well Number 6808331) with a diameter of 150 mm to a depth of 12.19 m and completed in the upper 1.5 m of weathered bedrock. The well is equipped with a submersible pump rated at 2.27 L/s. The well is allowed to extract a maximum of 197 cubic metres per day (m^3/d), although actual use is much less on an average daily basis. A maximum reported daily taking, 105 m^3/d, occurred in August 2011. Although the settlement area is expected to grow, the City of Hamilton does not plan to increase the number of connections to the municipal wellfield.

The new well, TW-2-13, was constructed in February 2013 approximately 370 m northeast of FDG01. The well was drilled to a depth of 21.67 m, penetrating 9.85 m into the bedrock. The well is cased for a depth of 12.5 mbgs and the remaining depth is a 152 mm open borehole (Stantec, 2014). While originally intended as a backup well, based on discussions with City of Hamilton staff, TW-2-13 will serve as the new primary well and FDG01 as the backup. Testing of the well in 2016, following acidification and well development, showed the well capable of sustaining 173 m^3/d (Lotimer & Associates, 2016). The well is planned to be brought into service in 2018.

Although not on municipal supply, projected water demand for the planned Greensville Elementary School/Library/Community Centre was considered when simulating effects of future increases in pumping in the Greensville area. At the time of this study, the project to construct the planned Greensville School had secured conditional approval from the City of Hamilton under the Planning Act (R.S.O. 1990), and was in the process of securing MOECC environmental compliance approval for the facility's sewage system. The target completion date for the new Greensville School is September 2018. The Greensville School is located on Harvest Road, approximately 120 m east of FDG01 (shown on Figure 3.1). The Greensville School will be supplied by a pumping well located just north of Harvest Road. The estimated takings from the well were assumed to be 23.2 m^3/d based on the design capacity of the facility's septic system.

3.2 *Existing, Allocated, and Planned Quantity of Water*

The Water Budget Guideline (2011) as updated by the December 2, 2013 Technical Memorandum (MOE, 2013) describe three components of water demand (see Figure 3.1 below):

- **Existing Demand**. Existing demand is estimated as the average pumping rate during the study period. Average pumping rates for the Greensville municipal wellfield were compiled from data provided by the City of Hamilton during the previous Tier 3 study. Municipal pumping rates from the existing well FDG01 were available for the period of 2007 to 2014, and are still considered to be valid for the present study given that system demands (connections) have not significantly increased. The average demand on the existing system between 2007 and 2014 was 40.9 m^3/d. Because TW-2-13 was not active at the time of this study, average daily rates from FDG01 were distributed between the two wells according to a 6-to-1 day cycle whereby FDG01 is pumped for one day for every six days that TW-2-13 is pumped. This operational assumption for the Greensville municipal wellfield was developed in consultation with City of Hamilton staff.

- **Committed Demand**. Committed demand refers to the incremental increase in the quantity of water provided by an existing drinking water system that would be required if the area served by the system were developed in accordance with the Official Plan for the area to an extent that would result in the greatest use of drinking water. Committed plus existing demand is termed the "allocated quantity of water" and cannot be greater than the permitted pumping rate. Any amount that exceeds the lawfully permitted value is considered to be a planned demand. As was noted, the City of Hamilton does not plan to increase the number of municipal supply connections.

Thus, there is no Committed Demand for the system and the allocated quantity of water is equal to the existing demand.

- **Planned Demand.** Planned demand for an existing well represents the incremental increase in water needed to reach the projected growth identified within a Master Plan or Class EA but is not already linked to growth within an Official Plan. As noted above, it also includes any amount of the demand needed to meet the Official Plan that exceeds the lawfully permitted value. The "planned quantity of water" for an existing system is equal to the existing plus committed plus planned demand. The planned quantity of water for a new well (planned well) includes any amount of water that meets the definition of a planned system under Ontario Regulation 287/07. As was noted, the City of Hamilton has added the new well, TW-2-13, simply to provide redundancy to the existing system and does not plan to increase takings. Therefore, there is no Planned Demand for the system, as summarized in

Table 3.1: Existing, allocated and planned demand for the Greensville Municipal Supply

Well	Existing Demand (m^3/d)	Committed Demand (m^3/d)	Allocated Quantity of Water (m^3/d)	Planned Demand (m^3/d)
FDG01	5.8	0	5.8	0
TW-2-13	35.1	0	35.1	0
Total	**40.9**	**0.0**	**40.9**	**0.0**

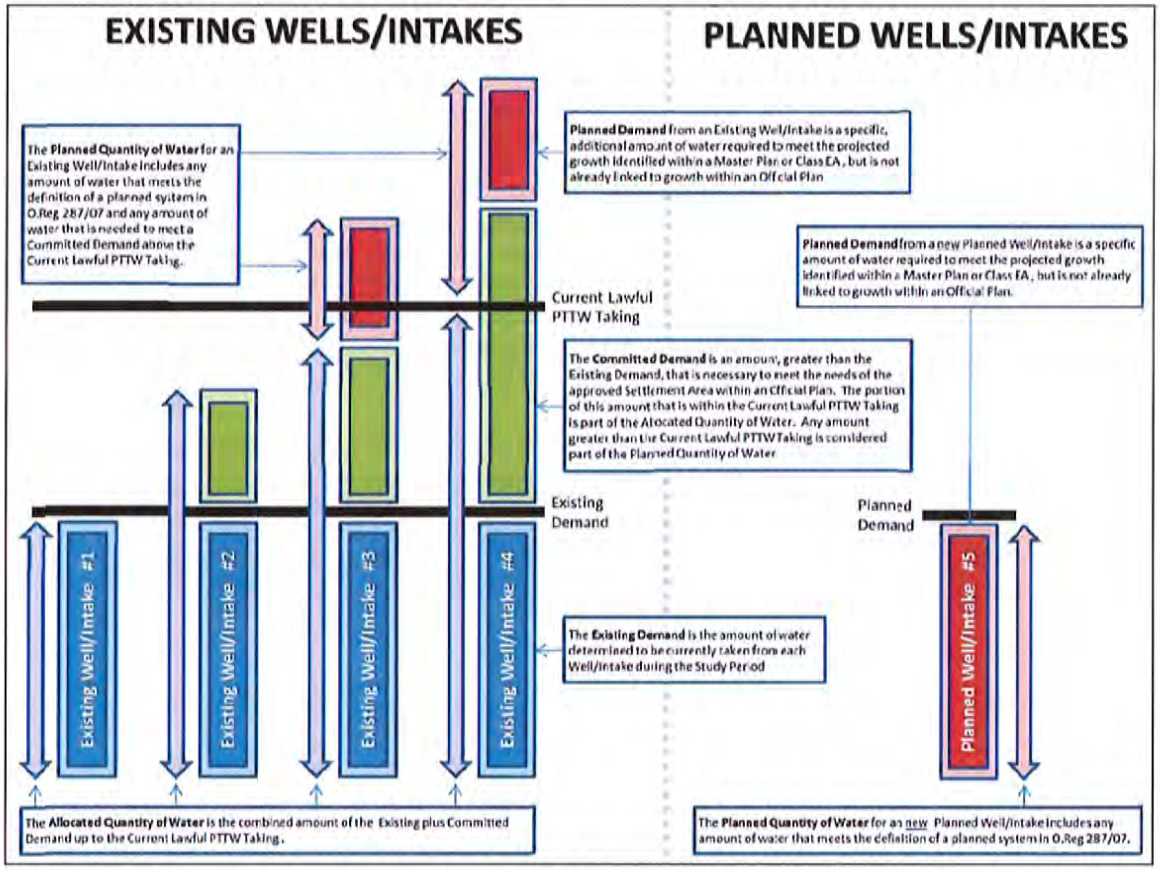


Figure 3.1: Allocated and Planned Quantities of Water (as defined by MOE Source Protection Branch, written comm., December 2, 2013).

3.2.1 Safe Additional Drawdown

The Water Budget Guide defines safe additional drawdown as the additional depth that the water level within a pumping well could fall and still maintain that well's allocated pumping rate. It is calculated as the additional drawdown that is available above the drawdown created by the pumping rate under existing conditions. To establish the safe additional drawdown for the Greensville municipal wells, the following components were evaluated:

Safe Water Level Elevation: This is the lowermost elevation at which the well can be pumped. This elevation may be limited to the well screen elevation, pump intake elevation or similar operational limitations, such as the top of a confined aquifer. The pump intakes FDG01 and TW-2-13 are assumed to be 10.5 mbgs and 19.3 mbgs, respectively, for the following Tier 3 Risk Assessment analyses. The pump setting for the existing well FDG01 was based on information provided by City of Hamilton staff during the previous Tier 3 study. No permanent pump setting presently exists for the new well TW-2-13; however, pump setting information was available from a pumping test performed by Lotimer & Associates (2016), for which the intake was set at 20 m below the top of casing (224.4 masl). Based on historical operation and testing, the safe water level for FDG01 has been identified as the pump intake elevation plus 1 m. A slightly different approach was applied to TW-2-13 where the safe water level was identified as the elevation of the uppermost water bearing fracture (Stantec, 2014).

Table 3.2: Safe additional drawdown for the Greensville municipal supply wells

Well	Ground Elevation (masl)	Average Pumped Water Level (masl)	Pump Setting (masl)	Top of Open Borehole Elevation (masl)	Minimum Safe Level (masl)	Available Drawdown (m)
FDG01	234.8	229.8	224.3	223.2	225.3	4.5
TW-2-13	243.8	233.1	224.4 [1]	231.2	227.4 [2]	5.8

Note: [1] No permanent pump setting yet identified for TW-2-13. Reported value based on intake setting for Lotimer & Associates (2016) pumping test.
[2] Minimum safe level for TW-2-13 based on elevation of uppermost water bearing fracture, as recommended by Stantec (2014).

Average Water Level in the Pumping Well: The average pumped water level was determined for the study period (2007 to 2012) from daily water level data provided by the City of Hamilton for Greensville well FDG01. While Greensville well TW-2-13 has yet to be brought online, logger data were provided for 2013 to 2015. While water level data from periods of pumping at TW-2-13 were available for this study, these periods correspond to well development and well efficiency testing (Stantec, 2014; Lotimer & Associates, 2015, 2016; and SNC Lavalin, 2016). Because no historical data were available on which to base the average (operational) water level in TW-2-13, the value used in this study (Table 3.2) was estimated by calculating the drawdown using a pumping rate of 35.8 m^3/d and specific capacity of 185.2 L/min/m, determined from results of the step-test conducted by SNC-Lavalin (2016).

Estimated Non-linear Well Losses in the Well: Additional drawdowns can occur within the well due to well inefficiencies (e.g., losses at the screen and around pump intakes). Well losses need to be considered as the additional available drawdown refers specifically to the water level in the well and not the average water level in the aquifer outside the well. Theoretical relations can be used to relate well losses to pumping rates (e.g., Jacob, 1950) as:

$$s_w = BQ + CQ^2$$

where S_W is the total drawdown, B is the formation loss coefficient as determined by a Theis (1935) or other analytical relation, Q is the pumping rate, and C is the well loss coefficient. The non-linear in-well loss coefficient (C) can be estimated by analyzing step test results using a graphical method developed by Jacob (1947). The inverse of the specific capacity, defined as the drawdown divided by the pumping rate, can be plotted versus the pumping rate. The slope of the best fit line through the data is equal to the well loss coefficient.

A step test on Greensville well FDG01 in April 2008 was analyzed to estimate the well loss coefficient. Figure 3.5 shows the pumping rate and recorded drawdowns over time. Figure 3.6 shows specific capacity versus pumping rate using average rates and drawdowns. A well loss coefficient of 9.8×10^{-6} $m/m^6/d^2$ was calculated from the graph. This means that, as an example, an increase in well pumping from 50 m^3/d to 100 m^3/d would yield an incremental well loss of 0.074 m based on Jacob (1947) where:

$$\Delta s = C\left[\left(Q_{existing} + \Delta Q\right)^2 - Q_{existing}{}^2\right]$$

Following initial development of well TW-2-13 by Stantec (2014), work was undertaken in August 2015 to improve well efficiency through acidification of the productive bedrock fractures (Lotimer & Associates, 2015). This first round of well-enhancement work was followed by a step-test to assess the improved capacity of the well. Using the results of this step-test, a well loss coefficient of 1.27×10^{-5} d^2/m^5 was calculated (shown in Figure 3.7). A more recent step test was completed for Greensville well TW-2-13 in June 2016 after the well was acidified and surged for a second time. A corresponding well loss coefficient of 1.48×10^{-5} d^2/m^5 was calculated from Figure 3.8. Non-linear well losses under existing and allocated demand are summarized in Table 3.3. The results indicate that the non-linear well losses are small and are unlikely to have a significant impact on the risk assessment results.

Table 3.3: Non-linear well loss corrections

Well	Existing Demand (m^3/d)	Allocated Quantity of Water (m^3/d)	Nonlinear Well Loss Coefficient (d^2/m^5)	Nonlinear Well Loss (m)	
				Existing	Allocated
FDG01	5.8	5.8	9.80×10^{-6}	0.00	0.00
TW-2-13	35.1	35.1	1.48×10^{-5}	0.02	0.02

Convergent Head Loss Corrections: The numerical model calculates the average water level in the grid cell containing the pumping well. Because the size of the well is small compared to the size of the model grid, water levels in the well will be different than the average cell value. The additional head loss calculated at the well compared to the average head loss simulated by the model at the grid scale is referred to as convergent head loss. A correction based on the Theis relationship was proposed first by Prickett and Lonnquist (1971) as:

$$s_w = \frac{Q}{2\pi T} \ln\left(\frac{0.208\Delta x}{r_w}\right)$$

where Q is the pumping rate, T is the aquifer transmissivity, Δx is the model grid spacing, and r_w is the effective well radius. The parameters and calculated values for convergent head loss are presented in Table 3.4 and are generally small because of the 12.5 m grid spacing used in the wellfield area. The Greensville supply wells have an allocated water demand of 40.9 m^3/d based on their reported historical pumping data. This pumping is split between the two wells where FDG01 is assumed to be pumped for one day for every six days TW-2-13 is on. Existing and allocated demand presented in Table 3.4 have therefore been normalized to account for the variable pumping amounts. Both supply wells are represented in the model as being in the centre of a 12.5 x 12.5 m cell. This results in a convergent head loss correction of 0.01 m and 0.08 of additional drawdown at FDG01 and TW-2-13, respectively.

Table 3.4: Convergent well loss corrections

Well	Well Radius (m)	Cell Size (m)	Trans-missivity (m^2/d)	Existing Demand (m^3/d)	Allocated Quantity of Water (m^3/d)	Convergent Well Loss (m)	
						Existing	Allocated
FDG01	0.2	12.5	194.4	5.8	5.8	0.01	0.01
TW-2-13	0.1	12.5	259.2	35.1	35.1	0.08	0.08

3.2.2 Well Characterization Graphs

The Tier 3 assessment requires a detailed characterization of the municipal wells, identifying their operating constraints relative to water levels in the wells. Figure 3.9 and Figure 3.10 present well system characterization information for Greensville municipal wells FDG01 and TW-2-13, respectively. The purpose of these graphs is to illustrate:

1) Well construction and pump setting information: To the left of the graphs is a schematic presentation of land surface, well depth, well screen interval and pump setting within the well;
2) In-well water levels: Water levels measured in the pumped well (where available) from pressure transducers or air-line measurements. The operating levels may be lower than the adjacent aquifer due to well losses.

3) Aquifer Water Levels: Average of water levels measured in nearby observation wells that characterize the heads in the aquifer outside the pumped well;
4) Pumping History: Individual well and total wellfield production are displayed to aid in the assessment of the water level data;
5) The level selected for determining the Safe Additional Drawdown (discussed below) and the calculated value (Safe Water Level); and,
6) Simulated low water levels from the transient numerical simulations showing the maximum drawdown in the well observed during the 10-year drought simulation (Risk Assessment Minimum Water Level) (discussed in Chapter 5).

Water level data and pumping history for FDG01 are shown for the 2007 to 2012 portion of the study period in Figure 3.9. Seasonal fluctuations can be observed in both the pumped water level and the pumping rate. Water levels remain considerably above the minimum safe levels defined in Section 3.2.1. It should be noted with respect to aquifer water levels (Item 3) that the closest bedrock monitoring well (MW04-D) is located approximately 500 m away from the municipal supply well and is subject to topographic changes of approximately 5 m. As a result, the ability to evaluate aquifer levels outside of the pumped well is limited.

An equivalent characterization plot is also presented for TW-2-13 (Figure 3.10). TW-2-13 was completed in 2013 and has not yet been put into service. Consequently, no long-term pumping data are available. Logger data are shown from July, 2013 to July, 2015 for the pumping well and two nearby monitoring wells, TW1-13 located 170 m to the north and TW-3-13 located 90 m to the south. The bedrock wells exhibit minimal seasonal trend; however, a considerable drop in the water levels was observed in November of 2013 due to a 72-hour constant rate pumping test (Stantec, 2014)

3.3 *Provincial Permitting and Water Use Data*

3.3.1 MOE PTTW Database

The MOECC maintains a database of permits to take water (PTTW) issued under the Ontario Water Resources Act for water takings larger than 50,000 litres per day (L/d). The PTTW database includes information on the maximum permitted water taking rates along with the maximum number of hours per day and days per year of permitted operation. The permits are classified by primary and secondary purposes (e.g., water supply/municipal or agriculture/field and pasture crops). While PTTW holders are required to report water use, actual water use information is not part of this database.

Other reporting or operational limits are sometimes included on the issued permit documents. Copies of the permits for the municipal wells were found in the reports provided by the City of Hamilton. Copies of a small percentage of the non-municipal permits were found on the MOE environmental registry (http://www.ebr.gov.on.ca/ERS-WEB-External/) and retrieved. The reporting and operational limits are generally not captured in the database but occasionally appear in the comment fields for some permits.

A complete copy of the MOECC PTTW database was downloaded from the Land Information Ontario (LIO) website in December of 2016. The last permit entered in the database had a date of September 2016. Each permit may have multiple sources (e.g., more than one well), with multiple purposes (e.g., a golf course can have one source for irrigation and another for drinking water at the clubhouse), and may include both groundwater and surface water takings (e.g., a well source and a stream source). The database was filtered to select only permits within the vicinity of the study area, resulting in a database extract of 569 records, each representing a single water taking identified under a Source ID and belonging to a permit number.

A key unknown is whether water takings are continuing for expired permits; that is, are there some users unaware that their permits have expired? To account for these takers in this study, any permit with an expiration date after January 1st, 2012 that had not been superseded by another permit was assumed to still be an active taking. It should also be noted that temporary permits related to construction or pumping tests were not included in the analysis.

Of the 569 source records, 96 sources have been included in the updated Tier 3 study based on the criteria discussed above. The included sources represent 54 unique permits in the model. Permitted surface water and groundwater sources included in the study are tabulated in Section 3.3.5.

3.3.2 MOE WTRS Database

Under the Ontario Water Resources Act, all PTTW holders are now required to report actual daily water takings to the MOECC for each source listed in the permit. To facilitate compliance, the MOECC developed the Water Taking Reporting System (WTRS) to accept self-reported information electronically over the Internet. Submission of data by e-mail or paper reporting forms is also acceptable.

There is no explicit link between PTTW and WTRS databases; however, matching can be done based on the PTTW permit numbers, the location coordinates, and the source description. Finding a match can be difficult, however, because permit numbers are often changed when the permit is renewed or when it is reissued with changes to conditions or water use rates. Location coordinates are also not always consistent between the two data sources. Furthermore, source descriptions are very brief and often generic (e.g., "well" or "pond") making it more difficult to re-link new permit numbers to older ones. It should be noted that there appear to be improvements in recently issued permits which tend to have more descriptive SourceIDs.

Of the 96 PTTW sources considered in this study, 66 sources could not be matched to any WTRS records in 2013 and 2014 for various reasons:

- Some of these permits (21) were newer than the most recent WTRS extract. These sources were assigned the maximum permitted pumping rate unless they replaced an older permit, in which case, the average reported rate from the previous permit was used.
- User failed to report takings to the WTRS in 2013 or 2014 regardless of whether or not they used water. This problem occurs because some water users are unaware that they have to report zero usage. For this study, we assumed that the user did take water and failed to report. If data were available, these sources were assigned the average of their previous reported takings. If no previous takings were reported, they were assigned the maximum permitted rate. An exception was made when the permitted water user reported water takings under the same permit number but for a different source; in this case, they were considered to be trustworthy in their reporting and the non-reported takings were left at zero.
- A match was not found between the PTTW and WTRS records due to some of the difficulties discussed above (e.g., mismatched source ID). In this circumstance, the maximum permitted rate was assumed.

The difficulties experienced in trying to accurately match the PTTW and WTRS databases have also been well documented in previous Tier 3 studies (e.g., Earthfx, 2016). It's important to note that WTRS compliance has improved steadily over the years and the quality of data will likely continue to improve.

3.3.3 MOE WWIS Database

The third key source of provincial information is the MOECC Water Well Information System (WWIS). This database summarizes well construction information reported by well drillers for water wells drilled in the province. The WWIS provides valuable information on the well location, well depth, screen setting, static water level, specific capacity, well yield and pump capacity, and well purpose. Each well is assigned a unique alphanumeric Well ID. The well information can be used to determine the aquifer from which the groundwater takings are drawn. Digital copies of the paper records (for data verification and location sketches) can be obtained from the MOECC website.

There is no direct link between the information in the WWIS and the PTTW databases although *occasionally* there is a reference to a specific Well ID in the PTTW Source description. Coordinates supplied for the permit location usually do not match exactly with those in the WWIS database. WWIS owner names and PTTW client names, which frequently change, also do not match exactly. The well purpose code categories used in the WWIS are similar but not identical to the PTTW records, and there are purpose codes in the PTTW database that are not represented in the WWIS data. However, using GIS techniques, it is sometimes possible to visually link the source location with a well record. An automated approach was used in this study to link the PTTW sources to the appropriate well records and thereby determine which hydrostratigraphic unit was being pumped (See Section 3.3.7).

3.3.4 Methodology for Compiling Provincial Water Use Data

The PTTW and WTRS databases described in Section 3.3 were used as the primary sources of groundwater and surface water use estimates. Some initial screening and reconciliation of the data was required, including:

- Water takings in the MOECC PTTW database are classified as "ground water", "surface water", or "surface and ground water". An example of the latter would be a shallow dug-out pond, where the pond is assumed to be excavated into the shallow water table. According to the MOECC PTTW database, 69 of the active permit sources were classified as groundwater, 18 as surface water, and the remaining 9 were classified as surface and groundwater. For the groundwater sources, approximately half listed "pond", "dugout", or other surface water source as their Source ID. To rectify inaccuracies in the permit source classifications, a search was conducted on all "surface water" and "surface and groundwater" locations for Source IDs with key words implying a groundwater source (e.g., "well", "pw", "sand point"). Any inconsistencies were corrected by reassigning the source to the "groundwater" classification. Similarly, all groundwater sources were filtered for key words relating to surface water sources, including "Spencer", "River", "Creek", "tributary", and "on-stream"; any such occurrences in the database were reviewed and reassigned as "surface water" takings.

- Reported takings within the WTRS datasets were linked to the sources in the PTTW database using a combination of the permit number and the Source ID. Within the 2013 and 2014 datasets, the coordinates are also included in the WTRS database (northings and eastings) allowing for additional link criteria when matching PTTW sources to their WTRS datasets.

- Of the active sources, many can be considered to be temporary and/or non-consumptive water uses. This includes permits with the primary or secondary purpose listed as conservation, dams and reservoirs, dredging, pipeline testing, pumping tests, and wildlife recreation. These have been flagged in the permit tables and omitted from the water use estimation and Tier 3 analyses.

3.3.5 Spatial Distribution of Water Takings

The location of active groundwater and mixed surface and groundwater permit sources, based on the PTTW and WTRS records, are shown in Figure 3.11. These records are also summarized in Table 3.5 and Table 3.6 for groundwater and surface water permits, respectively. Note that Table 3.5 contains both groundwater sources (i.e., wells) and mixed groundwater/surface water sources (i.e., dugout ponds). The majority of the permitted sources are located in the western portion of the model area, with additional permits sparsely distributed across the remaining model area. Active surface water permitted sources are also shown in Figure 3.11. Most permits are also concentrated in the western portion of the model area and draw from Middle and West Spencer Creek.

Two quarry-related water taking permits within the study area belong to the Flamboro Quarry (operated by Dufferin Aggregates) and the Lafarge operated North Quarry, South Quarry and Processing Area. The details of these permitted quarry takings are summarized in Table 3.7. The permits do not explicitly represent water takings such as a well or a stream diversion. Instead they represent combined surface and groundwater takings because surface water runoff and groundwater leakage are both collected and stored in sumps in the quarry floors. Water is taken from the sumps to control local groundwater heads within the quarry and for use in aggregate processing. The Lafarge permit also includes limits to off-site discharges (i.e., to Truck Fill, Beagle Club and Railway Cut) as well as to on-site circulation of water between sump ponds (i.e., North Quarry Sump and South Quarry Sump).

Table 3.5: Summary of simulated permitted groundwater takings.

MOE PTTW Number	Source Name	Primary Purpose	Secondary Purpose	Easting (m)	Northing (m)	Maximum Daily Permitted Taking (m^3/d)	Number of Permitted Days per Year	Mean Annualized Reported Daily Demand (m^3/day)	Reported Maximum Daily Demand (m^3/day)	Consumptive Use Factor
02-P-2003	Dugout pond	Agricultural	Nursery	585676	4803036	750	15	0	0	0.9
	Dugout pond	Agricultural	Nursery	585414	4802342	750	15	0	0	0.9
03-P-2180	Dugout pond	Agricultural	Field and Pasture Crops	576589	4797415	2,087	50	46	873	0.8
03-P-2181	Dugout Pond	Agricultural	Field and Pasture Crops	574266	4795481	2,087	50	6	262	0.8
03-P-2210	Dugout Pond	Agricultural	Field and Pasture Crops	575879	4796517	2,087	50	22	570	0.8
03-P-2390	Dugout pond	Agricultural	Field and Pasture Crops	579787	4801351	1,637	150	0	0	0.8
03-P-2395	Dugout Pond	Agricultural	Field and Pasture Crops	579846	4802598	1,637	180	0	0	0.8
0400-AB4J3S	Kikkert Pond	Agricultural	Nursery	576200	4791800	445	20	0	0	0.9
0661-9HAN4F	PW1	Commercial	Golf Course Irrigation	575170	4794410	491	214	210	314	0.7
	Irrigation Pond	Commercial	Golf Course Irrigation	575460	4793931	3,240	214	745	2,974	0.7
1033-7DPPAH	Pond	Agricultural	Nursery	579500	4801700	1,091	200	461	1,069	0.9
	Well	Agricultural	Nursery	579527	4801583	274	200	54	274	0.9
1245-9MHJ35	Pond 1	Agricultural	Other - Agricultural	572815	4792740	566	40	82	198	0.8
1333-968HG7	Chris' s Pond	Agricultural	Field and Pasture Crops	576846	4798358	2,087	50	917	1,744	0.8
1586-9SPK7B	PW 1	Commercial	Golf Course Irrigation	580170	4801559	368	185	0	0	0.7
	Pond 1	Commercial	Golf Course Irrigation	580050	4801508	398	215	211	315	0.7
1767-967QGH	Dave's Pond	Agricultural	Field and Pasture Crops	574321	4795574	2,084	50	447	2,577	0.8
2047-7D4PL7	Westover Farm Pond	Agricultural	Nursery	575010	4795669	450	4	1	307	0.9
2066-A3YQJ4	PW-1	Agricultural	Other - Agricultural	580910	4801997	111	365	44	110	0.8
	PW-2	Agricultural	Other - Agricultural	580858	4801988	52	365	31	51	0.8
	PW-3	Agricultural	Other - Agricultural	580949	4801993	125	365	46	125	0.8
	PW-4	Agricultural	Other - Agricultural	580932	4801985	170	365	52	154	0.8
2266-8FENER	Irrigation Pond	Agricultural	Field and Pasture Crops	581655	4800745	818	365	64	313	0.8
2476-9F5KM6	Greensville Well FDGO1	Water Supply	Municipal	582300	4792538	197	365	41	126	1
Greensville TW-2-13	Greensville (New) Well TW-2-13	Water Supply	Municipal	582377	4792896	130	366	0	0	1
2666-8AELTJ	Farm 1- Well # 2 (88-P-2009)	Agricultural	Field and Pasture Crops	576014	4794679	353	365	58	184	0.8

MOE PTTW Number	Source Name	Primary Purpose	Secondary Purpose	Easting (m)	Northing (m)	Maximum Daily Permitted Taking (m^3/d)	Number of Permitted Days per Year	Mean Annualized Reported Daily Demand (m^3/day)	Reported Maximum Daily Demand (m^3/day)	Consumptive Use Factor
	Farm 1 - Well # 3 (00-P-2388)	Agricultural	Field and Pasture Crops	576059	4794688	137	365	53	218	0.8
	Farm 2 TW1 (5428-6FUQN8)	Agricultural	Field and Pasture Crops	576453	4794732	504	365	133	510	0.8
3004-AD7MSR	Home Pond	Agricultural	Field and Pasture Crops	575874	4796589	2,087	80	0	0	0.8
3105-6P3LES	Fire Water Pond	Miscellaneous	Other - Miscellaneous	573851	4797825	75	6	0	1	1
	Fire Water Pond Make-up water well	Miscellaneous	Other - Miscellaneous	573870	4797821	75	6	1	34	1
3148-AB5M54	Pond 1	Agricultural	Nursery	579766	4801571	750	200	261	716	0.9
3708-9Z6PUA	Well #1	Industrial	Food Processing	580014	4793474	524	365	130	261	1
	Well #2	Industrial	Food Processing	579987	4793481	182	365	30	179	1
3752-8FQNWN	Ponds 1, 2, 3	Agricultural	Other - Agricultural	576528	4793127	1,146	275	246	511	0.8
4176-9T6QGR	Well PW1-98	Commercial	Golf Course Irrigation	582826	4790745	1,637	154	286	1,637	0.7
	Pond 3	Commercial	Golf Course Irrigation	582631	4790543	1,637	214	204	1,618	0.7
5032-7DDLBA	Pond D (lined)	Agricultural	Nursery	572790	4794105	500	120	0	159	0.9
	Well (WWR #6809976)	Agricultural	Nursery	573027	4794458	131	120	2	109	0.9
5112-83SJ4Q	Dugout Pond	Industrial	Aggregate Washing	573870	4794550	4,910	180	995	1,294	0.25
5651-92BRWG	Pond	Agricultural	Field and Pasture Crops	584291	4801487	909	50	0	0	0.8
5866-9QLL32	Well 1	Agricultural	Market Gardens / Flowers	583725	4797225	1,309	210	0	0	0.9
	Pond 1	Agricultural	Market Gardens / Flowers	583793	4797225	1,364	210	832	1,100	0.9
	Pond 2	Agricultural	Market Gardens / Flowers	583745	4797425	1,364	210	0	0	0.9
5884-8YXLJW	PW-1	Commercial	Golf Course Irrigation	575200	4794180	491	153	22	216	0.7
	Irrigation Pond	Commercial	Golf Course Irrigation	575466	4793925	1,287	180	625	1,285	0.7
	Clubhouse Well	Water Supply	Other - Water Supply	575429	4793822	43	365	0	0	0.2
6573-ACWHZD	Weston Pond	Agricultural	Field and Pasture Crops	575262	4794988	1,905	80	0	0	0.8
6781-8SNQ9Q	Krulikoski Pond	Agricultural	Nursery	585629	4803033	518	60	0	0	0.9
	Sproule Pond	Agricultural	Nursery	585290	4802418	518	60	0	0	0.9
6808-9PGP4V	Griffith Farm Pond A	Agricultural	Nursery	573648	4791020	900	214	0	0	0.9
	Griffith Farm Pond B	Agricultural	Nursery	573598	4791019	900	214	0	0	0.9
	Griffith Farm Old Pond	Agricultural	Nursery	573886	4790958	400	214	0	0	0.9

MOE PTTW Number	Source Name	Primary Purpose	Secondary Purpose	Easting (m)	Northing (m)	Maximum Daily Permitted Taking (m^3/d)	Number of Permitted Days per Year	Mean Annualized Reported Daily Demand (m^3/day)	Reported Maximum Daily Demand (m^3/day)	Consumptive Use Factor
7233-6Z3LQW	Kikkert Pond	Agricultural	Nursery	576200	4791800	445	20	1	285	0.9
7620-98RR56	Well	Agricultural	Market Gardens / Flowers	584105	4794603	173	140	0	0	0.9
	Pond 1	Agricultural	Other - Agricultural	583957	4796070	1,100	150	0	0	0.8
	Pond 2	Agricultural	Other - Agricultural	584072	4795705	1,100	180	0	0	0.8
7825-84BPPQ	Well B	Water Supply	Campgrounds	579397	4800756	262	184	27	169	0.2
8078-83KL59	New Supply Well WR 6813823	Water Supply	Communal	576900	4795300	150	365	0	0	0.2
	Park Well WR 6807806	Water Supply	Communal	576888	4795040	150	365	0	0	0.2
8488-8SLQ3S	Olga's Pond	Agricultural	Nursery	583608	4802934	518	60	0	0	0.9
8664-9TUQ4H	Pond A	Agricultural	Field and Pasture Crops	577101	4793786	996	120	46	65	0.8
	Pond BC	Agricultural	Field and Pasture Crops	577101	4793786	996	120	175	460	0.8
	Pond E	Agricultural	Field and Pasture Crops	577101	4793786	996	120	0	0	0.8
	Well 141	Agricultural	Field and Pasture Crops	577270	4793424	85	365	0	0	0.8
	Well 142	Agricultural	Field and Pasture Crops	577270	4793424	85	365	0	0	0.8
8777-8BZT2W	Pond A	Agricultural	Field and Pasture Crops	575386	4792307	2,726	100	515	6,352	0.8
	Pond B	Agricultural	Field and Pasture Crops	575150	4791463	2,726	100	203	2,298	0.8
	Washplant Well	Agricultural	Field and Pasture Crops	575001	4792167	197	365	58	144	0.8
8811-AB6QAH	Well #1	Water Supply	Other - Water Supply	578890	4794360	328	365	0	0	0.2
	Well #2	Water Supply	Other - Water Supply	578891	4795075	328	365	0	0	0.2
8868-7UPJ4P	1A	Agricultural	Nursery	577267	4793246	540	244	210	511	0.9
	1B	Agricultural	Nursery	577176	4793084	540	244	10	318	0.9
	2A	Agricultural	Nursery	577464	4792859	1,533	244	208	566	0.9
	2B	Agricultural	Nursery	577372	4792839	730	244	350	725	0.9
	2C	Agricultural	Nursery	577266	4792824	306	244	0	0	0.9
	3A	Agricultural	Nursery	577830	4792967	680	244	139	601	0.9
	3B	Agricultural	Nursery	577672	4792912	540	244	101	467	0.9

Table 3.6: Summary of simulated permitted surface water takings.

MOE PTTW Number	Source Name	Primary Purpose	Secondary Purpose	Easting (m)	Northing (m)	Maximum Daily Permitted Taking (m^3/d)	Number of Permitted Days per Year	Mean Annualized Reported Daily Demand (m^3/day)	Reported Maximum Daily Demand (m^3/day)	Consumptive Use Factor
2435-8XGPYV	Pond on Grindstone Creek	Agricultural	Field and Pasture Crops	585546	4800260	273	200	185	237	1
4082-8GSR7D	Hanes Pond	Agricultural	Other - Agricultural	573575	4794367	970	30	0	0	1
5032-7DDLBA	Ponds A and B (intermittent)	Agricultural	Nursery	572831	4794308	1,361	120	0	0	1
	Pond C (intermittent tributary of West Spencer)	Agricultural	Nursery	573055	4794223	500	120	0	0	1
5415-7JXSB7	West Farm Pond	Agricultural	Nursery	571916	4793612	450	5	0	0	1
5725-8NVQRL	Thompson Pond	Agricultural	Other - Agricultural	571632	4793826	654	20	203	211	1
6012-7QARLZ	Spencer Creek	Agricultural	Nursery	576307	4792881	2,700	365	374	2,544	1
7788-8GKMGE	Pond 1	Agricultural	Other - Agricultural	579200	4791800	455	20	147	234	1
	Pond 2	Agricultural	Other - Agricultural	579100	4791600	455	20	0	0	1
8168-8FUQW7	Reservoir A	Agricultural	Other - Agricultural	576188	4793185	1,188	275	480	545	1
	Reservoir B	Agricultural	Other - Agricultural	576137	4793083	768	20	0	0	1
8606-9RWHE9	Spencer Creek	Commercial	Golf Course Irrigation	577820	4792004	1,178	184	45	981	1
	Storage Pond	Commercial	Golf Course Irrigation	577865	4791638	1,309	184	66	1,309	1
	Irrigation Pond	Commercial	Golf Course Irrigation	577788	4791238	1,746	184	82	1,033	1
8704-7JXNYM	Christie Reservoir (Spencer Creek)	Agricultural	Field and Pasture Crops	577385	4792495	763	273	437	757	1
8777-8BZT2W	Spencer Creek	Agricultural	Field and Pasture Crops	575578	4792720	2,726	70	446	2,700	1
8841-73AL2E	Pond #22	Agricultural	Other - Agricultural	584417	4803109	518	30	0	0	1
	Pond #23	Agricultural	Other - Agricultural	584392	4802744	518	30	0	0	1

Table 3.7: Summary of Lafarge and Flamboro Quarry PTTW.

MOE PTTW Number	Source Name	Primary Purpose	Secondary Purpose	Easting (m)	Northing (m)	Maximum Daily Permitted Taking (m^3/d)	Number of Permitted Days per Year	Mean Annualized Reported Daily Taking (m^3/day)	Reported Maximum Daily Taking (m^3/day)	Consumptive Use Factor
1101-7K9QQL (LaFarge Quarry)	South Quarry Sump	Dewatering	Pits and Quarries	582166	4794426	50,406	365	5,470	10,135	0.25
	North Quarry Sump	Dewatering	Pits and Quarries	580920	4796354	15,720	365	11,801	18,557	0.25
	Truckfill	Dewatering	Pits and Quarries	–	–	3,032	365	2,881	–	0.25
	Railway Cut	Dewatering	Pits and Quarries	–	–	17,568	365	10,289	–	0.25
	Beagle Club	Dewatering	Pits and Quarries	–	–	1,368	365	456	–	0.25
6581-A48RAW (Flamboro Quarry)	Sump 1	Dewatering	Pits and Quarries	579659	4795966	17,502	365	4,405	13,618	0.25
	Sump 1 (Ready-Mix)	Industrial	Other - Industrial	579659	4795966	720	365	0	0	0.25
	Sump 2	Dewatering	Pits and Quarries	579302	4796066	0	365	0	0	0.25

Notes: 1) Truck Fill, Railway Cut and Beagle Club sources are not listed in the PTTW database. Data for 2006 to 2012 were obtained from LaFarge and Flamboro Quarry during the initial Tier 3 study (Earthfx, 2014). Water takings were averaged over that period and do not reflect updated rates from the WTRS extract.

3.3.6 Permitted Water use by Category

As noted above, permit sources are categorized by primary and secondary use. The model area is largely rural with the exception of two large quarry operations in the central portion of the study area and the urban settlements of Greensville and Dundas in the south. Irrigation of specialty crop is known to be prevalent in the area (de Loë et al., 2001) and is reflected in the large number of permitted agricultural sources (57 groundwater sources and 15 surface water sources). A breakdown of permitted water takings by primary and secondary use are provided in Table 3.8. Location of groundwater and surface water permits by primary source are shown in Figure 3.14 and Figure 3.15, respectively. Note that the below table and figures do not consider quarry operations. While permitted quarry-related takings are significant (See Table 3.7), they are not explicitly represented in the model as takings. Instead, discharges are estimated based on the simulated inflows into the pits. See Section 4.4.1.3 or Earthfx (2014) for more details.

Table 3.8: Permitted sources categorized by primary and secondary purpose.

Primary Purpose	Secondary Purpose	Study Area	
		Groundwater	*Surface Water*
Agricultural	Field and Pasture Crops	22	3
	Mkt. Garden/Flowers	4	0
	Nursery	23	4
	Other Agricultural	8	8
Commercial	Golf Courses	8	3
Industrial	Aggregate Washing	1	0
	Food Processing	2	0
Miscellaneous	All	2	0
Water Supply	Municipal	2	0
	All Other	6	0
Totals		**78**	**18**

Table 3.9 compares maximum permitted takings and reported water use within the model area by source and specific use. Tabulated values are summarized in the pie charts below (Figure 3.2), which show that agricultural use - primarily for nursery farms and field and pasture crops - dominate both permitted and reported takings. Excluding quarry-related takings, the total maximum permitted takings are about 37,000 m^3/d, while total reported takings are only about 2,370 m^3/d. While it is understood that agricultural needs can vary widely from year to year, there appears to be a general tendency to have permitted maximum takings far in excess of actual needs. It is our understanding that the MOECC has been attempting in recent years to re-issue permits with lower, more realistic limits. The comparison also shows the importance of using the WTRS data in assessing water use rather than the PTTW values, which were used in many of the previous studies and likely significantly over-estimate actual water use.

Table 3.9: Permitted and reported water use within the model area categorized by primary and secondary purpose for 2013 to 2014.

Source	Purpose	Specific Use	Annualized Permitted Taking (m³/d)	Reported 2013 Average Demand (m³/d)	Reported 2014 Average Demand (m³/d)	Average Reported Demand (m³/d)
Groundwater	*Agricultural*	Field and Pasture Crops	5,269	259	293	276
		Market Gardens / Flowers	2,389	0	0	0
		Nursery	5,520	513	378	445
		Other - Agricultural	1,515	0	3	1
	Commercial	Golf Course Irrigation	5,099	115	125	120
	Industrial	Aggregate Washing	2,422	324	399	361
		Food Processing	706	0	0	0
	Water Supply	Municipal	197	38	35	36
		Communal	300	0	0	0
		Campgrounds	132	20	23	22
		Other	699	0	0	0
	Miscellaneous	All	2	0.08	0.03	0.05
	Total		**24,249**	**1,269**	**1,255**	**1,262**
Mixed GW/SW	*Agricultural*	Field and Pasture Crops	3,294	320	288	304
		Nursery	762	283	327	305
		Other - Agricultural	863	61	76	69
	Total		**4,920**	**664**	**691**	**678**
Surface Water	*Agricultural*	Field and Pasture Crops	1,243	261	179	220
		Nursery	3,318	142	155	149
		Other - Agricultural	1,188	60	75	68
	Commercial	Golf Course Irrigation	2,134	0	0	0
	Total		**7,883**	**463**	**410**	**437**
Grand Total			***37,052***	***2,396***	***2,356***	***2,376***

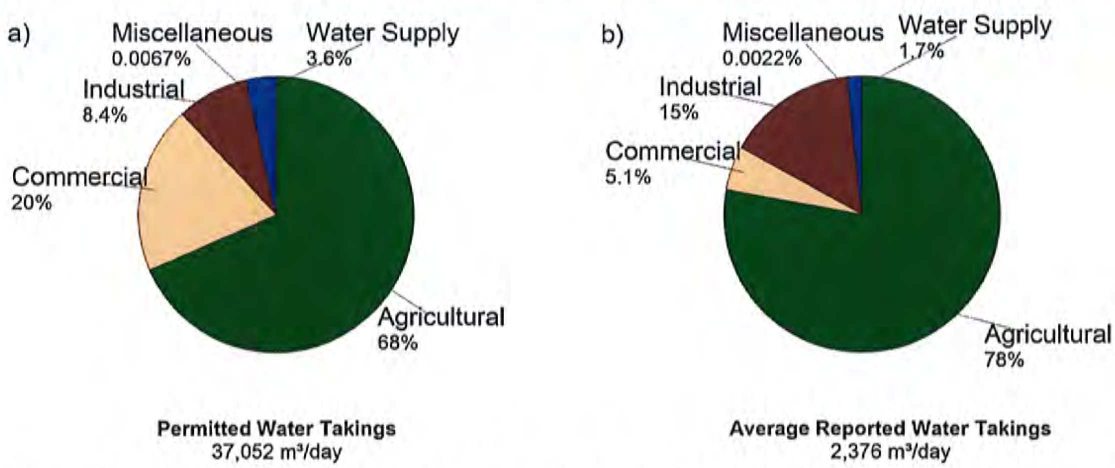


Figure 3.2: a) Permitted water takings and b) average reported takings (2013 to 2014) in the model area by primary use.

3.3.7 Water use by Aquifer

Neither the PTTW data nor the WTRS data provide any information linking a specific groundwater source to a known hydrostratigraphic unit. Information regarding details of well construction are available from the MOECC WWIS. However, these data, as previously discussed, lack explicit linkages to the PTTW and WTRS datasets. A significant effort was made to link these data in order to assign pumping wells to a specific hydrostratigraphic unit within the model. To accomplish this, a process was developed to jointly process the PTTW and WWIS databases, along with the hydrostratigraphic model layers within the Tier 3 model area. The approach relied upon a tiered approach, with increasing uncertainty, to assign groundwater permit sources to hydrostratigraphic units.

As illustrated in Figure 3.3, the highest level of certainty occurred where a perfect match could be made between a PTTW Source ID and a pumping well location. This was the case for the municipal wells with a number of years of detailed reporting and wellfield studies available to confirm the details. The second best match occurred when the PTTW Source ID could be linked to a specific WWIS well record number; however, without corroborating supplemental information such as a wellfield report, the location was based on PTTW and WWIS database locations. This tier has a slightly higher level of location uncertainty.

The third level of match was made when the PTTW Source ID was matched to the closest WWIS well within a 100 m radius. Finally, a fourth level of permit assignment was made when no well was located within 100 m. In this final case, the PTTW Source was assigned to the aquifer most commonly used within an 800 m radius of the source.

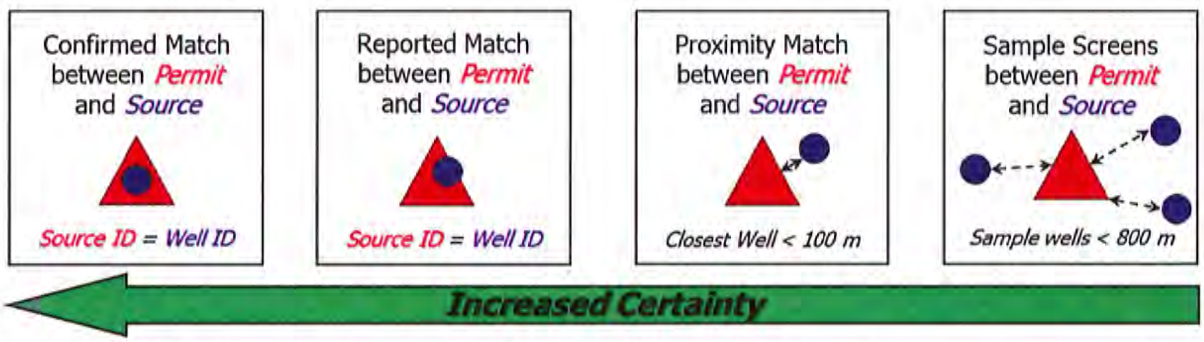


Figure 3.3: Schematic of tiered approach used to link groundwater permit sources to water well records, and related levels of certainty.

The distribution of groundwater sources, along with their interpreted source-type (bedrock or overburden) is presented in Figure 3.16. Table 3.10 contains reported average takings and simulated takings from individual hydrostratigraphic layers within the model area. The values in Table 3.10 include both groundwater and shallow water-table sources (i.e., dugout ponds). A review of the shallow overburden stratigraphic layers below each of the dugout ponds was undertaken so that these ponds could be assigned to the shallowest significant aquifer unit.

The majority of the reported water takings for agricultural use are from the surficial geology unit. The high concentration of water users drawing from the overburden in the western portion of the model mainly corresponds to the Norfolk Sand Plain physiographic region. This is shallowest unit making it the most economical option for irrigation applications. The Mackinaw Interstadial unit, weathered bedrock and Guelph Formation aquifers also have reported takings during 2013 and 2014 and are well represented in the simulated takings. While volumes taken vary, they are generally consistent between 2013 and 2014. Reported takings are considerably less than the simulated values. This may be a reflection of poor compliance with the WTRS, as discussed in Section 3.3.2. Some simulated takings were also linked to the Halton Till, Eramosa Formation, Vinemount Formation and Gasport Formations though these are generally small by comparison to the other units.

Table 3.10: Reported average water use by aquifer for 2013 and 2014.

Hydrostratigraphic Unit	WTRS Average Reported Water Taking (m^3/d)		Simulated Taking (m^3/d)
Year	2013	2014	
Surficial Material	1,240	1,159	17,692
Halton Till	--	--	286
Mackinaw Interstadial	283	327	3,211
Wentworth Till	--	--	--
Weathered Bedrock	142	150	3,397
Guelph Formation	230	310	1,485
Eramosa Formation	--	--	30
Vinemount	--	--	31
Goat Island Formation	--	--	--
Gasport/Amabel Formation	--	--	470

3.4 *Private Domestic Wells*

In addition to the permitted takings, a number of non-permitted takings of less than 50 m^3/d were included in the model to represent residential, commercial and industrial takings related to existing developments in the vicinity of the Greensville well. Following the installation of the Greensville municipal well in 1972, all new domestic water supply in the Greensville area was developed based on private wells. The cumulative impact of increased domestic water use was the subject of the Greensville Rural Settlement Area Groundwater Modeling Assessment (Earthfx, 2010). Well locations and pumping rates for the planned wells were estimated for the Rural Assessment study and incorporated into the Tier 3 model.

For the non-municipal residential wells, a GIS coverage was created whereby every property was assigned one "well" for each single family dwelling on it. Assumed locations of the non-permitted private wells are presented in Figure 3.11. A total of 950 wells were simulated and another 317 were identified as potential future locations. It is important to note that these "wells" do not represent actual existing or proposed supply well locations, rather a point representing a single family water use equivalence in close proximity to the dwellings. Properties with two or more dwellings were assigned two or more "wells". Industrial and commercial properties were assigned a number of "wells" depending on their size and operation. The "wells" were assumed to be sourcing water from the same bedrock interface aquifer used by the municipal well represented by model Layer 5. Each source was assigned a modelled pumping rate of 0.2 m^3/d, representing the consumptive portion of the private domestic water use at the well. This representation is consistent with the earlier City of Hamilton Greensville Rural Settlement Area Groundwater Modeling Assessment (Earthfx, 2010) study.

3.5 *Consumptive Demand*

"Consumptive demand" is defined in the Water Budget Guide (MNR, 2011) as the amount of water taken from a water source and not returned locally to the same source of water within a reasonable amount of time. While the actual takings can be determined from the WTRS data, estimating consumptive use is difficult and requires information on the locations, rates, and timings of the return flows.

It is likely that a portion of the water extracted by the Greensville municipal wells is returned to the soil zone through septic systems and activities such as lawn watering. However, to be conservative, municipal takings were treated as 100% consumptive in the Tier 3 analyses.

The PTTW database classifies the water takings based on general and specific purposes (e.g., Agriculture (Field and Pasture Crops) or Dewatering (Pits and Quarries)). Table 3-1 in the Water Budget Guide lists suggested consumptive use factors for different types of water takings. Table 3 in Appendix B of the Water Budget Guide lists an alternate set of suggested consumptive use factors specific to the primary and secondary purposes in the PTTW database. For this study, the Table 3 (Appendix B) consumptive use factors were applied to the actual daily takings in the WTRS data for groundwater sources (other than quarries) and used in the transient simulations. Table 3.5 presents a summary of all of the permitted groundwater takings and the consumptive factors simulated.

For the steady-state MODFLOW-only simulations, average pumping rates were applied. Averages of the reported actual takings from the WTRS database were used where available. Where no WTRS data were available, a conservative approach was applied whereby average daily withdrawals were assumed equal to the maximum permitted takings unless it was determined that they intended to report zero water use (See Section 3.3.2). The consumptive use rates used in the model for steady-state simulations are provided in Table 3.11.

There are 18 non-quarry related surface water takings in the study area represented by 12 unique permits. These are summarized in Table 3.6. To be conservative, a consumptive use factor of 1 was assigned to all surface water takings. Surface water takings are often related to diversions from a stream or lake into a storage pond. The storage ponds associated with these permitted surface water takers are assumed to be lined (to prevent infiltration of stored water) and the secondary taking from the ponds was not represented in the model to avoid double counting. Actual takings from the surface water bodies were estimated from the WTRS database for both the steady-state and transient simulations, wherever possible. The steady-state simulated rates are summarized in Table 3.12.

Two operating quarries are located within the study area; Flamboro Quarry (operated by Dufferin Aggregates) and the Lafarge Dundas Quarry (Table 3.7). The integrated GSFLOW model accounted for the full water balance in each quarry including both surface water and groundwater inflows and outflows. Both quarry operations discharge to surface water features. The two operations are adjacent to each other and are located to the north and west of the Greensville municipal wells. The Lafarge Dundas Quarry is the larger and deeper of the operations; it consists of the active North Quarry, the inactive South Quarry and the Processing Area (also a discontinued quarry). The North Quarry has been in development since 1996 and extraction is currently proceeding on the second of three planned benches. The current quarry floor is at an elevation of 230 to 231 masl.

Limited extraction has occurred in the South Quarry since 1990 and the site is mostly used for aggregate storage and accommodates a series of siltation ponds. An aggregate wash plant is located in the Processing Area which discharges to siltation ponds in the South Quarry. All three areas are actively dewatered, with the accumulated flow sent to a pond in the Processing Area. Water from this pond supplies the aggregate wash plant and the remainder is sent south through a disused railway cut into Lower Spencer Creek via Logie's Creek.

The smaller Flamboro Quarry, located west of the Lafarge operation is currently excavated to an elevation of 240 to 241 masl. The Flamboro Quarry is also actively dewatering, with the collected water discharged to the southwest to a tributary of Middle Spencer Creek.

Volumes discharged are primarily dependant on seasonal conditions and can vary significantly from year to year. Actual discharge data was provided by the quarries. As a result of dewatering operations, an average of 11,801 m^3/day of flow is directed south to the process area pond from Lafarge's North Quarry; 5,470 m^3/day of flow is discharged from the South Quarry. An average of 10,300 m^3/day is discharged offsite to the railway cut from the process pond. Dewatering operations at the Flamboro Quarry discharge 4,405 m^3/day offsite.

The takings were treated in the model as surface water "diversions" from the quarry ponds or sumps. However, it should be recognized that a portion of the discharge is from rainfall and direct runoff into the quarry and another portion is due to groundwater seepage through the quarry sides and floor, and into the quarry sump. Consumptive use factors were not applied because the processes that would both return and remove the diverted water from the surface water and groundwater systems are represented in the integrated model.

Table 3.11: Consumptive permitted groundwater takings in the steady-state simulations.

MOE Permit No.	Source ID	Layer	Row	Column	Consumptive Factor	Simulated Rate (m^3/day)
02-P-2003	Dugout pond	1	7	169	0.9	675
02-P-2023	Dugout pond	1	11	168	0.9	675
03-P-2180	Dugout pond	1	48	28	0.8	37
03-P-2181	Dugout Pond	1	87	17	0.8	5
03-P-2210	Dugout Pond	1	66	25	0.8	17
03-P-2390	Dugout pond	4	16	56	0.8	1,310
03-P-2395	Dugout Pond	3	10	57	0.8	1,310
0400-AB4J3S	Kikkert Pond	1	207	26	0.9	401
0661-9HAN4F	PW1	6	108	21	0.7	147
	Irrigation Pond	1	118	23	0.7	521
1033-7DPPAH	Pond	3	14	52	0.9	415
	Well	3	15	53	0.9	49
1245-9MHJ35	Pond 1	1	167	10	0.8	65
1333-968HG7	Chris' s Pond	1	35	30	0.8	734
1586-9SPK7B	PW 1	5	15	64	0.7	0
	Pond 1	5	15	61	0.7	148
1767-967QGH	Dave's Pond	4	85	17	0.8	357
2047-7D4PL7	Westover Farm Pond	1	83	21	0.9	1
2066-A3YQJ4	PW-1	11	13	79	0.8	35
	PW-2	7	13	78	0.8	25
	PW-3	11	13	79	0.8	37
	PW-4	11	13	79	0.8	41
2266-8FENER	Irrigation Pond	1	19	94	0.8	51
2476-9F5KM6	Greensville Well FDGO1	5	183	115	1	7
Greensville TW-2-13	Greensville (New) Well TW-2-13	6	155	122	1	34
2666-8AELTJ	Farm 1- Well # 2 (88-P-2009)	3	103	26	0.8	47
	Farm 1 - Well # 3 (00-P-2388)	6	103	26	0.8	42
	Farm 2 TW1 (5428-6FUQN8)	6	102	28	0.8	106
3004-AD7MSR	Home Pond	1	65	25	0.8	1,670
3105-6P3LES	Fire Water Pond	4	40	15	1	0
	Fire Water Pond Make-up water well	4	40	15	1	1
3148-AB5M54	Pond 1	3	15	56	0.9	235
3708-9Z6PUA	Well #1	6	127	61	1	130
	Well #2	7	127	60	1	30
3752-8FQNWN	Ponds 1, 2, 3	1	136	28	0.8	197
4176-9T6QGR	Well PW1-98	1	219	138	0.7	200
	Pond 3	3	221	134	0.7	143
5032-7DDLBA	Pond D (lined)	1	114	9	0.9	0

MOE Permit No.	Source ID	Layer	Row	Column	Consumptive Factor	Simulated Rate (m³/day)
	Well (WWR #6809976)	2	107	11	0.9	2
5112-83SJ4Q	Dugout Pond	1	105	15	0.25	249
5651-92BRWG	Pond	3	15	160	0.8	727
5866-9QLL32	Well 1	5	52	155	0.9	0
	Pond 1	1	52	155	0.9	748
	Pond 2	1	48	155	0.9	0
5884-8YXLJW	PW-1	1	113	21	0.7	15
	Irrigation Pond	1	118	23	0.7	438
	Clubhouse Well	6	120	23	0.2	0
6573-ACWHZD	Weston Pond	1	97	22	0.8	1,524
6781-8SNQ9Q	Krulikoski Pond	1	7	169	0.9	466
	Sproule Pond	1	10	167	0.9	466
6808-9PGP4V	Griffith Farm Pond A	1	216	14	0.9	810
	Griffith Farm Pond B	1	216	13	0.9	810
	Griffith Farm Old Pond	1	217	15	0.9	360
7233-6Z3LQW	Kikkert Pond	1	207	26	0.9	1
7620-98RR56	Well	5	104	159	0.9	156
	Pond 1	1	75	157	0.8	880
	Pond 2	1	82	158	0.8	880
7825-84BPPQ	Well B	5	19	51	0.2	5
8078-83KL59	New Supply Well WR 6813823	6	90	30	0.2	30
8078-83KL59	Park Well WR 6807806	6	96	30	0.2	30
8488-8SLQ3S	Olga's Pond	5	8	154	0.9	466
8664-9TUQ4H	Pond A	1	121	31	0.8	37
	Pond BC	1	121	31	0.8	140
	Pond E	1	121	31	0.8	0
	Well 141	1	128	32	0.8	0
	Well 142	1	128	32	0.8	0
8777-8BZT2W	Pond A	1	197	22	0.8	412
	Pond B	1	212	21	0.8	162
	Washplant Well	6	200	21	0.8	47
8811-AB6QAH	Well #1	6	109	46	0.2	66
	Well #2	11	95	46	0.2	66
8868-7UPJ4P	1A	1	132	32	0.9	189
	1B	1	140	31	0.9	9
	2A	1	158	33	0.9	187
	2B	1	159	32	0.9	315
	2C	1	161	32	0.9	0
	3A	6	149	36	0.9	125
	3B	6	154	34	0.9	91

Table 3.12: Consumptive surface water takings in the steady-state simulations.

MOE Permit No.	Source Description	Consumptive Factor	Simulated Takings (m^3/day)
2435-8XGPYV	Pond on Grindstone Creek	1	185
4082-8GSR7D	Hanes Pond	1	0
5032-7DDLBA	Ponds A and B (intermittent)	1	0
5415-7JXSB7	West Farm Pond	1	0
5725-8NVQRL	Thompson Pond	1	203
6012-7QARLZ	Spencer Creek	1	374
7788-8GKMGE	Pond 1	1	147
8168-8FUQW7	Reservoir A	1	480
8606-9RWHE9	Spencer Creek	1	45
8704-7JXNYM	Christie Reservoir	1	437
8777-8BZT2W	Spencer Creek	1	446
8841-73AL2E	Pond #22	1	0
5032-7DDLBA	Pond C on an intermittent tributary of West Spencer	1	0
7788-8GKMGE	Pond 2	1	0
8168-8FUQW7	Reservoir B	1	0
8606-9RWHE9	Storage Pond	1	66
8841-73AL2E	Pond #23	1	0
8606-9RWHE9	Irrigation Pond	1	82

3.6 *Other Water Uses*

The Tier 3 Risk Assessment must also consider whether the existing and planned quantity of municipal water demand can be met while maintaining the requirements of other water uses in the area. The analysis identified all other water uses and compiled reported or estimated water quantity requirements for them, where possible. As per the Technical Rules, "other water uses" include requirements for:

- waste water assimilation,
- navigation,
- recreation,
- aquatic habitat,
- provincially significant wetlands (PSW), and
- other water takings including agricultural, commercial and industrial water takings.

The first four items are discussed in more detail in the following sections. Other water takings were discussed in Section 3.3 and 3.4 of this report.

3.6.1 Wastewater Assimilation

Wastewater generated by residents within the Middle Spencer Creek subwatershed is managed by private septic systems. There are no communal wastewater treatment facilities and therefore no wastewater assimilation requirements for receiving watercourses.

An industrial wastewater treatment plant belonging to an animal rendering facility is located within the subwatershed on Highway 5, approximately 1.5 km northwest of the Greensville municipal wells. The Certificate of Approval (sewage) for the facility permits discharge of treated effluent to a wetland that flows into Christie Lake. Discharge of effluent is limited to the period from November through March, when the reservoir levels are drawn down and recreational use is limited. The issue of assimilative capacity is addressed through specifications of the quality of the effluent discharged, which is closely monitored and reported to the MOE on a regular basis.

3.6.2 Navigation

Portions of Middle Spencer Creek are recognized as being navigable waterways, including the main stem upstream of the Christie Dam; however, no specific requirements to maintain flows or lake stage at minimum levels to facilitate inland navigation have been identified.

3.6.3 Recreation

Christie Lake is a popular destination for water-based recreational activities; Hamilton Conservation Authority operates a beach along the north shore along with access points for kayaks, paddle boats and canoes. The high water levels that are maintained in the summer help to fulfill these secondary (recreational) uses for the reservoir, while it serves its primary purposes – to augment downstream flows during the dry summer months and provide flood control during the spring.

3.6.4 Aquatic Habitat

Cold water fisheries depend on a supply of groundwater discharge along the stream reach and are likely to be sensitive to reductions in groundwater discharge due to increased groundwater takings or decreases in groundwater recharge due to drought and/or land use change. As will be discussed further on, reductions in groundwater discharge to cold water streams above specified thresholds are considered as moderate or significant impacts in the scenario analyses. Groundwater contributes a smaller proportion of flow to warm water streams. No specific threshold has been assigned to reductions in baseflow to warm water fisheries other than the decrease in baseflow should not "constitute an unacceptable impact". Impacts to warm water streams are discussed later. To be conservative, all of the streams have been assumed to be cold-water reaches in the analysis of impacts to aquatic habitats.

3.6.5 Aquatic Habitat and Provincially Significant Wetlands

There are a large number of wetlands and wetland complexes within the study area, most of them located above the Niagara Escarpment (Figure 1.4). Three of the larger wetlands, which are recognized as provincially significant wetland (PSW) complexes, include:

- Sheffield-Rockton PSW;
- Hayesland-Christie PSW; and
- Logie's Creek-Parkside Drive PSW.

It should be noted that none of the PSWs are within the Middle Spencer Creek subwatershed. Wetlands within the Middle Spencer Creek subwatershed are riparian wetlands of Spencer Creek or its tributaries.

Wetlands in the study area act to detain overland runoff and streamflow. Standing water may be present in the wetlands for all or part of the year. Some wetlands also interact with the groundwater system as locations of groundwater recharge or discharge. As noted in the Water Budget Guide, local conditions can vary and one part of a wetland complex can act as a groundwater discharge area while another part of the same wetland complex can act as groundwater recharge area. Similarly, groundwater recharge and discharge processes may reverse during the year as wetland stage and groundwater levels fluctuate. Detailed modelling has shown that even within a single wetland, the upgradient end of a wetland can act as a groundwater discharge area while the downgradient end acts as a groundwater recharge area.

The Tier 3 scenario analyses, discussed further on, examined whether significant reductions in groundwater discharge to wetlands would occur as a consequence of increased pumping or decreased recharge due to drought and/or land use change.

3.7 *Land Use and Land Use Change*

The type of land cover has a strong effect on the water balance. Interception and evapotranspiration are directly influenced by vegetation type and cover density, which, in turn, affect runoff and infiltration rates. Conversion of natural or agricultural lands to urban land use (e.g., residential, commercial, industrial, or institutional) often increases the amount of impervious cover leading to increased evaporation from depression storage and increased runoff, reducing recharge potential. While at the same time, evapotranspiration and canopy interception, and soil zone storage are decreased as the vegetative and pervious cover is changed, increasing the recharge potential, making the net impact to groundwater recharge more difficult to predict intuitively.

The Tier 3 analysis must characterize projected land use changes within the context of potential impact to rates of groundwater recharge and the sustainability of municipal groundwater supplies. The Water Budget Guide identified the following steps to characterize potential recharge reductions:

1. *Create a map of existing land use.*
2. *Create a map of projected land use (Official Plan).*
3. *Identify areas of land use change by comparing projected land use against existing land use.*
4. *Estimate the projected change in imperviousness for each of the areas of land use change. This will require making assumptions relating to the imperviousness of land use categories.*
5. *Create a map of projected imperviousness changes for areas of land use change.*

The potential impact of stormwater management measures and low-impact developments are not to be accounted for when estimating imperviousness changes for projected land use.

3.7.1 Existing Conditions Land Use

The primary source for land coverage mapping in the Greensville Tier 3 study area is the MNR SOLRIS (v1.2) data with additional data provided by Conservation Halton. Figure 3.17 shows the distribution of land cover types across the study area. Some of the classifications have been combined for presentation purposes but each classification was treated separately in the model. Consistent parameter values were used for each unique land-cover type in the model (see Earthfx, 2014). Vegetation type was determined from the various land classifications. The predominant land use type is agricultural, which covers 59% of the model area. Natural areas, including forests and wetlands, cover 29%, while developed/settled areas (i.e., urban, rural, transportation, parks, industrial, commercial, etc.) cover another 9% of the study area.

3.7.2 Official Plan Land Use

The City of Hamilton completed the Rural Hamilton Official Plan in 2012, which provides direction and guidance on the management of land use change and physical development for the subsequent 30 years (City of Hamilton, 2011). Under the Rural Hamilton Official Plan (City of Hamilton, 2011), new residential developments to the west of the municipal well will be predominantly single-family, detached dwellings. There will be minimal infilling of existing settled areas. A phased approach to residential development is required under the Official Plan, whereby groundwater resources are closely monitored and unacceptable impacts can result in discontinuation of further development until impacts can be remediated and/or mitigated. No increases in commercial areas are included in the Official Plan. The areas of future residential development are shown in Figure 3.18 along with the existing areas of residential development.

Not all land in the existing residential areas has been developed. For the purpose of the simulations, these areas were converted to the rural residential land use classification, along with the areas within the future residential areas identified in the Rural Official Plan. Model parameters, such as percent impervious and vegetative type and cover densities, were updated for the cells representing these areas.

The most significant changes in land use within the study area are related to the continued expansion of the Lafarge and Flamboro quarries to the north of the Greensville municipal well. These quarries were assumed to continue their expansion until their maximum licensed extents. Full expansion conditions were represented in the model when simulating quarries in the future-conditions scenarios. The model representation of the quarries was revised (from existing conditions) by incorporating changes in ground elevation, removal of vegetative cover, and the construction and modification of drainage features. Representation of the quarries to satisfy the various risk assessment scenarios is discussed in more detail in Section 6.2.6.

3.8 *Figures*

Figure 3.4: Location of Greenville municipal wells and school well.

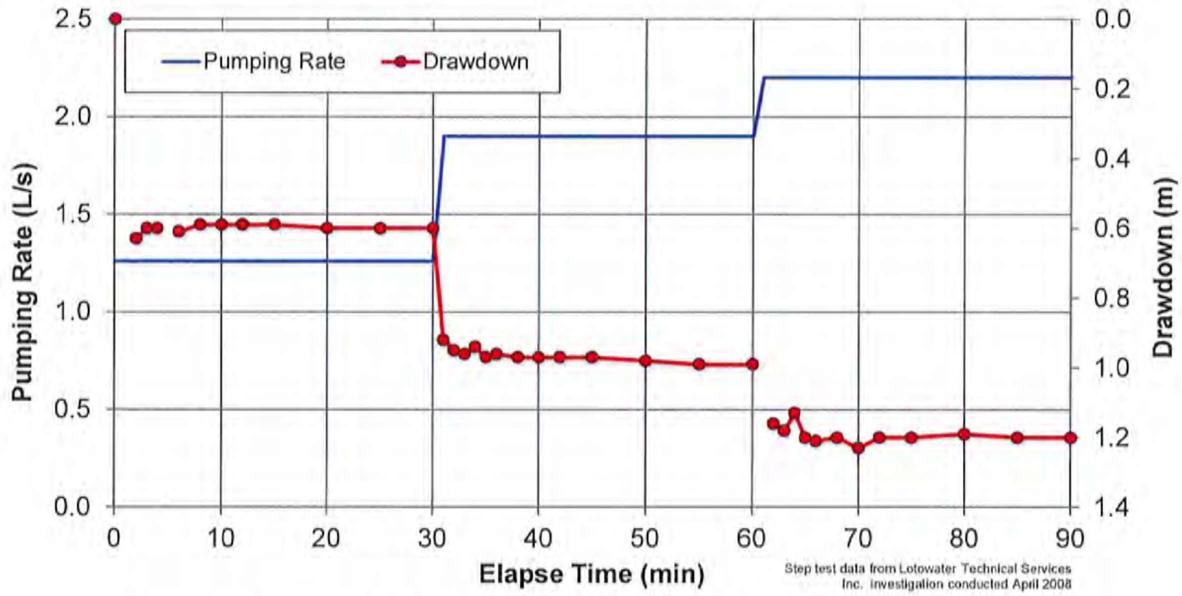


Figure 3.5: Step-test results at Greensville FDG01 – flow and drawdown vs. time.

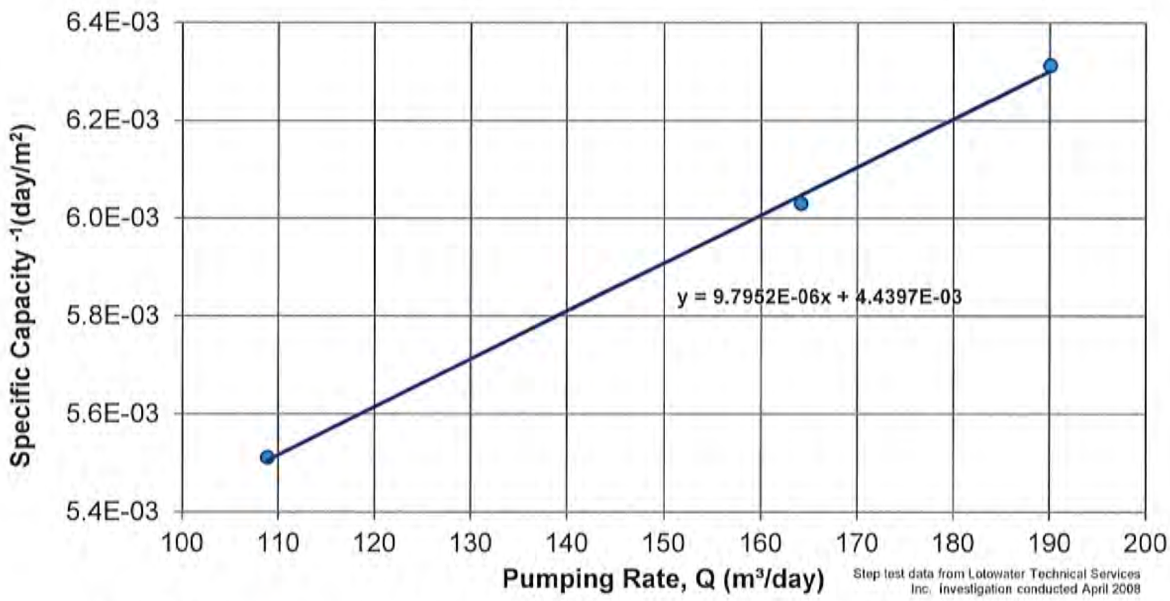


Figure 3.6: Step-test results at Greensville FDG01 – specific capacity vs. pumping rate.

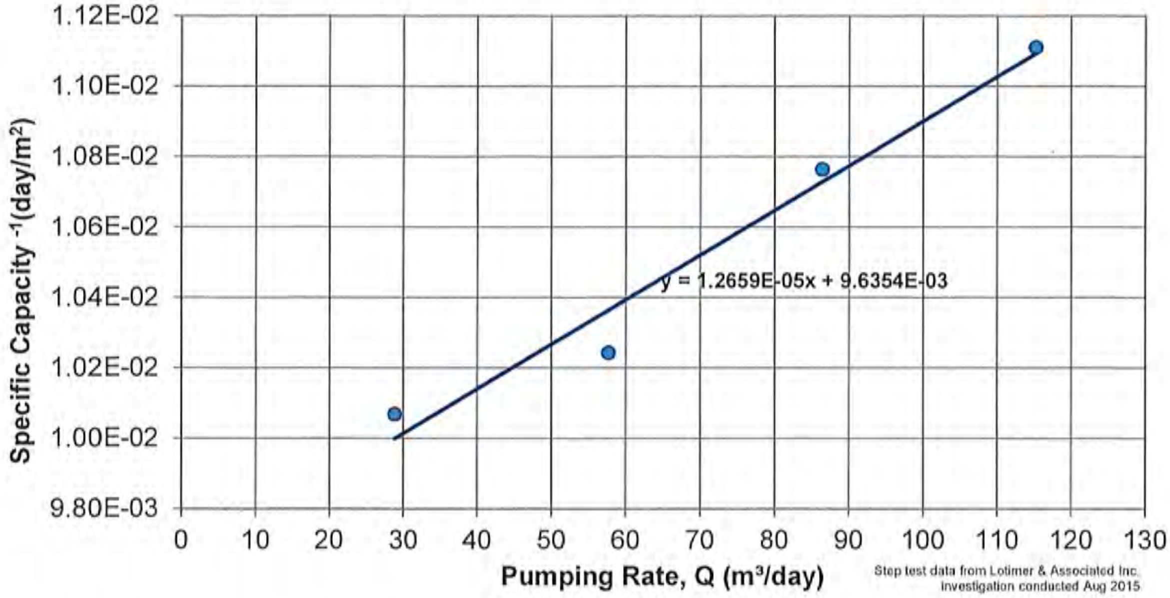


Figure 3.7: Step-test results at Greensville TW-2-13 after first round of well enhancement – specific capacity vs. pumping rate.

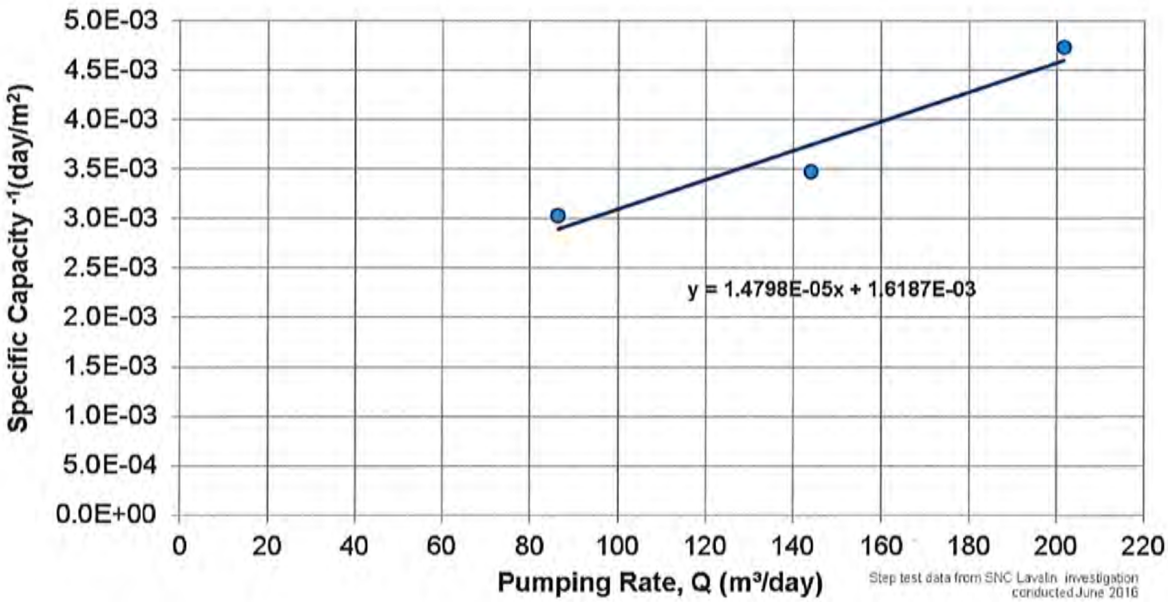
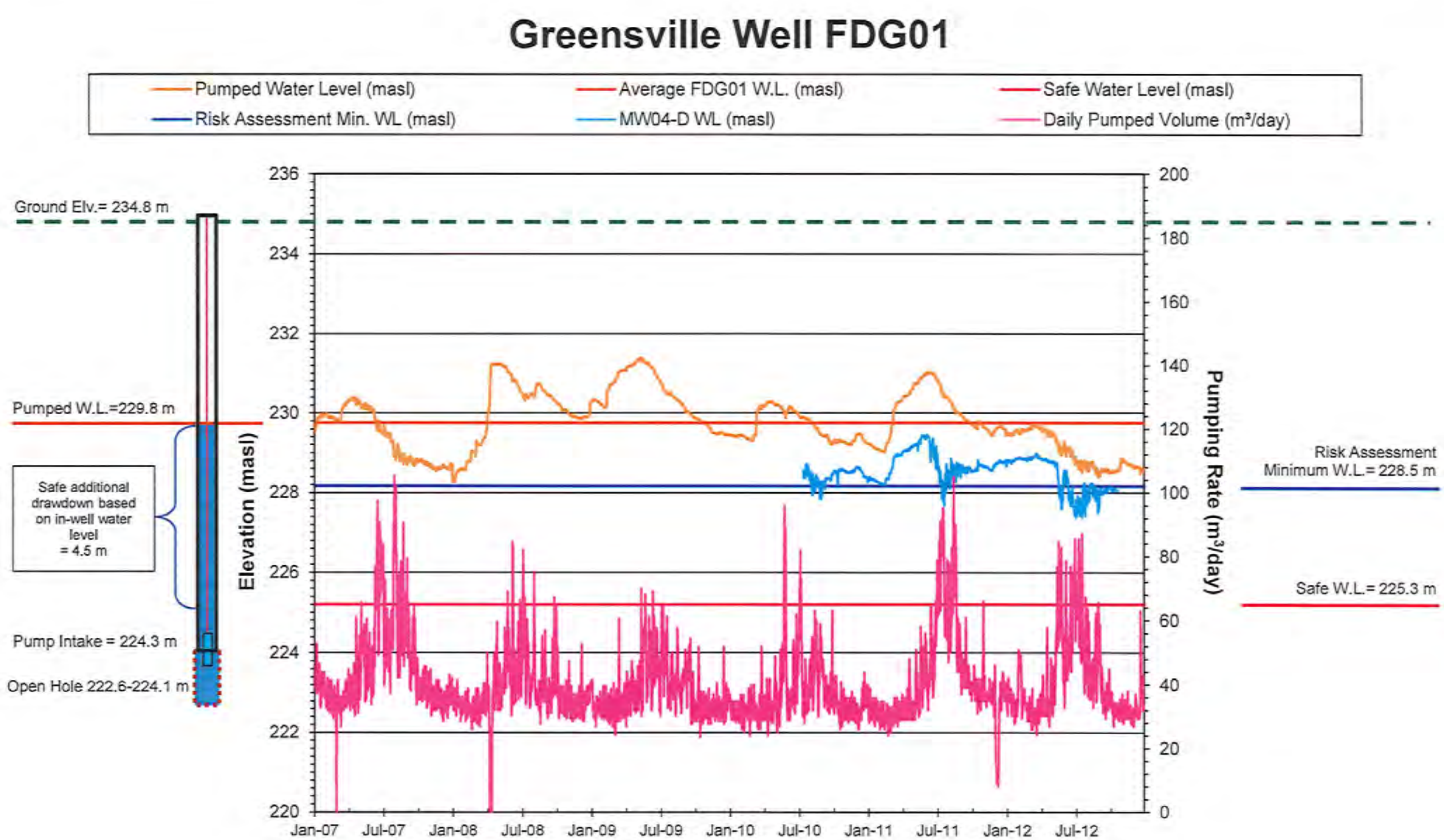
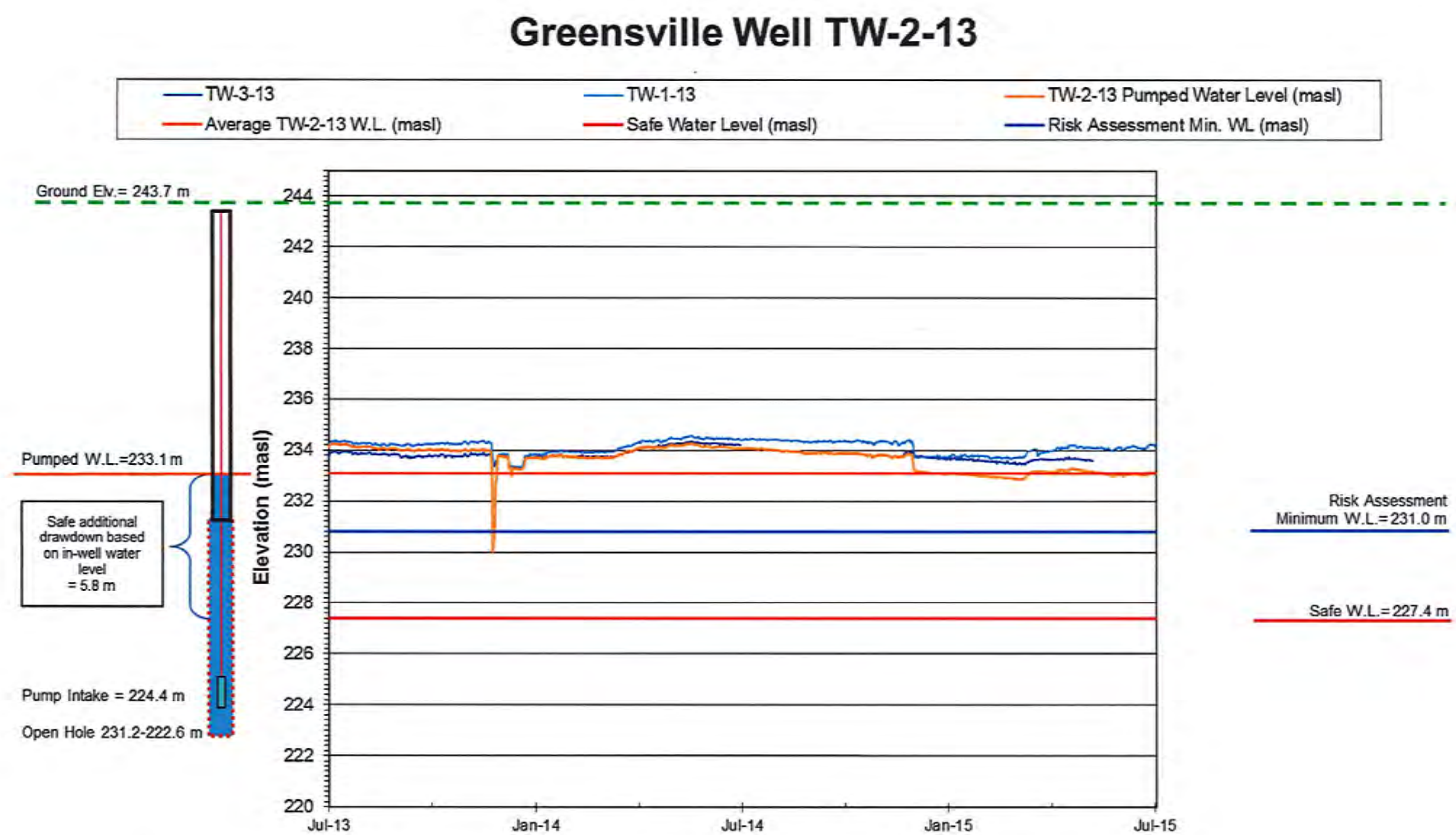


Figure 3.8: Step-test results at Greensville TW-2-13 after final round of well enhancement – specific capacity vs. pumping rate.



Note: Safe additional drawdown in the Greensville municipal supply well FDG01 is based upon a minimum operational water level limit of 1 m above the pump intake and the average in-well water level for the period of 2007 to 2012. Pump intake is set at a depth of 10.54 m below ground surface (Lotowater, 2008).

Figure 3.9: Well characterization plot for Greensville Municipal Well FDG01.



Note: Safe additional drawdown in the Greensville municipal supply well TW-2-13 is based upon a minimum operational water level limit remaining above the upper-most water bearing fracture found at 227.4 masl. Pump intake is set at a depth of 20 m below top of casing (Lotimer & Associates, 2016).

Figure 3.10: Well characterization plot for Greensville Municipal Well TW-2-1.

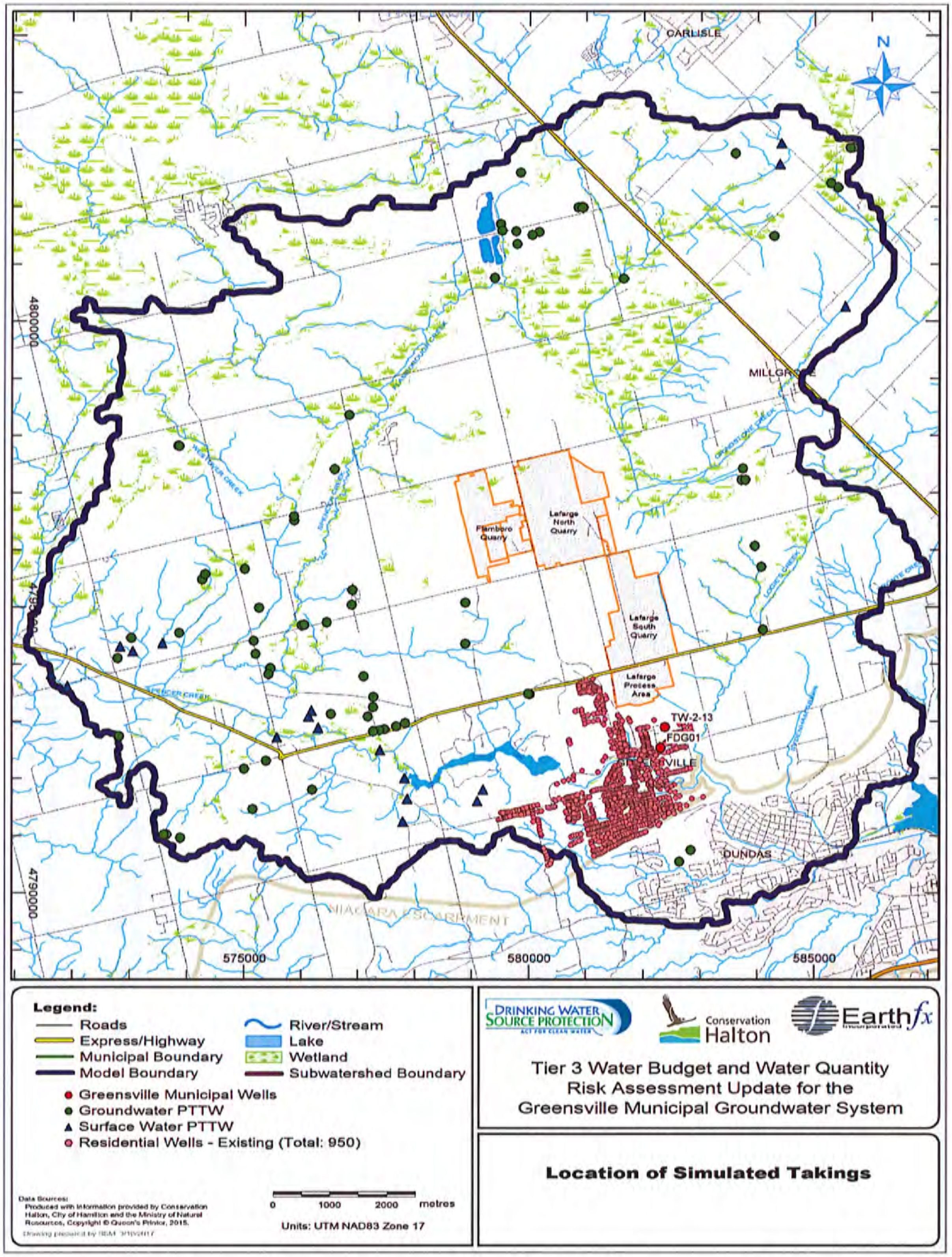


Figure 3.11: Location of simulated permitted and non-permitted (residential) takings.

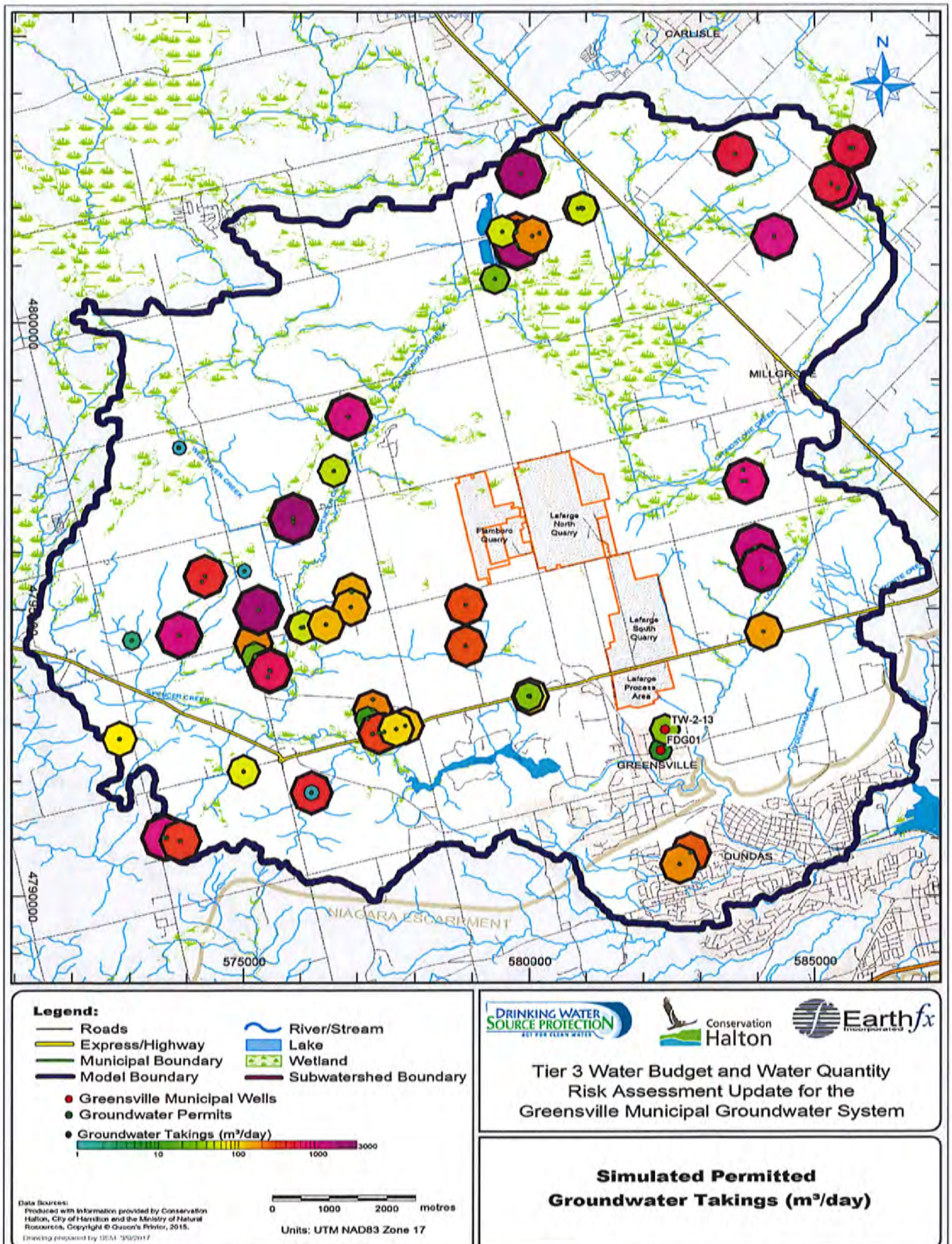


Figure 3.12: Simulated groundwater takings associated with PTTWs.

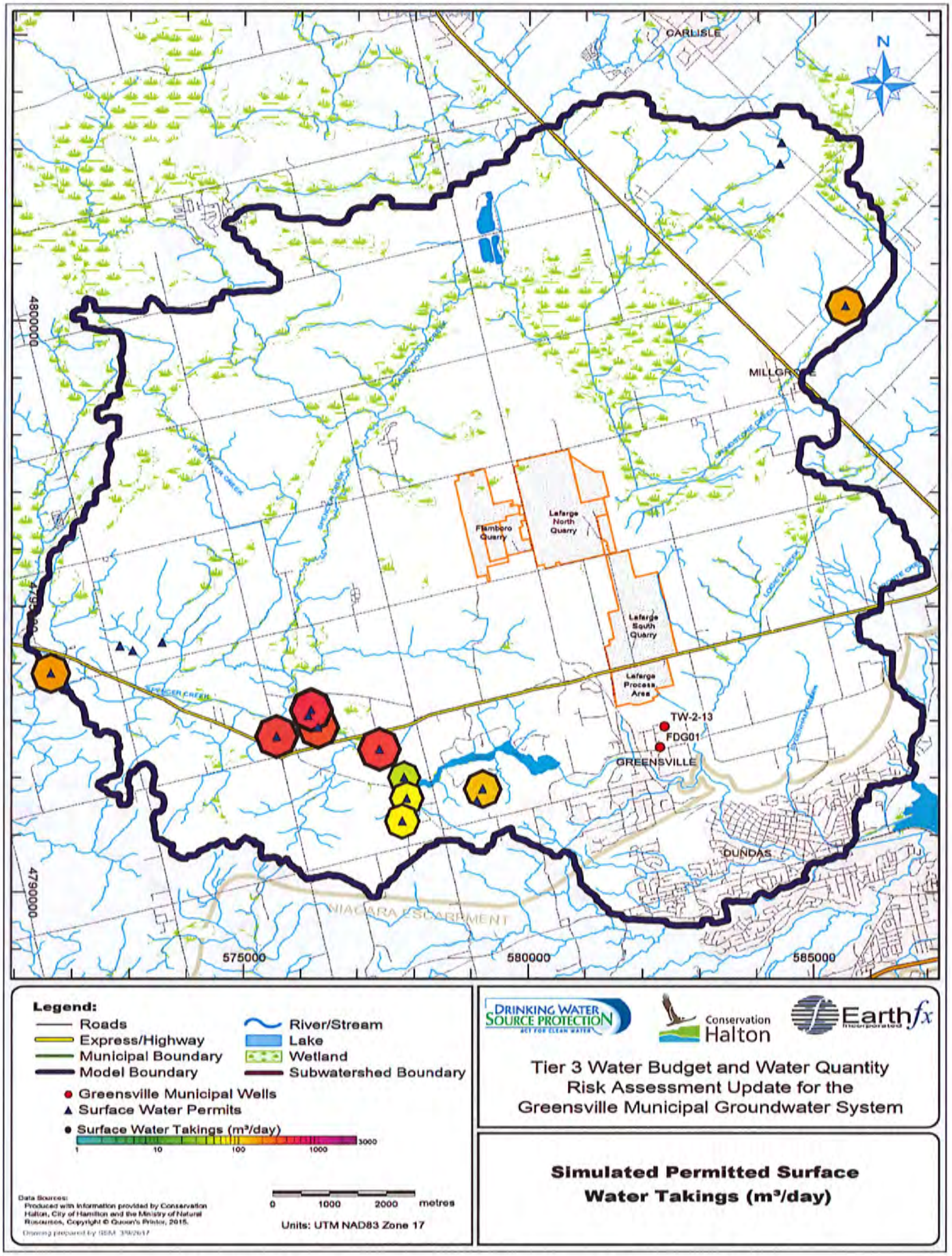


Figure 3.13: Simulated surface water takings associated with PTTWs.

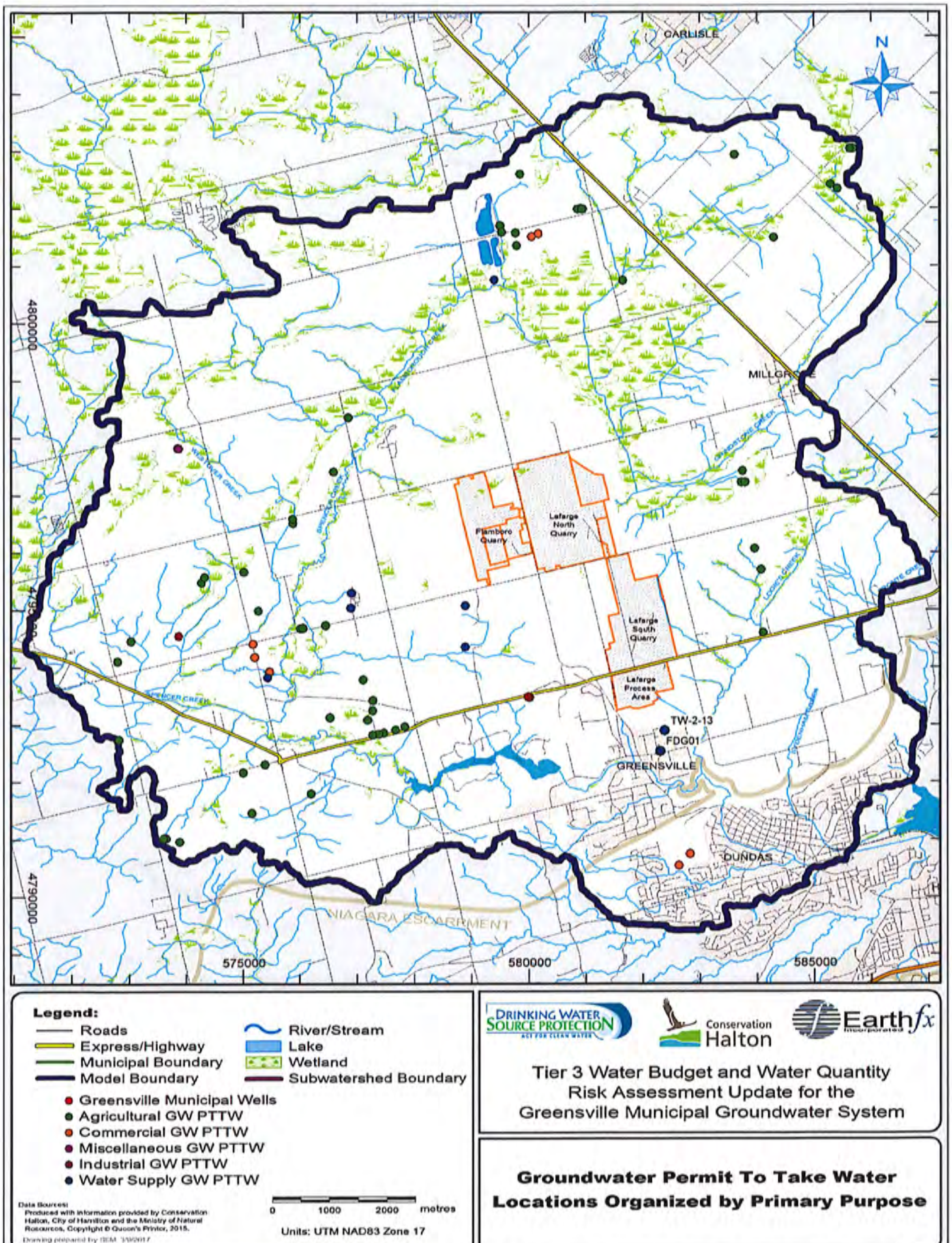


Figure 3.14: Groundwater and mixed groundwater / surface water PTTW locations organized by primary purpose.

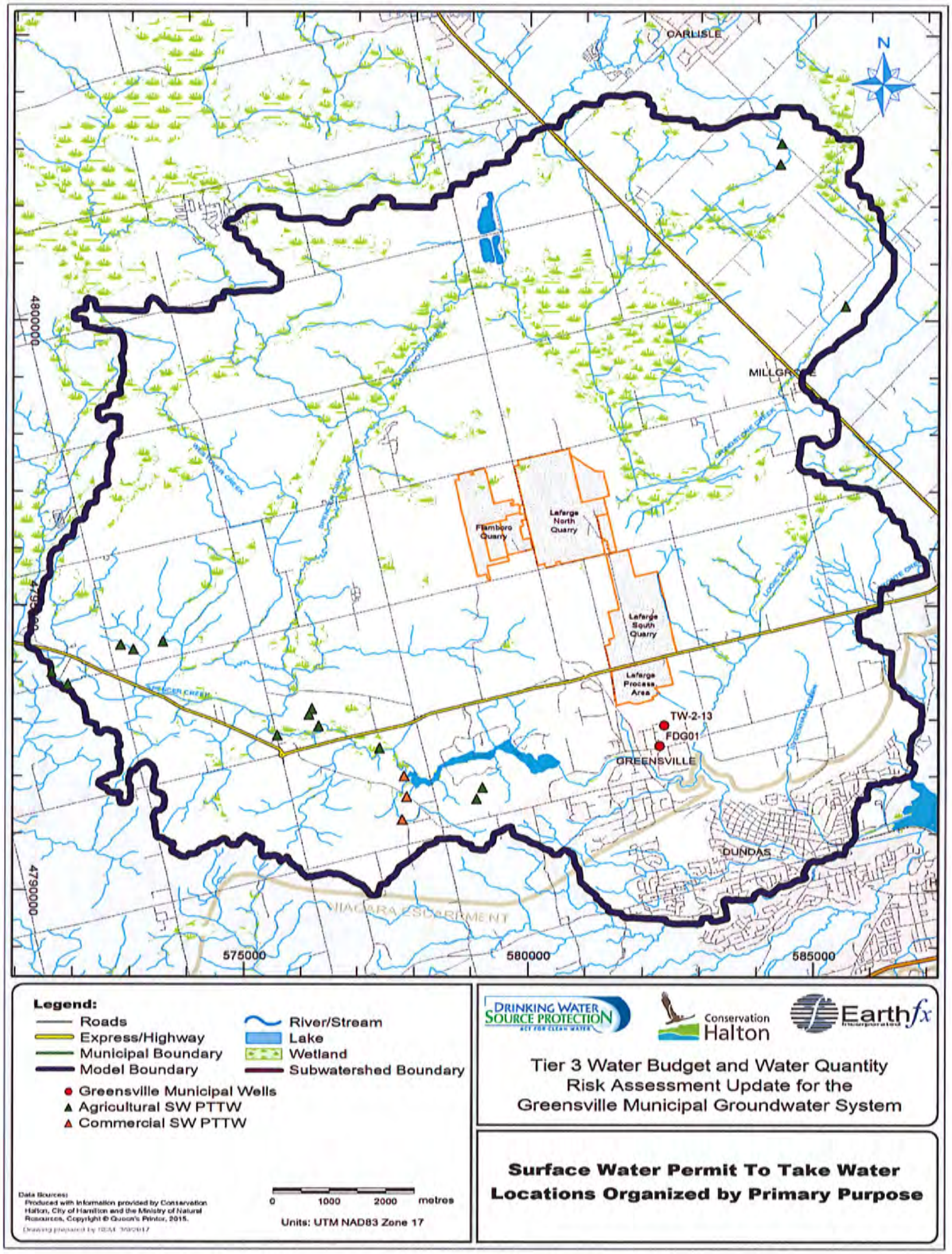


Figure 3.15: Surface water PTTW locations organized by primary purpose.

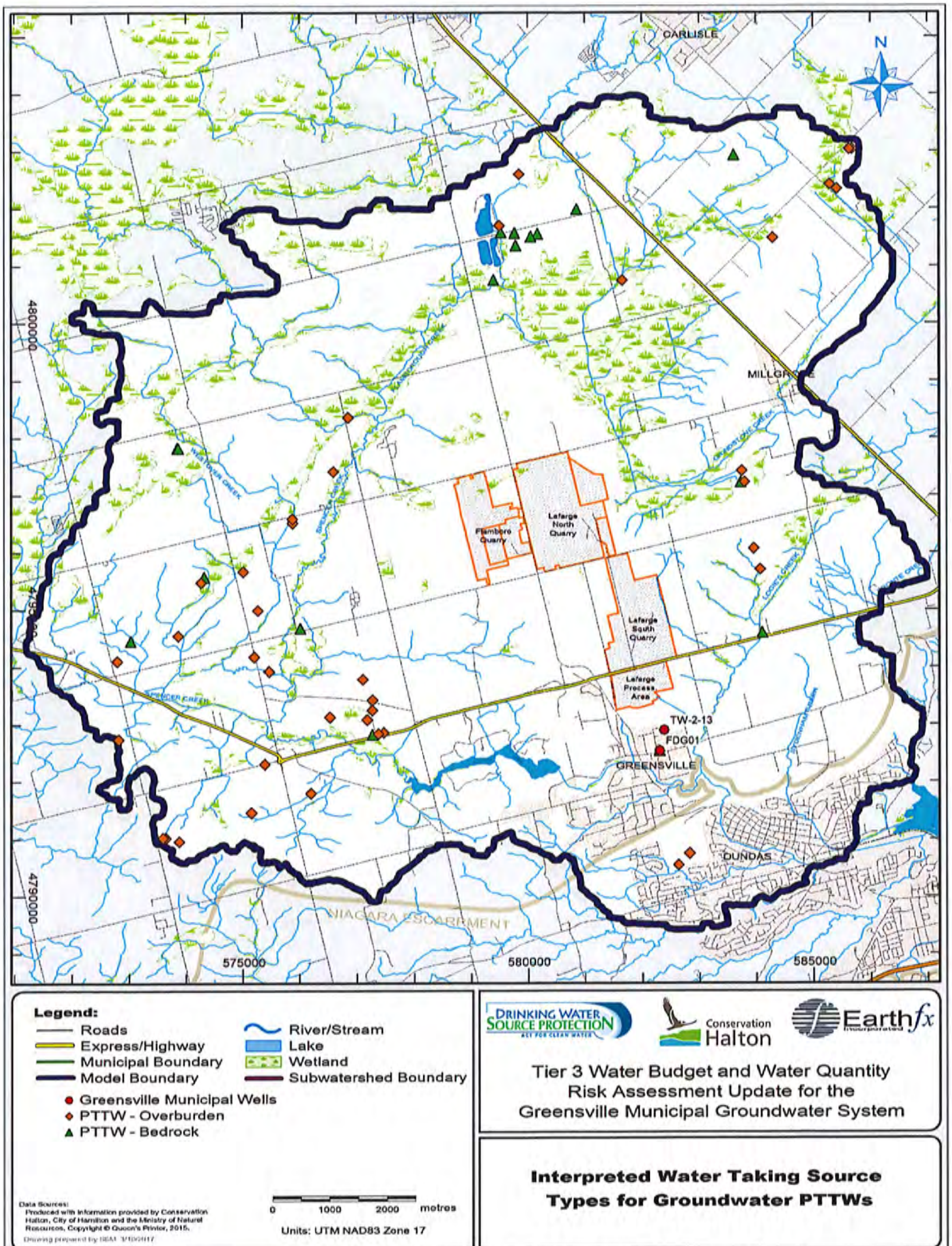


Figure 3.16: Interpreted water taking source types for groundwater PTTWs used in simulations.

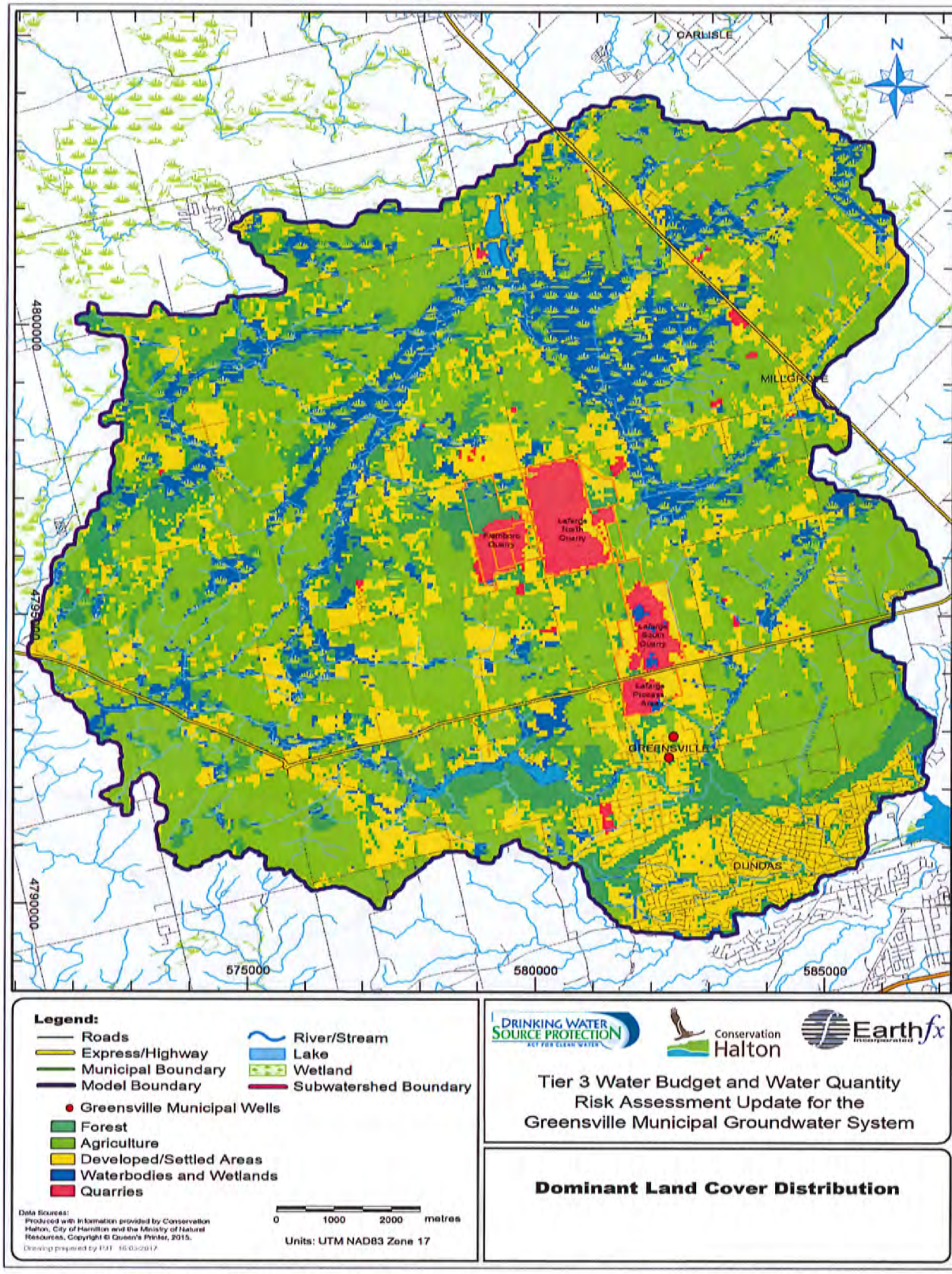


Figure 3.17: Detailed mapping of existing land cover in the wellfield vicinity (data provided by Conservation Halton).

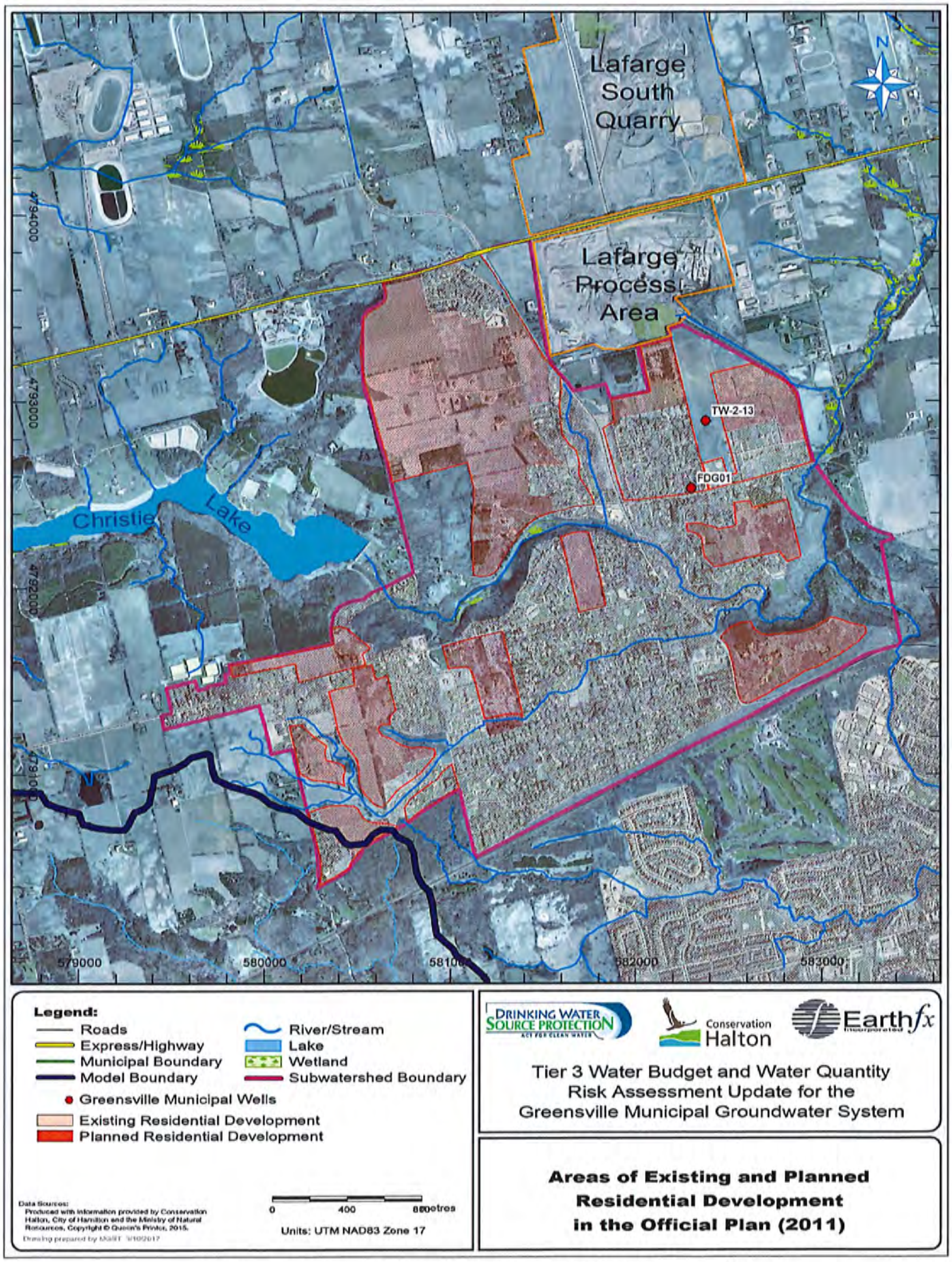


Figure 3.18: Areas of possible future residential development.

4 Tier 3 Model Update

After the new wellfield, quarry data, climate inputs, and water use information was analyzed and entered into the project database, we began the task of updating the Tier 3 conceptual geologic model as well as the numerical groundwater and hydrologic submodels that comprise the integrated GSFLOW model. The integrated model was then re-calibrated by comparing simulations results against the streamflow and groundwater level datasets updated with recently collected observations. The modifications and re-calibration process are described below.

4.1 *Stratigraphic and Hydrostratigraphic Model Update*

The Tier 3 conceptual stratigraphic model (Earthfx 2014) describing the geologic units underlying the study area was developed through a thorough review of borehole data and previous geologic reports. The conceptual geologic model comprises four Quaternary units overlying the clastic and carbonate sedimentary rocks of Middle Silurian to Late Ordovician age (Figure 2.1). This conceptual model was translated into an 11-layer stratigraphic model by mapping the tops of each geologic unit and then overlaying them to create a three-dimensional representation. The 11 layers of the stratigraphic model are summarized in Table 4.1, and are consistent with those used in the previous Tier 3 study.

As described in Earthfx (2014), unit surfaces were developed through a process of "picking" the unit tops in geologic boreholes across the site, which were then interpolated using a geostatistical technique known as "kriging". Post-processing of layers was conducted to identify and correct areas where surfaces crossed one another or the interpolation did not conform to the conceptual understanding of the local geology. As part of this Tier 3 update study, additional geologic borehole data from the Lafarge SQE and the municipal water supply exploration program (shown in Figure 2.4) were incorporated into a local revision to the 3-D surfaces of the stratigraphic model. In general, the existing surfaces were found to be consistent with the new geologic logs, requiring only minor alterations to some of the model surfaces. Figure 4.3 presents a north-south geologic section through the Greensville municipal wellfield.

The 11 layers of the stratigraphic model were then translated into a 12-layer hydrostratigraphic model according to the hydrogeologic conceptualization of the study area. The conceptual hydrostratigraphic model differs from the stratigraphic model in that geologic units were subdivided where properties differ (e.g., the weathered bedrock is separated from unweathered bedrock) and merged where units had similar properties. As shown in Table 4.1, the updated Tier 3 hydrostratigraphic model layers are generally consistent with those of the previous Tier 3 with one key exception.

Table 4.1: Stratigraphic and Hydrostratigraphic Model Layers from Previous and Updated Greensville Tier 3 studies.

<table>
<tr><th rowspan="2"></th><th rowspan="2">Layer</th><th rowspan="2">Tier 3 Stratigraphic Model</th><th colspan="4">Tier 3 Hydrostratigraphic Model</th><th rowspan="2">Role</th></tr>
<tr><th>Layer</th><th>Earthfx (2014/2015)</th><th>Layer</th><th>Updated Tier 3 Study</th></tr>
<tr><td rowspan="4">Overburden</td><td>1</td><td>Postglacial Deposits</td><td>1</td><td>Postglacial Deposits</td><td>1</td><td>Postglacial Deposits</td><td>Aquifer</td></tr>
<tr><td>2</td><td>Halton Till</td><td>2</td><td>Halton Till</td><td>2</td><td>Halton Till</td><td>Aquitard</td></tr>
<tr><td>3</td><td>Mackinaw Interstadial Sediments</td><td>3</td><td>Mackinaw Interstadial Sediments</td><td>3</td><td>Mackinaw Interstadial Sediments</td><td>Aquifer</td></tr>
<tr><td>4</td><td>Wentworth Till</td><td>4</td><td>Wentworth Till</td><td>4</td><td>Wentworth Till</td><td>Aquitard</td></tr>
<tr><td rowspan="9">Bedrock</td><td rowspan="2">6</td><td rowspan="2">Guelph Formation</td><td>5[1]</td><td>Weathered Bedrock</td><td>5[1]</td><td>Weathered Bedrock</td><td>Aquifer</td></tr>
<tr><td rowspan="2">6</td><td rowspan="2">Upper Lockport Aquifer</td><td>6</td><td>Guelph Formation</td><td>Aquifer</td></tr>
<tr><td>7</td><td>Upper Eramosa Formation</td><td>7</td><td>Upper Eramosa Formation</td><td>Aquifer</td></tr>
<tr><td>8</td><td>Lower Eramosa Formation</td><td>7</td><td>Lower Eramosa Formation</td><td>8</td><td>Lower Eramosa Formation</td><td>Aquitard</td></tr>
<tr><td>9</td><td>Vinemount member</td><td>8</td><td>Vinemount member</td><td>9</td><td>Vinemount member</td><td>Aquitard</td></tr>
<tr><td>10</td><td>Goat Island Formation</td><td>9</td><td>Goat Island Formation</td><td>10</td><td>Goat Island Formation</td><td>Aquifer</td></tr>
<tr><td>11</td><td>Gasport Formation</td><td>10</td><td>Gasport Formation</td><td>11</td><td>Gasport Formation</td><td>Aquifer</td></tr>
<tr><td>12</td><td>Clinton-Cataract Group/Queenston</td><td>11</td><td>Clinton-Cataract Group/Queenston</td><td>12</td><td>Clinton-Cataract Group/Queenston</td><td>Aquitard</td></tr>
</table>

Note: [1] Weathered bedrock contact aquifer corresponds to a model wide aquifer unit that occurs at the top of the bedrock surface, regardless of the sub-cropping Paleozoic bedrock unit.

As part of the Tier 3 update, the Upper Lockport aquifer unit of the previous study was subdivided into the Guelph and Upper Eramosa aquifers. These units were previously grouped together due to the similarities in aquifer properties of the two bedrock formations. As discussed in Section 2.2, new data from the Golder (2013) SQE study provided additional hydrogeologic insights that motivated the subdivision of this hydrostratigraphic unit in the updated Tier 3. Having an additional layer allows a more detailed representation of the groundwater flow and drawdowns in the vicinity of the quarry operations and the new municipal supply well TW-2-13.

To illustrate the translation of the stratigraphic model to the hydrostratigraphic model, a north-south section through the Greensville municipal wellfield area is shown in Figure 4.4. Additional sections were provided in Earthfx (2014) showing the regional geologic and hydrostratigraphic model layers, which have remained largely unchanged from the previous Tier 3.

Some additional refinement was done to transform the 12 hydrostratigraphic model layers into numerical model layers suitable for the GSFLOW model. This included, for example, ensuring that a minimum thickness was preserved for each layer. Unique hydraulic properties were assigned to the units represented by these surfaces (hydraulic properties were modified to preserve stratigraphic accuracy, where minimum thicknesses were maintained). The hydrostratigraphic model surfaces were also used in the assignment of well screens or pumped intervals used in the interpretation of groundwater levels (and calibration targets), discussed in Earthfx (2014) and in Section 3.3.7 .

4.2 Local Refinement of the MODFLOW Sub-Model Grid

A variable cell-size grid was designed for the MODFLOW submodel used in the original Greensville Tier 3 study. A very fine resolution, with square cells of 12.5 m to a side, was used around the Greensville municipal well FDG01, with coarser cells (up to 200 m on a side) being used towards the outer edges of the model. To accommodate the expanded municipal wellfield, the wellfield refinement area was extended to include TW-2-13 (as well as the other two test holes TW-1-13 and TW-3-13). Figure 4.5 presents the original (black) and the updated (red) finite-difference model grids for the updated Tier 3 model.

The portion of the model area occupied by the quarries was originally represented using cells with a maximum side length of 100 m. As part of this study, the model grid covering the majority of the quarry footprints were further refined to use cells with a maximum size of 50 m on a side. The updated model grid consists of 232 rows and 175 columns and contains 40,600 grid cells for each of the 12 model layers.

4.3 Recalibration of the Integrated GSFLOW Model

Following revisions of the stratigraphic and hydrostratigraphic model surface, the calibration of the integrated Tier 3 GSFLOW model was revisited. The main objectives of the re-calibration effort were to ensure the goodness-of-fit demonstrated in the original model calibration was preserved and that a reasonable match was achieved to the newly incorporated water level and streamflow data.

4.3.1 Steady-State Groundwater Flow Model Calibration Results

A preliminary re-calibration of the MODFLOW-NWT groundwater submodel was undertaken to refine the parameters assigned to the updated numerical model layers. This was done prior to re-calibrating the more computationally-intensive GSFLOW model. Water levels from new municipal and quarry monitoring locations were incorporated into the calibration dataset and used to verify the re-calibration of the steady-state groundwater submodel.

Table 4.2 summarizes the hydraulic conductivity parameters assigned to the various aquifers and aquitards represented in the Tier 3 model, along with the values used in the previous Tier 3 model. Figure 4.6 to Figure 4.17 present the distribution of horizontal hydraulic conductivity values assigned to each of the 12 numerical groundwater model layers. The horizontal hydraulic conductivity values are generally consistent with those used in the previous Tier 3 assessment, including those assigned to the newly incorporated Guelph and Upper Eramosa aquifer layers. More significant changes were made to the assigned vertical anisotropy values (K_H/K_V) in the bedrock aquifers and aquitards. The increases in the assigned vertical anisotropy values were made after a review of the aquifer test results and quarry monitoring data, which indicated a poor vertical connection across the intact bedding planes of the Guelph Formation (Golder, 2013).

In particular, an aquifer test was carried out by Golder (2013) at quarry monitor MW06-3C in May 2007 to assess the transmissivity of the Guelph and the Upper Eramosa bedrock units. The well was pumped at approximately 90 L/min over three days. Water from the well was interpreted to originate from horizontal partings in the Upper Eramosa member. A transmissivity of 5 m^2/d was determined based on the results of the test, with an equivalent bulk hydraulic conductivity of approximately 5×10^{-6} m/s based on an assumed unit thickness of 10 m. Vertical hydraulic conductivities were also estimated using the Neuman-Witherspoon method for the Guelph/Eramosa (6×10^{-7} m/s), Vinemount (3×10^{-8} m/s), and Goat Island units (7×10^{-7} m/s). These results suggested a high degree of vertical anisotropy in these units, with vertical hydraulic conductivities being 0.10 to 0.01 of the horizontal values.

Table 4.2: Comparison of numerical model layers and hydraulic conductivity values.

	Layer	Unit	Previous Tier 3		Updated Tier 3	
			K_H (m/s)	K_H/K_V	K_H (m/s)	K_H/K_V
Overburden	1	Postglacial Deposits Aquifer	1.0×10^{-8} to 1.0×10^{-5}	1	1.0×10^{-8} to 1.0×10^{-5}	1
	2	Halton Till Aquitard	6.0×10^{-7}	1	6.6×10^{-7}	5
	3	Mackinaw Interstadial Sediments Aquifer	4.0×10^{-6}	1	5.0×10^{-6}	1
	4	Wentworth Till Aquitard	9.0×10^{-7}	1	9.0×10^{-7}	5
Bedrock	5	Weathered Bedrock Aquifer	5.0×10^{-7} to 5.0×10^{-5}	10	6.0×10^{-7} to 6.0×10^{-5}	20
	6	Guelph Aquifer	1.0×10^{-5}	20	8.0×10^{-6}	300
	7	Upper Eramosa Aquifer			4.5×10^{-5}	20
	8	Lower Eramosa Aquitard	8.0×10^{-8}	90	8.0×10^{-8}	100
	9	Vinemount Aquitard	4.0×10^{-8}	100	4.0×10^{-8}	100
	10	Goat Island Aquifer	2.0×10^{-6}	70	5.0×10^{-6}	50
	11	Gasport Aquifer	1.0×10^{-6}	70	3.0×10^{-6}	50
	12	Clinton-Cataract Group /Queenston Aquiclude	5.0×10^{-8}	100	5.0×10^{-8}	100

Model calibration was conducted by adjusting aquifer properties and refining estimates of recharge provided by the PRMS model until a good match was achieved between the simulated and interpolated water levels. Figure 4.18 and Figure 4.19 provide comparisons between the simulated steady-state water levels and interpolated static water levels in the shallow weathered bedrock contact zone and deeper Goat Island-Gasport aquifer systems, respectively. A visual check indicates that good matches were achieved to the interpolated static water level data. Areas where the match was not as good also tended to be areas where observation data were sparse and the interpolated values are less certain.

Statistical analyses were applied to test the quality of the calibration. A scatterplot comparing the WWIS water levels to the simulated steady-state heads is shown in Figure 4.1. Ideally, all data points should fall on the 1:1 line shown on the plot. For the most part, the data points fall within the ±10 m error interval, defined by the dashed red lines.

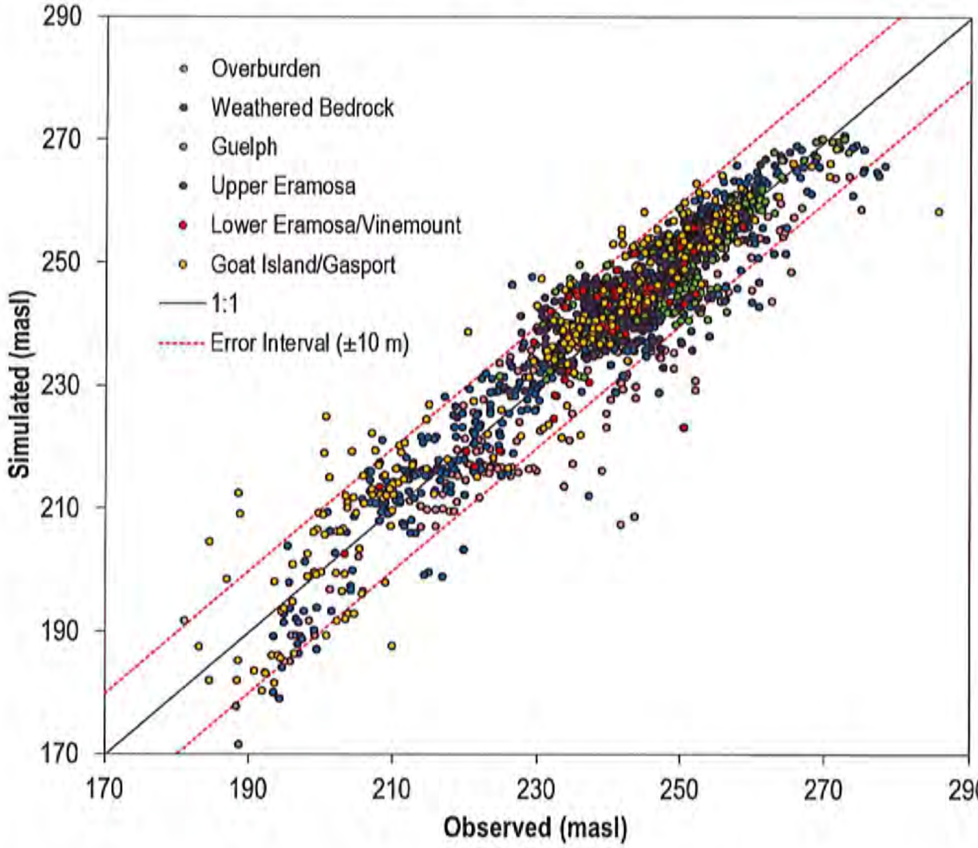


Figure 4.1: Scatter plot of observed versus simulated heads from steady-state groundwater model calibration.

Three calibration statistics were used to assess and demonstrate model accuracy: the mean error (ME), mean absolute error (MAE), and root mean squared error (RMSE). These are given by Anderson and Woessner (1992) as:

$$\text{Mean Error} = \frac{1}{n}\sum_{1}^{n}(h_O - h_S)_i \qquad \text{Equation 1}$$

$$\text{Mean Absolute Error} = \frac{1}{n}\sum_{1}^{n}\left|(h_O - h_S)_i\right| \qquad \text{Equation 2}$$

$$\text{Root Mean Squared Error} = \sqrt{\frac{1}{n}\sum_{1}^{n}(h_O - h_S)_i^2} \qquad \text{Equation 3}$$

where:

h_o = observed head;
h_s = simulated head; and
n = number of observations.

Calibration statistics for the 2,250 observed water levels are shown in the Table 4.3, along with separate statistics for each hydrostratigraphic unit. It should be noted that the overburden units, the Lower Eramosa and Vinemount member, and the Goat Island and Gasport Formations have been grouped together in Table 4.3.

Table 4.3: Calibration statistics for the steady-state groundwater sub-model.

Modelled System	Number of Wells (n)	ME (m)	MAE (m)	RMSE (m)	Range in Observations (m)	RMSE as % of Range (%)
Overburden	336	2.65	4.62	6.78	79.9	8.5%
Weathered Bedrock	905	-0.40	3.39	4.74	89.8	5.3%
Guelph Fm.	391	-0.11	2.85	3.96	52.2	7.6%
Upper Eramosa Fm.	270	-1.34	4.06	5.26	49.9	10.6%
Lower Eramosa Fm./Vinemount	81	-1.62	4.42	6.05	55.1	11.0%
Goat Island/Gasport Fm.	267	-2.20	5.23	7.08	104.6	6.8%
Overall	**2,250**	**-0.27**	**3.82**	**5.41**	**104.6**	**5.2%**

The negative value for the ME indicates that simulated values are generally higher than observed values by 0.27 m, and range from an average of 2.62 m above the observed values in the overburden to 2.2 m lower than the observed values in the Goat Island/Gasport aquifers. The MAE and RMSE provide good estimates of the average magnitude of the difference and variance between observed and simulated values. Overall, the calibrated groundwater submodel had a MAE of 3.8 m and a RMSE of 5.4 m.

Generally accepted guidelines indicate that the model is well calibrated when the RMSE is less than 10% of the range of water levels (Spitz and Moreno, 1996). The overall RMSE expressed as a percentage of the range in the static groundwater observation dataset was 5.2%, which is less than this calibration guideline. Using the same criterion to evaluate model performance within each hydrostratigraphic unit yields values ranging from 5.3% in the weathered bedrock to 11.0% in the Lower Eramosa/Vinemount aquitards. The MODFLOW mass balance error for the steady-state model was 0.36 percent.

The residuals analysis was repeated for the subset of monitors that includes only monitoring wells belonging to the Greensville municipal monitoring network, the Lafarge and Flamboro quarry operations, and the Provincial Groundwater Monitoring Network. A scatter plot comparing the average observed and simulated water level data for these data points is presented in Figure 4.2. Calibration statistics for these 192 higher-quality data points are presented below in Table 4.4.

Table 4.4: Calibration statistics for high quality average water level data points.

Number Of Wells (n)	ME (m)	MAE (m)	RMSE (m)	Range in Observations (m)	RMSE as Percent of Range (%)
206	-0.18	3.79	5.13	67.6	7.6%

The calibration error assessed using the high quality average water level dataset is in good agreement with the previous analysis with the larger dataset. The RMSE for the subset is 7.6% of the total range in observations, which is below the 10% maximum error limit recommended by Spitz and Moreno (1996). These results further support the conclusion that the groundwater sub-model is well calibrated with respect to available static groundwater level data.

Average recorded discharges for the Flamboro and Lafarge Quarries were compared to the simulated (baseflow) discharges for the sump ponds and off-site diversions in Table 4.5. It should be recognized when comparing the observed (average discharge) and predicted (simulated) discharges shown, that the simulated values represent only the accumulated contribution from groundwater seepage into the quarry and do not account for the portion of the reported discharge contributed by direct precipitation and runoff. The simulated values are low, as expected (from a steady state groundwater-only pre-calibration), ranging from 86% to 36% of the average total discharge at the Flamboro and North Quarry Sump, respectively. A more comprehensive water balance analysis was performed with the updated integrated GSFLOW model (presented in Section 5.2).

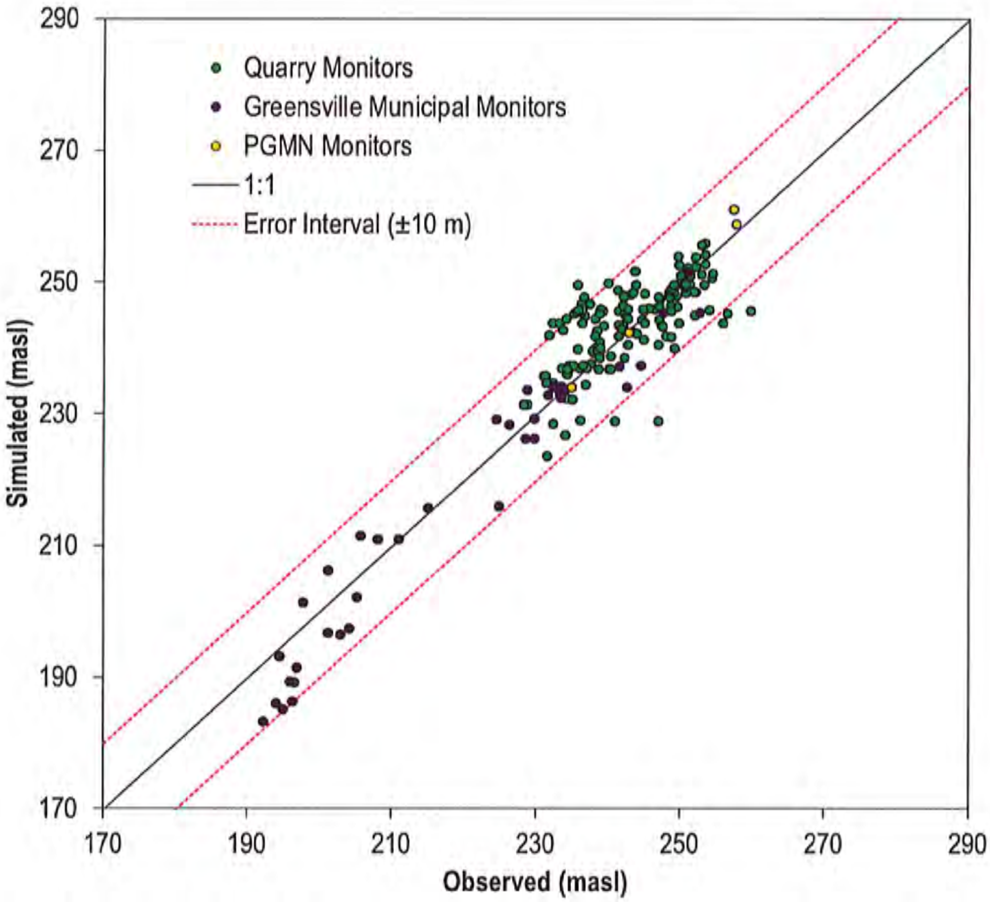


Figure 4.2: Scatter plot of observed versus simulated heads for high quality groundwater calibration points.

Table 4.5: Simulated and reported average discharge from the quarries.

Quarry	Source	Reported Average Discharge (m^3/d)[1]	Simulated Average Discharge (m^3/d)[2]
Flamboro Quarry	Main Sump	2,950	2,550
Lafarge Quarry	North Sump	11,851	4,300
	South Sump	4,048	3,100
	Railway Cut	10,675	8,900

Notes:
[1] Average quarry discharges are based on reported values from 2006 to 2011.
[2] Simulated discharge based results of steady-state groundwater submodel calibration.

The simulated average baseflows at 02HB023 and 02HB007 on Spencer Creek are 0.91 m^3/s and 1.08 m^3/s, respectively. This represents simulated values that are approximately 9% and 15% higher than the estimated average baseflow values at the two gauges (0.826 m^3/s and 0.922 m^3/s, respectively).

It should be noted that both WSC gauges are highly influenced by flow regulation from Valen and Christie Reservoirs and 02HB007 is affected by quarry discharge. Such a high degree of alteration to the natural flow regimes compromises the accuracy of conventional baseflow separation techniques and introduced uncertainty into the estimated values. This, in turn, reduced the utility of these values as secondary calibration points (especially 02HB007 which is directly influenced by Christie Reservoir and quarry discharge) for the steady-state model. Total flow measurements at these gauges were very important for calibration of the GSFLOW model, which accounted for runoff, upstream inflows, and outflows from the reservoirs and quarry.

4.3.2 Updates to the PRMS Submodel

As discussed in Section 2.3.5, the PRMS submodel inputs were updated to create a single unified climate dataset. Beyond this update to the forcing functions, no substantive changes were made to parameterization of the PRMS submodel. All model values remained consistent with the parameter values presented and discussed in Earthfx (2014).

4.4 *GSFLOW Model Calibration Results*

After completing the submodel pre-calibration work, the integrated GSFLOW model calibration was performed using an iterative process in which results of successive model runs were reviewed and used to improve the estimates of model parameters. As noted earlier, the model revisions were focused on the groundwater submodel, with no changes made to the hydrologic submodel parameters. Specifically, the groundwater submodel parameters (hydraulic conductivity and storage properties) were subject to additional refinement during the transient calibration.

The updated GSFLOW model was re-calibrated to the nine-year period from October 2006 to September 2015. The calibration period covers an extreme dry year ($_{WY}$2012) and a number of relatively wet years. This period also contains the largest amount of transient water level data for calibration purposes including PGMN and other continuous water level monitoring data, WTRS data for actual water takings, and WSC streamflow data. The continuous groundwater level data and the WSC streamflow data served as the primary calibration targets. Discharge from the quarries served as additional checks on the calibration.

4.4.1 Groundwater Calibration

Checks on the calibration were done by visual comparison of hydrographs of simulated and observed groundwater levels. The groundwater results are presented here as comparisons between relative potentials, as differences in the absolute elevations were found despite the close match obtained with the steady-state model. The hydrographs show results of the GSFLOW simulations as red lines and observed groundwater levels in blue.

4.4.1.1 *Calibration Municipal and PGMN Wells*

The focus of the groundwater model calibration update was on the Greensville municipal supply wells FDG01 and TW-2-13. The PGMN wells provide for a broader assessment of the regional model calibration, while calibrating to the Greensville municipal supply wells improves confidence in the model's suitability for undertaking the Tier 3 Risk Assessment.

The hydrograph for FDG01 (Figure 4.20) shows a very close match to absolute water level elevations at the start and end of the simulation; however, simulated water level deviated noticeably from the observed levels between the spring of 2008 until the fall of 2010. Further inspection of the reported data showed apparent shifts in the observed data that corresponded to well rehabilitation work conducted on the municipal well by Lotowater in April 2008. The dramatic improvements to well efficiency are evidenced by the positive 1 to 2 m offset of the observed data from the simulated levels immediately following this work.

Despite the offset caused by changes in well efficiency, patterns in the observed data are still captured in the simulated water levels.

The hydrographs for the three instrumented test holes completed as part of the municipal water supply exploration program are compared to simulated levels in Figure 4.21 (TW-2-13), Figure 4.22 (TW-1-13) and Figure 4.23 (TW-3-13). Due to the recent completion of these wells, there are limited data with which to compare the simulated water levels in the new municipal supply well and accompanying test holes. Despite the limited data coverage, a reasonable match to the timing of water level fluctuations was achieved. As shown in Figure 4.21, the observed recession in water levels in TW-2-13 appears to be more gradual than in the simulated levels. It should be noted that the observed data corresponds to the period prior to undertaking well enhancement in 2015 and 2016, which are expected to have resulted in increased local hydraulic conductivities. This locally enhanced hydraulic conductivity was considered in the model update, which could have contributed to the apparent discrepancy between the simulated and observed water levels in the new municipal well. The water level data compilation for the new municipal wells was subject to the data transfer period of the previous Tier 3 study (terminating in mid-2015); as a result, water levels following the enhancement activities on TW-2-13 were not available for use in model calibration.

Hydrographs for PGMN wells W0000002, W0000295, and W0000005 are presented in Figure 4.24, Figure 4.25 and Figure 4.26, respectively. All of these wells are interpreted to be completed in the weathered bedrock contact aquifer (model layer 5), or (in the case of W000005) within coarse-grained overburden sediments that are assumed to be in good hydraulic communication with the shallow bedrock contact aquifer. As can be seen, the transient model reasonably matches the relative timing and amplitude of seasonal fluctuations. Simulated levels at wells W0000295 and W0000002 were higher than observed values, but generally within 2 m of the targets throughout the simulated period.

4.4.1.2 *Calibration to SQE Groundwater Monitors*

Simulated water levels at new SQE area monitors (shown in Figure 2.3) are compared against observed data in Figure 4.27 to Figure 4.37. Monitoring well nests MW06-1 (Figure 4.30 to Figure 4.32) and MW06-4 (Figure 4.33 to Figure 4.34) are the closest to the existing North Quarry and South Quarry excavations, respectively. In general, the simulated levels show a good match to the vertical water level profiles at these locations, with a perched water table in the Guelph and depressed water levels in the Lower Eramosa that closely resemble those observed in the deeper Goat Island/Gasport aquifer.

Calibration to water levels in the Eramosa and Goat Island/Gasport aquifers at MW06-1 are shown in Figure 4.31 to Figure 4.32, and those for MW06-4 are shown in Figure 4.33 through Figure 4.34. Observed water levels in the Lower Eramosa and Goat Island/Gasport are approximately 15 to 20 m below the Guelph water levels at these locations, which is consistent with the predicted vertical water level difference between these units, ranging from 13 m to 16 m. The smaller range in the simulated results is due to over-prediction of water levels in the Lower Eramosa and Goat Island/Gasport aquifers.

Monitoring well nest MW06-3 is the furthest of the new SQE monitors from the edges of the active quarry. Simulated water levels are compared to observed values in Figure 4.35, Figure 4.36 and Figure 4.37 for the Upper Eramosa, Lower Eramosa, and Gasport, respectively. Simulated heads show a good match to absolute water level elevations as well as seasonal variation in the observed data. It was noted that water level fluctuations in the Gasport monitor MW-06-3-B-1 (Figure 4.37) are high relative to the predicted response, bearing a close similarity to those observed in the Lower Eramosa monitor MW-06-3-B-3 (Figure 4.36). This may be explained by a greater degree of vertical hydraulic communication across the Vinemount aquitard than is represented in the model at this location.

4.4.1.3 *Calibration to Quarry Sumps and Discharge*

Figure 4.38 through Figure 4.45 present hydrographs of the simulated and reported discharges from the Lafarge Quarry. Figure 4.38 presents the daily discharge from the North Quarry. Both the simulated and daily rates are quite variable, but as the figure shows, the model is able to capture the timing of the increases and decreases in discharge. This indicates that the simplified operating rules developed for the simulating quarry discharge are reasonable. As was discussed in the Earthfx (2014), the model estimated discharge based on simulated inflows rather than using reported discharge values; this allowed the model to be used for predictive simulations for periods when the discharges would be unknown (e.g., under future conditions or during simulated droughts). Figure 4.39 presents monthly average discharge values for the North Quarry. The figure confirms that the model is able to match the seasonality of the discharges but the simulated volumes are generally lower than the reported.

Figure 4.40 presents the simulated and reported daily discharge from the South Quarry. Discharge rates are more variable than for the North Quarry. The simulated monthly volumes, shown in Figure 4.41, are closer to the reported values, yet over-predict slightly. Figure 4.42 and Figure 4.43 show the simulated and reported daily and monthly discharge from the Processing Area to the Railway Cut. The predicted total volumes discharged offsite through the Railway Cut are also in agreement with reported discharge from the Processing Area.

It should be noted that recycling of water within the Lafarge Quarry is not captured in the model. In addition, site water management practices are complex and have been simplified to allow simulations in time-periods without reported values. Operational seasonality is also not strictly represented; it was assumed in the model that the quarry is constantly and consistently dewatered. Despite these limitations, the simulated heads around the quarry match the observed heads and the total volume of simulated water discharged offsite also matches reported values.

Figure 4.44 and Figure 4.45 show simulated and reported daily and monthly discharge from the Flamboro Quarry. Dewatering of the quarry is simulated by floor drains and on-site storage is not simulated. Additionally, operation seasonality is not captured as anecdotal evidence suggests that the quarry floor is allowed to flood in the winter (with correspondingly higher flows in the spring to dry out the quarry floor). This explains the discrepancy between the observed discharges during the winter and spring.

4.4.2 Streamflow Calibration

Two streamflow calibration gauges were employed during the GSFLOW model update and recalibration: Spencer Creek at Dundas (WSCID: 02HB007) and Spencer Creek at Highway No. 5 (WSCID: 02HB023). The characteristics of these stations were described in the watershed characterization (Earthfx, 2014). Two calibration periods were considered during the model recalibration phase: 1) October 2009 through September 2013 which covers changes to the Lafarge Quarry footprint and depth of excavation and the 2010 SOLRIS land coverage and 2) October 2005 through September 2011 which corresponds to the calibration period considered in the original Tier 3 efforts. Newer data, collected after 2012 and the conclusion of the original Tier 3 calibration efforts were treated as validation data.

Three objective functions were used to optimize the calibration of the model with respect to matching observed streamflow: Nash-Sutcliffe (1970) efficiency, Log Nash-Sutcliffe efficiency, and the observed percent volume difference between the observed and simulated streamflow. The Nash-Sutcliffe efficiency (NSE), given by:

$$\text{Nash Sutcliffe Efficiency} = 1 - \frac{\sum_{n=1}^{nobs}(Q_o - Q_s)^2}{\sum_{n=1}^{nobs}(Q_o - \overline{Q}_o)^2} \qquad \text{(Equation 4)}$$

where Q_o is the observed flow and Q_s is the simulated flow. The NSE can range from 1 to minus infinity, with 1 being a perfect fit. It should be recognized that the model simulates flow on a daily basis and would not be expected to achieve perfect matches with observed mean daily flows. Additionally, daily climate observations are made on a synoptic interval which ends at 0600Z (1:00 a.m. in EST) rather than at midnight. Because of the importance of matching baseflow and low flows in this study, the Log Nash-

Sutcliffe, which is considered a better measure of the model calibration to low flows (Krause *et al.*, 2005), was given particular emphasis.

Overall calibration statistics for the two gauges within the Spencer Creek subwatershed are provided in Table 4.6 for the various simulation periods. The model achieved NSEs between 0.62 and 0.63 for the daily values during the primary calibration period. Log NSEs compare favourably, with values in excess of 0.70, suggesting a good match to low flow conditions. Daily results were aggregated over each month, and monthly NSEs from 0.64 to 0.65 were calculated, with monthly Log NSEs generally showing similar results when compared to the non-transformed metrics. NSEs at both gauges are similar to values reported in Earthfx (2014), with a slight reduction in Log NSE (for example, from 0.82 at Spencer Creek near Dundas to 0.72 in the recalibrated model). Log NSEs at Spencer Creek at Highway No. 5 in the recalibrated model remain close to previously reported values (0.80 versus 0.81). Statistics calculated for the validation periods demonstrates a similar level of quality during the 2005 through 2011 period.

Table 4.6: Streamflow calibration statistics for the revised integrated GSFLOW model.

Gauged Basin	Daily		Monthly	
	Nash-Sutcliffe	*Log Nash-Sutcliffe*	*Nash-Sutcliffe*	*Log Nash-Sutcliffe*
***Calibration Period 1** Revised Tier 3*	*(October 2009 – September 2013)*			
Spencer Creek at Dundas (02HB007)	0.62	0.70	0.64	0.70
Spencer Creek at Highway No. 5 (02HB023)	0.63	0.72	0.65	0.76
***Calibration Period 2** Original Tier 3*	*(October 2005 – September 2011)*			
Spencer Creek at Dundas (02HB007)	0.58	0.72	0.57	0.75
Spencer Creek at Highway No. 5 (02HB023)	0.57	0.80	0.56	0.80
***Validation Period** New Data*	*(October 2011 – September 2015)*			
Spencer Creek at Dundas (02HB007)	0.53	0.66	0.55	0.70
Spencer Creek at Highway No. 5 (02HB023)	0.56	0.71	0.55	0.72

Daily streamflow calibration hydrographs are presented on Figure 4.46 and Figure 4.47 for Spencer Creek near Dundas and Spencer Creek at Highway No. 5, respectively. Model performance appears good at Spencer Creek near Dundas. The match to low flows is consistent with the performance exhibited by the original Tier 3 model. A good match to peaks is generally observed, however spring freshet volumes are under-predicted. It was noted during the original Tier 3, however, that the simplified rule curve employed in the simulation of operations at Christie Reservoir maybe affecting the match to spring stream flows. Spring performance is improved at Spencer Creek at Highway No. 5 which sits above the reservoir. Future work might consider employing actual reservoir operations within the model to improve overall model calibration.

4.5 *Figures*

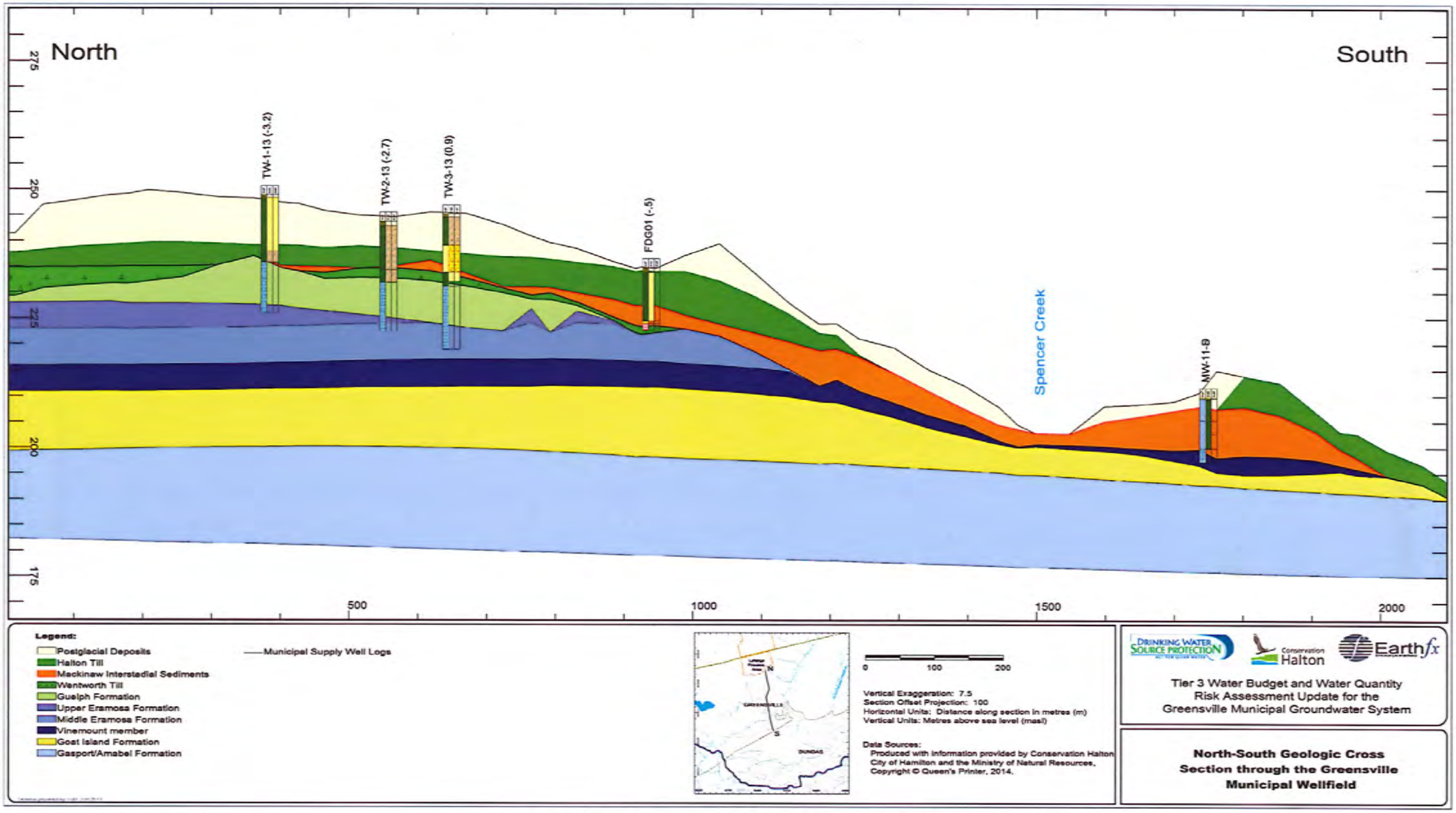


Figure 4.3: North-south geologic cross section through the Greensville municipal wellfield.

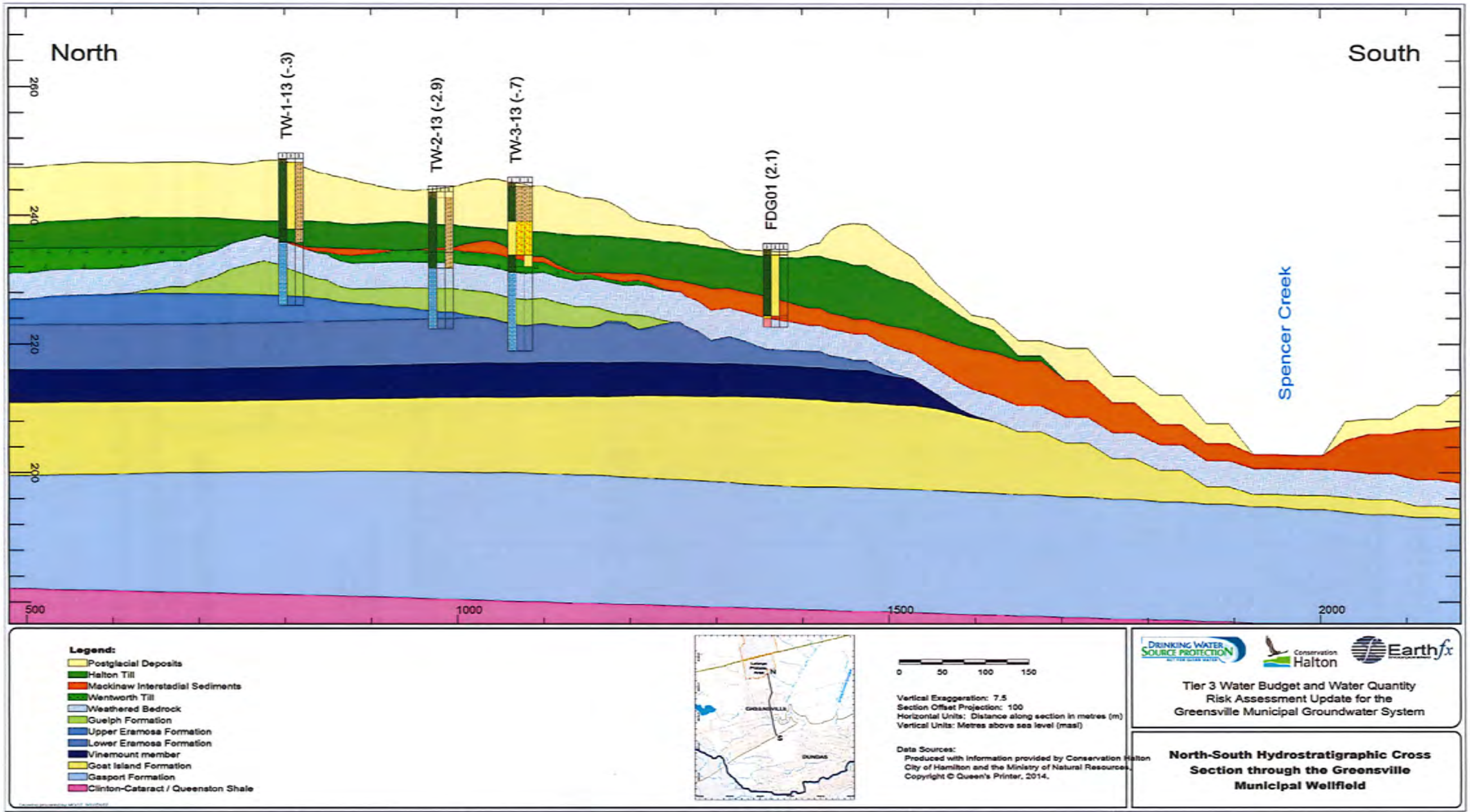


Figure 4.4: North-south hydrostratigraphic section through the Greensville municipal wellfield.

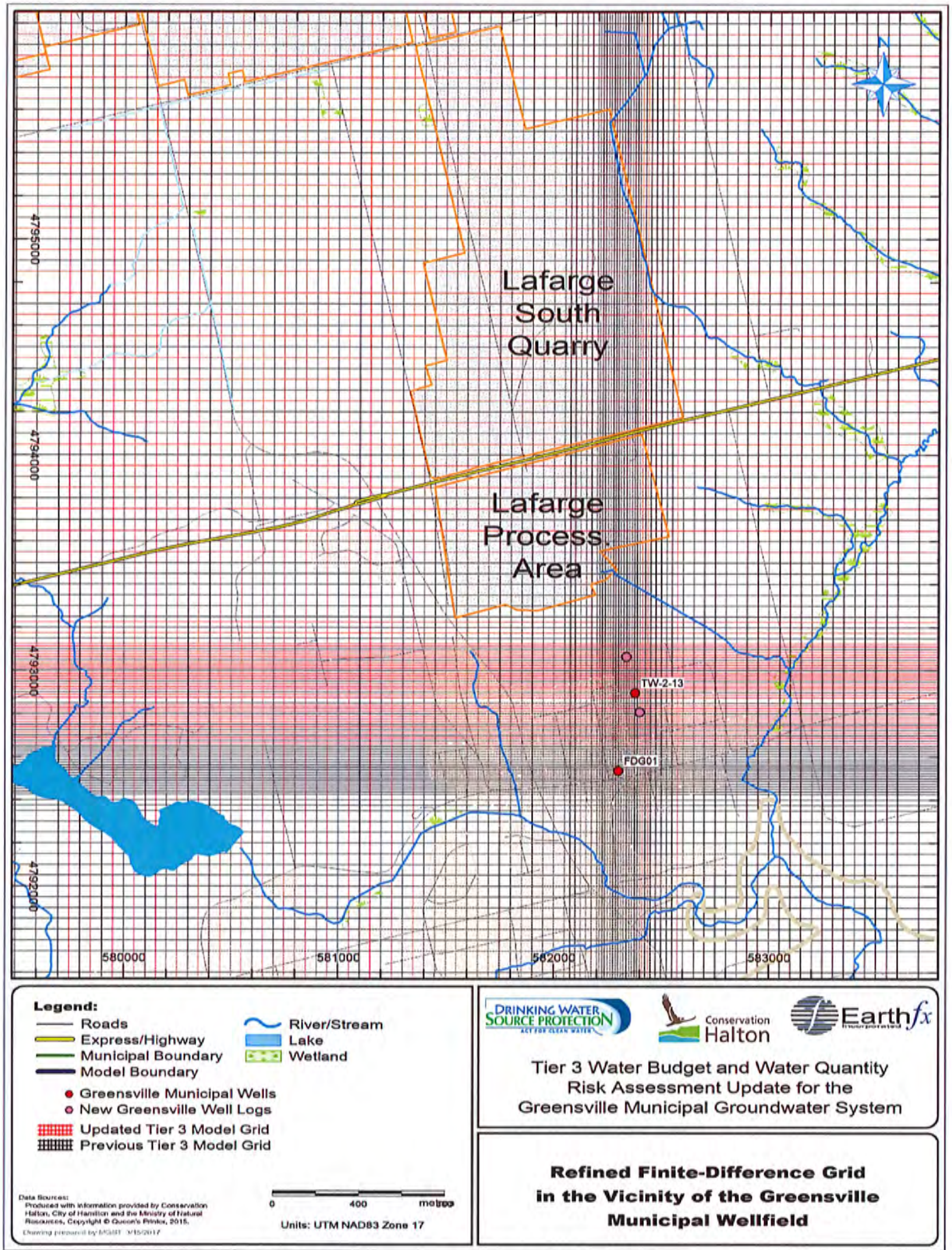


Figure 4.5: Refinement of groundwater model grid in vicinity of the Greensville municipal wellfield.

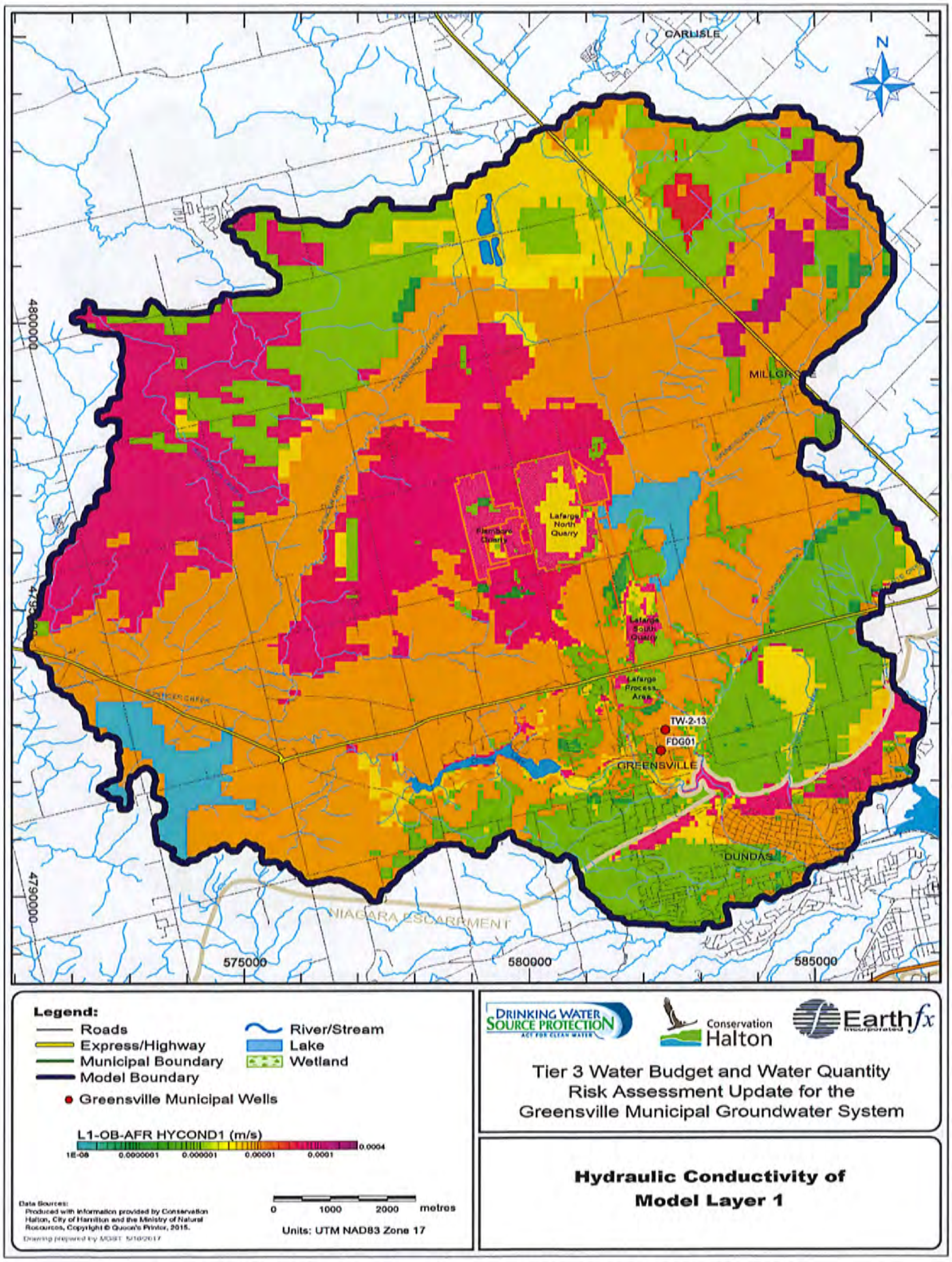


Figure 4.6: Hydraulic conductivity distribution assigned to model layer 1.

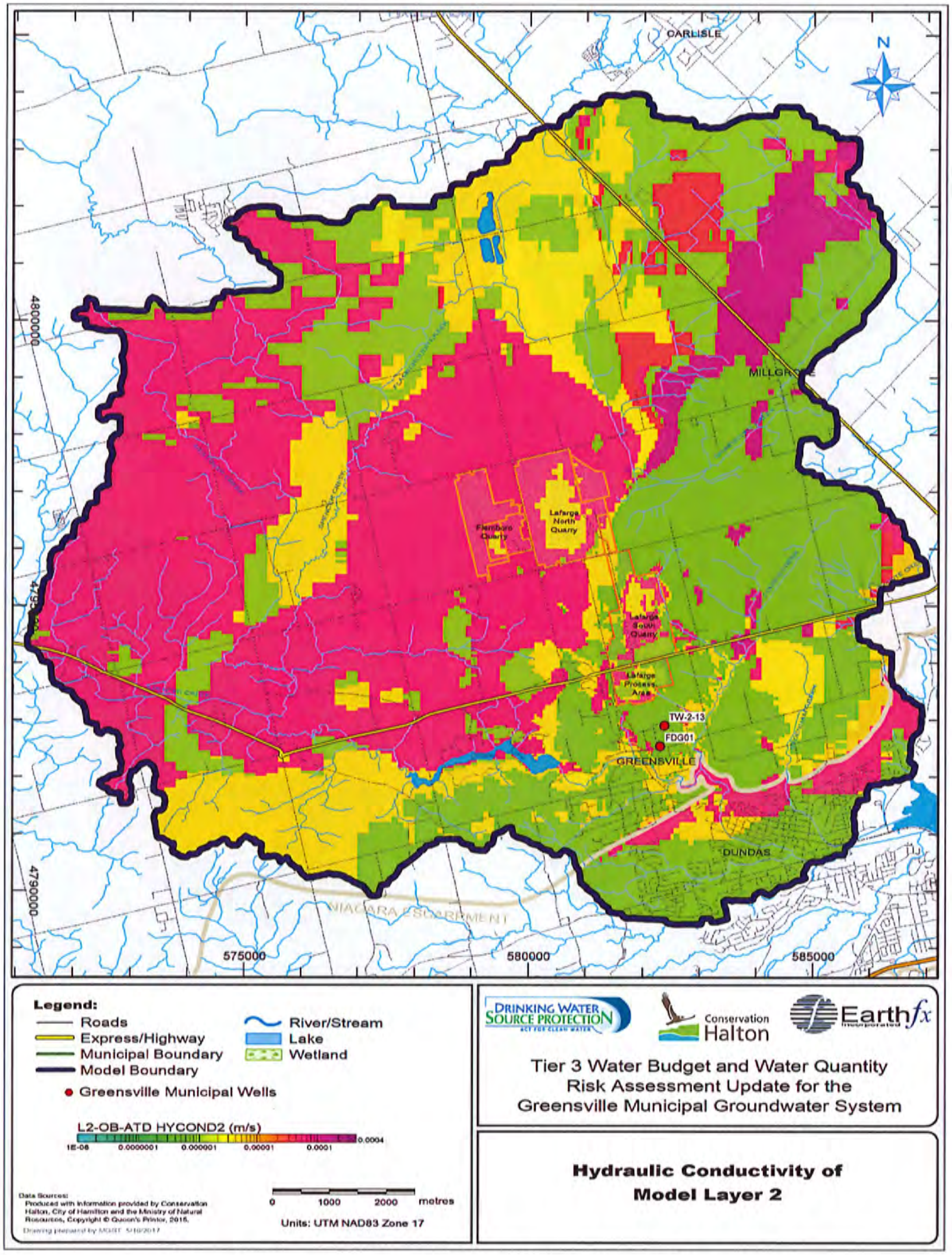


Figure 4.7: Hydraulic conductivity distribution assigned to model layer 2.

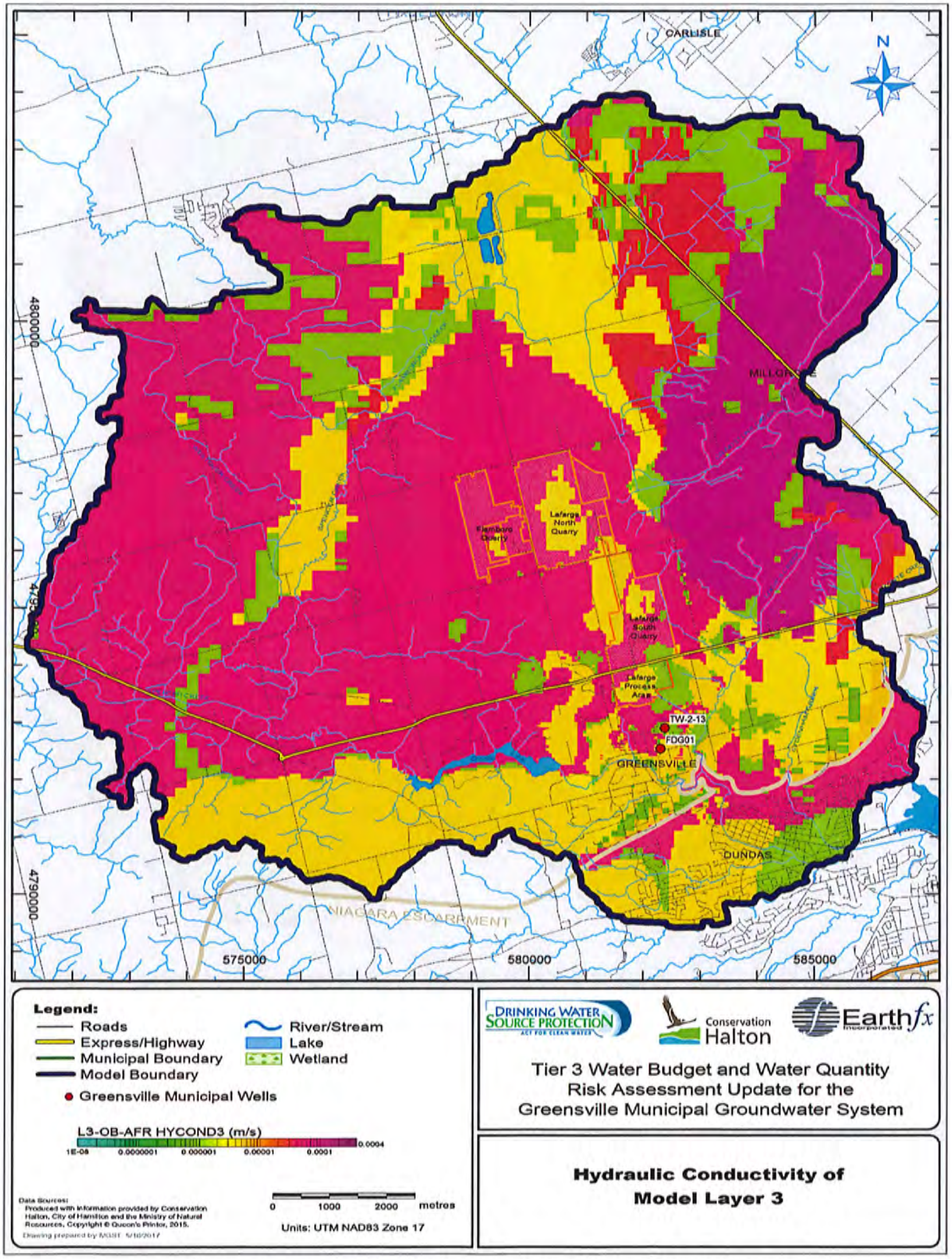


Figure 4.8: Hydraulic conductivity distribution assigned to model layer 3.

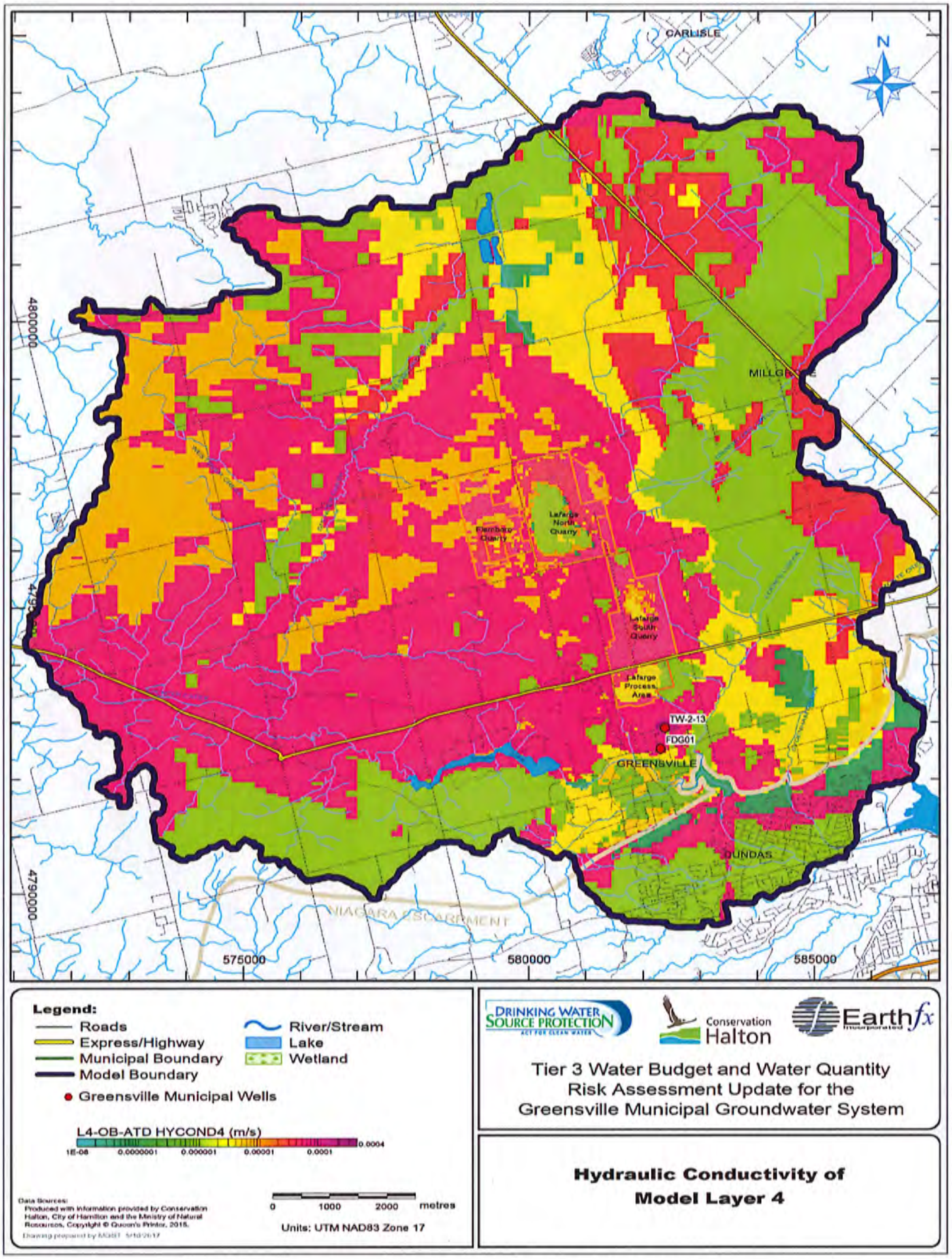


Figure 4.9: Hydraulic conductivity distribution assigned to model layer 4.

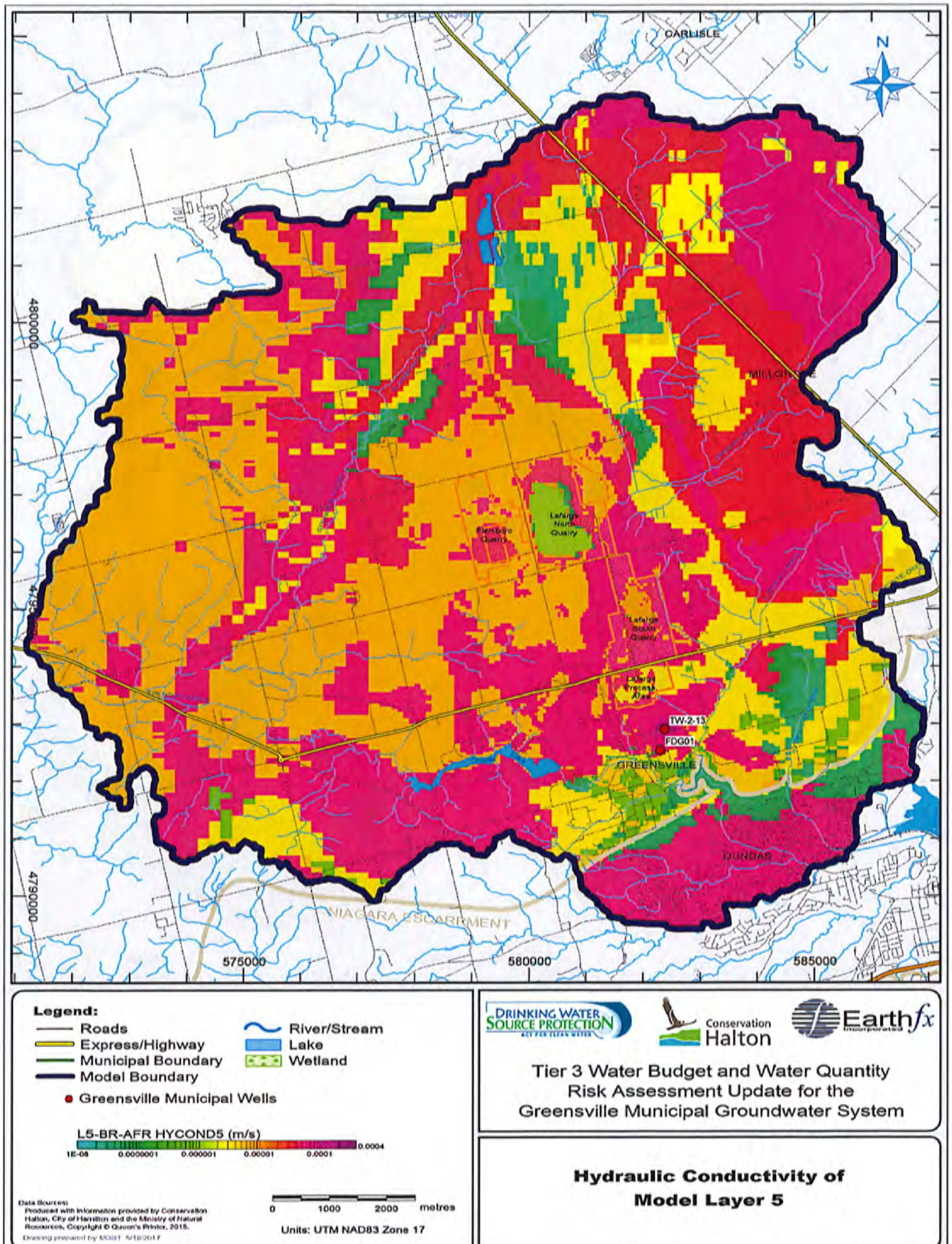


Figure 4.10: Hydraulic conductivity distribution assigned to model layer 5.

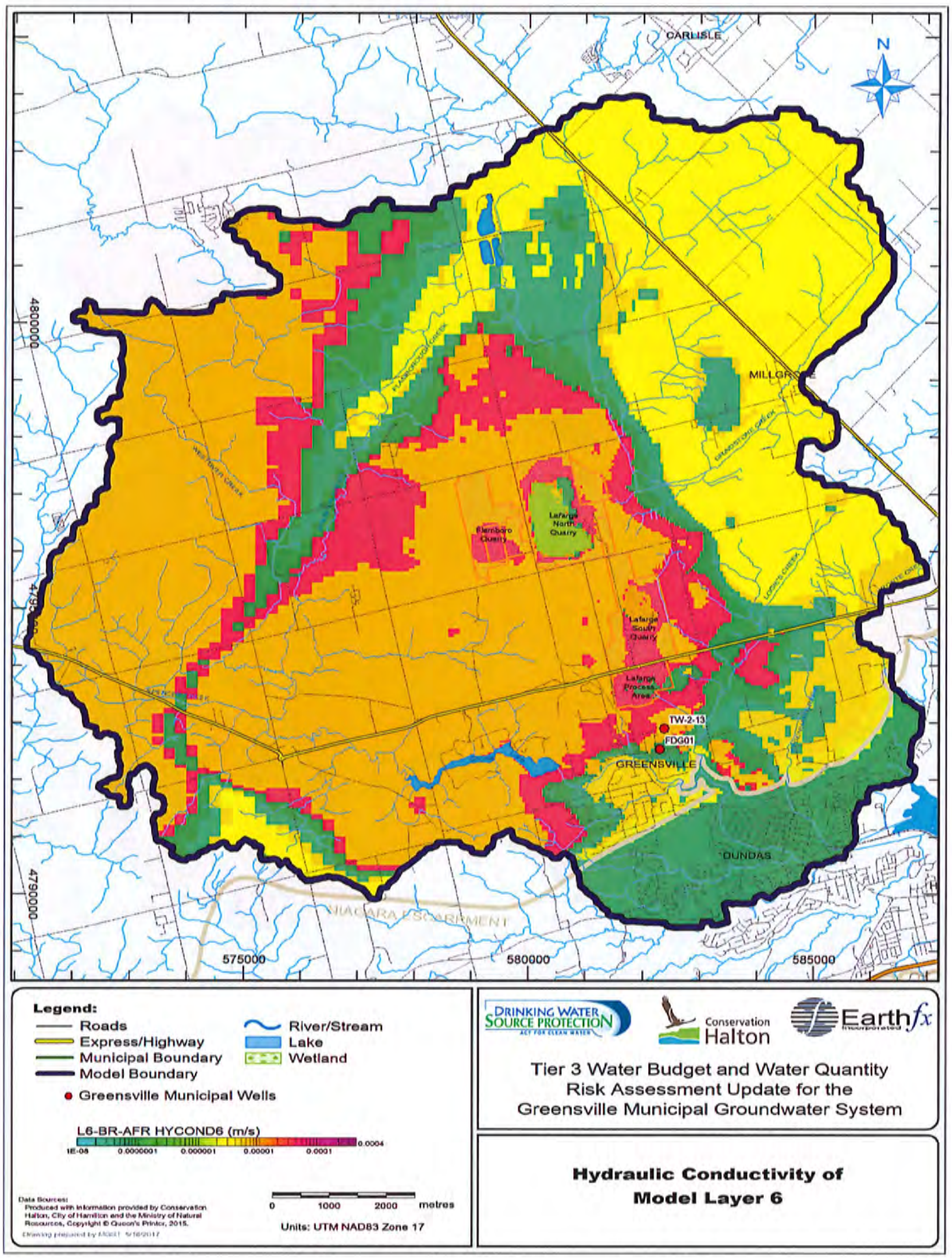


Figure 4.11: Hydraulic conductivity distribution assigned to model layer 6.

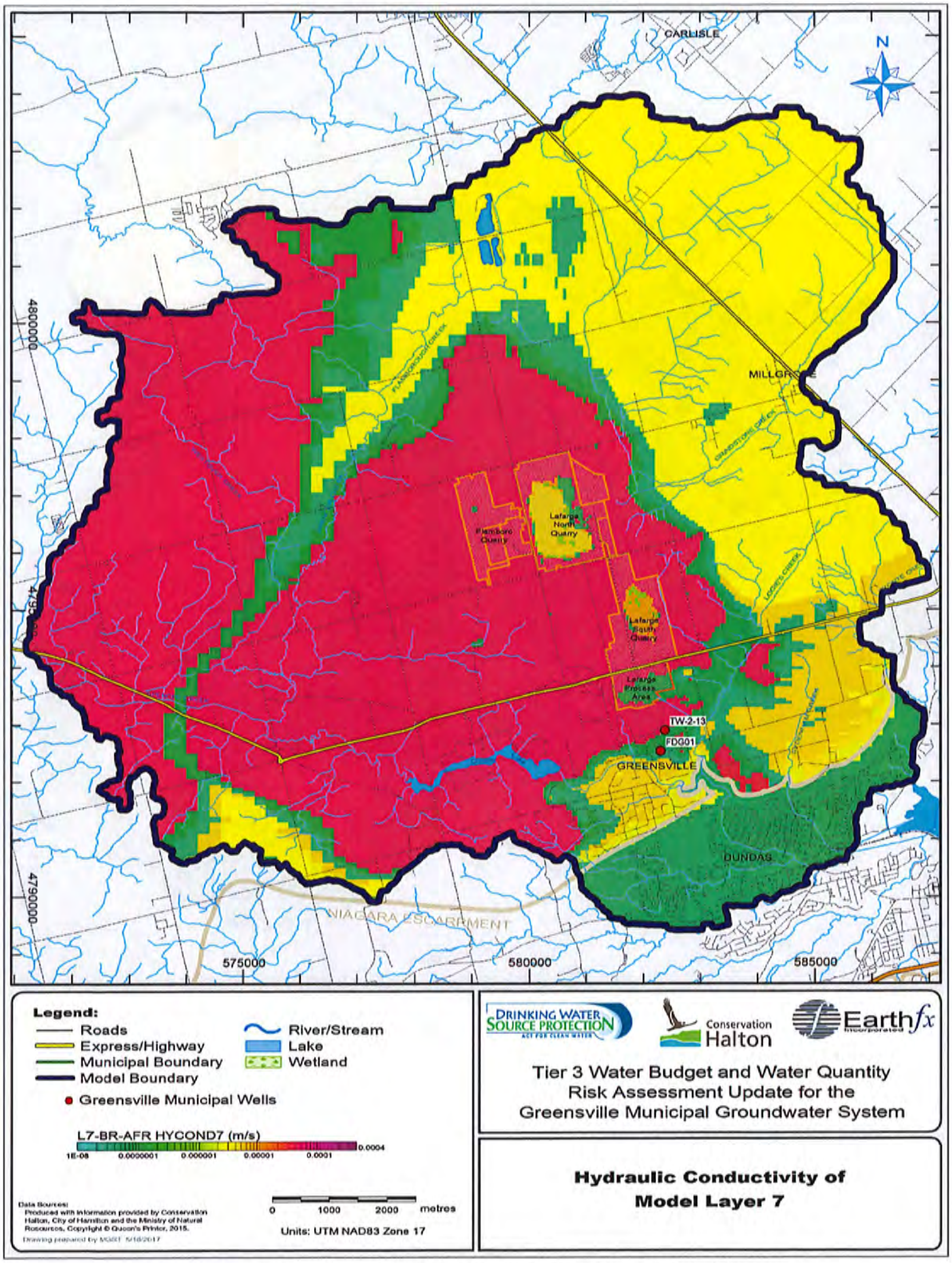


Figure 4.12: Hydraulic conductivity distribution assigned to model layer 7.

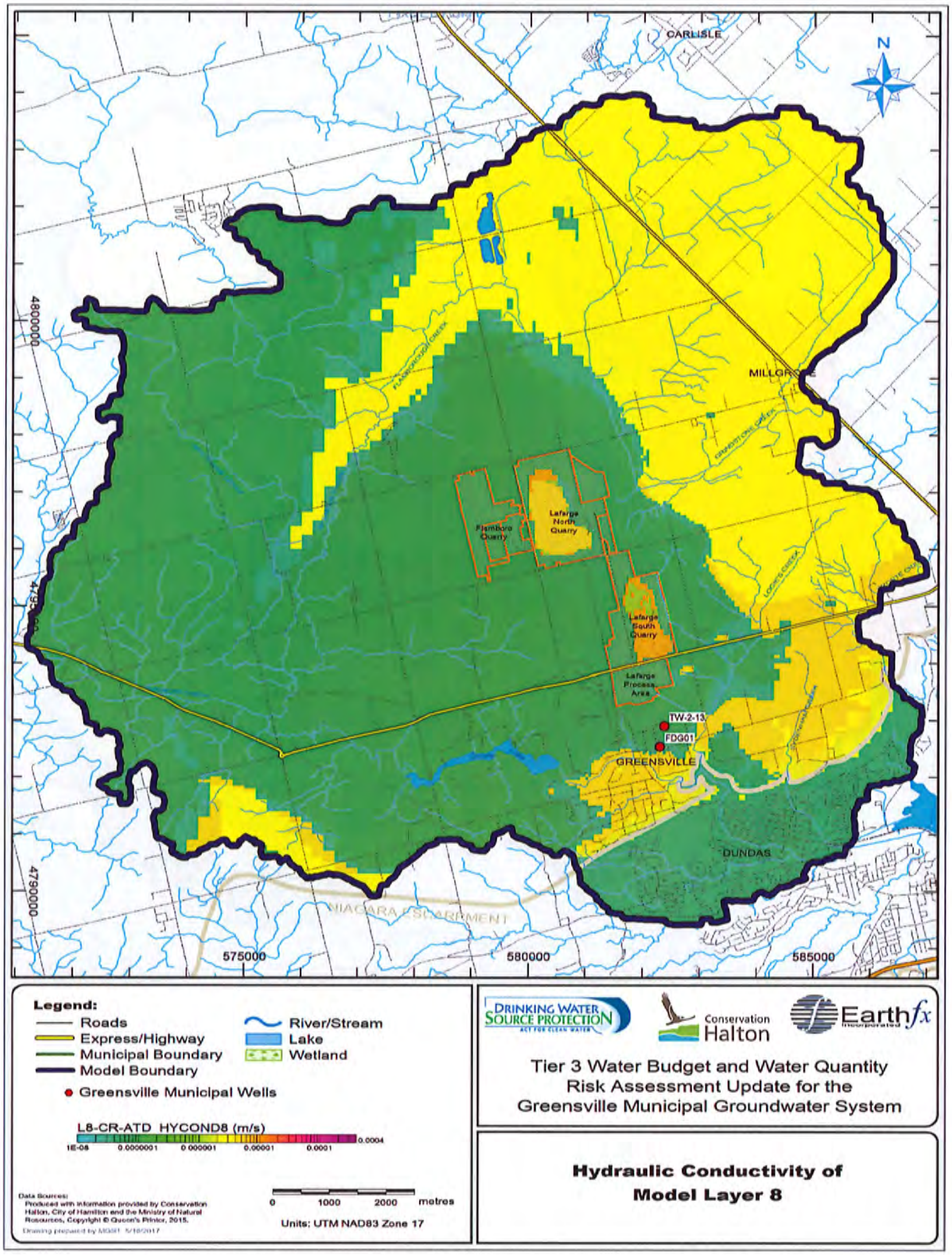


Figure 4.13: Hydraulic conductivity distribution assigned to model layer 8.

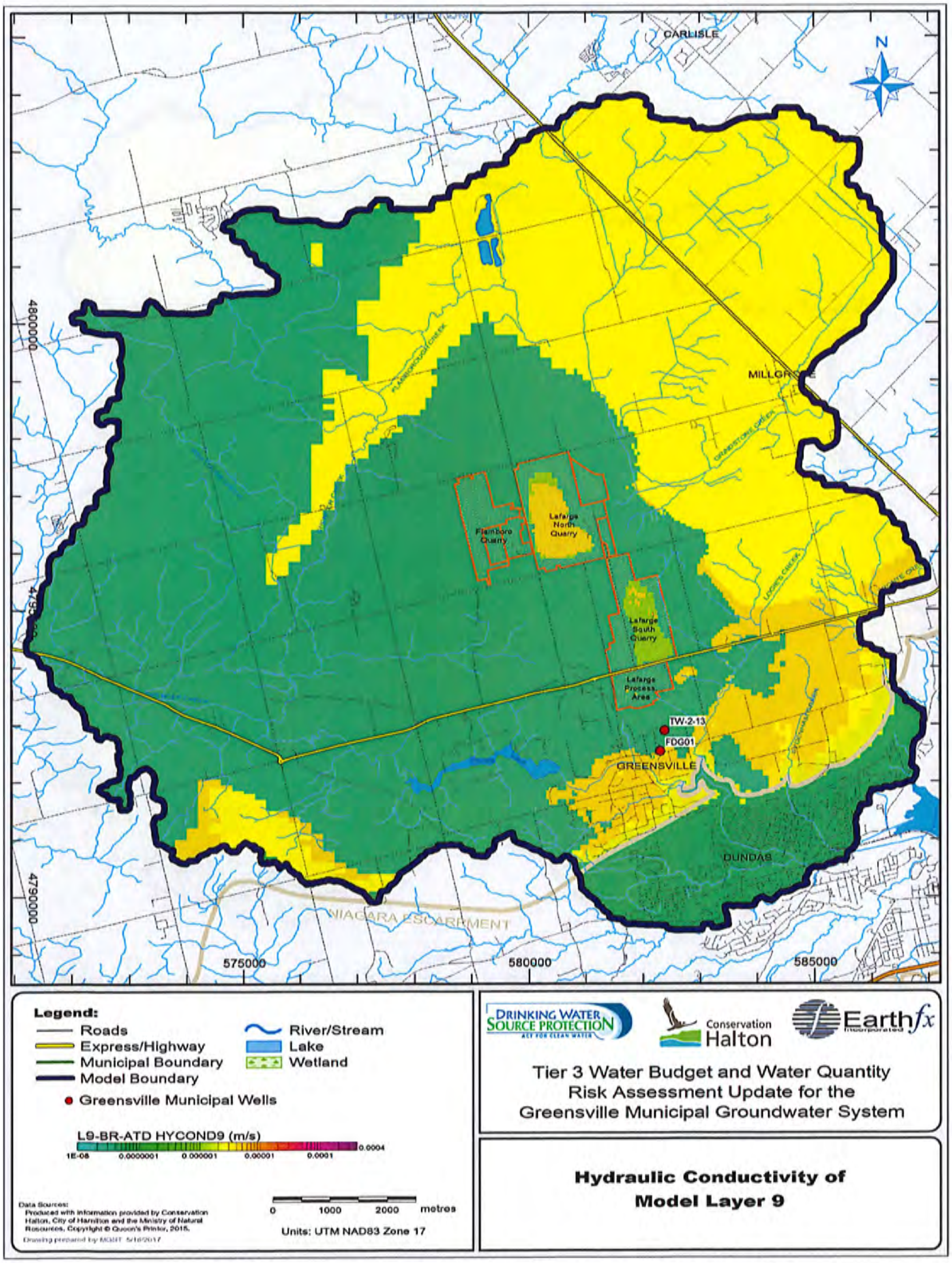


Figure 4.14: Hydraulic conductivity distribution assigned to model layer 9.

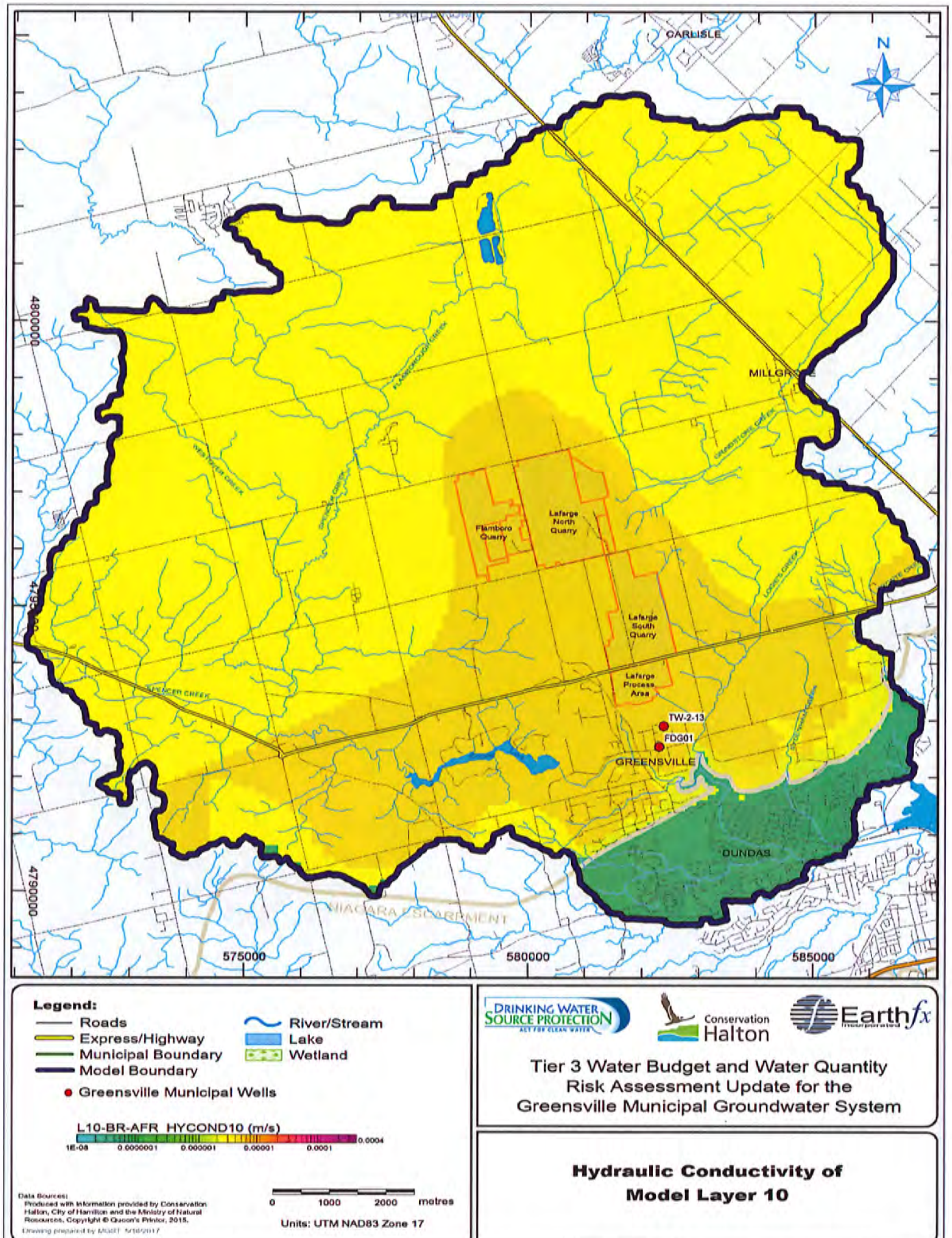


Figure 4.15: Hydraulic conductivity distribution assigned to model layer 10.

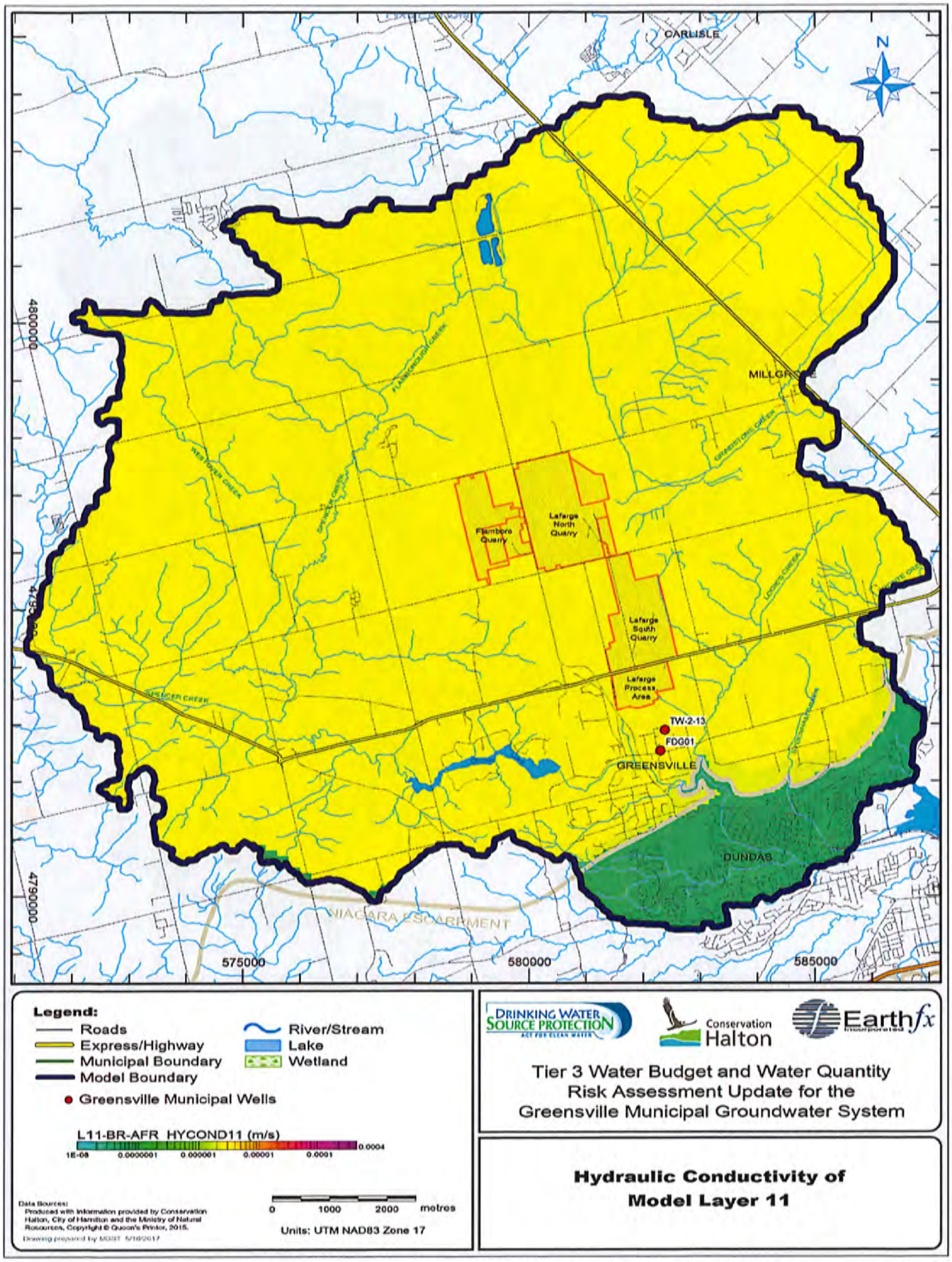


Figure 4.16: Hydraulic conductivity distribution assigned to model layer 11.

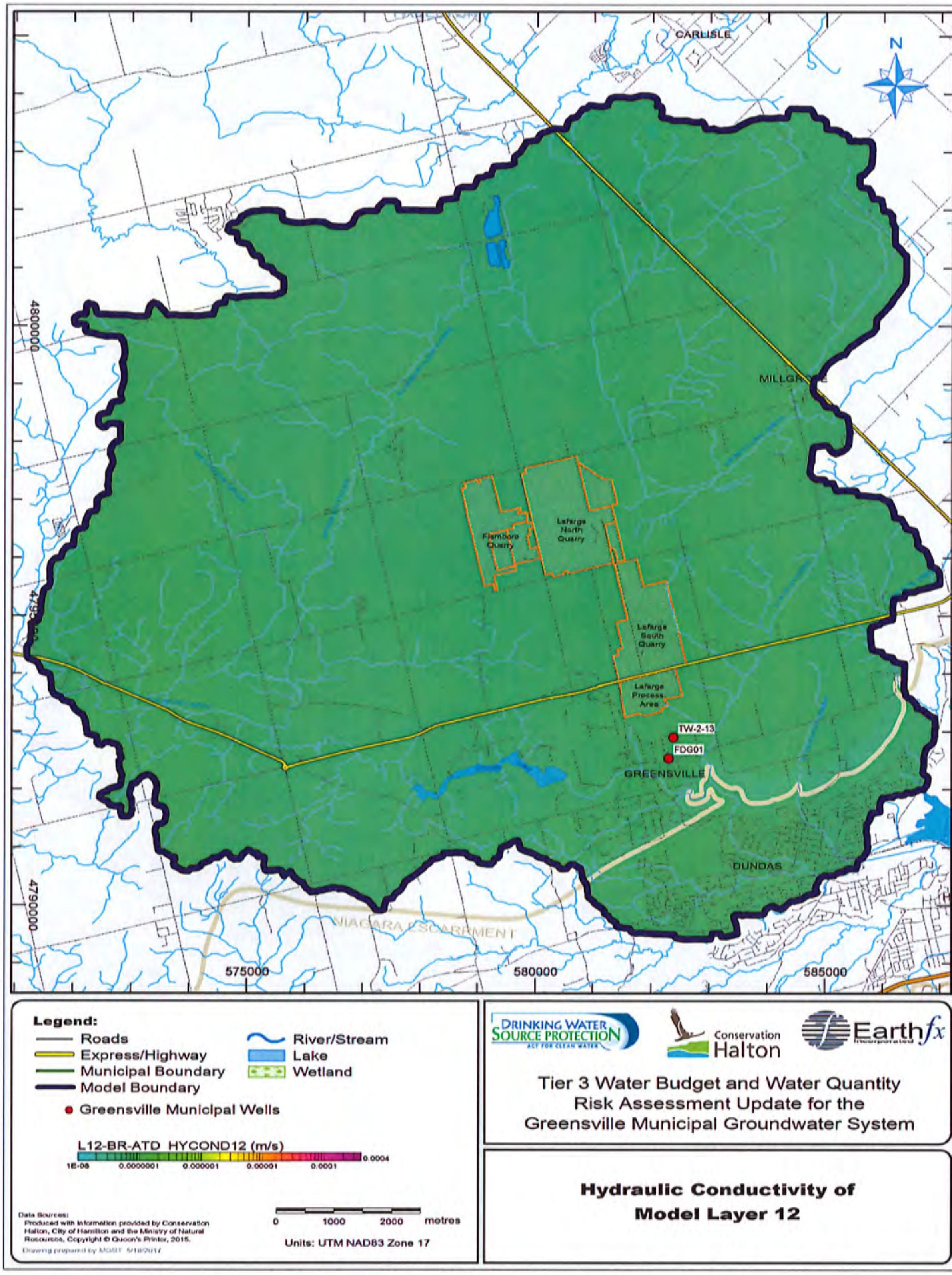


Figure 4.17: Hydraulic conductivity distribution assigned to model layer 12.

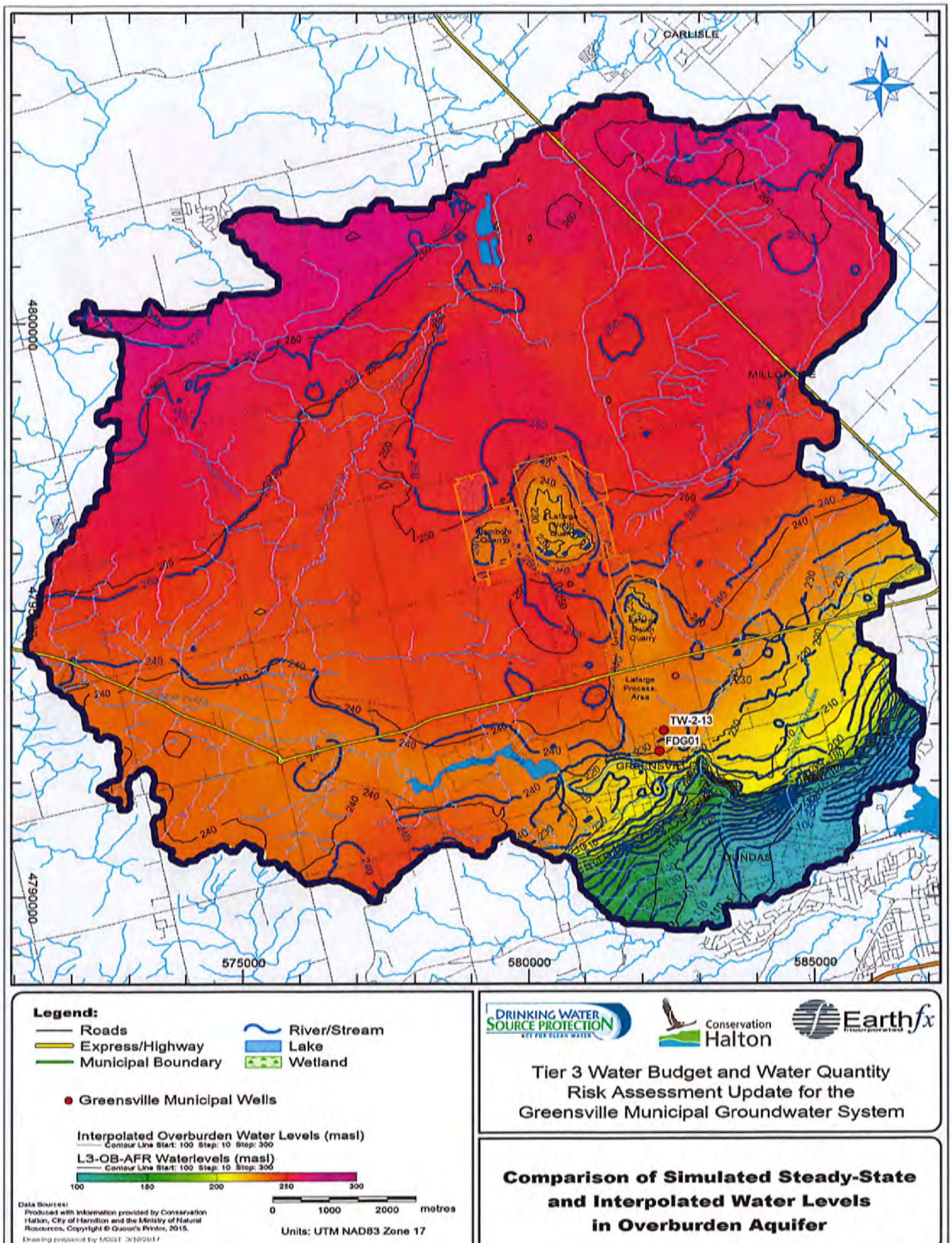


Figure 4.18: Comparison of simulated (black) and interpolated static water levels (blue) in the overburden aquifer system.

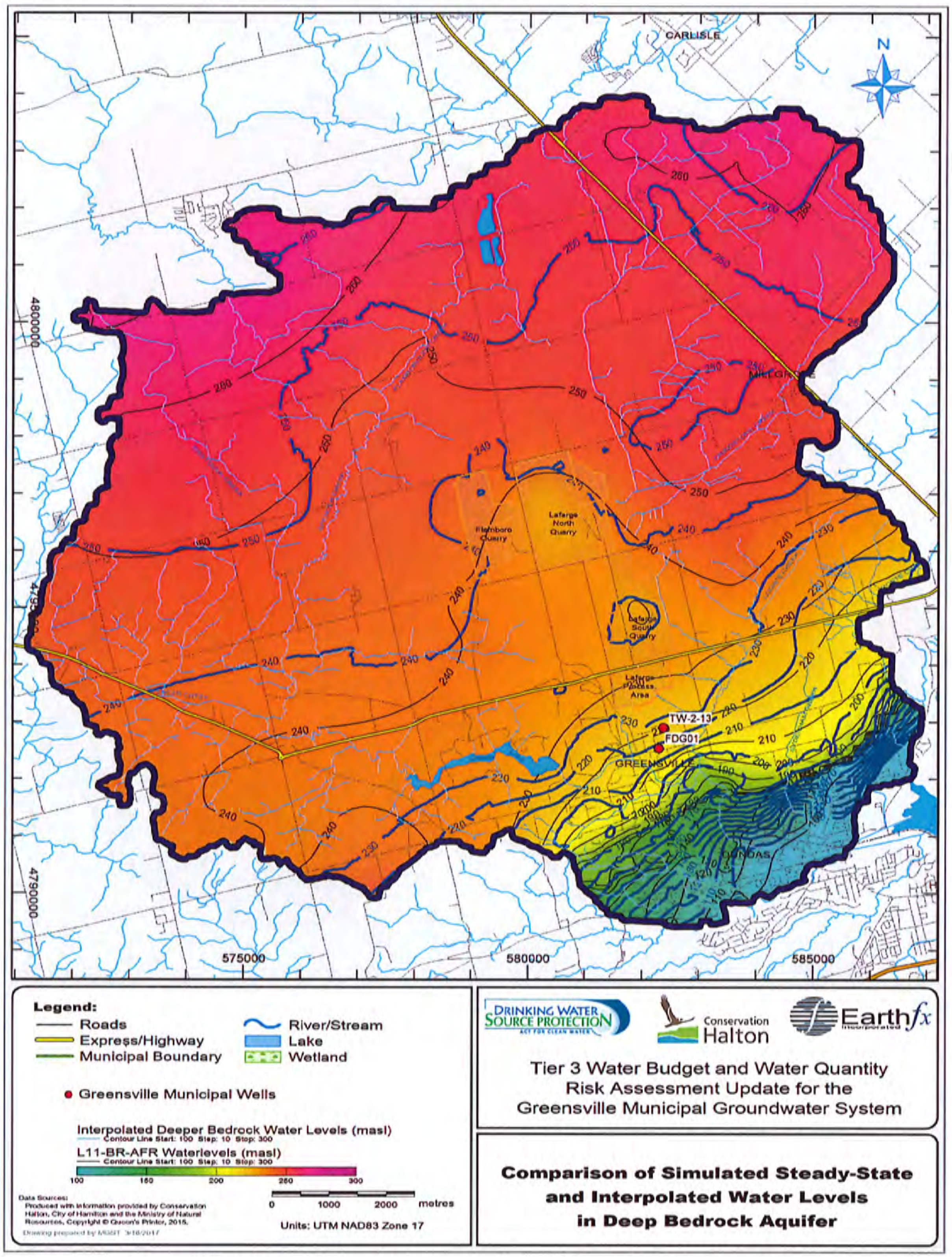


Figure 4.19: Comparison of simulated (black) and interpolated static water levels (blue) in the deep aquifer system.

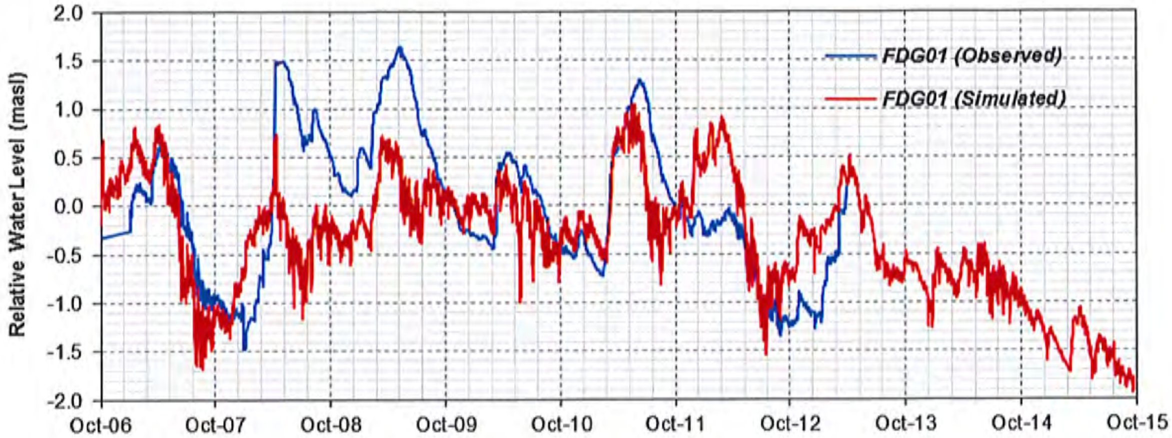


Figure 4.20: Simulated and observed heads at municipal supply well FDG01.

Figure 4.21: Simulated and observed heads at future municipal supply well TW-2-13.

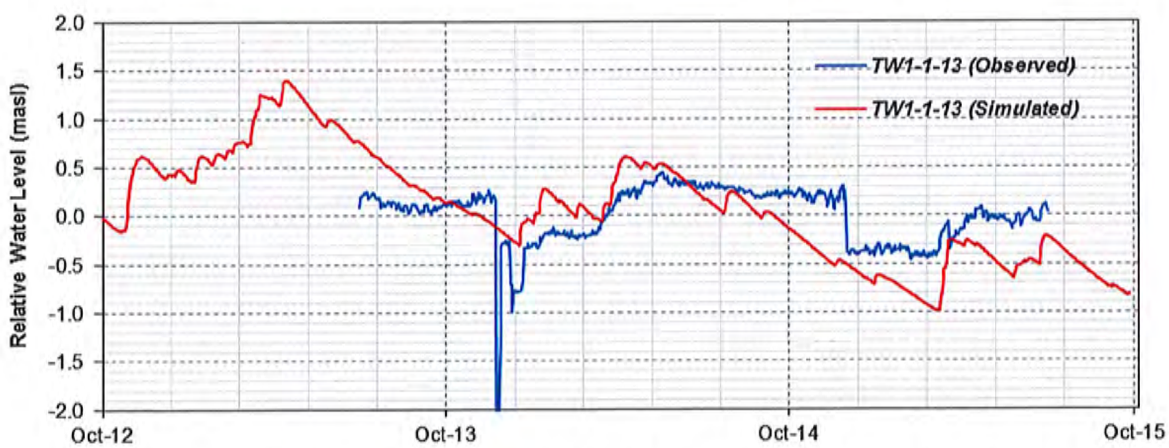


Figure 4.22: Simulated and observed heads at municipal test hole TW-1-13.

Figure 4.23: Simulated and observed heads at municipal test hole TW-3-13.

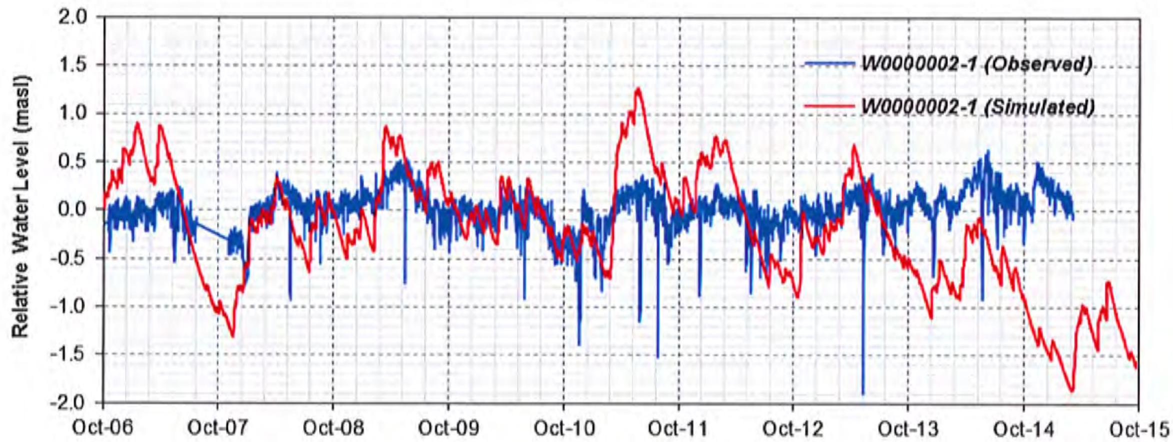


Figure 4.24: Simulated and observed heads at PGMN well W0000002-1

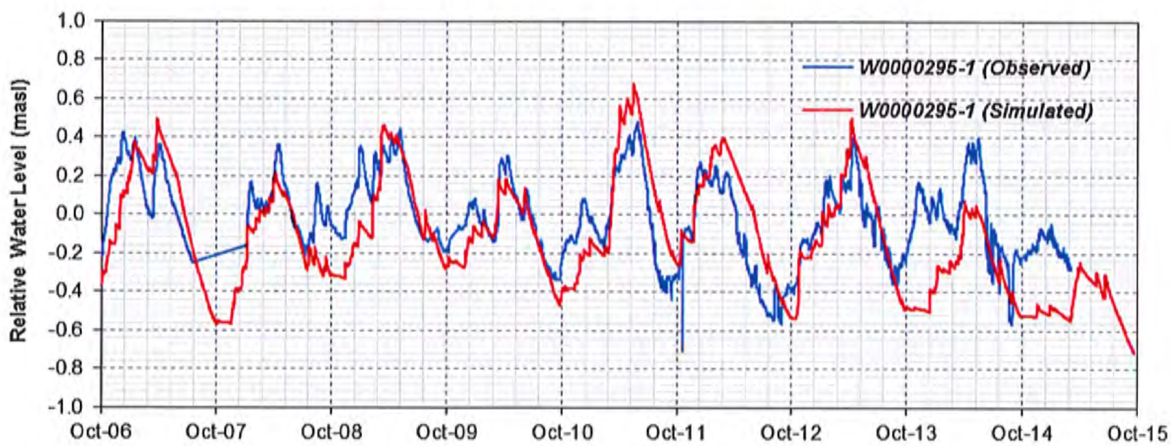


Figure 4.25: Simulated and observed heads at PGMN well W0000295-1.

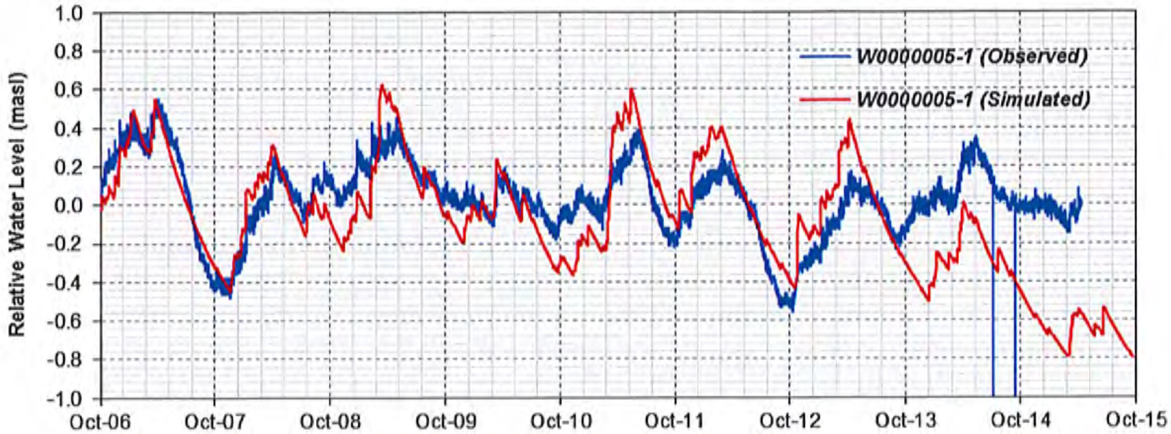


Figure 4.26: Simulated and observed heads at PGMN well W0000005-1.

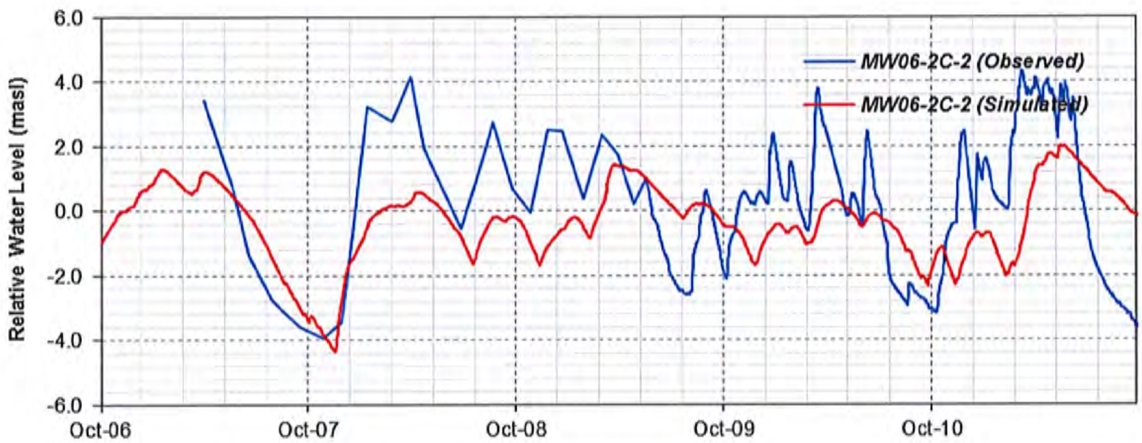


Figure 4.27: Simulated and observed heads at SQE well MW06-2C-2 (Guelph).

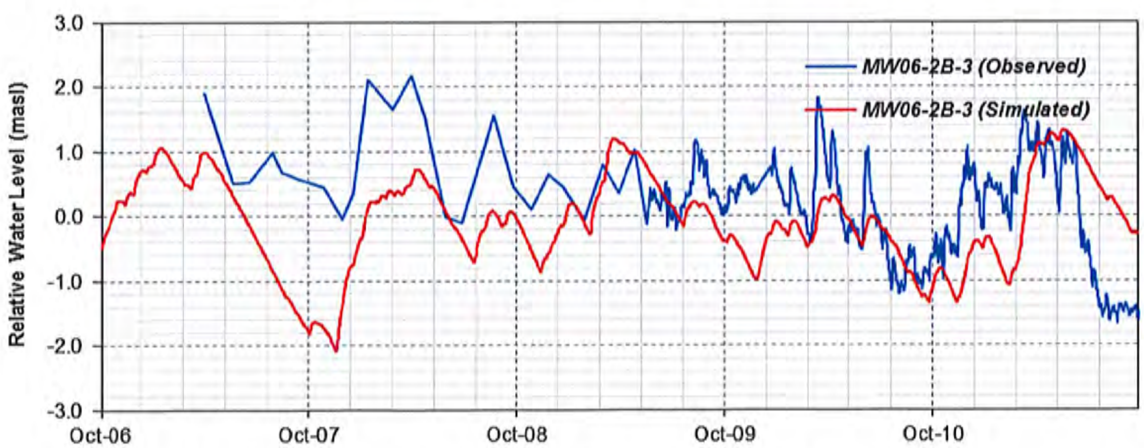


Figure 4.28: Simulated and observed heads at SQE well MW06-2B-3 (Lower Eramosa).

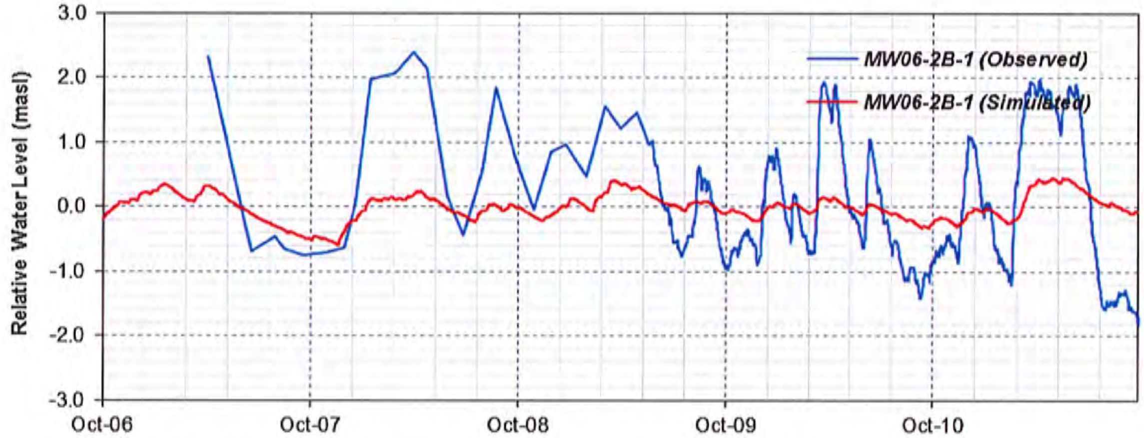


Figure 4.29: Simulated and observed heads at SQE well MW06-2B-1 (Gasport).

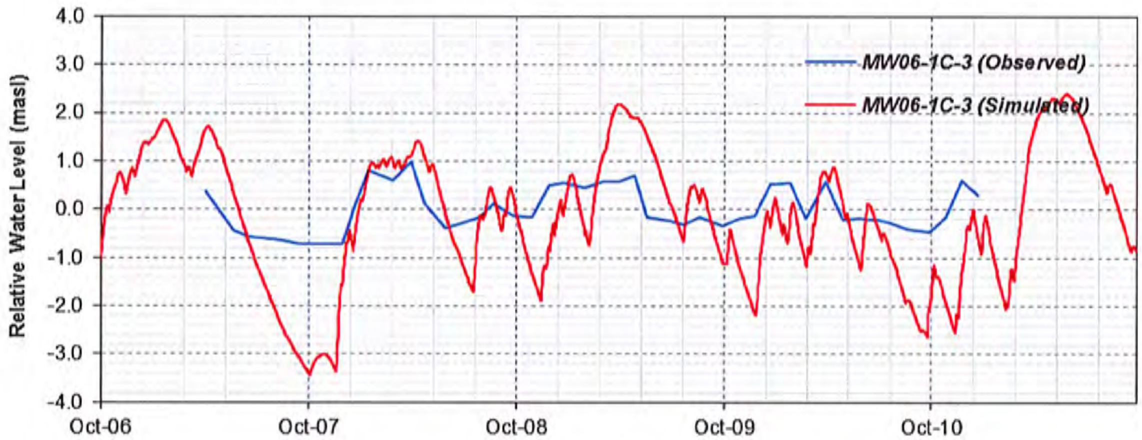


Figure 4.30: Simulated and observed heads at SQE well MW06-1-C-3 (Guelph).

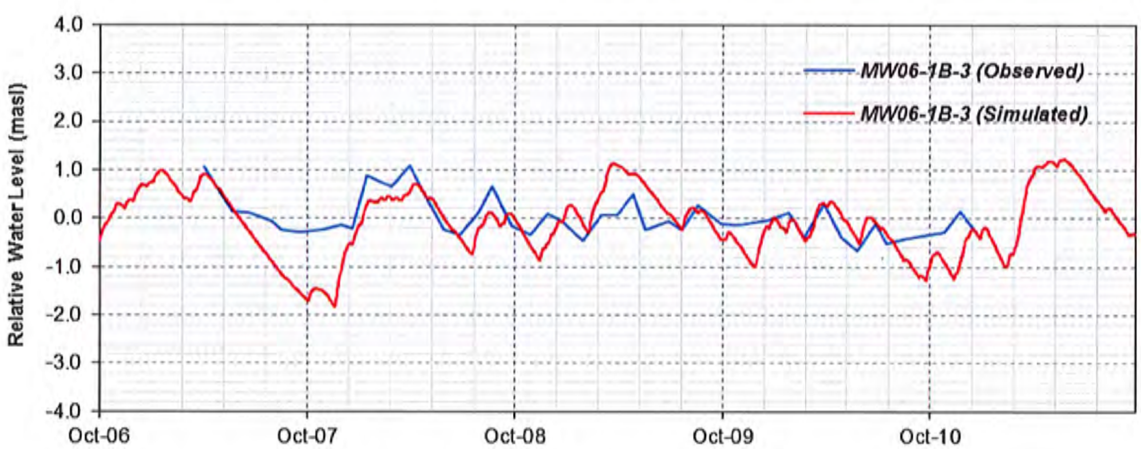


Figure 4.31: Simulated and observed heads at SQE well MW06-1-B-3 (Lower Eramosa).

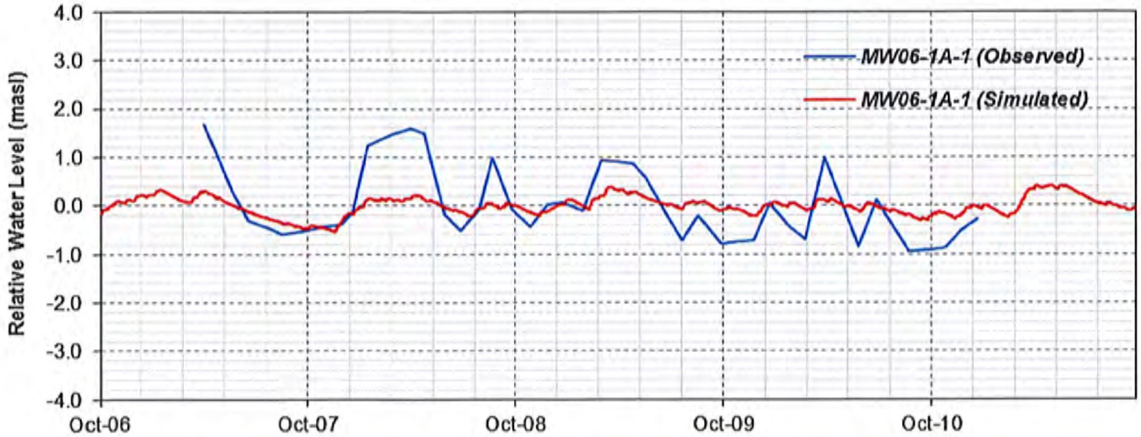


Figure 4.32: Simulated and observed heads at SQE well MW06-1-A-1 (Gasport).

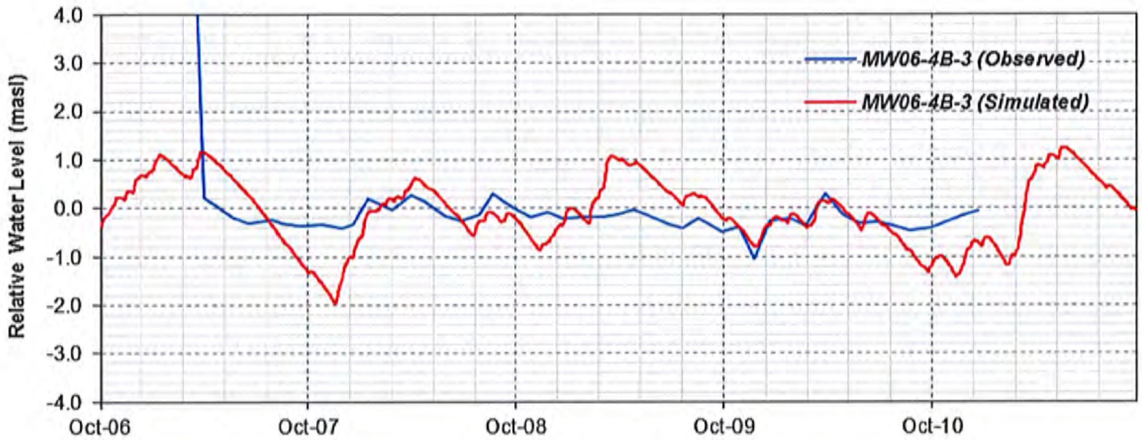


Figure 4.33: Simulated and observed heads at SQE well MW06-4-B-3 (Lower Eramosa).

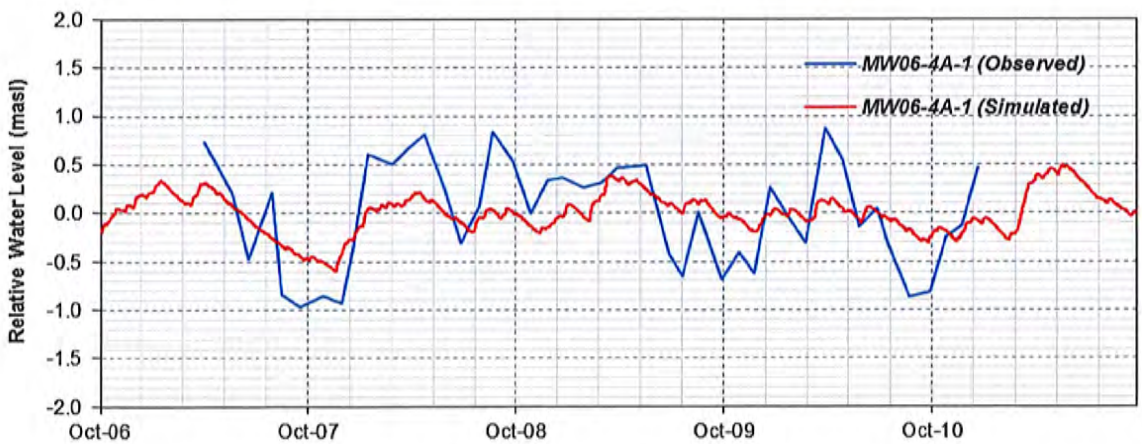


Figure 4.34: Simulated and observed heads at SQE well MW06-4-A-1 (Gasport).

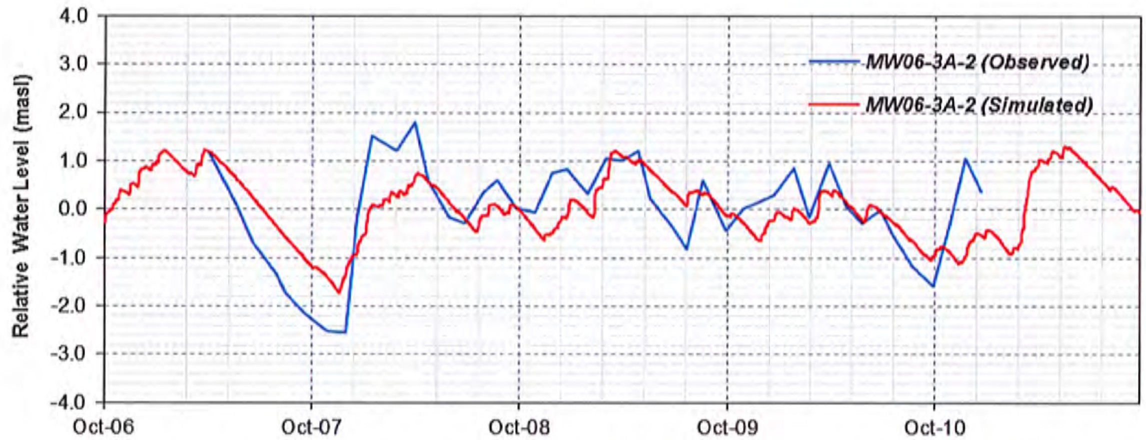


Figure 4.35: Simulated and observed heads at SQE well MW06-3-A-2 (Upper Eramosa).

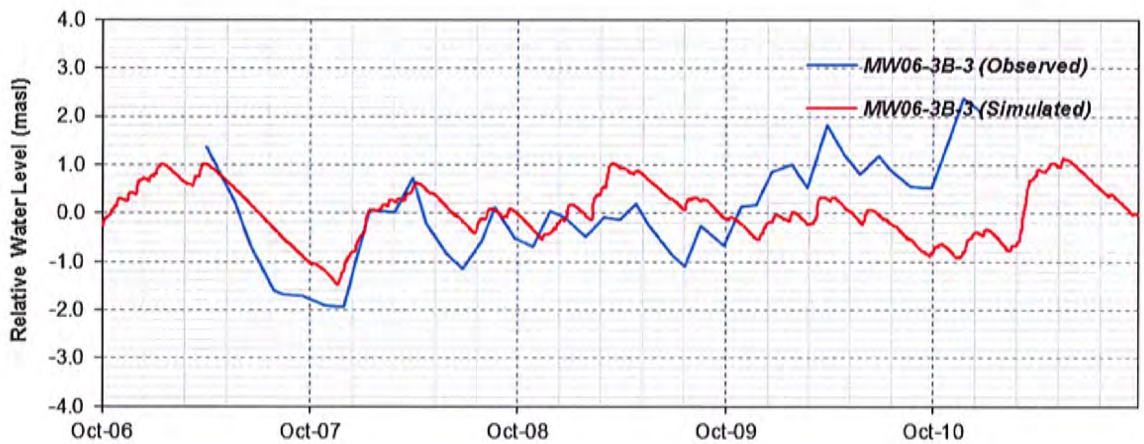


Figure 4.36: Simulated and observed heads at SQE well MW06-3-B-3 (Lower Eramosa).

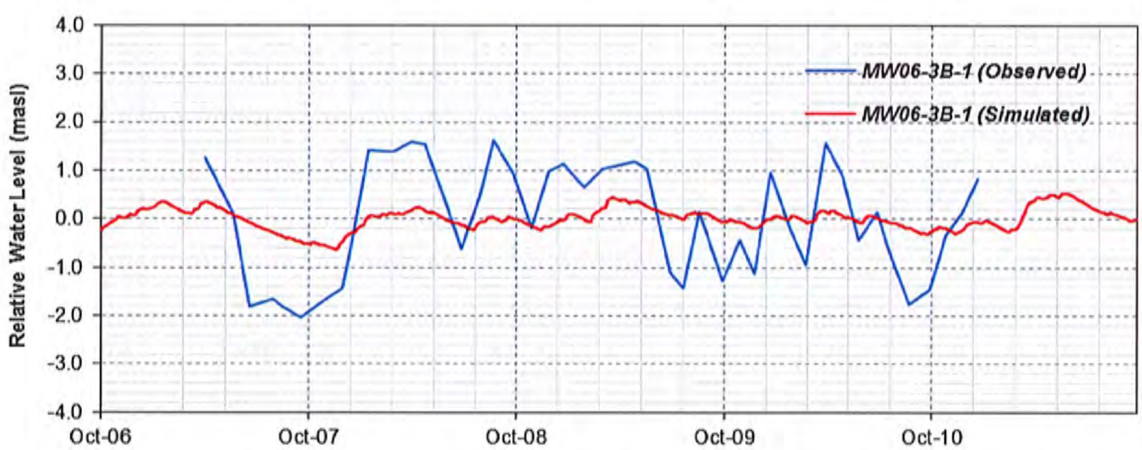


Figure 4.37: Simulated and observed heads at SQE well MW06-3-B-1 (Gasport).

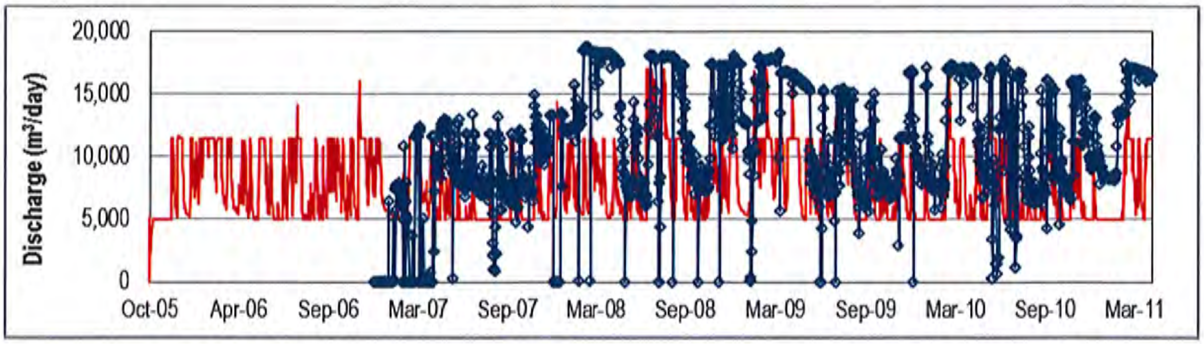


Figure 4.38: Simulated and reported daily discharge from the Lafarge North Quarry.

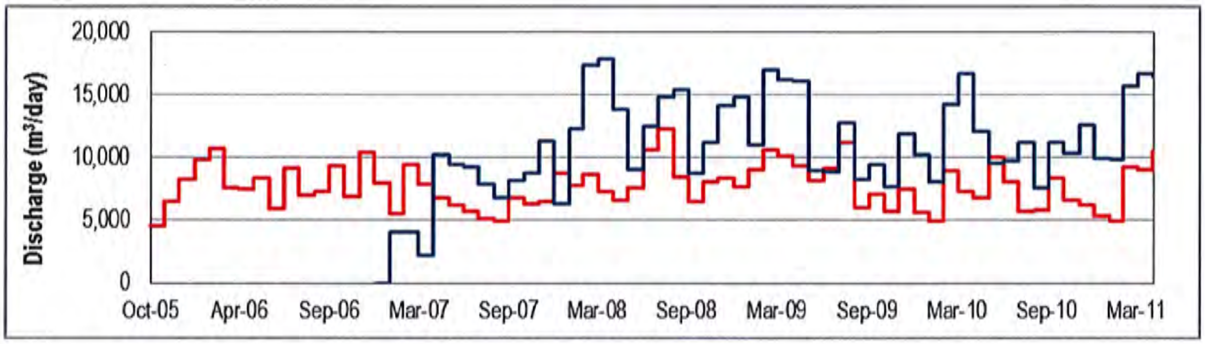


Figure 4.39: Simulated and reported monthly discharge from the Lafarge North Quarry.

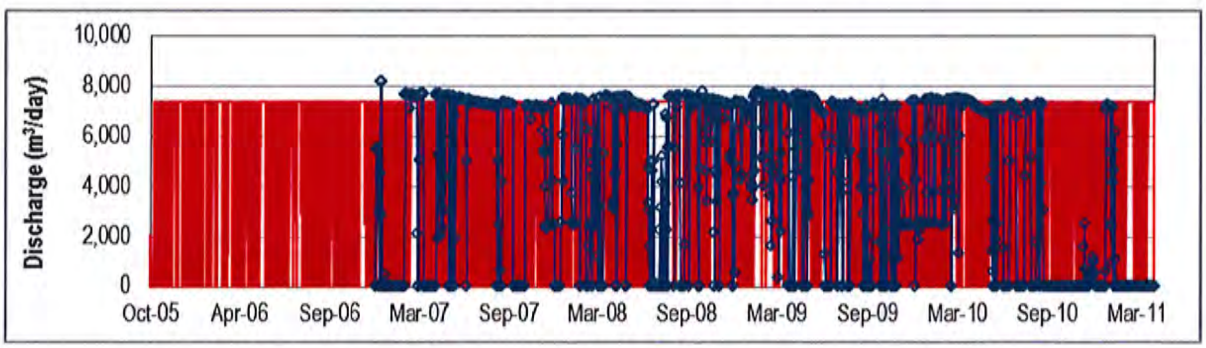


Figure 4.40: Simulated and reported daily discharge from the Lafarge South Quarry.

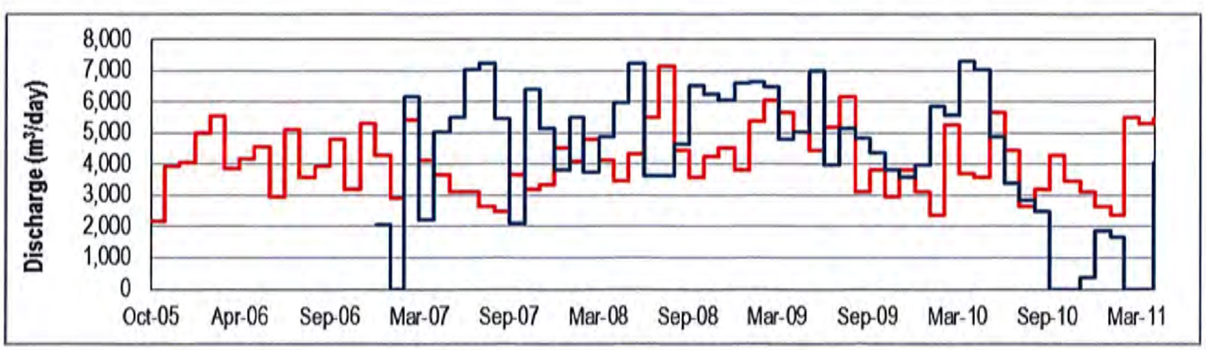


Figure 4.41: Simulated and reported monthly discharge from the Lafarge South Quarry.

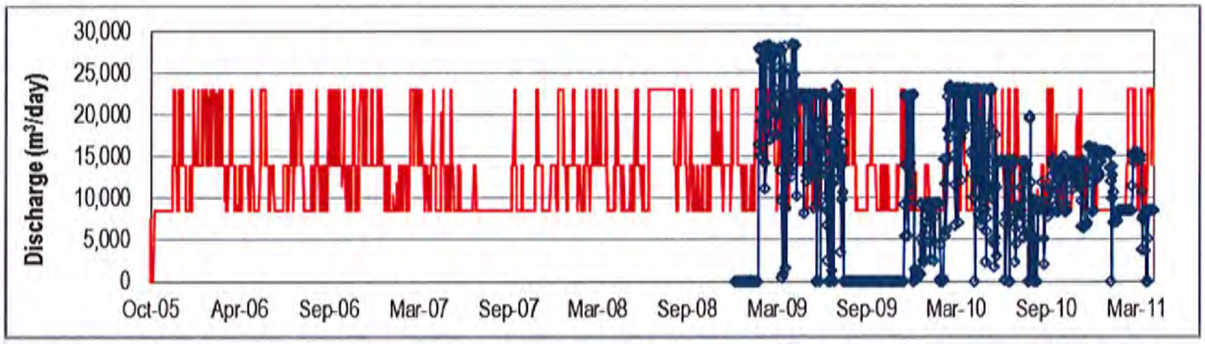


Figure 4.42: Simulated and reported daily discharge from the Lafarge Processing Area through the Railway Cut.

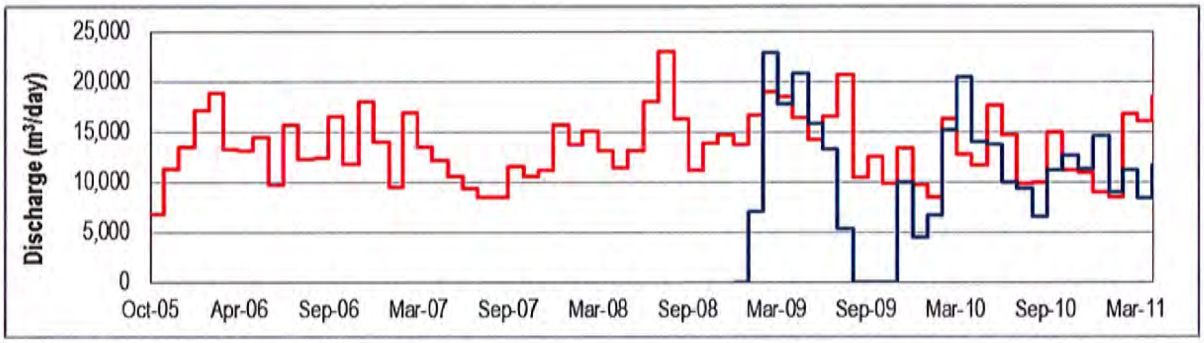


Figure 4.43: Simulated and reported monthly discharge from the Lafarge Processing Area.

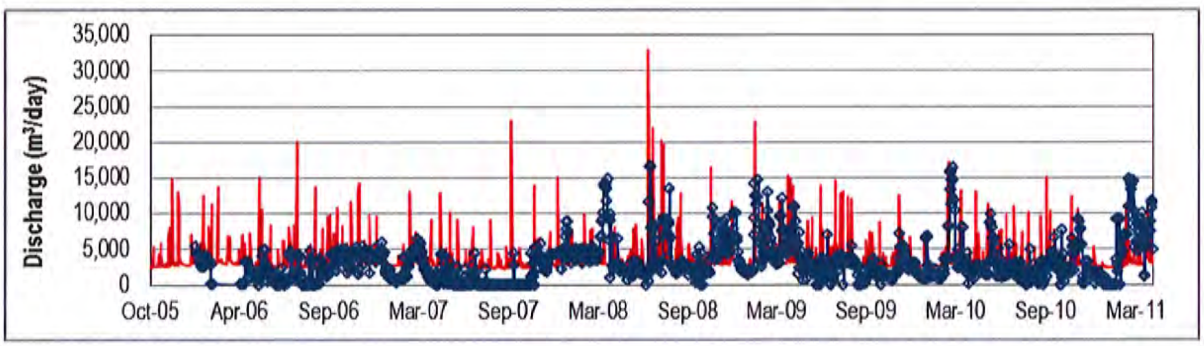


Figure 4.44: Simulated and reported daily discharge from the Flamboro Quarry sump.

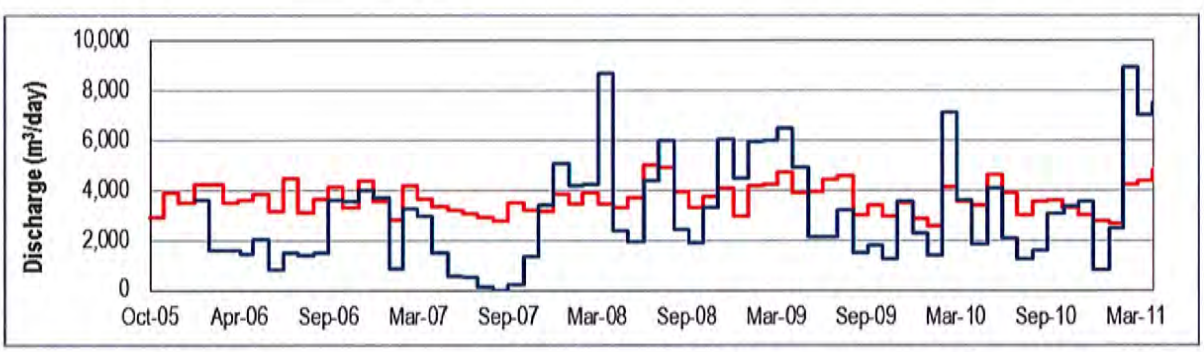


Figure 4.45: Simulated and reported monthly discharge from the Flamboro Quarry sump.

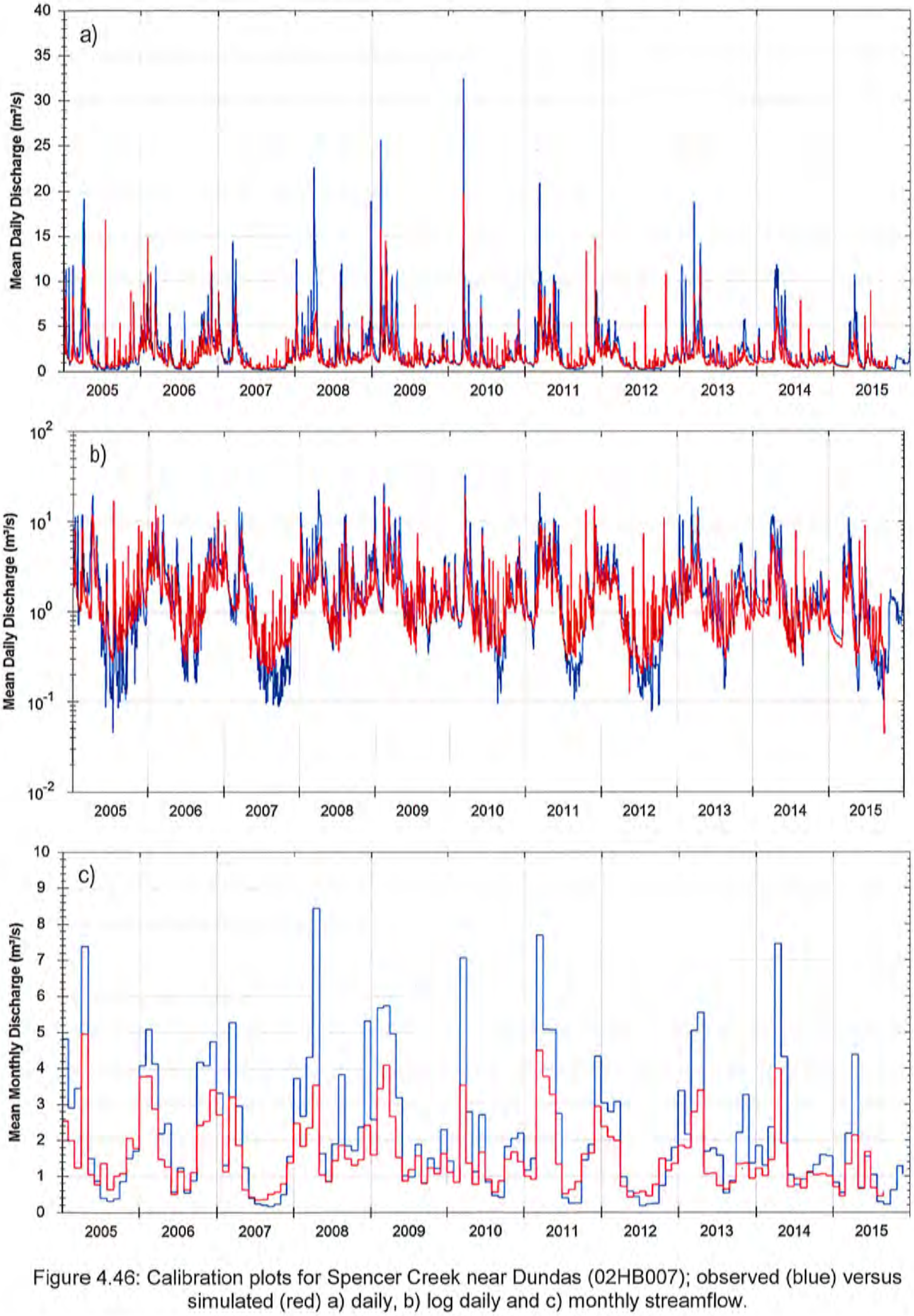


Figure 4.46: Calibration plots for Spencer Creek near Dundas (02HB007); observed (blue) versus simulated (red) a) daily, b) log daily and c) monthly streamflow.

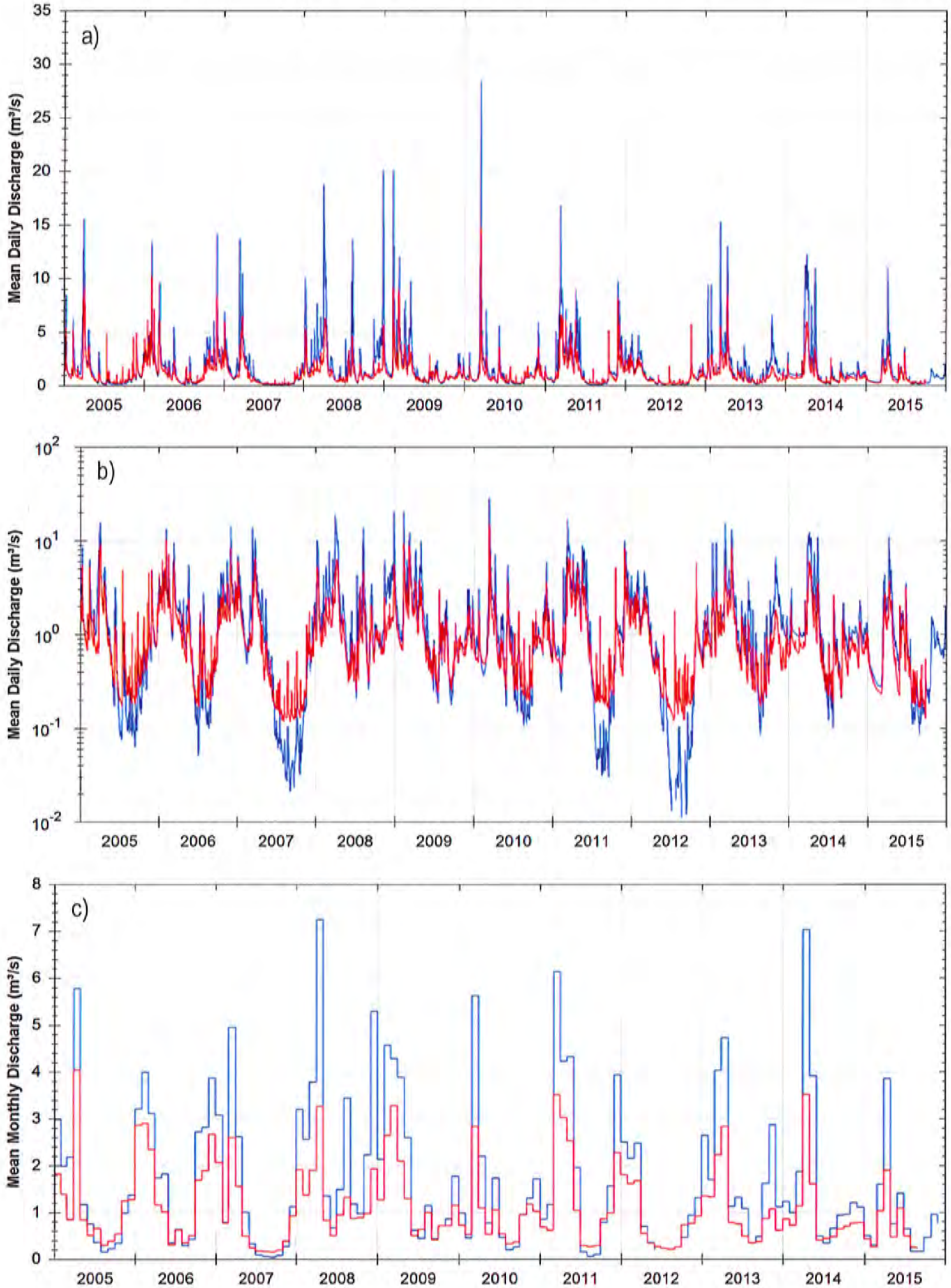


Figure 4.47: Calibration plots for Spencer Creek at Highway No. 5 (02HB023); observed (blue) versus simulated (red) a) daily, b) log daily and c) monthly streamflow.

5 Tier 3 Water Budget Update

5.1 *Introduction*

As part of the Greensville Tier 3 update, the integrated surface water/groundwater model was used to update the water budget elements for the Middle Spencer Creek subwatershed. The coupling of the surface water and groundwater systems through the model makes the integrated model an ideal tool for obtaining detailed quantitative estimates of all water budget components.

GSFLOW model outputs are similar to those generated for the PRMS and MODFLOW submodels but with a number of significant enhancements. Over 85 different groundwater and surface water flow components can be output on a cell-by-cell basis each simulation day. Earthfx has added additional components to the output and combined some flow components so that local (cell-based) and subcatchment-based water balances can be easily obtained. Daily results are aggregated into monthly, yearly, and long-term averages. All results presented in the following water budget analysis are expressed as annual averages.

5.2 *Middle Spencer Creek Water Budget*

The water budget components for Middle Spencer Creek were calculated in two steps: first, an overall water balance was calculated, taking into account the inputs and outputs to the hydrologic system as a whole; second, a groundwater-only water budget was estimated. The key difference, between the two methods is that exchanges between the surface water and groundwater system (e.g., groundwater recharge or surface leakage) are fully contained within the overall water balance and do not represent net losses to the system. True losses to the system are therefore only possible via processes that move water out of the subwatershed, such as lateral groundwater flow or ET.

The overall water budget for the Middle Spencer Creek subwatershed is presented in Table 5.1. Precipitation represents more than half of the inputs to the subwatershed (52%), with an average annual precipitation rate of 858 mm/yr. Figure 5.1 presents the spatial distribution of annual average actual ET (AET) values. Actual ET includes the evaporation losses from canopy interception and detention storage reservoirs. While potential ET (PET) values vary within a narrow range, AET rates show more spatial variability due to the differences in rainfall, vegetative cover, soil type, depth to water, overland runoff, and infiltration which can limit the amount of soil water available for ET in the summer months. Actual ET rates are high within the wetland areas where soil water is readily available and rates approach PET demand.

Inflows and outflows from overland runoff across the watershed boundaries are estimated to be quite minor. Figure 5.2 shows the spatial distribution of long-term average net overland runoff over the model area. The primary influence on the magnitude of overland runoff appears to be the distribution of rainfall and the variation in topography, land use, and soil type (as represented by the CN values, (see Earthfx, (2014)). Higher runoff values are associated with impervious surfaces, which are associated with commercial, horse racing, and industrial facilities. Large amounts of runoff also occur along the steeply-sloped quarry sides.

Lateral fluxes of groundwater across the subwatershed boundary represent relatively minor components of the overall subwatershed budget, making up 3% and 2% of the inflows and outflows, respectively. Pumping (excluding quarry water use) represents approximately 1% of the total outflow from the Middle Spencer Creek subwatershed.

Table 5.1: Overall Water Budget for the Middle Spencer Creek Subwatershed, as simulated by the Updated Tier 3 GSFLOW Model.

Water Budget Component	Inflows (m^3/day)	Inflows (mm/yr)	% of Total Inflows
Precipitation	116,700	858	52%
Overland Runoff In	458	3	0%
Lateral GW Flow In	5,684	42	3%
Stream Inflow	100,902	742	45%
Total Inflow:	**223,745**	**1,645**	**100%**
Water Budget Component	**Outflows (m^3/day)**	**Outflows (mm/yr)**	**% of Total Outflows**
Evapotranspiration	80,013	588	36%
Overland Runoff Out	407	3	0%
Well Pumping	2,721	20	1%
Lateral GW Flow Out	4,776	35	2%
Stream Outflow	135,790	998	61%
Total Outflow:	**223,706**	**1,645**	**100%**

Because the subwatershed receives water from five other subwatersheds (West Spencer, Westover Creek, Flamborough Creek, Upper Spencer, and Logie's Creek), these inflows were added to the GSFLOW water budget. Average annual stream inflows from contributing subwatersheds around Middle Spencer Creek are estimated to be approximately 100,902 m^3/d (or equivalent to 742 mm/yr when averaged over the subwatershed area).

Due to the importance of groundwater resources within the study area, a stand-alone water budget for the groundwater system was undertaken. The water budget components related to the groundwater system were determined for the Middle Spencer Creek subwatershed using the updated MODFLOW submodel. Major water budget components include areal recharge, surface leakage, lateral groundwater flows, stream/lake fluxes in and out of the groundwater system, and well pumping (Table 5.2). It should be noted that in some locations, a portion of the water supplied as recharge to the upper model layer may be immediately returned to the soil zone as "rejected recharge". The rejected recharge is included within the "surface leakage" which also accounts for groundwater seepage to the soil zone in areas of high water table. As such, traditional definitions of the groundwater recharge and surface leakage components of the water budget are somewhat limiting, as water can move back and forth between those systems in complex and highly varied pathways. From the values for groundwater recharge (449 mm/yr) and surface leakage (327 mm/yr), the groundwater system receives an average annual net flux of 122 mm/yr in the Middle Spencer Creek subwatershed.

Table 5.2: Water budget for the Middle Spencer Creek Subwatershed (groundwater system only), as simulated by the MODFLOW submodel.

Water Budget Component	Inflows (m³/day)	Inflows (mm/yr)	% of Total Inflows
Groundwater Recharge	61,093	449	88%
Lateral GW Flow	5,684	42	8%
Stream Leakage	2,545	19	4%
Lake Leakage	32	0	0%
Total Inflows	**69,353**	**510**	**100%**
Water Budget Component	**Outflows (m³/day)**	**Outflows (mm/yr)**	**% of Total Outflows**
Surface Leakage	44,515	327	64%
Lateral GW Flow	4,776	35	7%
Stream Leakage	16,702	123	24%
Lake Leakage	424	3	1%
Well Pumping	2,721	20	4%
Total Outflows	**69,137**	**508**	**100%**

Groundwater inflows within the subwatershed area are dominated by recharge (Figure 5.3), which make up 88% of the water entering the area, followed by lateral groundwater inflows at 8%. The remainder of the inflows (approximately 4%) are composed of streambed and lakebed losses to the groundwater system. As shown in Figure 5.4, fluxes from the streams to the groundwater system are relatively localized and generally associated with quarry discharges which result in enhanced downward gradients across the streambeds due to a combination of depressed water levels and augmented stream flows.

The simulated water budget for the Middle Spencer Creek subwatershed shows the system to be generally well-drained by the network of streams and rivers, which account for about one quarter (24%) of the outflows from the subwatershed. Adding to the outflow is discharge to lakes and surface leakage (i.e., groundwater discharge to the soil zone), both of which are eventually routed to the receiving stream network in the model, accounting for a further 1% and 64% of the water budget, respectively.

Lateral groundwater outflow from the Middle Spencer Creek subwatershed represent only 7% of the water budget (4,776 m^3/d or 35 mm/yr) and roughly balance the lateral inflow from other catchments (5,684 m^3/d or 42 mm/yr). Constant head boundary conditions were not present along the edges of the Middle Spencer Creek subwatershed. Some lateral outflow occurs across the Niagara Escarpment, to the southeast of the municipal wells, where the subwatershed boundary follows the brow of the escarpment but the model extends to the Dundas Valley. Additional outflow occurs across the west watershed boundary toward the Lafarge South Quarry due to the local depression in groundwater heads caused by the dewatering operations.

The water budget presented in this report updates the estimates provided in the previous Tier 3 study. As noted, traditional definitions of the surface water and groundwater components of the water budget are somewhat limiting, as water can move back and forth between those systems in complex and highly varied pathways. In addition, quarry water use was not simulated as a withdrawal from the model, but in a more realistic manner as discharge to drains and ultimately surface water flow, therefore, groundwater discharge from the quarry is not included in the "well pumping" term, but is included within other components of the overall subwatershed budget (i.e., surface discharge along the quarry floor, and groundwater discharge to the quarry sump).

5.3 Figures

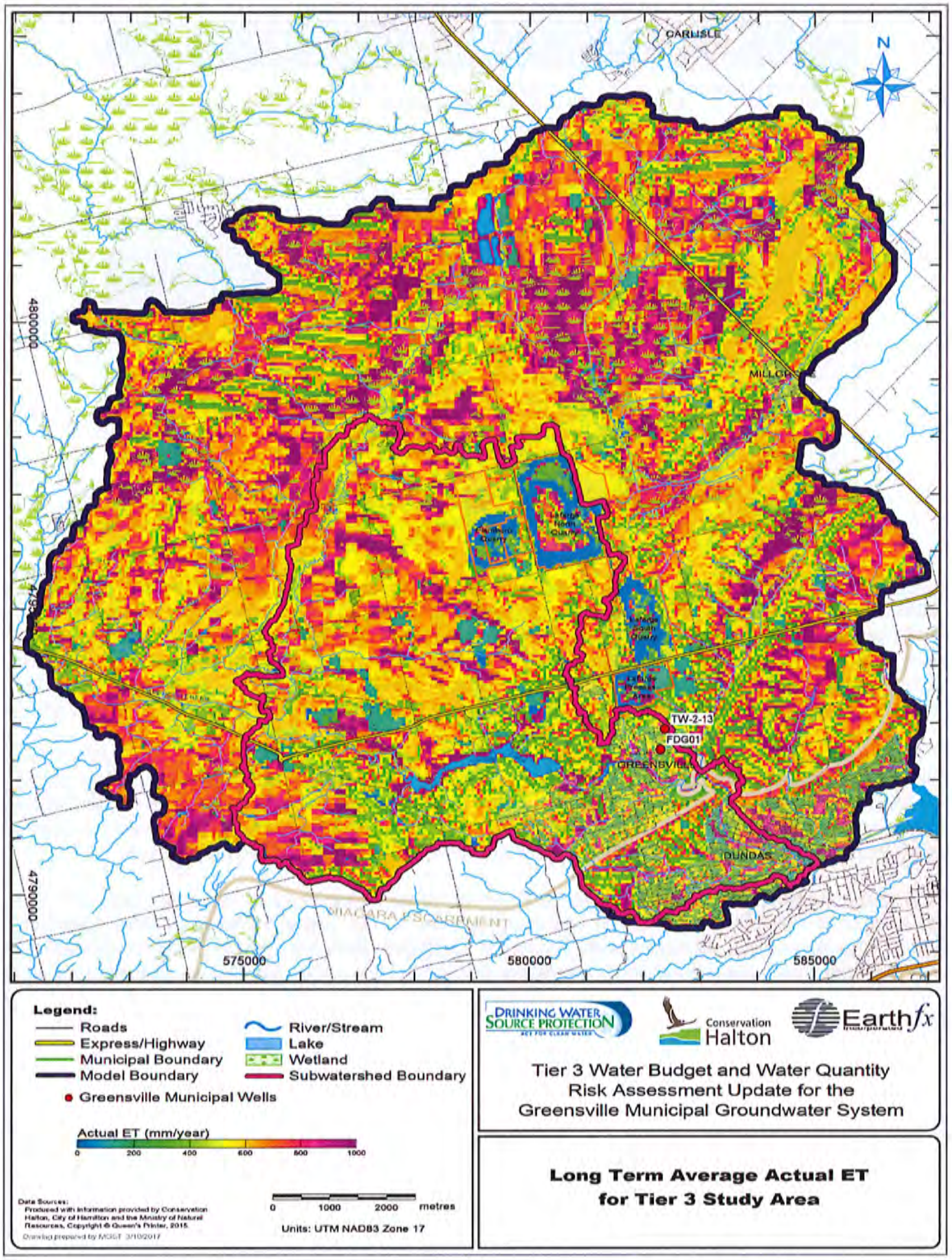


Figure 5.1: Long-term average actual evapotranspiration (with existing land use).

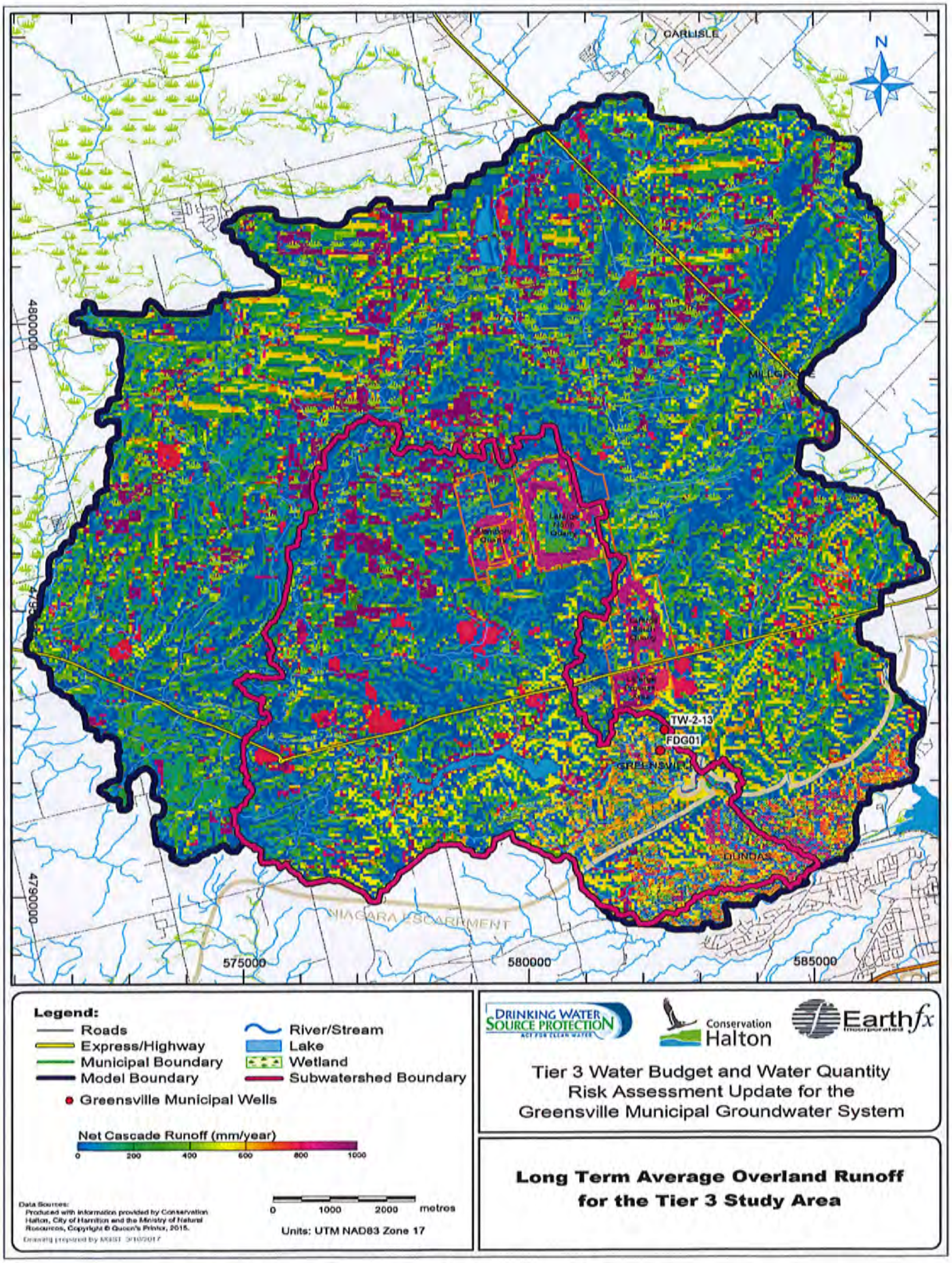


Figure 5.2: Long-term average net overland runoff (with existing land use).

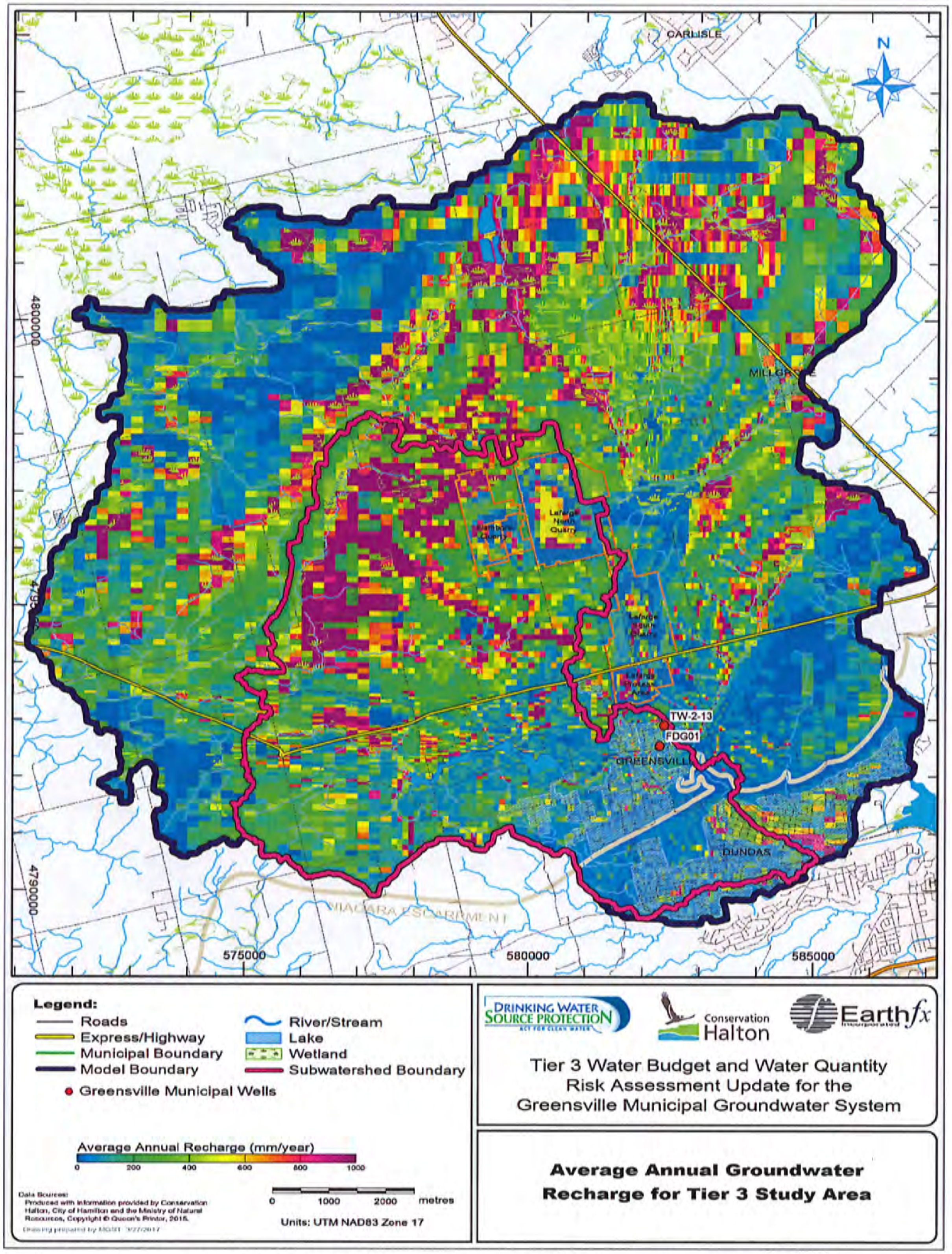


Figure 5.3: Long-term average annual groundwater recharge (with existing land use).

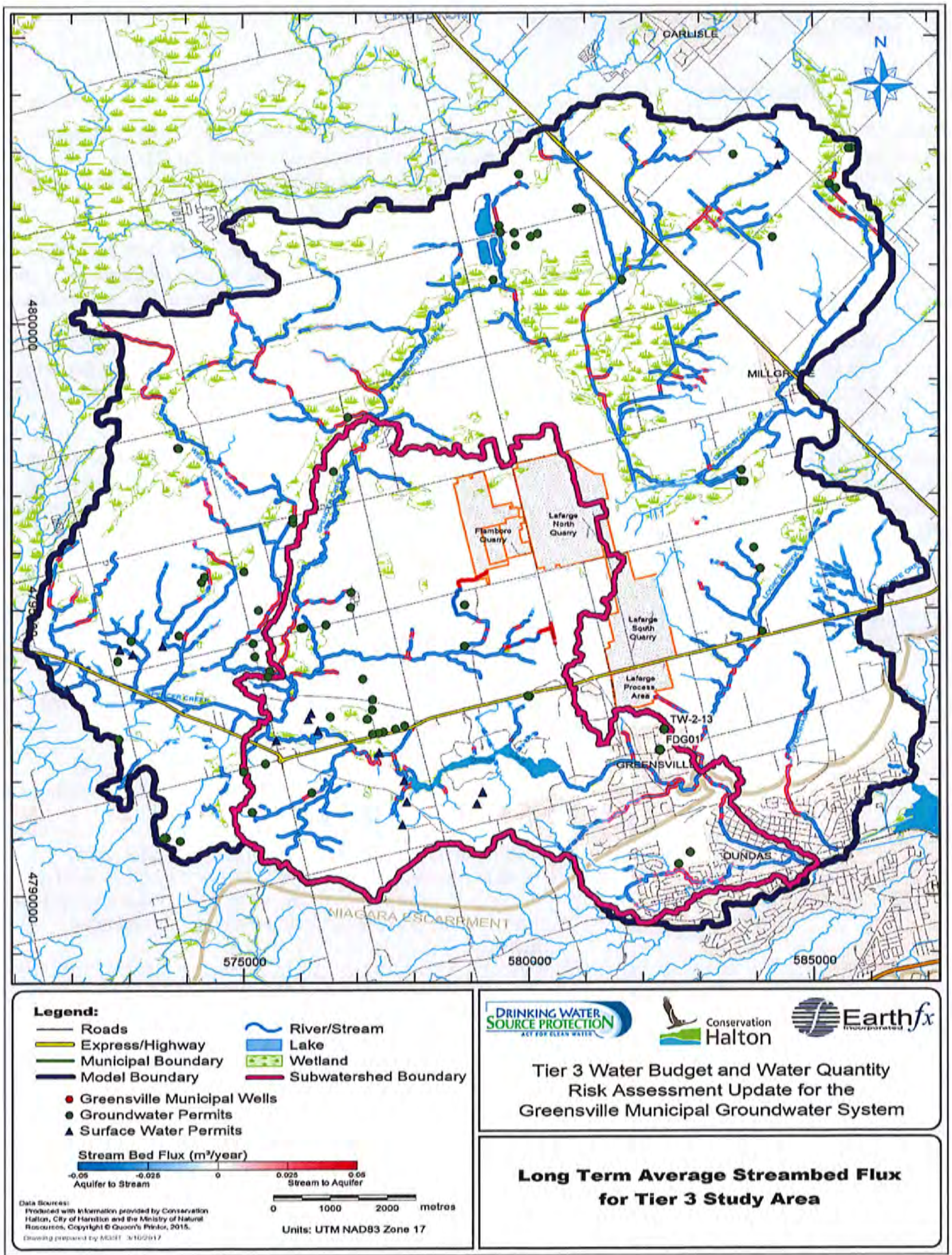


Figure 5.4: Long-term average groundwater discharge to streams (with existing land use).

6 Local Area Risk Assessment

6.1 *Introduction*

A Tier 3 Risk Assessment is undertaken to determine if a groundwater supply is able to meet the needs of a municipality under a variety of water demand, drought, and land development conditions. Two key tasks listed in the Water Budget Guide (MNR, 2011) are applicable to the local area risk assessment and are listed below:

- **Delineate vulnerable areas:** The groundwater quantity vulnerable areas, WHPA-Q1 and WHPA-Q2, are delineated using the Tier 3 water budget model. A 'local area' is also delineated around the municipal wells for the risk assessment analysis. These terms are defined in the next section.
- **Evaluate risk scenarios**: A set of scenarios are prescribed in the Technical Rules and in the Water Budget Guide which consider the impact of changes in allocated quantity of water, long-term average climate, drought conditions, and changes in land use on the sustainability of the municipal wells.

The risk assessment scenarios, summarized in Table 6.1, are evaluated in terms of the ability to maintain pumping at the municipal well. Impacts to other water uses (e.g., waste assimilation, other water takings, wetlands, and aquatic habitat) are evaluated in Scenario G only. Specific details related to each scenario and subsets of each scenario are listed in Table 6.2.

Table 6.1: Summary of risk assessment scenarios (from MNR, 2011).

Scenario	Time Period	Data
C	Period for which climate and streamflow data are available for the local area	Data related to average daily pumping rates for water takings and land cover are reflective of conditions during the study period.
D	Ten-year drought period	Data related to average daily pumping rates for water takings and land cover are reflective of conditions during the study period.
G	Period for which climate and streamflow data are available for the local area	Data related to average daily pumping rates for water takings and land cover are reflective of conditions during the year in which the planned system or an existing system with a committed demand is operating at its allocated quantity.
H	Ten-year drought period	Data related to average monthly pumping rates for water takings and land cover are reflective of conditions during the year in which the planned system or an existing system with a committed demand is operating at its allocated quantity.

Table 6.2: Risk assessment scenario details (from MNR, 2011).

Scenario	Time Period	Model Scenario Details		
		Land Cover	Municipal Pumping	Model Simulation
C	Average conditions	Existing	Existing	Steady-state model with average annual recharge
D	10-year drought	Existing	Existing	Transient model (1956-1966)
G(1)	Average conditions	Planned Future Land Use	Allocated quantity of water	Steady-state model with average annual recharge
G(2)		Existing	Allocated quantity of water	
G(3)		Planned Future Land Use	Existing	
H(1)	10-year drought	Planned Future Land Use	Allocated quantity of water	Transient model (1956-1966)
H(2)		Existing	Allocated quantity of water	
H(3)		Planned Future Land Use	Existing	

Note:

[1] Planned quantity of water is equal to existing plus committed plus planned demand. No planned demand is anticipated for the Greensville municipal supply system; future pumping scenarios reflected increased demand due to quarry dewatering, additional private residential wells in the Greensville Rural Settlement Area, and the projected pumping from the new Greensville School/Library/Community Centre.

6.1.1 Updates from Previous Tier 3

All of the Risk Assessment scenarios summarized above in Table 6.2 were completed with the updated Greensville Tier 3 model using the same methodology as the previous Greensville Tier 3. Major updates to the scenarios include:

- Addition of the new Greensville municipal supply well TW-2-13 for both the existing and future pumping scenarios. As discussed in Section 3.2, current demand will be split between the two wells in a 6:1 ratio, favouring the new municipal well. In practice (and in transient simulations), this translates to 7-day cycles whereby TW-2-13 is pumped for 6 days followed by pumping from FDG01 for one day.
- Updates were made to the conceptual geologic model, conceptual hydrostratigraphic model, and the numerical model to incorporate recent geologic and hydrogeological information obtained from municipal water supply exploration work and hydrogeologic investigations conducted on behalf of Lafarge (Golder, 2013).
- Water use estimates were revised to incorporate data from new versions of the MOECC PTTW and WTRS databases. In addition, estimated water takings associated with the planned Greensville Elementary School/Library/Community Centre (referred to herein as the Greensville School) were included under the planned demand scenarios G(1), G(2), H(1) and H(2). The estimated takings from the Greensville School well of 23.2 m^3/d were based on the design capacity of the facility's septic system, as recommended by City of Hamilton staff.

6.2 *Wellhead Protection and Local Area Delineation Methodology*

6.2.1 WHPA-Q1 Methodology

According to the Technical Rules for the Assessment Report (MOE, 2009), a WHPA-Q1 is defined as:

> *...being the combined area that is the cone of influence of the well and the whole of the cones of influence of all other wells that intersect that area.*

The Water Budget Guide (MNR, 2011) presents a method for defining the WHPA-Q1 as:

> *[The] WHPA-Q1 is delineated by estimating the cone of influence for the existing land use and existing plus committed plus planned pumping rates scenario. The cone of influence is estimated by calculating the maximum water level drawdown for the scenario as compared to the aquifer drawdown under non-pumping conditions. The drawdown cone used to delineate the WHPA-Q1 should be based on the existing plus committed plus planned pumping rates for the existing and planned municipal wells. In addition, the drawdown cone will be intersected with the drawdown cone from all other consumptive water users in the study area.*

The 'cone of influence' is considered to be the areal projection of the depression (or drawdown) created in the water table or potentiometric surface caused by groundwater pumping at a well compared to baseline (non-pumping) conditions. In theory, the cone of influence extends outward until a natural hydrologic boundary (such as a lake) is intersected. For practical purposes, a 1.0 m drawdown threshold was selected as the measureable limit of the cone of influence for the Greensville Tier 3 Assessment. This value was established through a thorough review of seasonal variations in monitoring wells with continuous data, as documented in Earthfx (2015) and is consistent with many other Tier 3 studies.

The cone of influence was determined using the updated groundwater flow model developed for this study. The simulated steady-state potentials (heads) under the future consumptive demand scenario were subtracted from the simulated steady-state heads under the non-pumping or "baseline" scenario. Where the cones of influence of the Greensville municipal wells intersect those of other users, the WHPA-Q1 defines the combined cones of influence. The definition of "baseline conditions" for this analysis is presented in Section 6.2.4.

6.2.2 WHPA-Q2 Methodology

The WHPA-Q2 is defined in the Technical Rules for the Assessment Report (MOE, 2009) as:

> *...being the [WHPA-Q1] area and any area where a future reduction in recharge would significantly impact that area.*

The Water Budget Guide (MNR, 2011) discusses the method for defining the WHPA-Q2:

> *When identifying an area where a future reduction in recharge might occur, reference must be made to a municipality's Official Plan (OP) to identify lands where new development could occur. Furthermore, consideration must also be given to the maximum amount of recharge reduction that might result from these developments without the influence of stormwater best management practices (e.g., Low Impact Development). In order for an area to be delineated as WHPA-Q2 outside of the WHPA-Q1, it must be shown through the scenarios that recharge reductions in that area might result in a measurable impact on water levels at the municipal pumping wells.*

Based on examples provided in the Water Budget Guide, the WHPA-Q1 can be expanded to include the map outline of future land developments, identified in a municipality's Official Plan (OP), only if they are

(1) outside of or straddle the WHPA-Q1 boundary, and (2) could decrease natural groundwater recharge to a point that it would have a measurable impact on water levels at the municipal pumping wells.

To delineate the WHPA-Q2 and estimate the impact of recharge reduction on the municipal well, the simulated steady-state heads with the adjusted recharge rates under future land use conditions were subtracted from the simulated steady-state heads under baseline conditions. Pumping from the municipal wells and other wells remained unchanged in the two steady-state scenarios.

6.2.3 Local Area Methodology

The local area for a municipal supply well is defined in the Technical Rules for the Assessment Report (MOE, 2009) as the combination of the following areas:

i) *the cone of influence of the municipal supply wells;*
ii) *the cones of influence resulting from other water takings where those cones of influence intersect those of the municipal supply wells; and*
iii) *the areas where a reduction in recharge would have a measureable impact on the cone of influence of the municipal supply wells.*

In the case of a Tier 3 Assessment with only groundwater-supplied municipal systems, the local area is equivalent to the WHPA-Q2.

The next sections of the report present a more detailed description of the scenarios simulated for the WHPA-Q1 and WHPA-Q2 analyses, including the baseline "non-pumping" conditions scenario, the future consumptive demand scenario, and the future land use scenario. Results of the WHPA-Q1 and WHPA-Q2 (and local area) delineation are presented in Section 6.3.

6.2.4 Simulation of Baseline Conditions

The baseline scenario considered a non-pumping condition with groundwater recharge rates based on existing land use. For this simulation, all permitted takings, including the municipal wells and residential private wells were removed from the steady-state model.

While it is relatively straightforward to undertaken a "non-pumping" scenario with wells and surface water diversions, representing the quarries under the baseline conditions scenario presented a conceptual challenge. In the truest sense, the drawdown caused by the quarry since its existence would be calculated using a baseline representing the site under pristine conditions prior to the start of material excavation. This option was rejected due to the limited data available to represent, with a reasonable degree of confidence, the subsurface conditions going back as far as the 1900s.

As an alternative, it was decided to accept the quarries at their current extents as being part of the landscape for the study area, while simultaneously recognizing that the Tier 3 baseline scenario would need to represent "non-pumping" conditions. If discharge from the quarry is halted, the quarries would refill and act as groundwater-fed lakes. While this still represents an altered-state, as opposed to pre-quarry (i.e., natural conditions), it was decided that allowing quarries to fill to their natural levels represented a fair and defensible approach for defining baseline conditions for this part of the study. This representation of the baseline conditions scenario was used the previous Tier 3 study, and developed in consultation with the Peer Review Team, MNRF, CH, HCA, and the City of Hamilton.

To simulate re-filling of the existing quarry excavations (shown in Figure 6.7), the quarry drainage and active discharge mechanisms simulated in the model under current conditions (discussed in the Phase I Report (Earthfx, 2014)) were removed and MODFLOW lake cells were extended to cover the full extents of the (flooded) quarry footprints. A comparison of the quarry representations for the current conditions model and the flooded (baseline) scenario is illustrated in Figure 6.1.

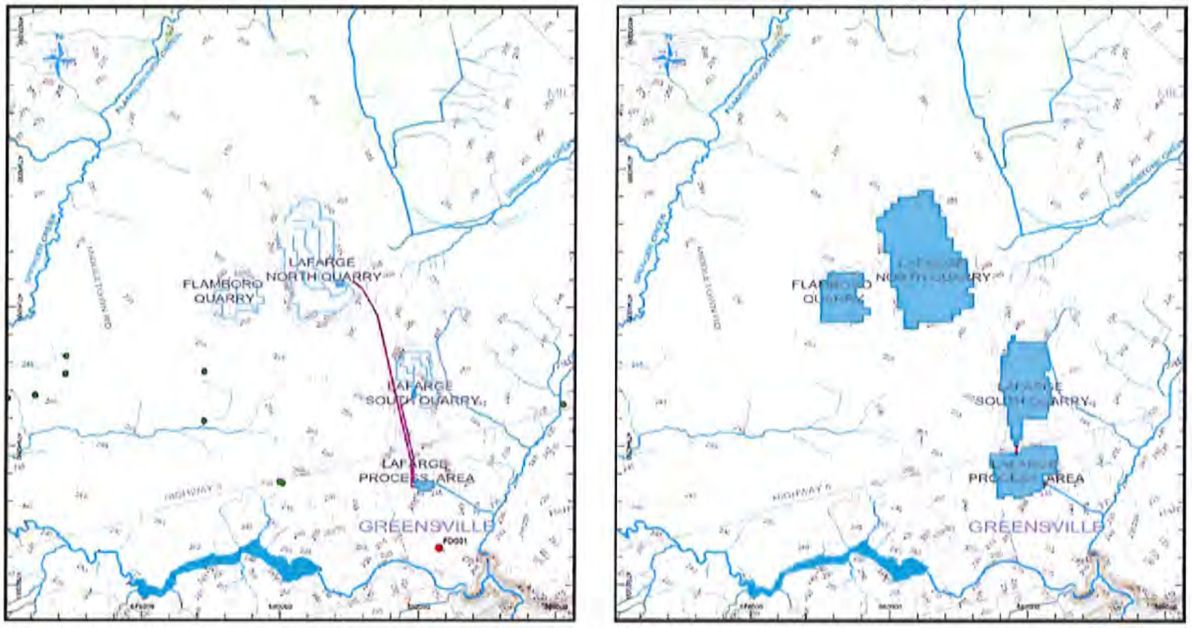


Figure 6.1: Quarry drainage representation under current condition (left), and flooded baseline conditions with no drainage or active discharge (right).

Table 6.3 summarizes the drainage of the Lafarge and Dufferin Flamboro Quarries in the baseline scenario. For the Lafarge South Quarry, the hauling road that passes under Highway 5 allows the "South Quarry Lake" to drain into the Process Area via the hauling road assuming an invert elevation of 236 masl. Similarly, the "Process Area Lake" will naturally drain south to Logie's Creek once water levels exceed the invert elevation of 233 masl for the Railway Cut in the southeast corner of the Processing Area. This passive drainage was simulated in the model by adding MODFLOW diversions with weir elevations set to the invert elevations of the outlets to the South Quarry and Process Area lakes.

The Lafarge North Quarry and Flamboro Quarry have no natural outlets. Stage in these quarry lakes is controlled by the natural water balance between precipitation, runoff, groundwater seepage and evaporation. The water balance for each lake is simulated by GSFLOW.

Table 6.3: Summary of Represented Quarry Conditions under Baseline Scenario

	Drainage	**Bottom Elevation**
Lafarge North Quarry	Lake with no natural outlet that receives groundwater, runoff, and precipitation.	230 masl based on existing quarry depth.
Lafarge South Quarry	Southward drainage to Process Area via Highway 5 underpass (invert elevation at 235.9 masl).	230 masl based on existing quarry depth.
Lafarge Process Area	Southward drainage to Logie's Creek via Railway Cut (invert elevation at 233.0 masl).	230 masl based on existing quarry depth.
Dufferin Flamboro Quarry	Lake with no natural outlet that receives groundwater, runoff, and precipitation.	239 masl based on existing quarry depth.

6.2.5 Baseline Simulation Results

Figure 6.8 presents the simulated heads in the weathered bedrock aquifer under the baseline scenario with drainage and lake conditions set as discussed above. Even when allowed to fill with water, the quarries represent a local depression in the potentiometric surface because the invert elevations are lower than groundwater levels in the surrounding area.

Quarry lake stage in the baseline scenario is shown in Figure 6.9 along with the simulated water table elevations. Water levels in the Lafarge South Quarry and Process Area lakes are controlled by invert elevations (as described above), approaching 236 masl and 233 masl, respectively. Water levels in the Lafarge North Quarry and Flamboro Quarry lakes are controlled by the water balance between groundwater seepage, precipitation, and evaporative loses. The simulated lake stage in the Lafarge North Quarry is 237 masl representing a lake depth of approximately 7 m, as shown on Figure 6.9. The simulated lake stage in the Flamboro Quarry is 246 masl.

6.2.6 Simulation of Future Conditions

The future conditions scenarios consider the likely changes to hydrologic and hydrogeological conditions within the study area and their potential impacts on municipal supply and other water uses. Specific changes considered in the Tier 3 Risk Assessment include future increases in water demand, build-out of quarries, and changes in land use and are discussed below. Simulating the build out of the quarries considered the effects of (a) increased pumping to dewater the expanded and deepened quarry footprints, as well as (b) increased interception of rainfall in the quarry ditch network that would otherwise recharge the underlying aquifer. Distinguishing between these two aspects did not affect how the quarries were simulated in the analysis, only how the impacts were assessed under the guidelines for the risk assessment.

6.2.6.1 *Future (Planned Demand) Pumping*

With regards to future (Planned Demand) pumping, no increases in demand are anticipated for the Greensville municipal wellfield because future developments in the Greensville settlement area are to be supplied by private systems (City of Hamilton, 2011). The expanded Greensville municipal system, as represented by the operational practice of splitting the pumping between FDG01 and TW-2-13 according to a 6-to-1 day cycle, was unchanged between the current and future pumping scenarios.

Projected water demands for the Greensville School were represented in the future pumping scenarios. The Greensville School is located on Harvest Road, approximately 120 m east of FDG01 (shown on Figure 3.4). The Greensville School will be supplied by a pumping well located just north of Harvest Road. The estimated takings from the well were 23.2 m^3/d, as discussed in Section 6.1.1.

An additional 317 private wells (estimated to be needed to supply future residential developments as per the Greensville Rural Settlement Area assessment) were represented in the future scenarios along with the 950 existing private wells which were represented in the current conditions scenario. The assumed distribution of these wells is presented in Figure 6.10.

6.2.6.2 *Full Quarry Build-out*

The full build-out of the Lafarge North and South quarries and the Flamboro Quarry represents future conditions with the maximum likely “groundwater taking” rate. The future configurations of the quarries, as represented in the model, are summarized in Table 6.4. The projected footprints of the fully built-out quarries are shown on Figure 6.11. It should be noted that the built-out quarries correspond to the currently licensed footprints only, and do not include the quarry expansions currently under review, including the Lafarge South Quarry Extension.

The Lafarge North Quarry, along with the recently approved North Quarry Extension, were simulated as having a combined footprint of approximately 260 ha. It was assumed that the lower bench of the quarry would leave a 3 m thickness of bedrock above the Vinemount member. Accordingly, the lower benches of the North Quarry and North Quarry Extension have bottom elevations ranging from 226 to 234 masl.

The full build-out of the Lafarge South Quarry was simulated as having a footprint of approximately 150 ha. The lower quarry bench was assumed to leave 3 m of bedrock above the Vinemount member, resulting in base elevations ranging from 216 to 226 masl (Golder, 2007; Golder, 2013). The Flamboro Quarry excavation was assumed to extend to a uniform lower bench elevation of 230 masl within the licensed quarry area. Construction of the proposed South Quarry Extension was not represented.

Table 6.4: Quarry conditions under full build-out as represented in the future scenarios.

	Drainage	Bottom Elevation
Lafarge North Quarry and North Quarry Extension	Runoff and groundwater seepage into excavation collected in ditches and routed to sump. Sump discharges south into Process Area pond.	226 to 234 masl based on an assumed 3 m of bedrock thickness left above the Vinemount member.
Lafarge South Quarry	Runoff and groundwater seepage into excavation collected in ditches and routed to sump. Sump discharges south into Process Area pond.	216 to 226 masl based on an assumed 3 m of bedrock thickness left above the Vinemount member.
Lafarge Process Area	Process Area sump pond discharges via Railway Cut south to Logie's Creek.	230 masl based on existing quarry excavation.
Dufferin Flamboro Quarry	Runoff and groundwater discharge collected in ditches and routed off-site south to a tributary of Spencer Creek.	230 masl based on licensed quarry depth.

6.2.6.3 *Future Land Use Changes (Reductions in Recharge)*

Under the Tier 3 Risk Assessment, the impact of projected land development on groundwater recharge rates and, more specifically, the effect on water levels in the municipal wellfield, are considered. As discussed in Section 3.7, the most significant changes in land use within the study area are related to the continued expansion of the Lafarge and Flamboro quarries to the north of the Greensville municipal wellfield. Increases in residential development, as identified in the Rural Official Plan, are also discussed.

To simulate changes in rates of groundwater recharge caused by future rural residential developments, the agricultural, field, and bare soil land cover classifications within the areas shown in Figure 3.18 were replaced with the rural residential classification. Model parameters, such as percent impervious, vegetative type, and cover densities, were adjusted for the cells representing these areas. The GSFLOW model was run again to produce a modified groundwater recharge map, shown in Figure 6.12, for use in future conditions scenarios. Results indicated that recharge increased slightly for converted agricultural areas despite an increase in imperviousness. The increases occurred because less water was lost to canopy interception and ET due to the assumed reduction in vegetative cover density.

6.3 *Wellhead Protection and Local Area Delineation Results*

The WHPA-Q1 was delineated by determining the change in simulated heads within each aquifer between the following two model scenarios:

1. Steady-state "baseline scenario" using existing land use, no municipal or non-municipal pumping, and quarry excavations allowed to fill to levels regulated by topographic controls (as described in Section 6.2.4, above) to estimate the heads that would exist without active dewatering; and

2. Steady-state scenario using existing land use, allocated pumping rates (existing plus committed demand) for the municipal wells, pumping from existing and future residential wells, and full build-out of the quarries with active dewatering.

Simulated heads in the weathered bedrock aquifer (Layer 5) under future pumping and full quarry build-out are shown in Figure 6.13. Simulated steady-state heads in the aquifer under future conditions were subtracted from those under baseline scenario (Figure 6.8) to determine the drawdowns shown in Figure 6.14. Figure 6.15 shows the drawdowns in the vicinity of the municipal well. The 1.0 m drawdown contours in the weathered bedrock aquifer caused by the two Greensville municipal wells coalesce with the drawdown cone from the Greensville School supply well to form a combined drawdown cone with a total area of approximately 0.25 km². The drawdown cone extends 160 m north of TW-2-13; however, it does not intersect the 1 m drawdown contour due to quarry operations.

The largest drawdowns (approximately 25 m) occur in the parts of the North Quarry and the north end of the South Quarry where additional material is extracted between baseline and future conditions. The simulated 1 m drawdown extends up to 1.3 km from the edge of the future quarry excavation. Simulated drawdowns in the Lafarge Process Area are small (between 1 and 2 m) relative to those in the other quarry excavations. This is because no further quarrying of the bedrock is anticipated for the Process Area and lake levels in the Process Area under baseline conditions are controlled by the Railway Cut invert, which is approximately 1 m higher than the current sump pond control elevation.

Water levels in the deeper Goat Island/Gasport aquifers under future conditions are shown in Figure 6.16. The aquifers are used for private water supply in the study area and are represented by Layers 10 and 11 in the model. The depression around the dewatered future quarry excavations is clearly illustrated by the drawdown contours shown in Figure 6.17. Maximum drawdowns of approximately 20 m are predicted in the South Quarry under the full build-out condition. The 1 m drawdown contour in the Gasport/Goat Island aquifer extends between 900 m and 3200 m out from the quarry excavation. The 1 m drawdown encompasses an area 43.9 km^2 and, most significantly, includes the Greensville municipal wells.

The drawdowns in the Goat Island/Gasport aquifers are caused by depressurizing the confined aquifers through the removal of the Lower Eramosa, and dewatering of the bedrock surrounding the quarry excavation. Although the overlying Vinemount member was assumed to be left in place, the competency of the unit as a confining layer was assumed to be compromised as a result of blasting and removal of material. This effect has been observed in the monitoring wells, as discussed in Section 2.2 and in Earthfx (2014).

6.3.1 WHPA-Q1 Results

The WHPA-Q1 represents a land surface area defined by the change in water levels between baseline (no-pumping) and pumping conditions. As in other Tier 3 studies (e.g., Earthfx (2013) and Earthfx (2014)), the WHPA-Q1 was defined by the maximum extent of the drawdown in all aquifers. This is consistent with the approach employed during the previous Greensville Tier 3 study. As discussed in Earthfx (2015), the recommended drawdown threshold for WHPA-Q1 delineation is 1 m. The maximum extent of the drawdown using this threshold is shown in Figure 6.18.

The use of the maximum extent of the drawdown in all aquifers has the effect of increasing the size of the WHPA-Q1 considerably. The 1 m drawdown contour in the weathered bedrock aquifer encompasses the two municipal supply wells FDG01 and TW-2-13, as well as the planned Greensville School well (120 m east of FDG01). Alone, the 1 m drawdown in the weathered bedrock contact aquifer includes an area of approximately 0.25 km² and does not intersect the 1.0 m drawdown contour around the quarry. The 1 m drawdown in the deeper Gasport/Goat Island aquifer fully covers the quarries and the Greensville municipal wellfield, and increases the extent of the final WHPA-QA1 to an area of 43.9 km^2.

6.3.2 WHPA-Q2 Results

Areas of future land use change were identified from the Rural Hamilton Official Plan as discussed in Section 6.2.6.3 and shown in Figure 3.18. All changes in land use within the model boundary were simulated in the WHPA-Q2 analysis to determine their effect on the Greensville municipal wellfield. The inputs to the GSFLOW model were adjusted to account for increased surface imperviousness and decreases in vegetation cover density associated with the infilling of the rural residential land within the Greensville Settlement Area. In accordance with the Water Budget Guide, no best management practices for stormwater management were considered. A new future annual average groundwater recharge rate map was determined through a 25-year GSFLOW model simulation, as shown in Figure 6.12.

Future land use changes in the Greensville Rural Settlement Area along with the WHPA-Q1 area are presented in Figure 6.19. Under the Water Budget Guide, only areas of land use change that straddle or are outside of the delineated WHPA-Q1 area need to be considered in the WHPA-Q2 assessment. For the purposes of this study, all areas of land use change were considered simultaneously with the assumption that if no measurable impacts on the municipal wells were predicted, no further analysis of individual impacts from land use changes outside the WHPA-Q1 area would be required.

As noted, to be included in the WHPA-Q2, it must be shown that the reduction in recharge will cause a measureable impact on the water levels in the municipal wells. To be consistent with the WHPA-Q1, a drawdown threshold of 1 m was used to determine whether the impact was "measurable". To isolate the incremental impacts of land use changes in the vicinity of the municipal wellfield, the incremental drawdowns at the two municipal wells between Scenario C (existing conditions) and Scenario G(3) (future land use change) were assessed (Scenario G(3) is discussed further on in Sections 6.4.3 and 6.5.3). At both FDG01 and TW-2-13, the water level drawdowns were predicted to be well below this 1 m threshold. The proposed land use changes are, therefore, not anticipated to have a measureable impact on the Greensville municipal well. Accordingly, the WHPA-Q2 area is coincident with the WHPA-Q1 area.

6.3.3 Local Area Results

For Tier 3 Assessments with municipal groundwater systems, the local area is equivalent to the WHPA-Q2. In this study, the WHPA-Q2 and the local area were determined to be identical to the delineated WHPA-Q1 as shown in Figure 6.18.

6.4 *Risk Assessment Scenario Methodology*

A series of risk assessment scenarios, listed in Table 6.2, were simulated to assess the impact of increases in water use, drought conditions, and land use change on the sustainability of the municipal wells. The scenarios were evaluated using the updated Greensville Tier 3 numerical model. Sustainability of the Greensville municipal supply wells was measured in terms of the simulated change in water levels within the municipal wells relative to the safe additional drawdown thresholds (developed in Section 3.2.1). Where required, the impact to other water uses was also considered.

6.4.1 Scenario C: Existing Land Use and Pumping under Average Climate

Scenario C is intended to verify the ability of the municipal water supply wells to maintain existing pumping rates under average climate conditions. This simulation used the updated Tier 3 numerical model with existing pumping rates for the municipal supply wells and for other, non-municipal permitted groundwater takers. This scenario is similar to that undertaken in the model calibration. Average pumping rates for the period of 2007 to 2014 were selected for quantifying existing demand. The long-term average annual groundwater recharge rate (Figure 6.20) was estimated from a 25-year GSFLOW simulation undertaken as part of the previous Tier 3 study.

The numerical model simulates the heads in all aquifers across the study area. Simulated head in the cells containing the two municipal water supply wells were extracted from the model results for use as reference water level conditions for comparing the results of the other steady-state stress assessment scenarios. Results for the other scenarios are expressed as additional (incremental) drawdown with respect to Scenario C.

In addition, the simulated streamflows, groundwater discharge to streams, and stage and groundwater levels in the wetland areas for Scenario C were extracted and used as reference conditions for evaluating the effects of municipal pumping on other water uses.

6.4.2 Scenario D: Existing Land Use and Pumping under Drought Conditions

Scenario D evaluates whether the municipal water supply wells are able to pump at the existing pumping rates under drought conditions. The drought period was selected after a careful review of data from AES climate stations within a 30 km distance of the study area for the period of 1950 to 2012. A total of 88 climate stations were available for this time period, of which 10 are located within the model area.

An in-depth analysis of historical climate data was undertaken as part of the previous Tier 3 study (Earthfx, 2015) to identify an appropriate drought period for use in the Tier 3 risk assessment. Based on this analysis, climate data from $_{WY}$1953 (Oct. 1952) to 1967 (inclusive) was selected. Average annual precipitation spatially averaged (see Section 2.3.2) over the study area was during this period was 815 mm/yr, interpolated annual volumes are provided in Figure 6.2. This 15-year period provided a four-year model "start-up" during which precipitation is near the historical average annual value. The four-year start-up period is followed by a three-year period of progressively drier years between 1957 and 1959. This is followed by seven more years, of which five have below-average precipitation volumes (including 1965, a dry year). Data coverage is good during this period, with between 50 and 80 climate stations available for spatial interpolation of climate data in the immediate vicinity of the model area. This spatial coverage provides a realistic representation of regional precipitation patterns.

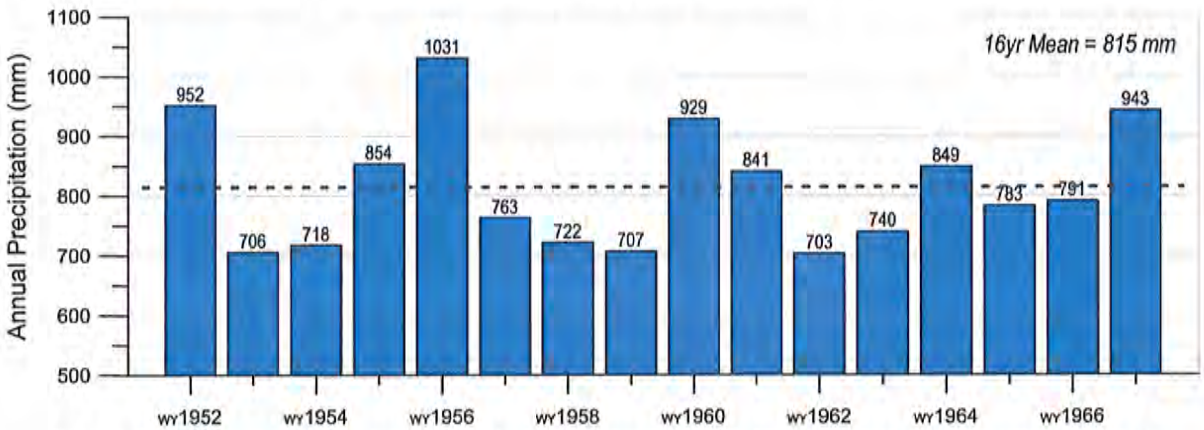


Figure 6.2: Annual spatially averaged study area precipitation for the drought assessment period.

The drought scenario was simulated using the updated Tier 3 integrated groundwater/surface water model using climate data from the MNRF infilled climate dataset (MNR, 2008). Pumping rates were varied in the transient drought simulations on a daily basis by cycling average daily extraction rates based on the reported values from the available WTRS and municipal datasets between 2007 and 2014. Because TW-2-13 was not active at the time of this study, average daily rates from FDG01 were distributed between the two wells according to a 6-to-1 day cycle whereby FDG01 was pumped for one day for every 6 days that TW-2-13 was pumped. This operational assumption for the Greensville municipal wellfield was developed in consultation with City of Hamilton staff. Extraction rates for non-municipal wells were adjusted using the consumptive use factors discussed in Section 3.5. Discharge from the quarries varied during the drought simulation based on the internally-calculated daily water balance. Results of the drought analysis are discussed further on in this section.

6.4.3 Scenario G: Future Land Use and Future Pumping under Average Climate

Scenario G evaluates the ability of the municipal wells to maintain their allocated water pumping rates (existing plus committed demand) under average climate conditions. This scenario was simulated using the updated Tier 3 numerical model under steady-state conditions with long-term average annual groundwater recharge. As per the Water Budget Guide (MNR, 2011), Scenario G was subdivided into three scenarios to isolate the impacts of municipal pumping from those of future changes in land use. The subsets of Scenario G are described as follows:

Scenario G(1) - Allocated (Future) Water Demand and Future Land Use: Scenario G(1) evaluates the combined impact of increased consumptive water demand and changes in recharge due to future land use (Section 6.2.6). For the Greensville Tier 3, this scenario evaluated the combined impacts of future quarry build-out, additional private water wells in the rural settlement area, and changes in groundwater recharge caused by conversion of agricultural lands to rural (estate) residential. The future annual average groundwater recharge rate map, determined through a 25-year GSFLOW model simulation with adjusted land-use based model parameters (see Section 6.3.2), was used in this simulation.

Scenario G(2) - Allocated (Future) Water Demand and Existing Land Use: Scenario G(2) evaluates only the impact of increased consumptive water demand (as discussed above) on the municipal water supply wells and other water uses. The average annual groundwater recharge rate was based on existing land use conditions (Figure 6.20). Scenario G(2) is equivalent to the WHPA-Q1 scenario, as discussed previously.

Scenario G(3) - Existing Pumping and Future Land Use: Scenario G(3) evaluates only the impact of future land use changes on the municipal water supply wells and other water uses. Average existing pumping rates for municipal water supply wells and non-municipal permitted groundwater takings were used in this scenario.

6.4.4 Scenario H: Future Land Use and Future Pumping under Drought Conditions

Scenario H evaluates whether the municipal wells are able to operate under a 10-year drought with future consumptive water use rates for nearby takings and with future land use. The drought time period used for Scenario H is the same as that discussed under Scenario D.

As per the Water Budget Guide, Scenario H was subdivided into three scenarios to isolate the impacts of increased consumptive water demands from those of changes in land use. The subsets of Scenario H are described as follows:

Scenario H(1) - Allocated Water Demand and Future Land Use under Drought Conditions: Scenario H(1) evaluates the combined impacts of increased consumptive water demand and reductions in recharge due to future land use changes under drought conditions. As in Scenario G(1), this includes the future quarry build-out, the addition of the Greensville School well, additional private water wells in the rural settlement area, and changes in groundwater recharge caused by conversion of agricultural lands to rural residential.

As in Scenario D, municipal and non-municipal pumping rates were varied in the model on a daily basis by cycling average daily extraction rates according to the reported values from the available WTRS and municipal datasets between 2007 and 2014. Because no increases in demand are anticipated for the Greensville municipal system, the pumping rates remained unchanged for this scenario. Detailed discussion of the future conditions with respect to additional residential wells, full quarry build-out, and changes in land use were presented in Section 6.2.6.

The drought scenario was simulated using the updated Tier 3 integrated groundwater/surface water model using climate data from the MNRF infilled climate dataset (MNR, 2008). Land-use based model parameters were adjusted (i.e., percent impervious and vegetative type and cover densities) to represent future land use change. The results of this analysis are discussed further on in this section.

Scenario H(2) - Allocated (Future) Water Demand and Existing Land Use under Drought Conditions: This scenario evaluates only the impact of increased consumptive water demand on the municipal wells during the 10-year drought period. Land use was assumed to represent existing conditions, as in Scenario D.

Scenario H(3) - Existing Pumping and Future Land Use under Drought Conditions: This scenario evaluates only the impact of future land use change on the municipal water supply wells during the 10-year drought period. Average daily extraction rates were applied as in Scenario D. Daily groundwater recharge rates with future land use changes within the Greensville Rural Settlement Area were calculated as in Scenario H(1).

6.5 *Risk Assessment Scenario Results*

6.5.1 Scenario C (Steady-State): Existing Pumping and Current Land Use under Average Climate

Simulated groundwater heads in the weathered bedrock aquifer are presented in Figure 6.21. The deeper Goat Island/Gasport aquifers are not used by the Greensville municipal well and, for brevity, maps and hydrographs for these units are not shown. Simulated drawdown from the non-pumping baseline conditions simulation in the weathered bedrock aquifer is shown in Figure 6.22 and drawdowns at the municipal wells are listed in Table 6.5. As noted previously, Scenario C, which represents existing conditions, served as a baseline for calculating the additional (incremental) drawdown for Scenario G. In Scenario C, the Lafarge and Flamboro quarries were simulated at their current (dewatered) extents.

Table 6.5: Simulated drawdowns at the Greensville Municipal Wellfield for Scenario C and Scenario G.

Well	Aquifer	Safe Additional Drawdown (m)	Scenario C Drawdown [1] (m)	Additional Drawdown (m) [2][3]		
				Scenario G(1)	Scenario G(2)	Scenario G(3)
Greensville FDG01	Weathered Bedrock Aquifer	4.5	0.68	0.55	0.58	-0.03
Greensville TW-2-13	Weathered Bedrock Aquifer	5.8	1.61	0.43	0.47	-0.04

Notes:

[1] Scenario C drawdown calculated using non-pumping baseline scenario as reference water level.
[2] Additional drawdown values calculated using Scenario C results as reference water level.
[3] Values include corrections for convergent head losses and non-linear head losses.

6.5.2 Scenario D: Future Water Demand and Future Land Use under Drought Conditions

Results from the drought Scenario D include the simulated stage, heads, and numerous other water budget components, which were produced by the Tier 3 integrated model on a cell-by-cell basis for each day of the simulation. Daily model values were summed and averaged on a monthly basis to simplify the presentation of some model results. Simulated average monthly aquifer heads for September 1956, prior to the start of the 10-year drought ($_{WY}$1953 to $_{WY}$1955), were taken to represent a pre-drought reference condition for the rest of the drought scenarios.

Hydrographs of daily head at the Greensville municipal wells are presented in Figure 6.3. The daily model outputs were examined to determine when the simulated heads reached their lowest values for the two wells during the drought period. The hydrographs indicate that the maximum decrease in simulated water levels occurs in July 1962 for FDG01. This corresponds to the two-year period of below average annual precipitation between 1962 and 1963 (Figure 6.2), combined with increased water demand during the summer months. The simulated maximum drawdown for TW-2-13 occurs in November 1963, at the end of the 1962-1963 low precipitation period.

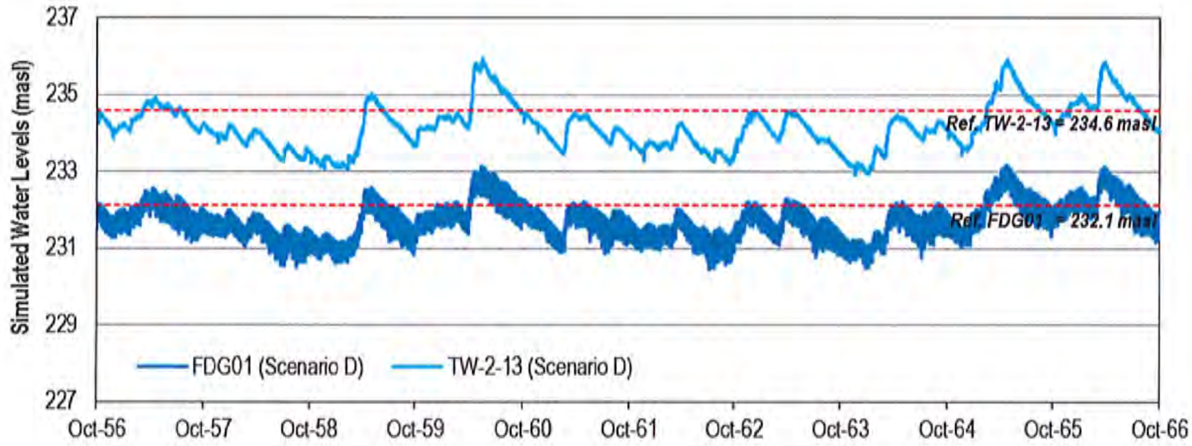


Figure 6.3: Simulated water levels at Greensville municipal wells FDG01 and TW-2-13 during Scenario D.

The maximum drawdowns under Scenario D were compared to the safe additional drawdowns at the municipal wells as presented in Table 6.6. Drawdowns were calculated relative to the average monthly head for September 1956, which served as a baseline. Drawdowns for FDG01 and TW-2-13 were both approximately 1.7 m, representing approximately one third of the respective safe additional drawdowns. These results indicate that the wells are fairly resistant to drought.

Table 6.6: Simulated drawdown at the Greensville Municipal Wellfield for Scenarios D and H

Well	Aquifer	Safe Additional Drawdown (m)	Additional Drawdown (m)			
			Scenario D	Scenario H(1)	Scenario H(2)	Scenario H(3)
Greensville FDG01	Weathered Bedrock Aquifer	4.5	1.68	2.14	2.19	1.64
Greensville TW-2-13	Weathered Bedrock Aquifer	5.8	1.68	2.03	2.13	1.58

Notes:

[1] Additional drawdown values calculated using monthly average simulated head for September 1956 (pre-Drought) of Scenario D as reference water level.

[2] Values include corrections for convergent head losses and non-linear head losses.

Simulated change in monthly average head between the pre-drought water levels and July 1962 are shown in Figure 6.23 for the weathered bedrock contact aquifer. The figure only shows declines greater than 1.0 m, the threshold for significant impact. The drawdowns in Figure 6.23 suggest that the groundwater system is generally resilient to drought. Simulated reductions in head in the semi-confined weathered bedrock aquifer are less than 1 m across much of the model area, with noticeable exceptions in areas of steep changes in bedrock topography along the Niagara Escarpment and at the edges of the quarries. A steeper decline along the escarpment is due to the large topographic relief across this feature, which acts as a site for seepage (drainage) of the groundwater system; this, combined with the low aquifer storage values associated with fractured bedrock aquifers, results in more pronounced reductions in groundwater heads during periods of drought.

The area to the south of the Lafarge North Quarry and to the west of the Lafarge South Quarry had simulated reductions in groundwater head of up to 2 m. Heads in the area representing the western extent of the recessional Waterdown Moraine are more sensitive to drought because groundwater levels are sustained by locally high recharge in an area that is otherwise well-drained by the quarry excavations to the east and by a tributary of Spencer Creek to the west.

6.5.3 Scenario G (Steady-State): Future Water Demand, Future Land Use, Average Climate

Additional drawdowns at the Greensville municipal wells under long-term (steady-state) conditions with future water demand and future land use (Scenario G simulations) were calculated and compared to the safe additional drawdown at the Greensville municipal wells (Table 6.5). As discussed previously, no future increases to pumping at the Greensville municipal water supply system are anticipated; thus, allocated demand is equal to the existing demand. However, increases in future takings related to the build-out of the quarries were considered along with the future pumping from the Greensville School well and the private residential wells in the Greensville Rural Settlement Area. Estimated drawdowns due to pumping at the municipal wells were corrected for convergent head loss and non-linear in-well losses as per the Water Budget Guide (MNR, 2011).

Scenario G(1): Simulated drawdowns at the municipal wells for Scenario G(1), which considers future pumping and future land use change, are less than the safe additional drawdown (Table 6.5). This indicates that pumping of the municipal wells to meet allocated demand is sustainable under average climate conditions with the future build-out of the quarries and the increases in nearby water demands and land use changes in the Greensville Rural Settlement Area.

Simulated additional drawdown in the weathered bedrock aquifer relative to reference conditions (Scenario C) is presented in Figure 6.24. Only drawdowns greater than the 1 m threshold are shown. The most significant increases in drawdown are focused around areas of the licensed quarry excavation that were not yet mined out under Scenario C, such as the Lafarge North Quarry Extension and the north end of the Lafarge South Quarry (both with approximate additional drawdowns of 25 m). Incremental drawdowns extend outward from these areas of future quarry build-out approximately 1300 m in the east toward Grindstone Creek and 1000 m to the west of the Lafarge South Quarry.

Additional drawdowns to the south of the quarries are below the 1 m drawdown threshold, and therefore do not appear on Figure 6.24. Because no future build-out is planned for the Lafarge South Process Area, dewatering requirements in that area are not expected to increase. The additional drawdown caused by the future private residential wells was small and did not exceed the 1 m threshold. In addition, the slight increase in groundwater recharge caused by the change in land use from agricultural row crops to rural residential estates likely offsets some of the drawdown created by the new private wells.

Scenario G(2): The simulated additional drawdowns for Scenario G(2), which considers the impacts of future increases in nearby pumping as well as future build-out of the quarries, are shown in Figure 6.25 for the weathered bedrock aquifer. The pattern of additional drawdowns in the vicinity of the quarries are consistent with those seen under Scenario G(1). Additional drawdowns to the south of the quarries are below the 1 m threshold.

The additional drawdowns at the Greensville municipal wells (Table 6.5) are slightly larger than under Scenario G(1), because there is no increased recharge associated with the switch from agricultural land use to residential estates. Simulated drawdowns caused by the additional private residential wells were still less than the safe additional drawdown and, therefore, did not compromise the sustainability of the municipal supply wells.

Scenario G(3): Scenario G(3) considers future land use change only. Simulated additional drawdown in the weathered bedrock aquifer relative to baseline conditions (Scenario C) is presented in Figure 6.26. Changes in groundwater levels in the semi-confined weathered bedrock aquifer exceed the 1 m threshold in a single area to the west of Brock Road, South of Highway 5. Simulated groundwater levels increased by approximately 1.2 m due to the change in land use from agricultural to rural residential estates. The slight increase in the water levels in the weathered bedrock aquifer at the municipal wells for Scenario G(3) are presented as small negative drawdowns, as shown in Table 6.5.

6.5.4 Scenario H (Transient Drought): Future Water Demand and Future Land Use under Drought Conditions

Results from the Scenario H transient drought simulations include stream stage, aquifer heads, and numerous other water budget components calculated on a cell-by-cell basis for each simulation day. The maximum additional drawdowns at the Greensville municipal wells under transient drought conditions (Scenario H) were compared to their respective safe additional drawdowns. Drawdowns were calculated relative to the simulated average monthly aquifer heads for September 1956 prior to the start of the 10-year drought from Scenario D, which served as a reference condition. Results are presented in Table 6.6.

Scenario H(1): As in Scenario G, three different simulations were run to identify the separate and combined contributions of increased pumping and future land use change on the simulated drawdowns. Scenario H(1) simulates drought conditions and considers future pumping as well as future land use change. Hydrographs of simulated daily heads for the Greensville municipal wells are presented in Figure 6.4, along with daily water levels from Scenario D (shown in blue) for comparison. Differences between the Scenario H(1) and Scenario D hydrographs are small. As in Scenario D, the maximum decrease in simulated head occurs in July 1962 for FDG01 and in November 1963 for TW-2-13.

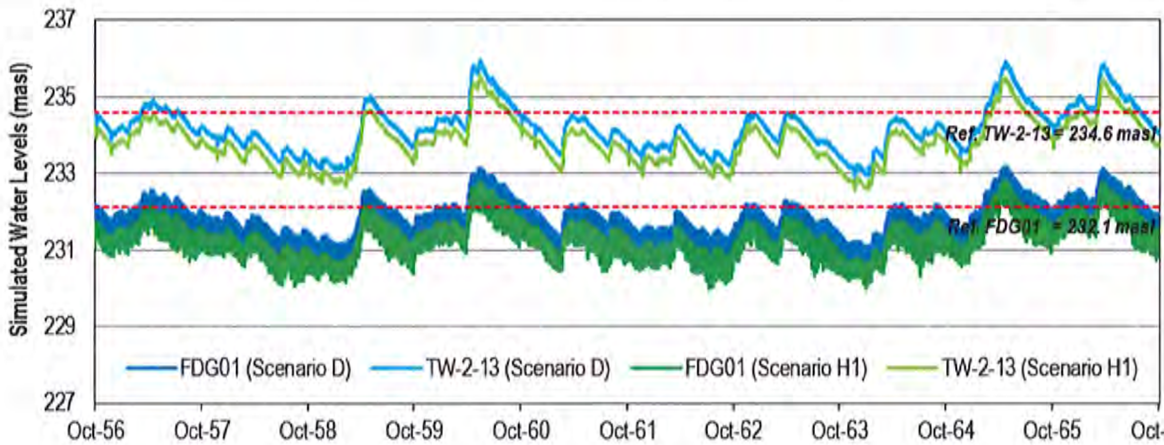


Figure 6.4: Simulated water levels at Greensville municipal wells FDG01 and TW-2-13 during Scenario H(1).

Monthly average heads for July 1962 were used to represent the most severe drought conditions in comparative mapping. The additional drawdown between the pre-drought period (September 1956 for Scenario D) and the average heads for July 1962 are shown in Figure 6.27 for the weathered bedrock aquifer. High drawdowns around the quarry are due to the effects of increased dewatering at quarry build-out, which was not considered in Scenario D.

The maximum additional drawdowns at the Greensville municipal wells for Scenario H(1) are presented in Table 6.6. The additional drawdowns were found to be less than the safe additional drawdown thresholds for both FDG01 and TW-2-13. This indicates that the Greensville municipal wells are capable of meeting the system water demands under drought conditions with future quarry build-out, additional pumping from surrounding private residential wells, the Greensville School, and future land use.

Scenario H(2): Scenario H(2) simulates the response of the municipal wells under drought climate conditions and considers only increased pumping. Hydrographs of simulated daily heads at the Greensville municipal wells are presented in Figure 6.5 for Scenario H(2) along with hydrographs of simulated water levels from Scenario D (shown in blue) for comparison. The maximum additional drawdown at the municipal wells for Scenario H(2) are presented in Table 6.6 and were found to be less than the safe additional drawdown values.

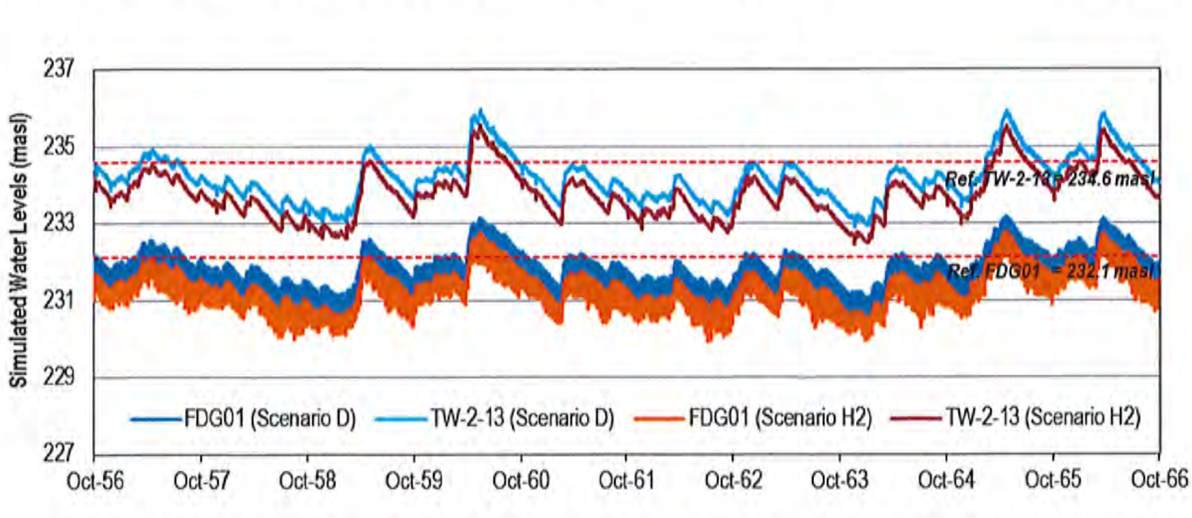


Figure 6.5: Simulated water levels at Greensville municipal wells FDG01 and TW-2-13 during Scenario H2.

Monthly average heads during July 1962 were taken to represent the most severe drought conditions. The incremental drawdown between the reference levels (September 1956 for Scenario D) and the average heads for July 1962 are shown in Figure 6.28 for the weathered bedrock aquifer. As in Scenario H(1), the high drawdowns around the quarry are due to the effects of increased dewatering at quarry build-out which was not considered in Scenario D.

Scenario H(3): Scenario H(3) simulates the response of the municipal wells under drought climate conditions and considers only the future land use changes in the Greensville Rural Settlement Area. Hydrographs of simulated daily heads at the Greensville municipal wells are presented in Figure 6.6, along with simulated water levels from Scenario D (shown in blue) for comparison. The maximum additional drawdowns at the municipal wells for Scenario H(3) are presented in Table 6.6. The small change in transient heads between Scenario H(3) and Scenario D suggest that the municipal wells are largely unaffected by the planned land use changes.

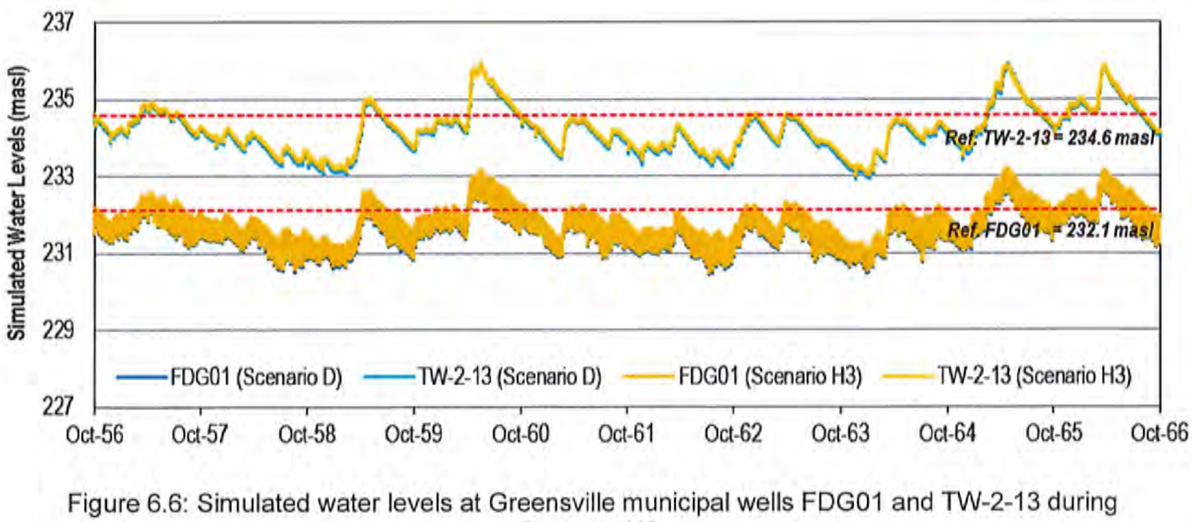


Figure 6.6: Simulated water levels at Greensville municipal wells FDG01 and TW-2-13 during Scenario H3.

6.6 *Impacts to Other Uses*

One of the goals of the Scenario G series of simulations is to develop a better understanding of the threats to other water uses posed by the municipal wells. Other water uses, as defined in the Clean Water Act, are discussed in Section 3.6.

Under the Technical Rules (MOE, 2009), as clarified by the MOE (written communication, December 2, 2013), the assignment of risk level to the local area based on threats to other users is related only to impacts caused by increased pumping at the municipal wells to meet a Planned Quantity of Water (see Figure 3.1). Because there are no anticipated increases in demand for the Greenville municipal wells, the risk level assigned through consideration of impacts to other uses is therefore 'low'.

Nevertheless, the response of other water uses listed above to future conditions represented in the Greensville Tier 3 model are discussed in the following sections. While these simulated impacts will not change the assigned risk level, the results of this analysis offer insight into areas of potential concern moving into the future.

6.6.1 Effects on Aquatic Habitat

With respect to aquatic habitat, the Technical Rules (MOE, 2009) provide specific thresholds to be used in evaluating the impact to cold water stream reaches. Impacts are measured in terms of changes in average monthly baseflow discharge to streams. A reduction by an amount that is greater than either of the following two criteria is deemed significant (when caused by increases in municipal pumping from a planned system):

- 20% of the existing estimated streamflow that is exceeded 80% of the time (Q_{p80}), or
- 20% of the existing estimated average monthly base flow of the stream.

In addition to the 20% threshold for significance, a moderate risk level occurs if the reduction is between 10 and 20%. The first criterion can be used where the Q_{p80} values are estimated from gauged flows. The second criterion is more applicable to ungauged streams and was selected for use in this study because it is more compatible with the steady-state analyses completed for Scenarios C and G.

Groundwater exchange between the aquifer system and each stream reach is computed by the SFR2 module in the updated Greensville Tier 3 steady-state model. A stream reach, for modelling purposes, is defined as the length of the stream within a model cell; a stream segment is defined as the length of stream between junctions. The rate of discharge from each groundwater cell is based on: (1) the difference in head between the aquifer and the water level (stage) in the stream, (2) the hydraulic conductivity and thickness of the streambed, and (3) the wetted area of the stream within the cell. The stage in the stream is calculated based on the baseflow accumulated from all upstream reaches.

Cell-by-cell discharge values can be used to identify stream reaches where there is significant groundwater discharge. The Tier 3 model accumulates and routes the groundwater discharge through the stream network using the surface water routing capabilities of the SFR2 module. In addition, groundwater discharge to land surface and reject recharge are also routed to the stream network according to topographic drainage area. Combined, these model components provide an approximate estimate of accumulated baseflow. As the flows move downstream, they can re-infiltrate in reaches where the stream stage is higher than the underlying aquifer head.

Simulated steady-state baseflow is shown in Figure 6.29 for the Scenario C baseline conditions (existing pumping). Figure 6.30 presents Scenario G(1) groundwater discharge to streams, considering future pumping to meet allocated demand. The figures use a logarithmic color scale to better present the large range in simulated values. A value of 0.001 m^3/s was used as a cut-off in these figures and in subsequent calculations to reflect the numerical precision of the model.

The change in simulated groundwater discharge to streams between Scenario C and G(1) (in m^3/s) is shown in Figure 6.31. The same information is presented in terms of percent decrease between Scenario G(1) and Scenario C in Figure 6.32. The results indicate that a tributary to Spencer Creek in the vicinity of the Greensville municipal wellfield experiences reductions of between 10 and 15%. The impacts to this tributary (approximately 475 m west of FDG01) are attributed to the addition of up to 150 new private residential wells immediately west of the affected stream reaches.

Reductions in streamflow ranging from 10 to 100% are expected to the east of the Lafarge Quarry, particularly in the tributary of Logie's Creek running along the east edge of the South Quarry, and in small tributaries that drain into the Grindstone Creek wetland east of the North Quarry. It should be noted that in the approved Lafarge North Quarry Extension plan, flows to the Grindstone Creek wetland are to be augmented by water from the quarry sumps; though this was not represented in the model.

Simulation results for the westward flowing tributary to Spencer Creek that has its headwaters between the three quarries show a significant percent reduction in flows approaching the quarry footprints. The most significant percent reductions are found in the portion of the tributary leading to the outfall of the Flamboro Quarry dewatering discharge. The simulation results show that at full build-out of the quarries, the deeper Lafarge North Quarry will intercept a portion of the incoming groundwater seepage to the neighboring Flamboro Quarry. The combination of reduced groundwater inflows into Flamboro Quarry (translating to a reduction in sump discharge) along with steeper hydraulic gradients near the edges of the future quarry footprint results in a reduction in flows to the receiving Spencer Creek tributary.

Isolated reaches outside of the Local Area delineation also indicate streamflow reductions ranging from 5 to 100% associated with non-municipal surface water and groundwater permits. Agricultural permits on Grindstone Creek, southwest of Millgrove, and on Logie's Creek, west of the Lafarge South Quarry result in local reductions in the adjacent stream reaches. In both cases, the reaches are lower-order headwater tributaries, which are likely to be more sensitive to nearby groundwater takings. Furthermore, the simulated reductions could be the result of cumulative impacts from both the local agricultural takings and the influence of quarry dewatering.

As discussed previously, the Tier 3 Risk Assessment methodology for impacts to other uses is based upon impacts caused by increased pumping at the municipal wells. As no increase in demand is anticipated for the Greensville municipal supply system, no increase in the risk level is warranted.

6.6.2 Assessment of Impacts to Provincially Significant Wetlands

Thresholds for evaluating risk to Provincially Significant Wetlands (PSW) are not specified in the Technical Rules. One approach, used in other Tier 3 studies, identifies wetlands subject to more than a 1 m drawdown in groundwater levels beneath the PSW as being at risk (e.g., Earthfx, 2013). Drawdowns were determined by subtracting simulated steady-state heads in the weathered bedrock aquifer under Scenarios C from those under the Scenario G(1).

Figure 6.33 shows the incremental 1 m drawdown in the weathered bedrock aquifer under Scenario G(1) and the location of the PSWs. Five wetland features fall within the 1 m drawdown contours within the local area. Two of these wetlands are located northwest and northeast of the Lafarge North Quarry, the latter of which forms part of the Hayesland-Christie Wetland Complex along Grindstone Creek. Three other smaller PSWs located east of the Lafarge South Quarry footprint fall either entirely or partially within the 1 m incremental drawdown contour and are therefore considered to be partially impacted by future, non-municipal water takings. The 1 m incremental drawdown does not encompass the Greensville municipal wellfield and, therefore, the threat to PSWs posed by the municipal wells is negligible.

6.6.3 Assessment of Impacts for Recreational Water Use

The simulations have demonstrated that Christie Reservoir has little to no hydraulic connection with the Greensville municipal wellfield, located 1.7 km away in the weathered bedrock contact aquifer. The 1 m incremental drawdown in Scenario G(1) (compared to simulated water levels in Scenario C) caused by future conditions at the quarry and in the Greensville Rural Settlement Area does not intercept the Christie Reservoir (Figure 6.24). Therefore, the threat to recreational water use posed by the municipal wellfield is considered to be negligible.

6.6.4 Assessment of Impacts for other Permitted Groundwater Takings

The impacts to other permitted groundwater takings were assessed based on the simulated reduction in the water levels at nearby non-municipal wells used for agricultural, commercial and industrial, or recreational water takings under Scenario G (outlined in the Technical Bulletin: Part IX Local Area Risk Level). As discussed previously, there is no anticipated increase in pumping at the Greensville municipal wells, so impacts to other permitted groundwater takings due to future conditions are being assessed for informational purposes rather than to assign a risk level.

Figure 6.34 shows the extent of the 1 m incremental drawdown in the weathered bedrock and Goat Island/Gasport aquifers, determined by subtracting simulated heads in the respective model layers for Scenario C and Scenario G(1). Also shown in Figure 6.34 are the locations of the permitted water takings. Of the 41 non-municipal groundwater permits, three permits (comprising 6 groundwater sources), as well as the future Greensville School well, are located within the combined 1 m incremental drawdown area. Table 6.7 presents the simulated incremental drawdown at each of these wells for Scenario G.

Table 6.7: Simulated additional drawdown at non-municipal wells - Scenario G.

Permit No.	Source	Aquifer	Additional Drawdown (m)		
			Scenario G(1)	Scenario G(2)	Scenario G(3)
8811-AB6QAH	Well #1	Guelph Formation	0.04	0.04	0.00
8811-AB6QAH	Well #2	Gasport Formation	3.13	3.34	0.00
3708-9Z6PUA	Well #1	Guelph Formation	0.35	0.36	<0.01
3708-9Z6PUA	Well #2	Upper Eramosa	0.38	0.40	<0.01
7620-98RR56	Well	Weathered Bedrock	0.08	0.08	0.00
Greensville School Well	Well	Weathered Bedrock	1.24	1.27	<0.01

Based on the results of Scenario G, simulated additional drawdowns were typically less than 1 m. Two exceptions are Well #2 of permit 8811-AB6QAH, which is screened in the Gasport Formation and is predicted to experience an additional drawdown of up to 3.3 m. Based on an interpreted top of the Goat Island/Gasport aquifer of 214.7 masl and the Scenario C water level of 235.2 masl, there is an estimated 20 m of available drawdown at this location. In this context, the additional impacts to this non-municipal permit could be considered acceptable; however, confirmation of the precise conditions at Well #2 of PTTW No. 8811-AB6QAH should be assessed to conclusively establish the significance of any dewatering impacts to this well.

Drawdowns at the future Greensville School well also exceeded the 1 m threshold for Scenario G(1) and Scenario G(2). As this well was not pumped under the current conditions scenario (Scenario C), the incremental drawdowns presented in Table 6.7 represent to the total anticipated drawdown as a result of pumping the Greensville School well. The well record shows that the well is completed in the same productive weathered bedrock contact aquifer as the nearby municipal supply wells. Assuming similar safe additional drawdowns as the municipal wells (ranging from 4.5 to 5.8 m), the predicted drawdowns for the Greensville School well are within the acceptable range of available drawdown. Once again, confirmation of the conditions at this well should be assessed in a future study, as only limited information about this water supply system was available at the time of this study.

6.7 *Local Area Risk Assessment Results*

Using the various stress assessment scenarios, a Water Quantity Risk Level Classification was performed for the local area delineated for the Greensville municipal wells (Figure 6.18). The methodology for assessing and assigning a risk level to local areas was clarified in the Technical Bulletin: Part IX Local Area Risk Level (MOE, 2010). The assignment of risk was conducted based on the circumstances presented in Table 6.8.

6.7.1 Tolerance

The Tier 3 assessment also considers a municipal water system's tolerance to risk. The Technical Rules state that "tolerance is evaluated to determine whether an existing system is capable of meeting peak demand". Under Rule 100:

> *For the purposes of evaluating the groundwater scenarios C and D in Table 4B, a tolerance level shall be assigned to the existing type I, II or III system which the local area relates that is the subject of evaluation in accordance with the following:*
>
> *(1) A tolerance level of high if the existing system is capable of meeting peak demand during all assessment periods.*
>
> *(2) A tolerance level of low if sub-rule (1) does not apply to the existing system.*

To evaluate the tolerance of the system under existing conditions and average climate (Scenario C), the pumping rates at the municipal wells FDG01 and TW-2-13 were increased to 11.6 and 70.2 m^3/day, respectively. This represents a sustained increase of 100% from the average daily system takings of 40.9 m^3/d. The results of the steady-state simulation using the increased long-term pumping rate suggest that the existing municipal supply wells are capable of meeting estimated peak demands. Results of the tolerance assessment for the Greensville municipal wells are summarized in Table 6.9.

Table 6.8: Risk Assessment Scenarios and Circumstances

Local Area – Significant Risk	
Scenarios	**Circumstances**
Scenario C Existing demand, existing land use, average climate **Scenario D** Existing demand, existing land use, drought climate	The tolerance of the groundwater supply system within the local area is considered to be low, based on its inability to meet the existing demand at any time during the transient drought or steady-state scenarios (Scenario C and Scenario D). Failure to meet the simulated demand occurs when the safe additional drawdown for a given well is exceeded. The assignment of a tolerance of low is generally associated with an existing municipal system that has historically experienced water quantity shortages.
Scenario G(1), (2), (3) Existing and/or planned demand, existing and/or planned land use, average climate **Scenario H(1), (2), (3)** Existing and/or planned demand, existing and/or planned land use, drought climate	At any time during the transient or steady-state scenarios (Scenario G and Scenario H), the groundwater supply system within the local area is unable to meet the simulated demand (existing or Existing plus Committed plus Planned). Failure to meet the simulated demand occurs when the safe additional drawdown for a given well is exceeded.
Scenario G(1), (2), (3) Existing and/or planned demand, existing and/or planned land use, average climate	The Existing plus Committed plus Planned demand at the municipal wells within a local area would result in a measurable and unacceptable impact to other uses. With respect to aquatic habitats classified as cold water streams, an impact is considered to be unacceptable if it results in: i) a reduction in groundwater discharge that is more than 20% of the existing monthly stream flow that is exceeded 80% of the time (Qp80); or ii) a reduction in groundwater discharge that is more than 20% of the existing estimated average monthly base flow of the stream.

Local Area – Moderate Risk	
Scenarios	**Circumstances**
Scenario G(1), (2), (3) Existing and/or planned demand, existing and/or planned land use, average climate	The Existing plus Committed plus Planned demand at the municipal wells within a local area would result in a measurable and *potentially* unacceptable impact to other uses. With respect to aquatic habitats classified as cold water streams, an impact is considered to be *potentially* unacceptable if it results in: i) a reduction in groundwater discharge that is more than 10% but no larger than 20% of the existing monthly stream flow that is exceeded 80% of the time (Qp80); or ii) a reduction in groundwater discharge that is more than 10% but no larger than 20% of the existing estimated average monthly base flow of the stream.

Table 6.9: Evaluation of Greensville Municipal Wellfield tolerance.

Well	Aquifer	Existing Demand (m^3/day)	Assessed Peak Demand Rate (m^3/day)	Safe Additional Drawdown (m)	Additional Drawdown under Peak Demand (m)
Greensville FDG01	Weathered Bedrock Contact Aquifer	5.8	11.6	4.5	0.53
Greensville TW-2-13		35.1	70.2	5.8	1.60

To further support the assignment of a high tolerance level to the Greensville municipal system, it should be noted that the existing well FDG01 has proved a reliable source of drinking water since its construction in 1972, prior to the introduction of TW-2-13 (planned for 2017). Preliminary hydraulic testing results of TW-2-13 by SNC Lavalin (2016) indicated that the new well has a capacity of 90 L/min (130 m^3/d), and is capable of meeting the peak demands of the Greensville water supply system on its own. As such, the addition of TW-2-13 provides a 100% redundancy factor to the Greensville municipal wellfield, thereby establishing a high system tolerance to peaks in demand as well as temporary shutdown of one of the wells due to maintenance requirements or equipment malfunction.

6.7.2 Risk Classification

The results of the stress assessment scenarios (G and H) suggest that the Greensville municipal wellfield is capable of meeting existing and allocated water demands for current and future land use conditions during both average climate and drought. The Greensville local area was therefore assigned a risk level of low (Table 6.10). No impacts to other uses including aquatic habitat, PSWs, and other permitted takings are anticipated because pumping rates at the municipal wells are not expected to increase. The tolerance of the Greensville municipal wellfield is high because there is sufficient additional drawdown in the wells even when pumping is sustained at peak rates, and either well could handle the total system demands should there be a failure at one of the wells.

Table 6.10: Assigned Risk Levels

Local Area	Tolerance	Risk Level
Greensville Well FDG01	High	Low
Greensville Well TW-2-13	High	Low

6.7.3 Uncertainty Analysis of Risk Level Assignment

According to the Technical Rules (Rule 108) and MOE (2010) after assigning a risk level to a local area, an uncertainty analysis must be conducted that considers the following factors:

(1) the distribution, variability, quality and relevance of the data used to evaluate the scenarios;
(2) the degree to which the methods and models used to evaluate the scenarios accurately reflects the hydrologic system of the local area for both steady state and transient conditions;
(3) the extent and level of calibration and validation achieved for any groundwater and surface models used or calculations and general assessments completed;
(4) the quality assurance and control procedures used in evaluating the scenarios.

These factors were considered to determine whether the uncertainty underlying the risk assignment should be characterized as high or low:

Distribution, variability, quality and relevance of data used to evaluate the scenarios -- The distribution of data used to describe the geologic, hydrogeologic, and hydrologic setting of the study area was discussed at great length in the Phase 1 report (Earthfx, 2014), and remains largely applicable to this Tier 3 update. Data from a variety of sources, including climate records, streamflow measurements, static and transient groundwater levels, geologic logs, pumping data, reservoir stage, and quarry discharge, were collected, reviewed, and synthesized for the original study. Additional data from the Lafarge hydrogeologic study (Golder, 2013) and the recent City of Hamilton supply well exploration program supplemented this already extensive database.

The new data were used to update the conceptual stratigraphic and hydrostratigraphic models and formed the basis for the updated integrated model used in this study. Time series data (climate data, upstream inflows, and pumping) were supplied as input to the numerical model while the groundwater level, streamflow, and lake stage data served as calibration targets.

Overall, the data coverage for the study area is comparable to and, in some cases, exceeds that for other Tier 3 studies. In particular, the number of transient groundwater monitors is high especially in the vicinity of the municipal wells. The subwatershed has two WSC stream gauges and there are a relatively high number of AES and other climate stations within and adjacent to the study area. NEXRAD data are also available. Although there were issues noted with the quality, temporal coverage, and spatial coverage of the MOECC WWIS groundwater level data used in the steady-state calibration and gaps in the climate data record, on the whole, the amount of data available, the overall quality of the data, and the density of coverage in the vicinity of the municipal well is quite good. There is always a degree of uncertainty, however, with regards to hydrogeologic properties and model assumptions needed to extrapolate available data such as the extent and thickness of the Vinemount aquitard and the effect of blasting on the integrity of the Vinemount aquitard.

Degree to which the methods and models used to evaluate the scenarios accurately reflects the hydrologic system of the local area for both steady state and transient conditions -- The Greensville Tier 3 conceptual model represented a significant localized refinement of the North Hamilton Conceptual Model used for the Tier 1 and 2 assessments. The geologic conceptual model was updated for the original Tier 3 study to reflect new insights from the OGS, local drilling programs from previous studies, and a considerable refinement of the overburden stratigraphy.

Both the hydrologic and groundwater flow submodels are deterministic, distributed, physically-based models. The hydrologic model considered climate inputs, topography, soils, and land use data in computing dynamic soil water balances, groundwater recharge, and overland flow to streams on a daily basis. The groundwater flow model represents groundwater flow, unsaturated flow, streamflow, and lake water balances. These models have been employed in a rigorous manner to model the surface and subsurface hydrologic processes within the local area with specific focus on the interaction and feedback between the groundwater and surface water systems as required under the Technical Rules. The representation of the surface water system, groundwater and surface water interaction, and the effect of quarry and reservoir operations as well as water takings is much improved over previous Tier 1 and 2 models and builds upon the robustness of the original Tier 3 modelling work.

By integrating the two submodels in GSFLOW, the feedback mechanisms between the groundwater and surface water systems are well represented. The reasonableness of submodel outputs (e.g., groundwater recharge values from the hydrologic model and groundwater discharge to the soil zone from the surface water model) and the overall water budget can be tested much more rigorously than with a separate, non-integrated model. Overall, the integrated model used to evaluate the local conditions significantly exceeds similar impact studies. Although no model can perfectly match the observed behaviour due to inherent simplifications and incomplete information, it is felt that the conclusions based on model results are reasonable and scientifically sound.

The extent and level of calibration and validation achieved for any groundwater and surface water model used or calculations and general assessments completed -- The original Tier 3 model was calibrated to a wide range of conditions, including extreme wet and dry-year conditions, and across multiple seasons. A high degree of confidence in model results was obtained. The updated Tier 3 integrated model underwent another phase of calibration, this time with a focus on achieving a match to new hydrogeologic datasets in the vicinity of the municipal wellfield and the Lafarge quarries. The original

Tier 3 was considered to be well-calibrated and the calibration has been improved through this update. The updated Tier 3 model is therefore suitable for determining the sustainability of the municipal wellfield as part of the Tier 3 Risk Assessment.

Quality assurance and control procedures used in evaluating the scenarios -- The models require a large amount of data analysis and preparation prior to initiating a scenario analysis. Great care was used in setting up, documenting, and conducting each risk assessment scenario. Multiple levels of internal review were conducted to ensure that the input data preparation programs produced correct input files. All model outputs were saved and reviewed through visual inspection of hydrographs, digital mapping, and animations.

Based on the above-noted factors, it is felt that there is a low uncertainty in the assignment of the risk levels to the local area.

6.8 *Water Quantity Threats*

Under the Technical Rules (MOE, 2008), local areas classified as having a risk level of significant or moderate have the water quantity threats identified that may limit the sustainability of the municipal water supply wells. Drinking water quantity threats are defined as: 1) an activity that takes water from an aquifer or a surface water body without returning the water taken to the same aquifer or surface water body; or 2) an activity that reduces the recharge of an aquifer.

The updated local area of the Greensville municipal wellfield was assigned a risk level of low, consistent with the results from the previous Tier 3 study. As a result, no water quantity threats have been identified.

6.9 Figures

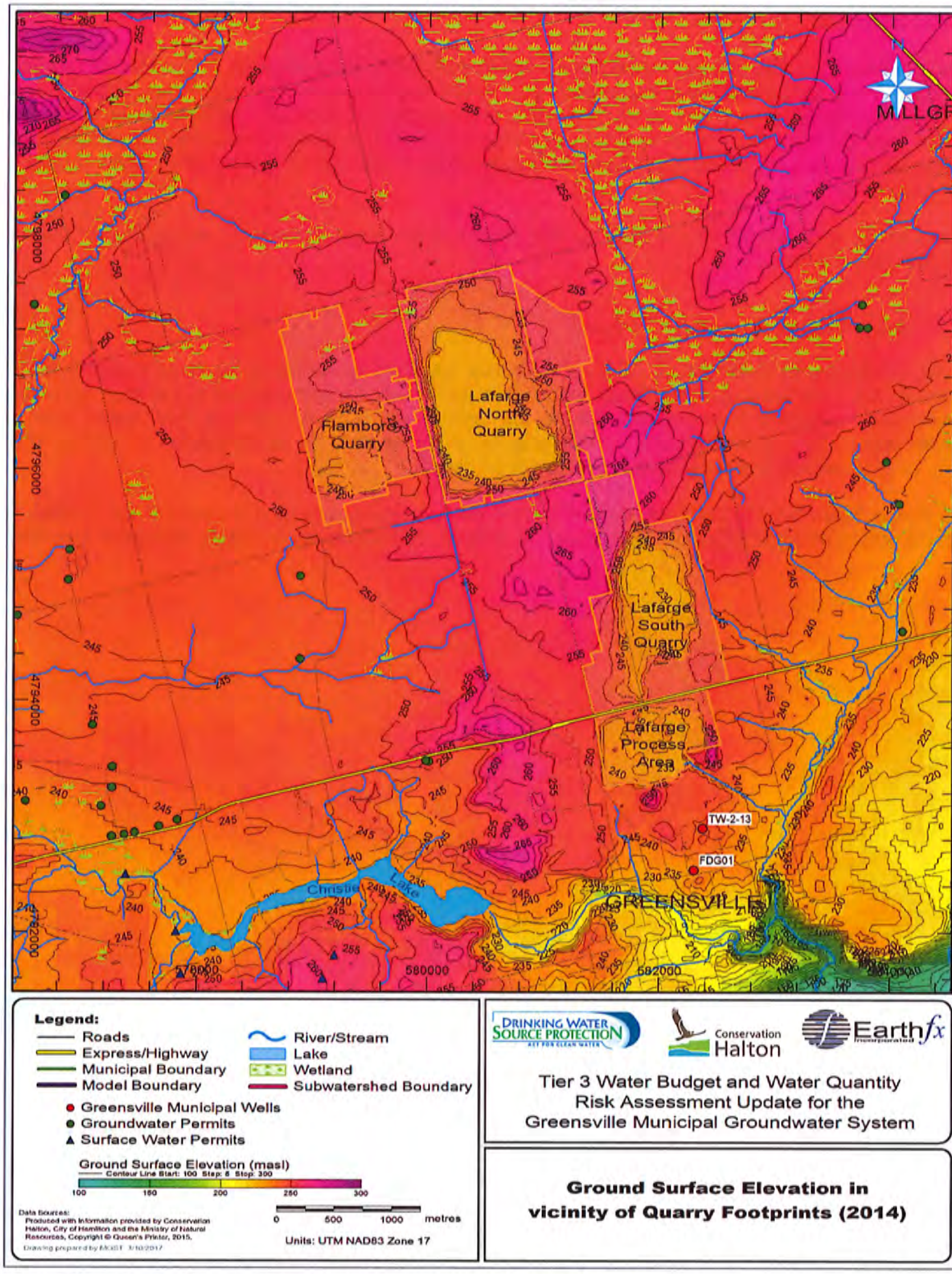


Figure 6.7: Ground surface elevation in vicinity of quarry footprints under current conditions.

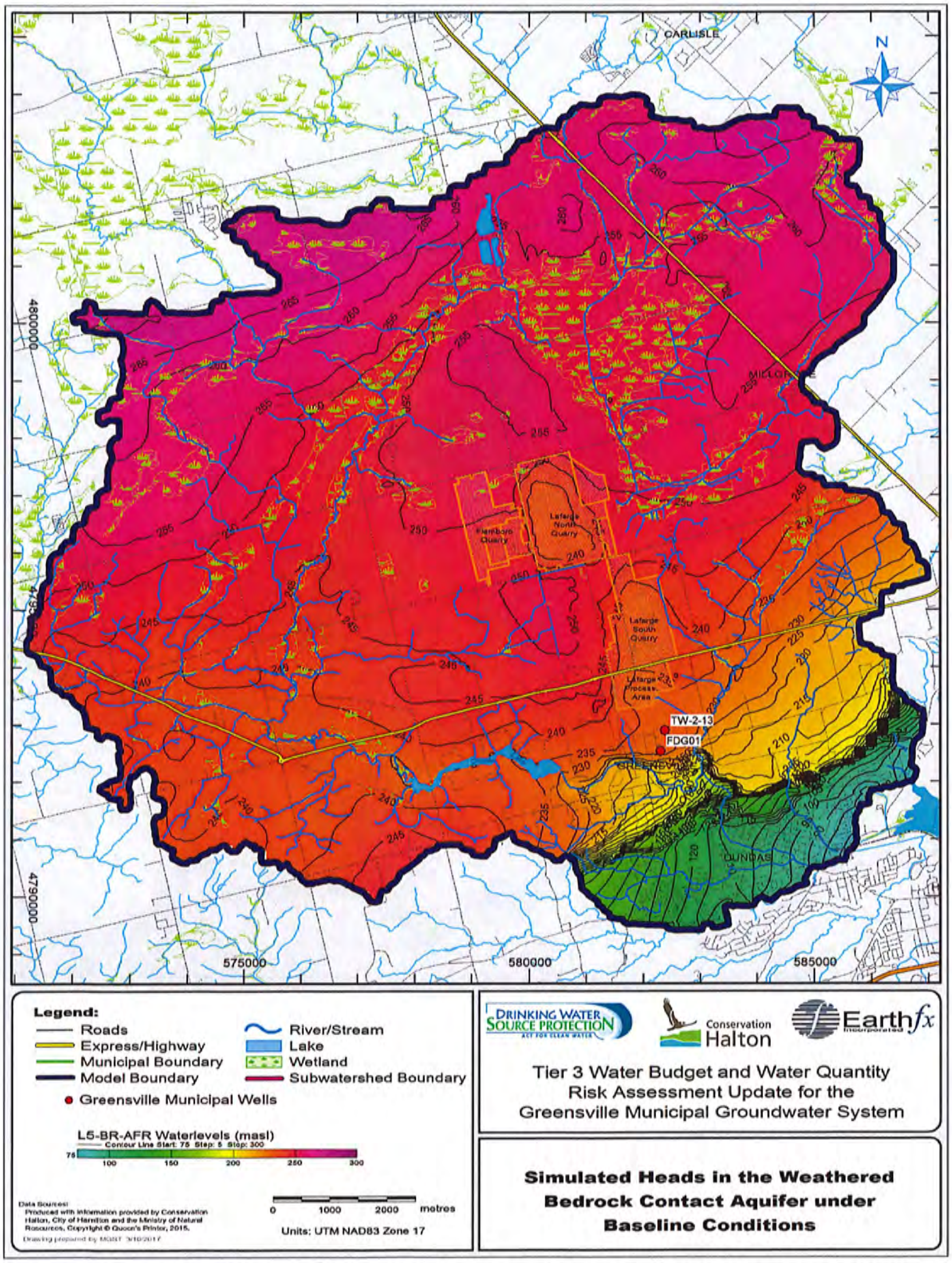


Figure 6.8: Simulated heads in the weathered bedrock aquifer (Layer 5) under baseline conditions.

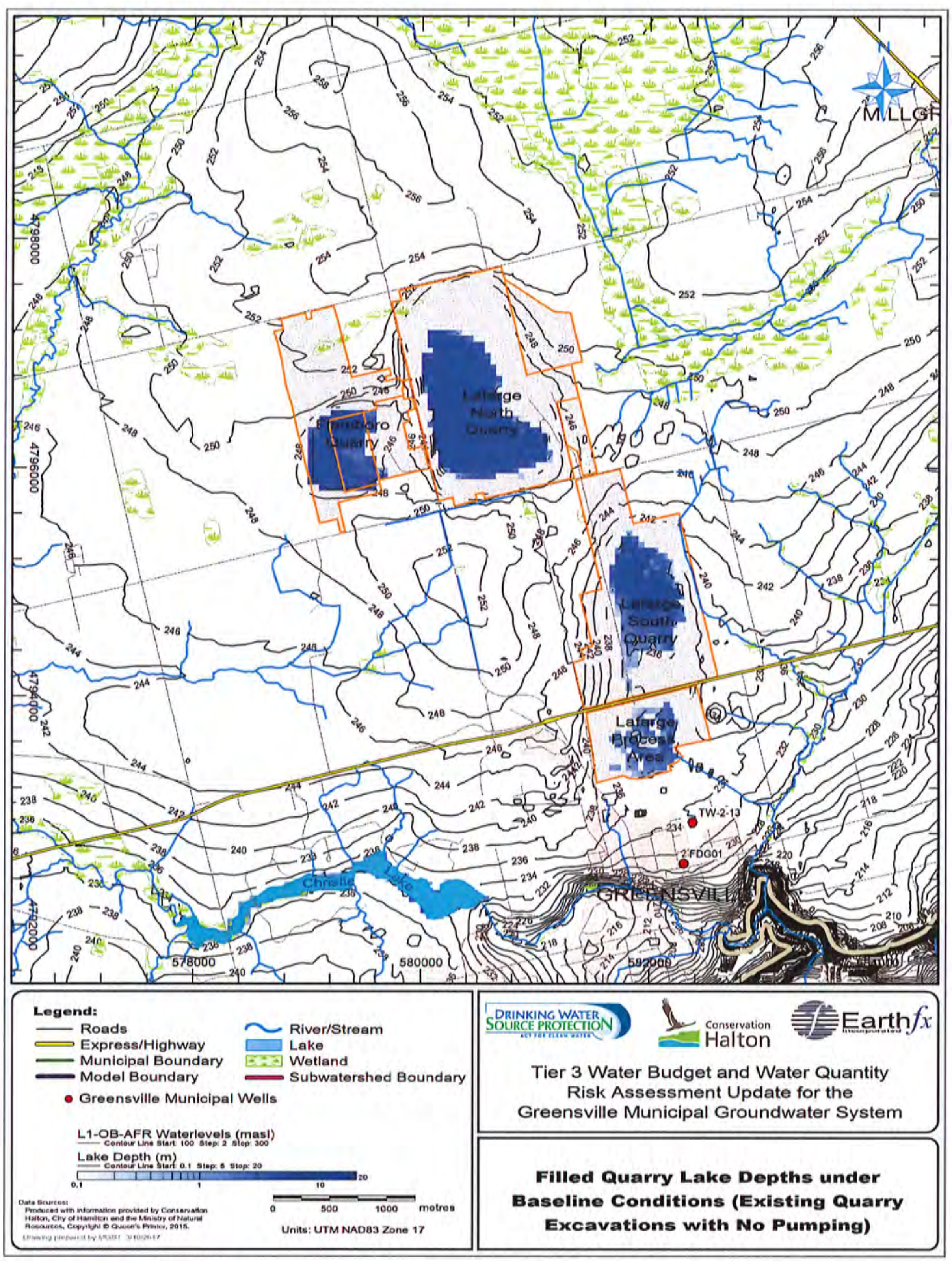


Figure 6.9: Simulated water table, lake stage, and quarry lake depth under baseline conditions.

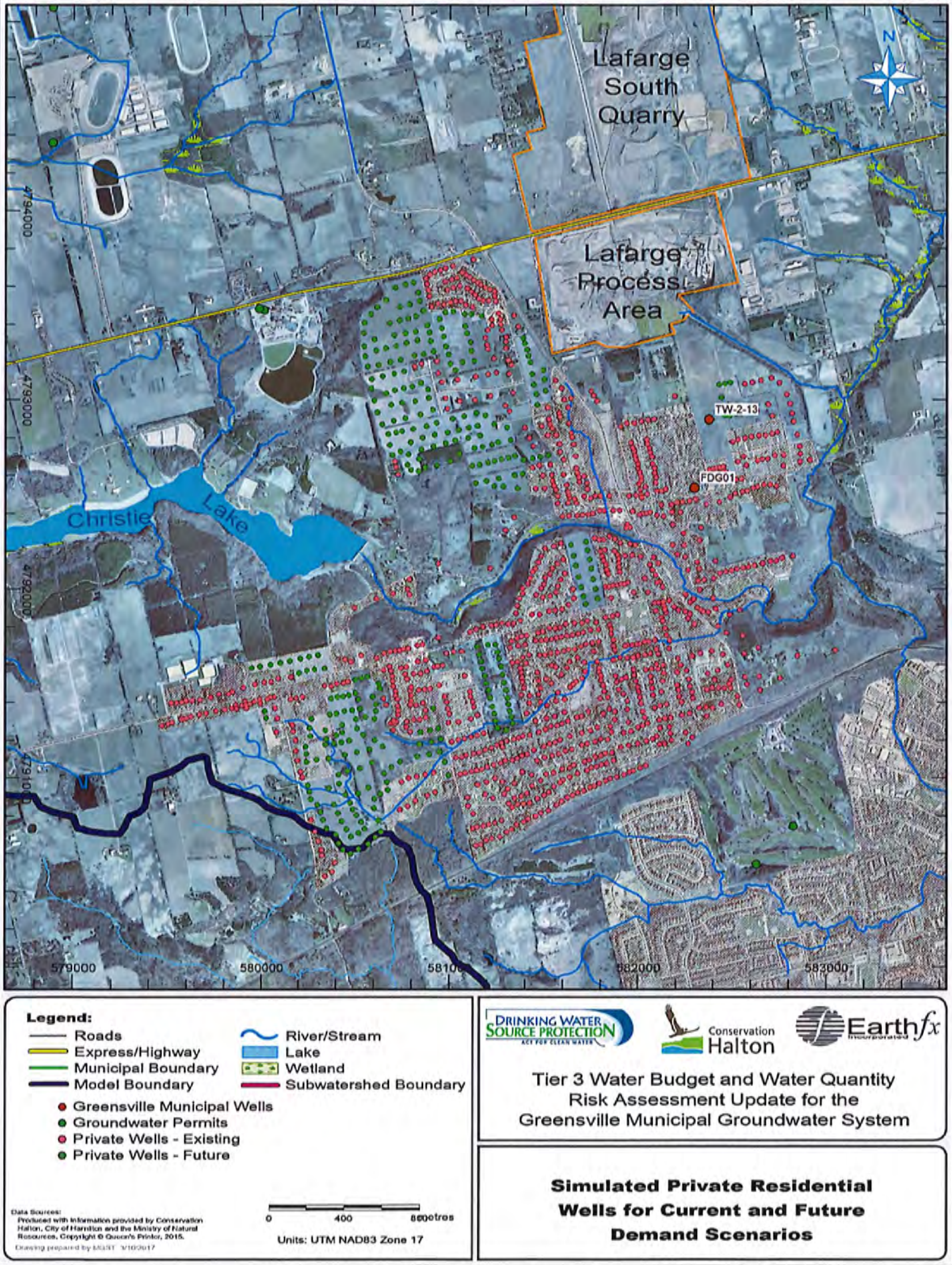


Figure 6.10: Locations of private residential wells simulated in the current and future demand scenarios.

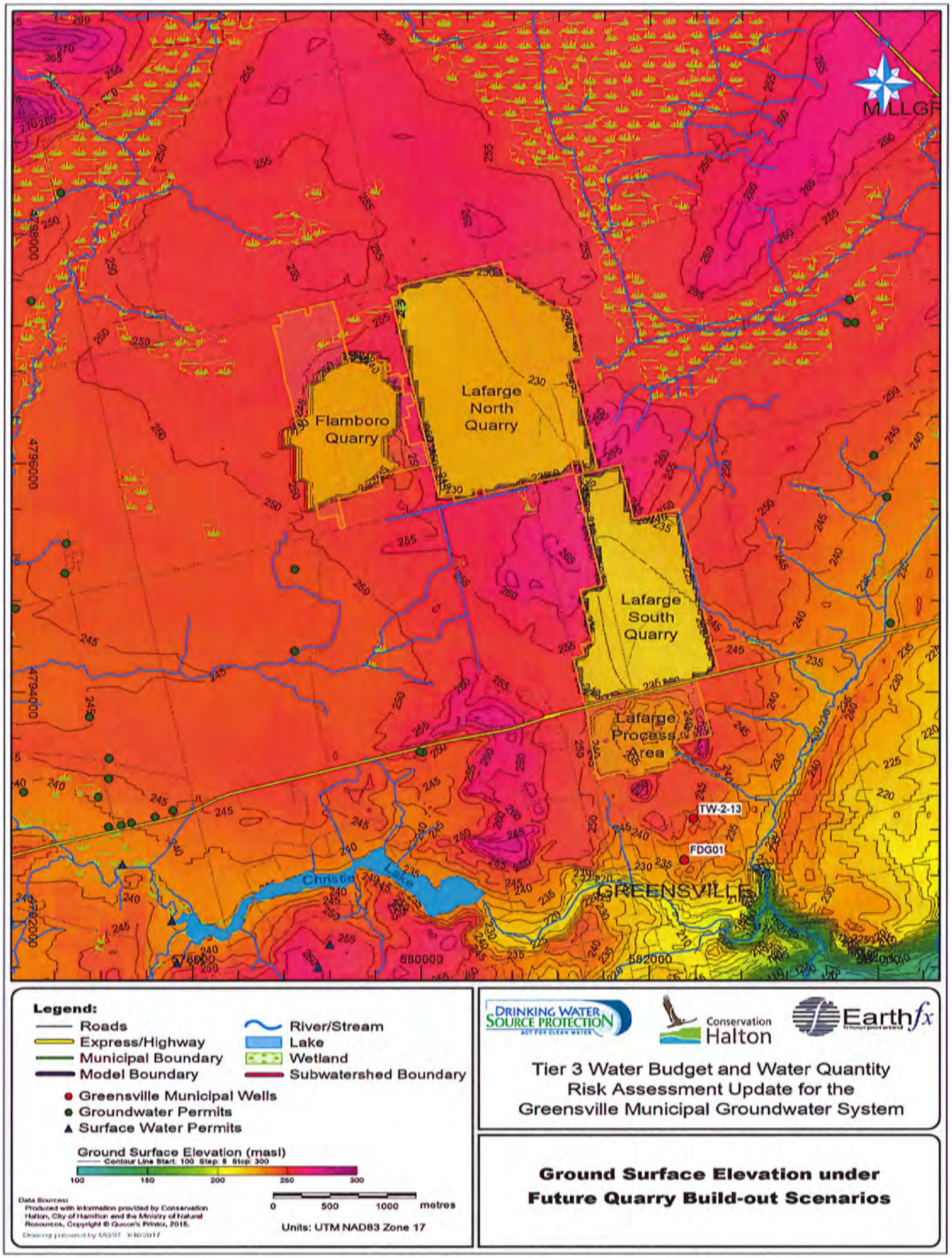


Figure 6.11: Projected extents of quarry excavations at full build-out.

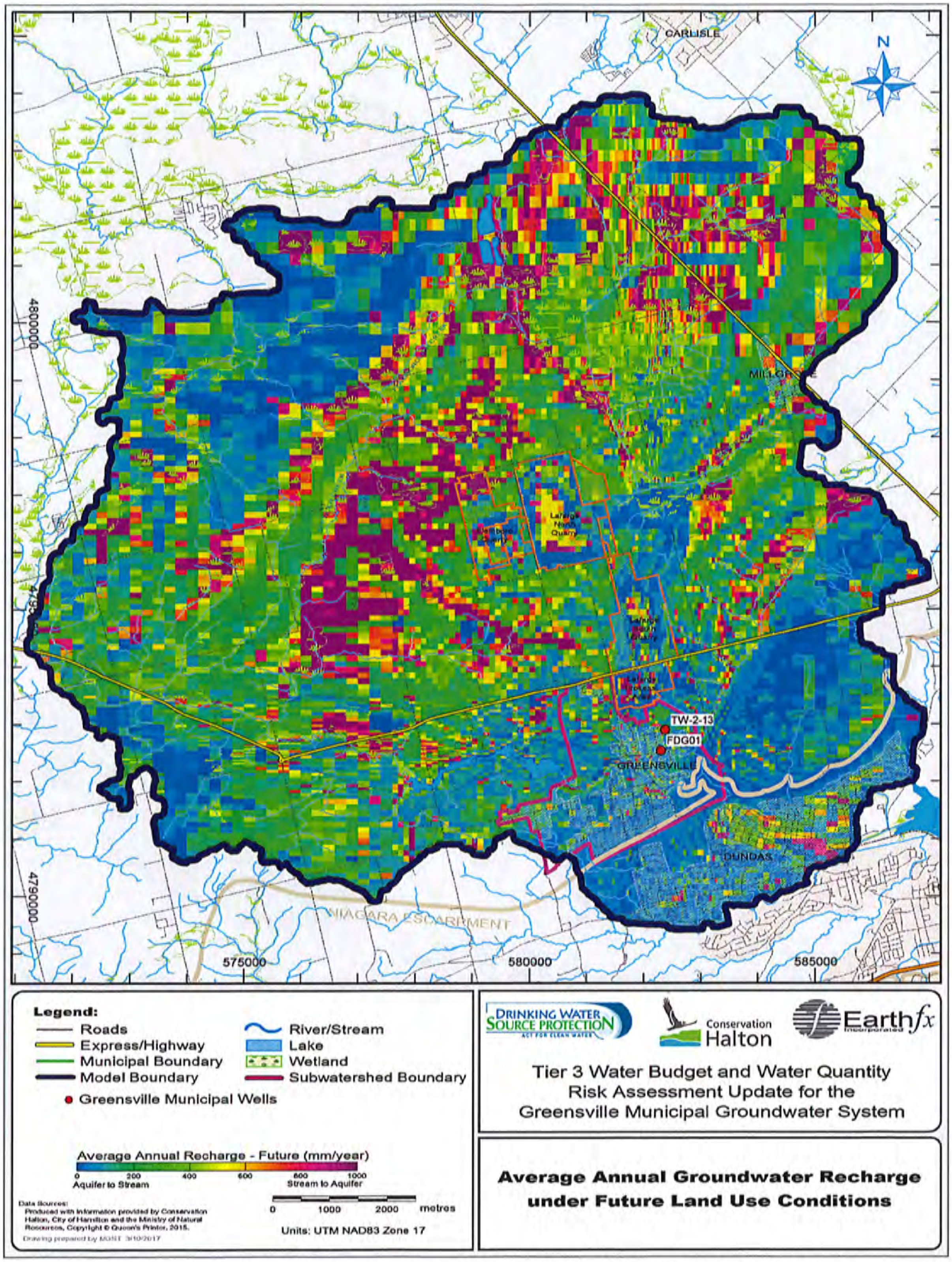


Figure 6.12: Average annual groundwater recharge with future land use change.

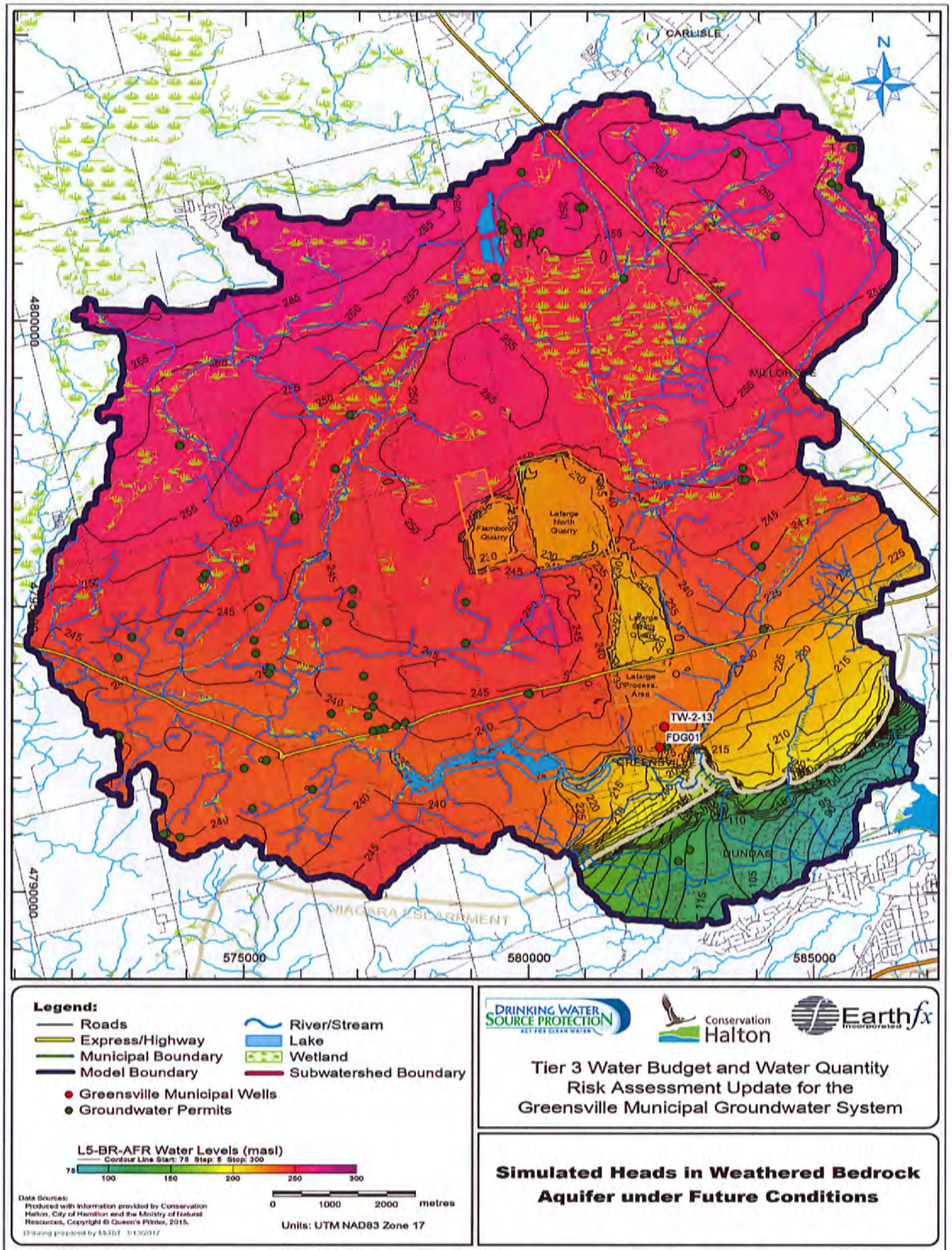


Figure 6.13: Simulated heads in the weathered bedrock contact aquifer under future conditions.

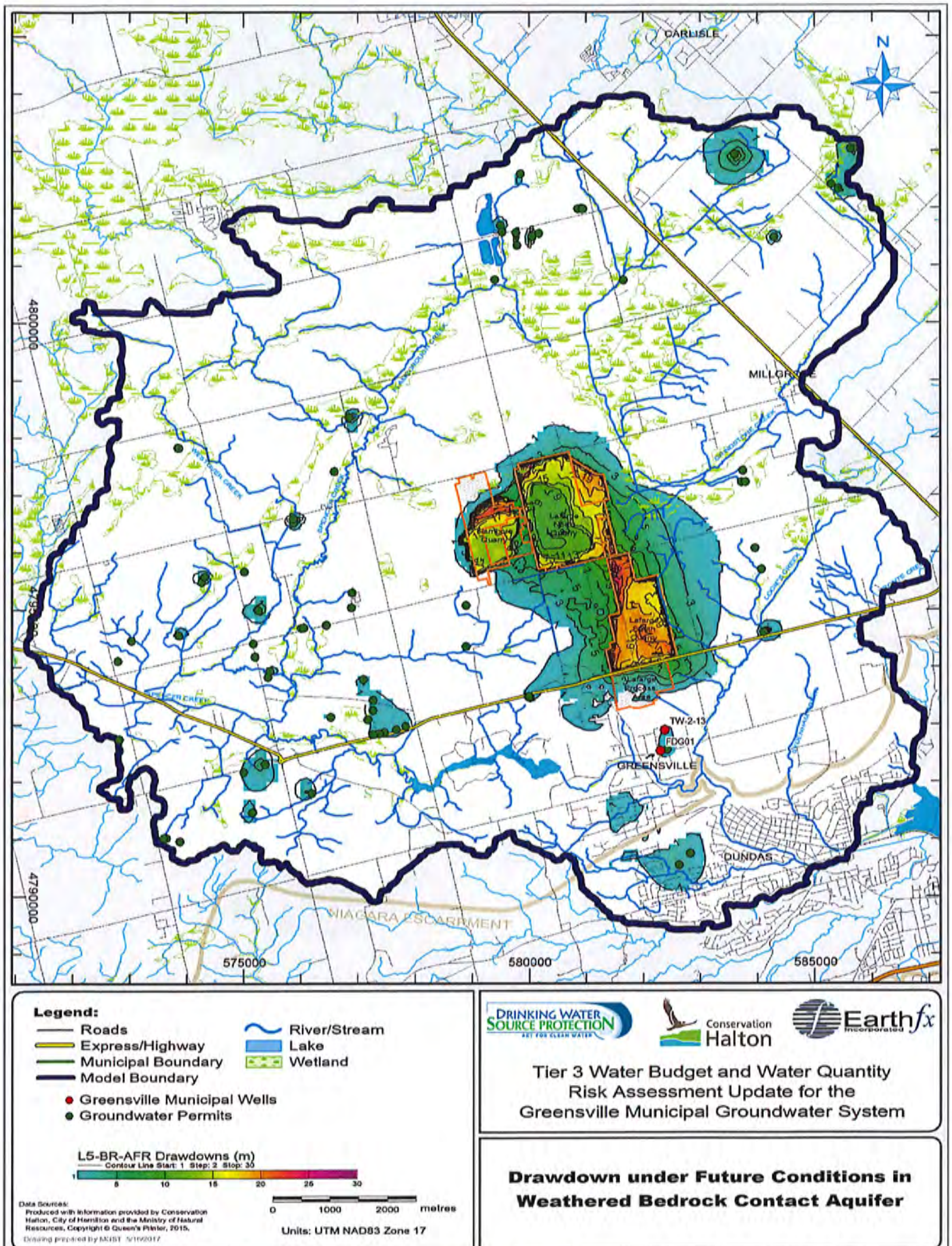


Figure 6.14: Drawdown in the weathered bedrock contact aquifer under future conditions.

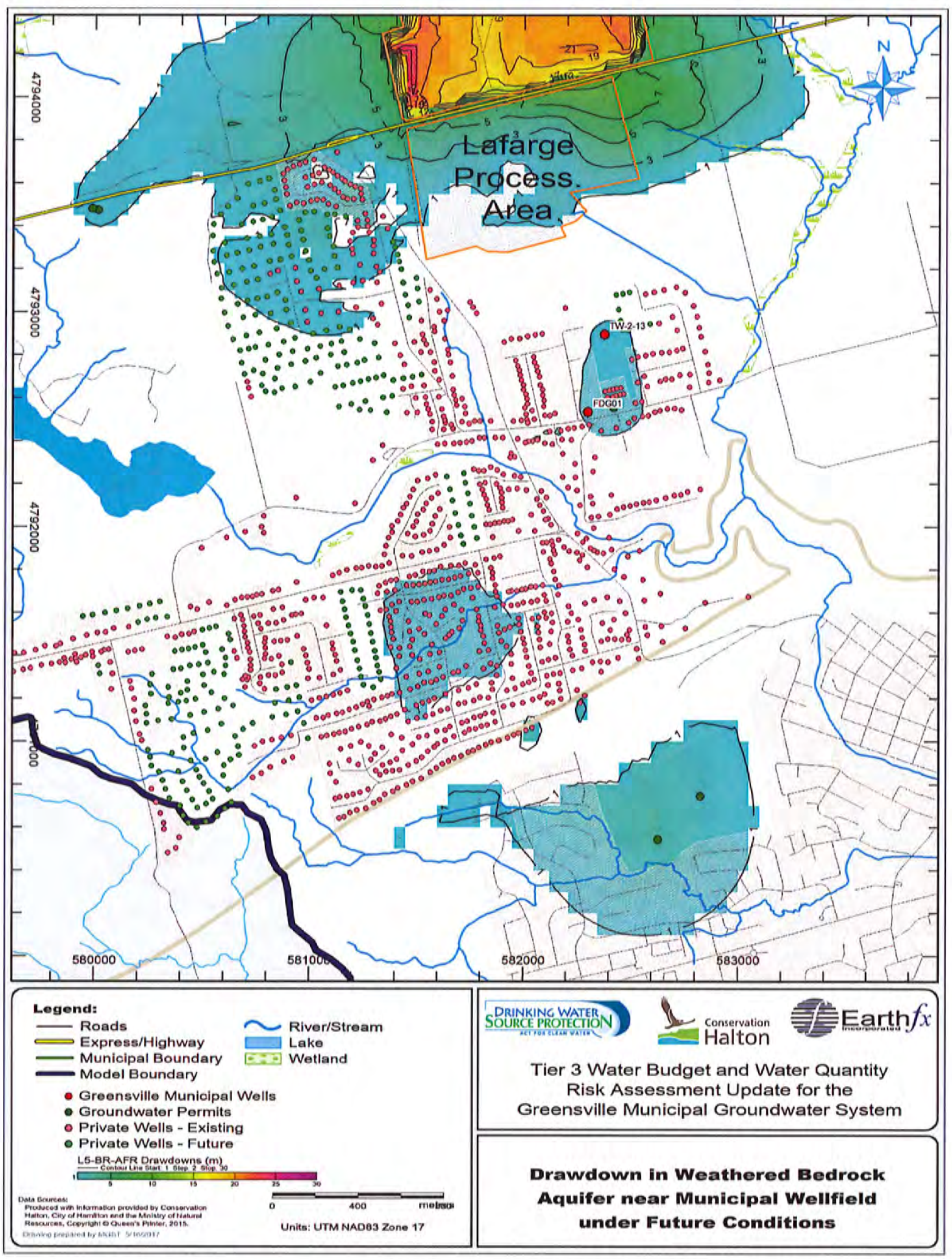


Figure 6.15: Simulated drawdowns in the weathered bedrock contact aquifer near the municipal wellfield under future conditions.

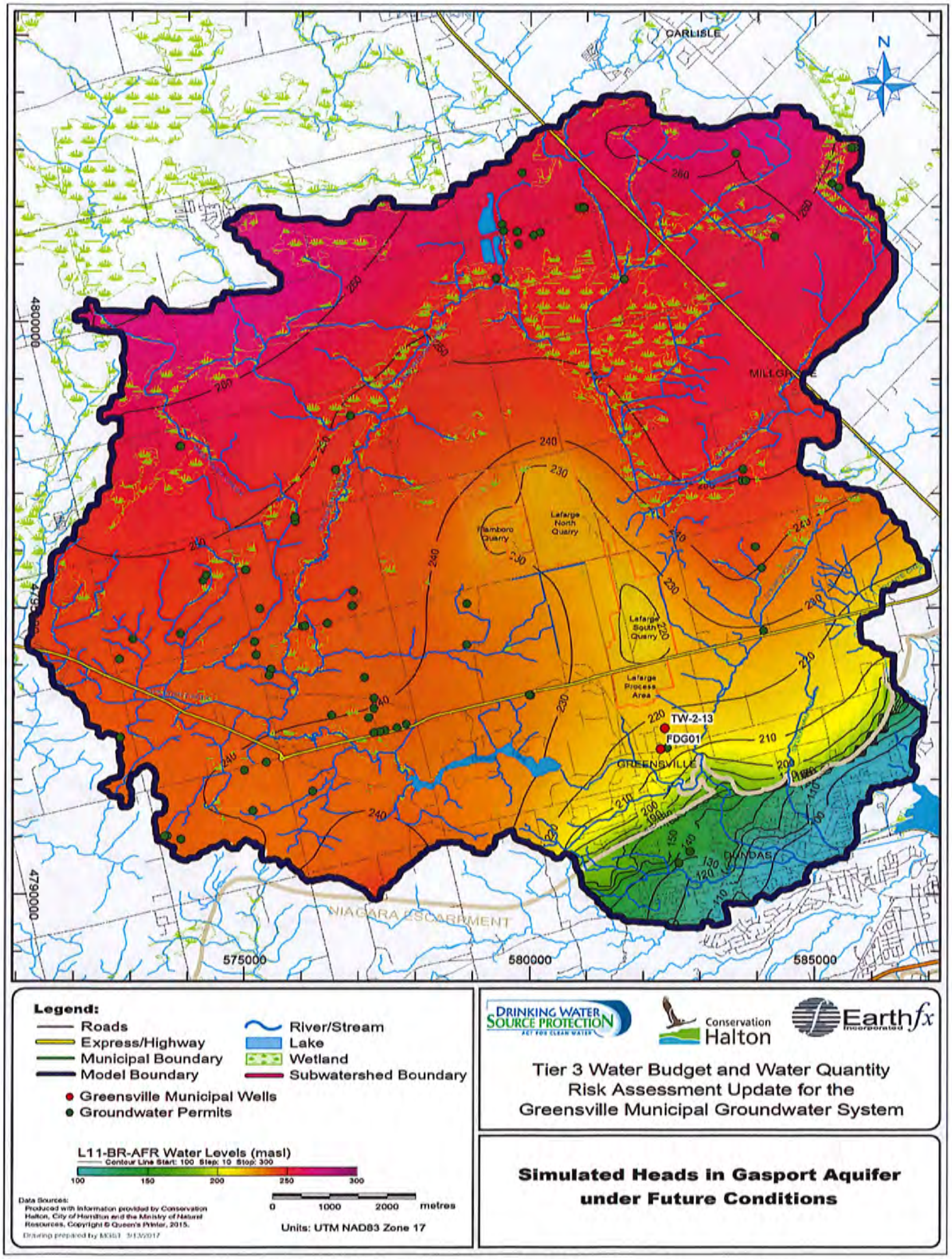


Figure 6.16: Simulated heads in the Gasport aquifer under future conditions.

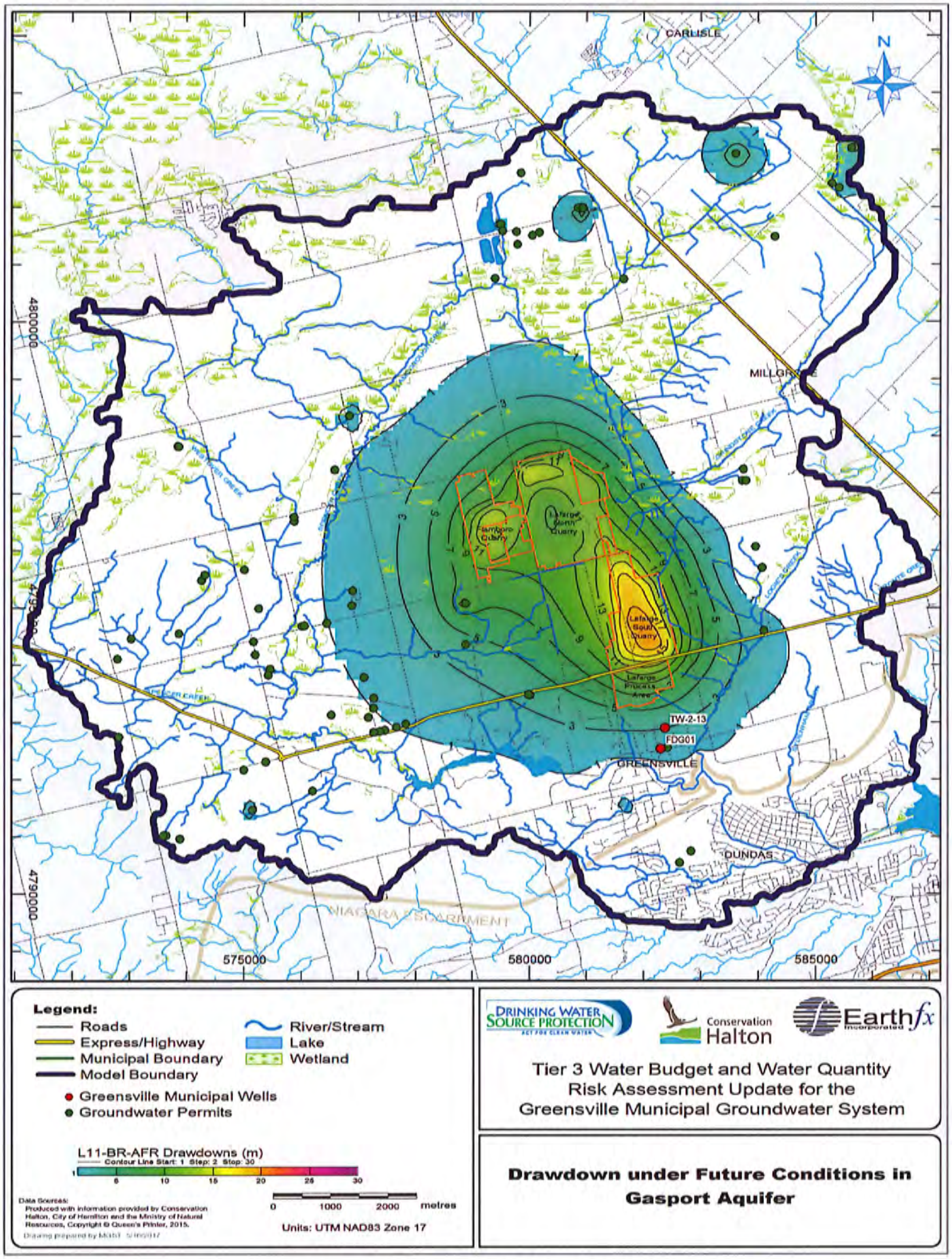


Figure 6.17: Drawdown in Gasport aquifer under future conditions.

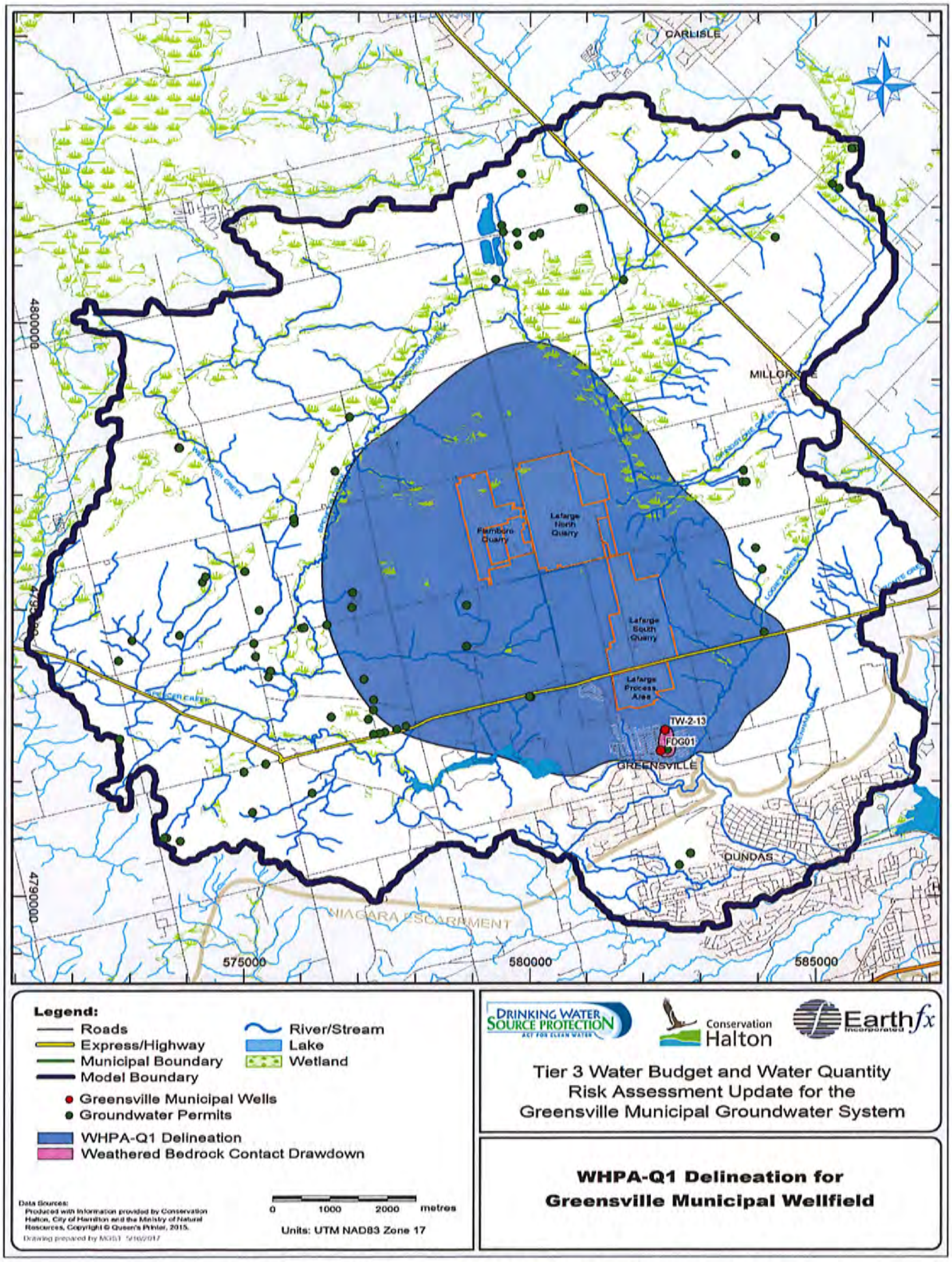


Figure 6.18: WHPA-Q1 delineation for Greensville municipal wellfield.

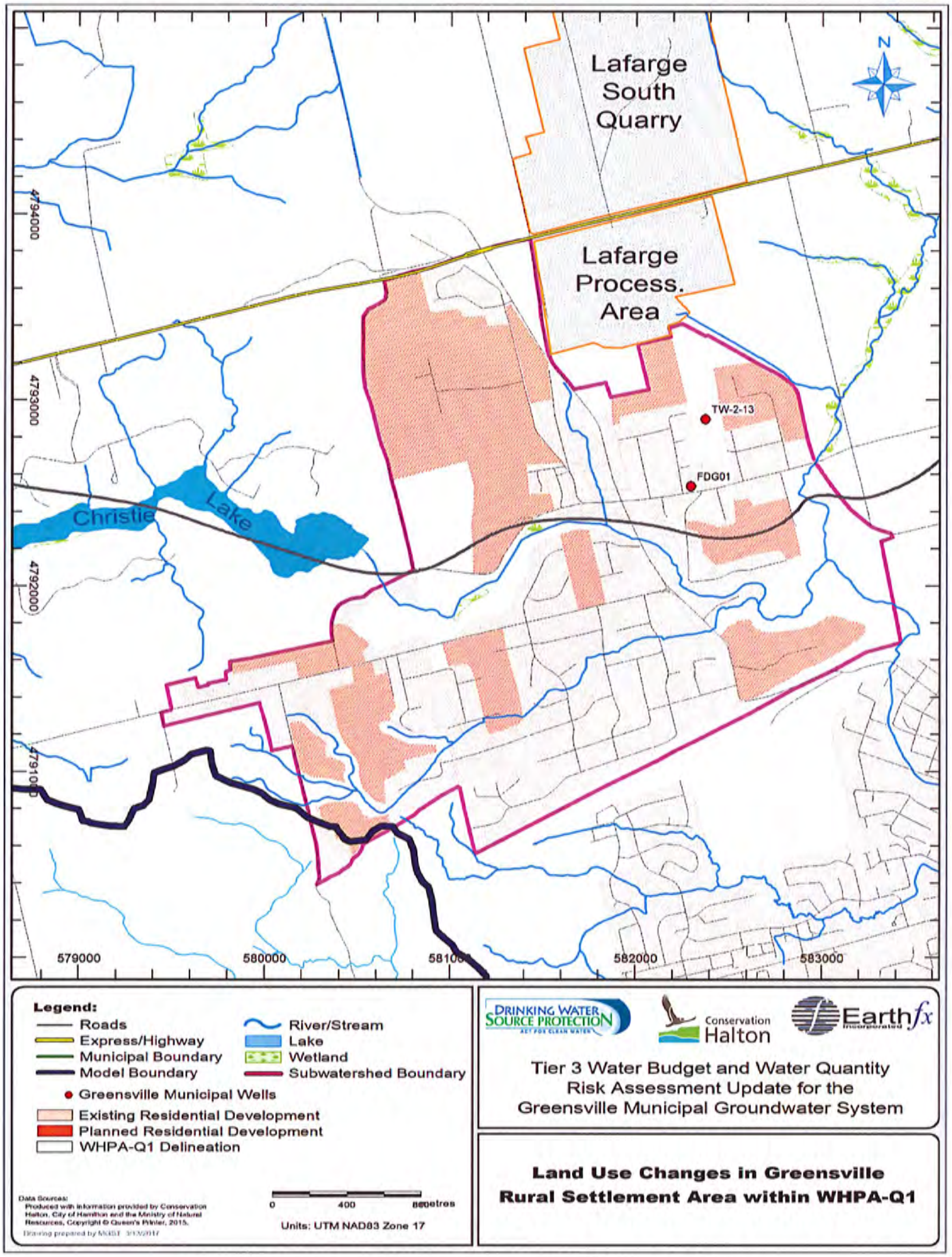


Figure 6.19: Future land use changes within the WHPA-Q1 area.

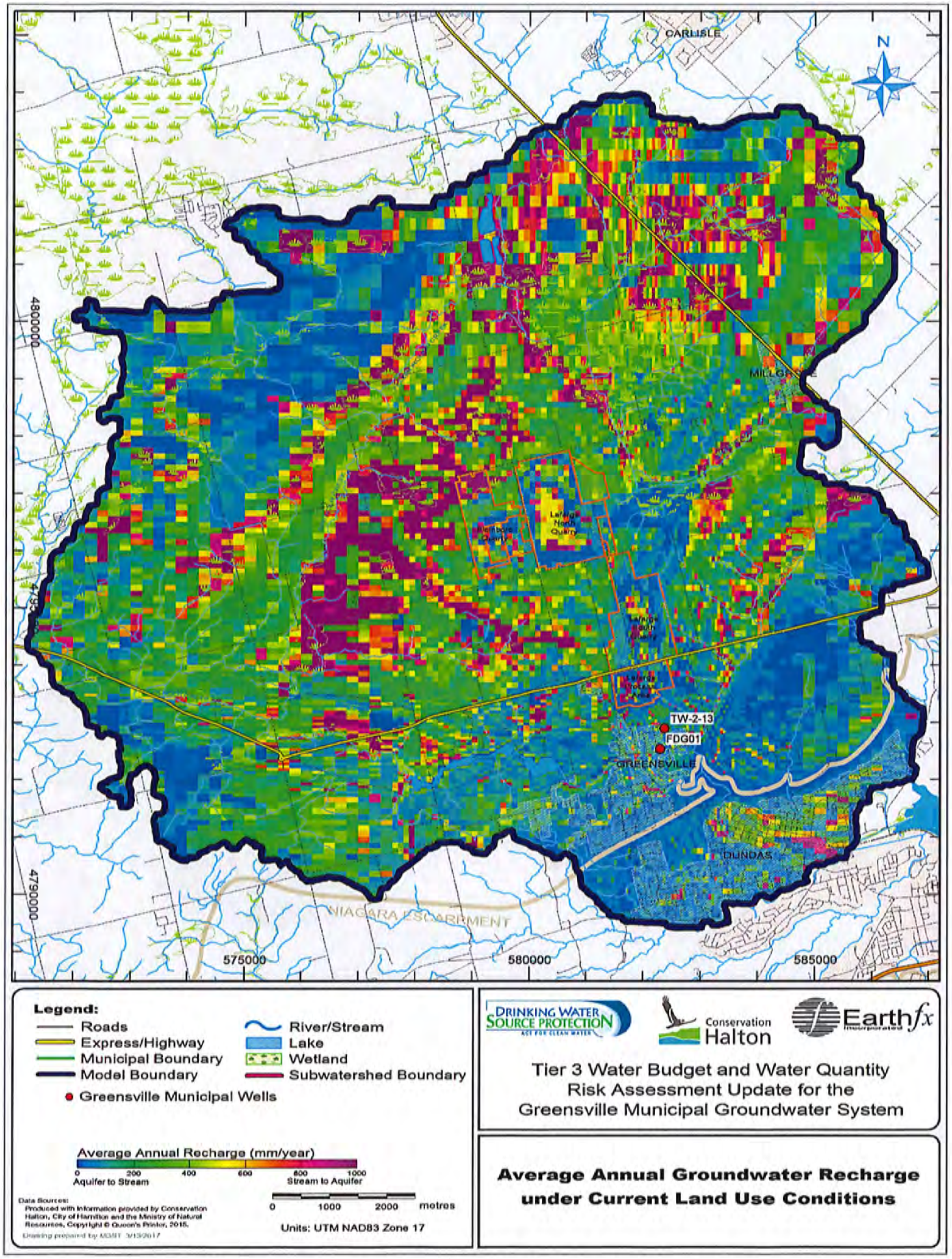


Figure 6.20: Average annual groundwater recharge under current land use conditions.

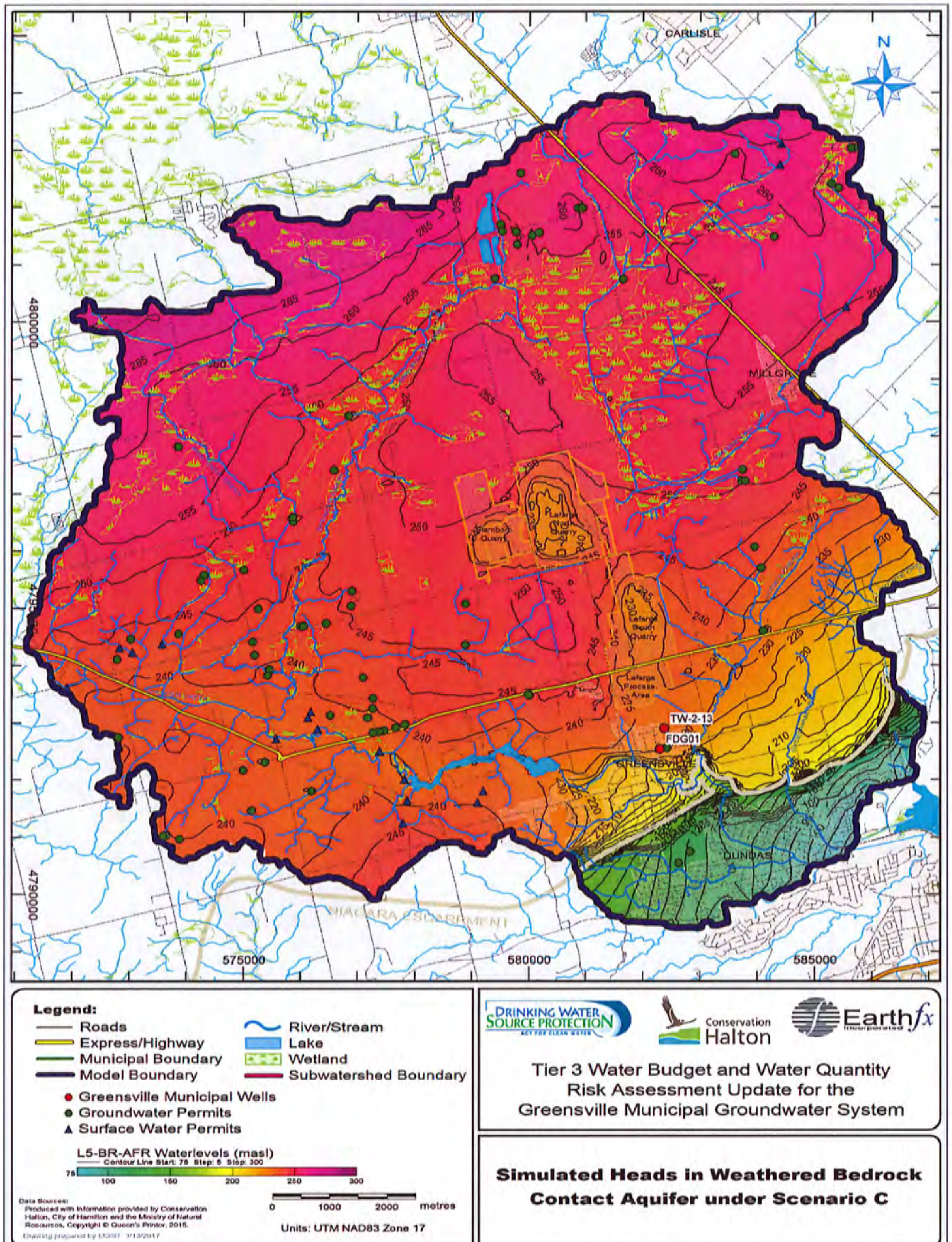


Figure 6.21: Simulated heads in weathered bedrock contact aquifer under current conditions (Scenario C).

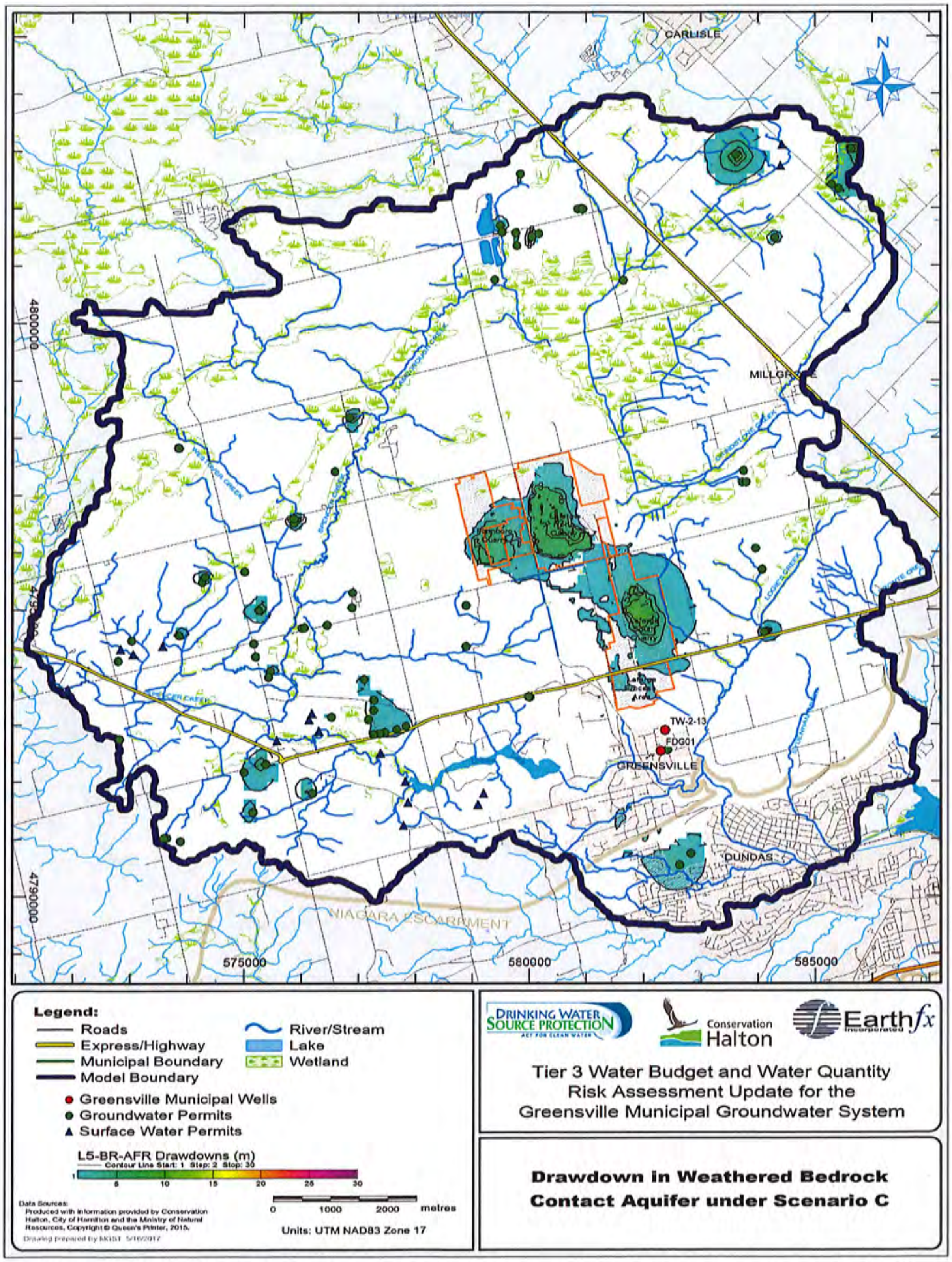


Figure 6.22: Simulated Drawdown in weathered bedrock contact aquifer between Scenario C and non-pumping scenario.

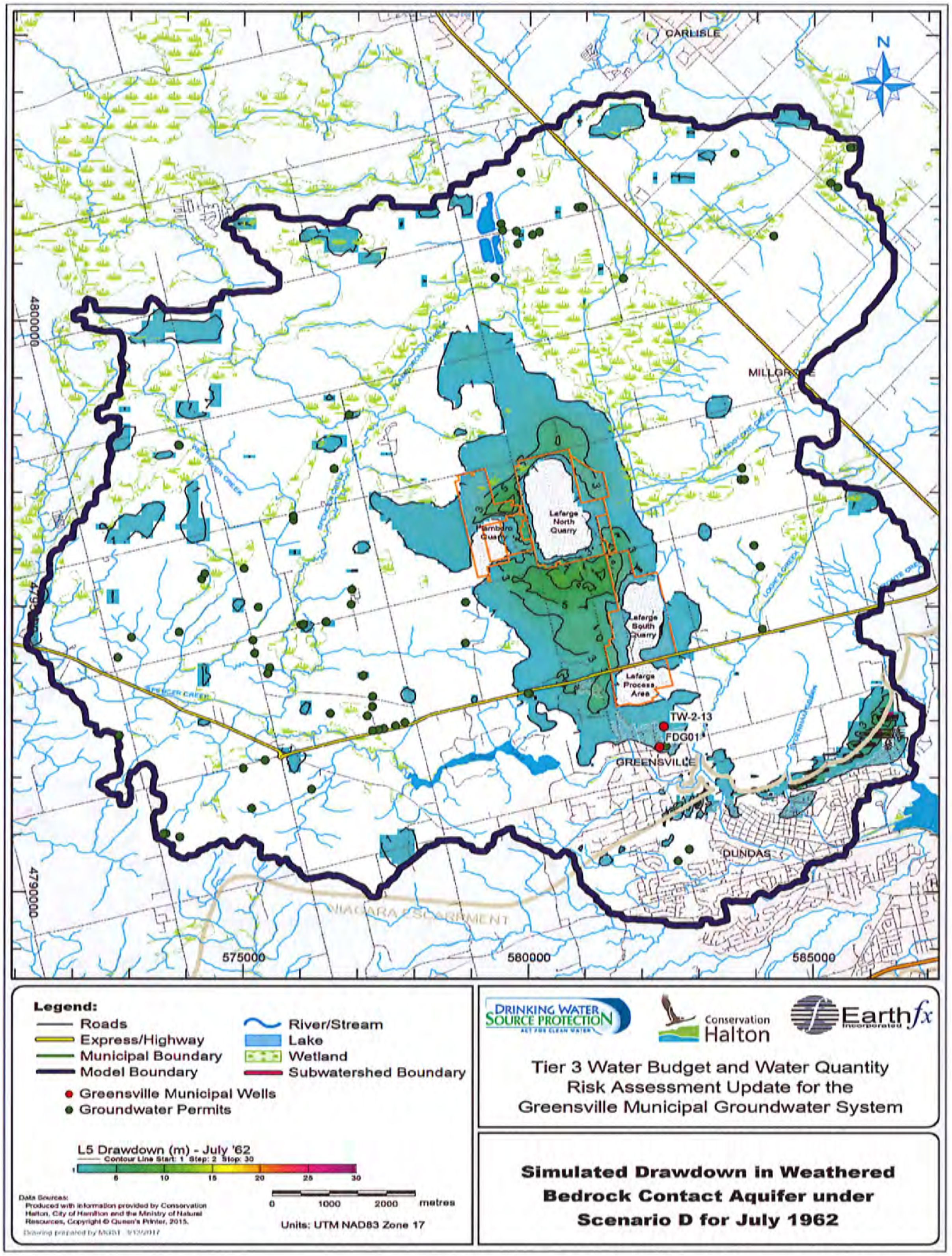


Figure 6.23: Change in head in weathered bedrock contact aquifer between pre-drought and July 1962 - Scenario D.

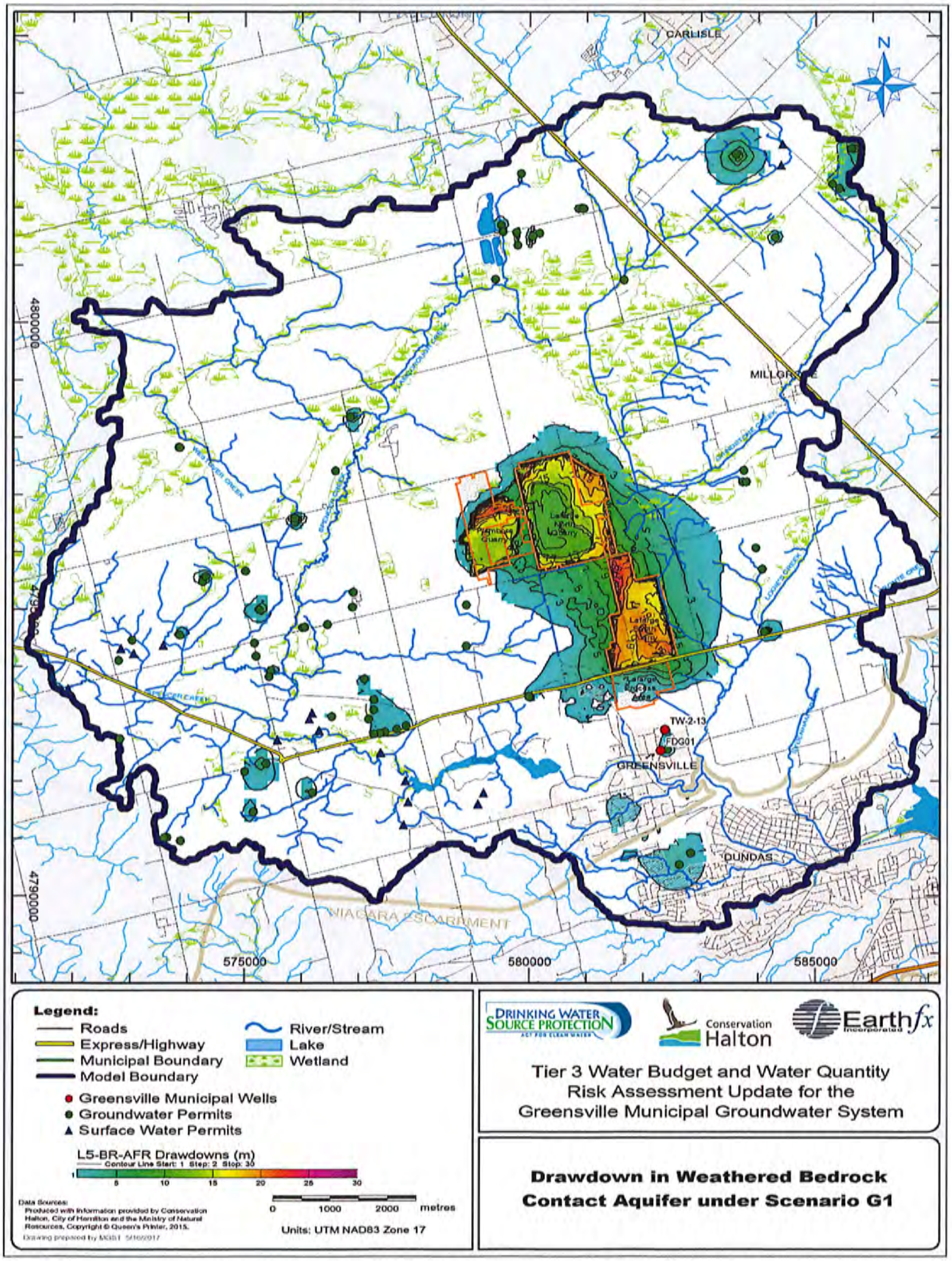


Figure 6.24: Simulated additional drawdown in weathered bedrock contact aquifer under Scenario G(1).

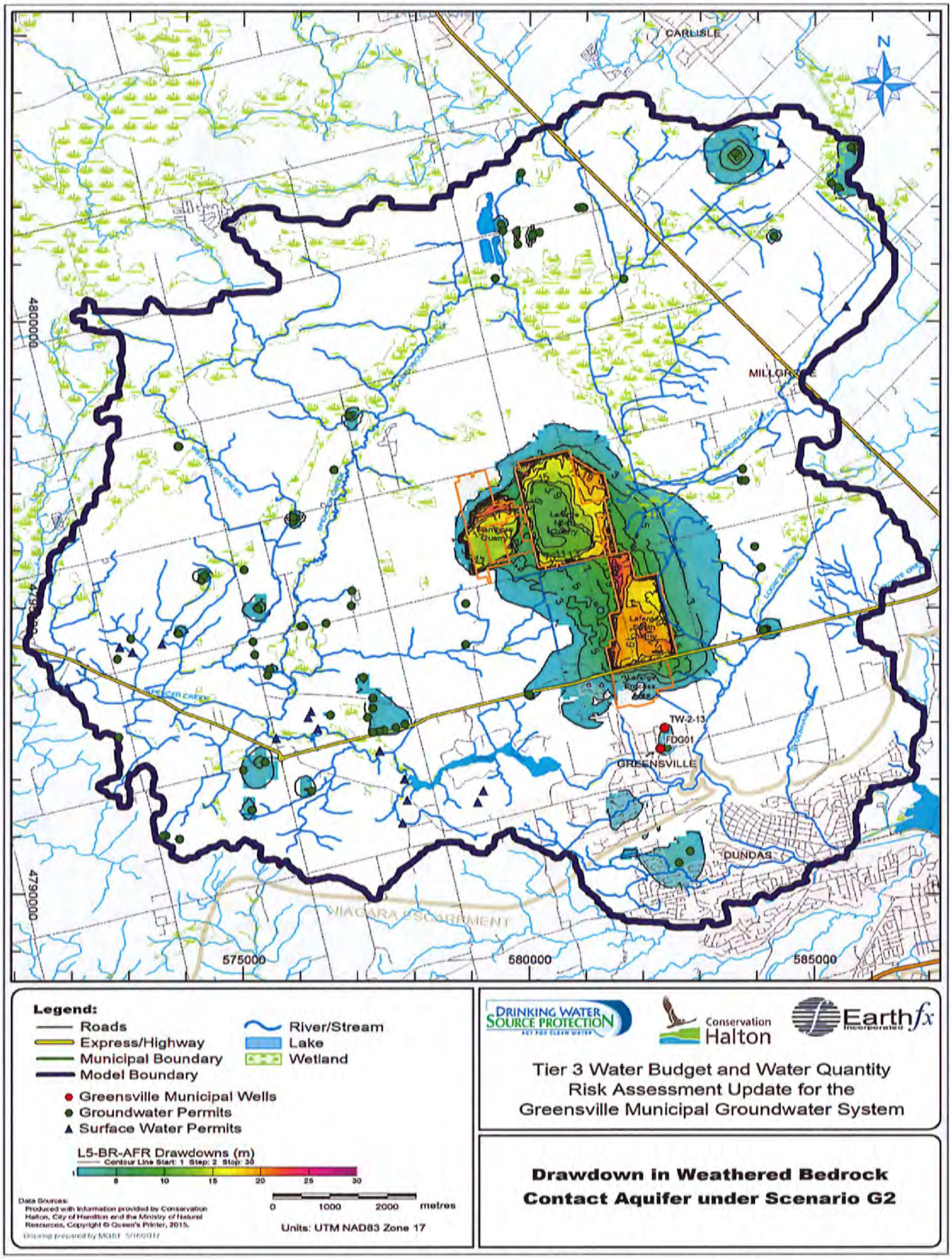


Figure 6.25: Simulated additional drawdown in weathered bedrock contact aquifer under Scenario G(2).

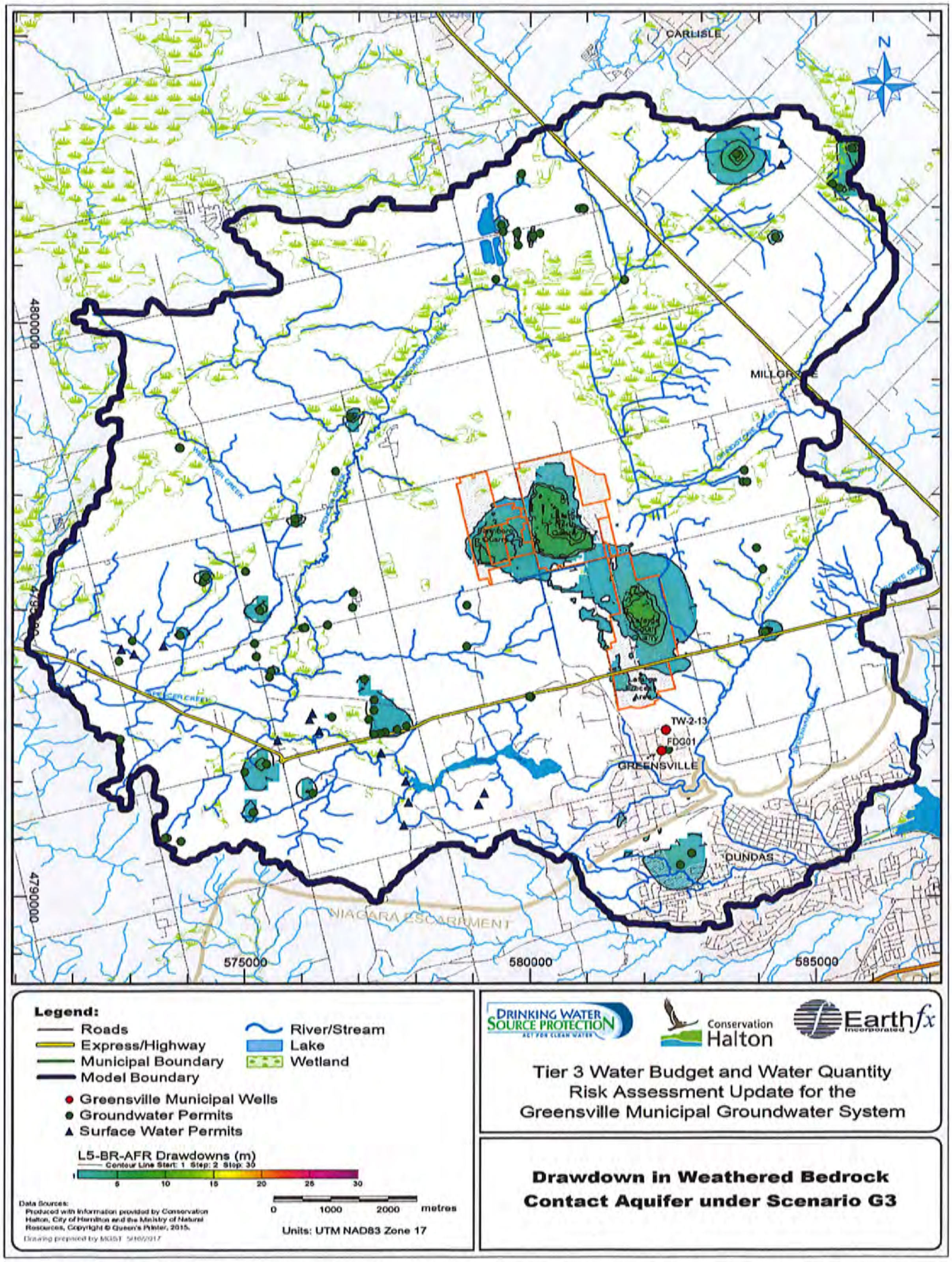


Figure 6.26: Simulated additional drawdown in weathered bedrock contact aquifer under Scenario G(3).

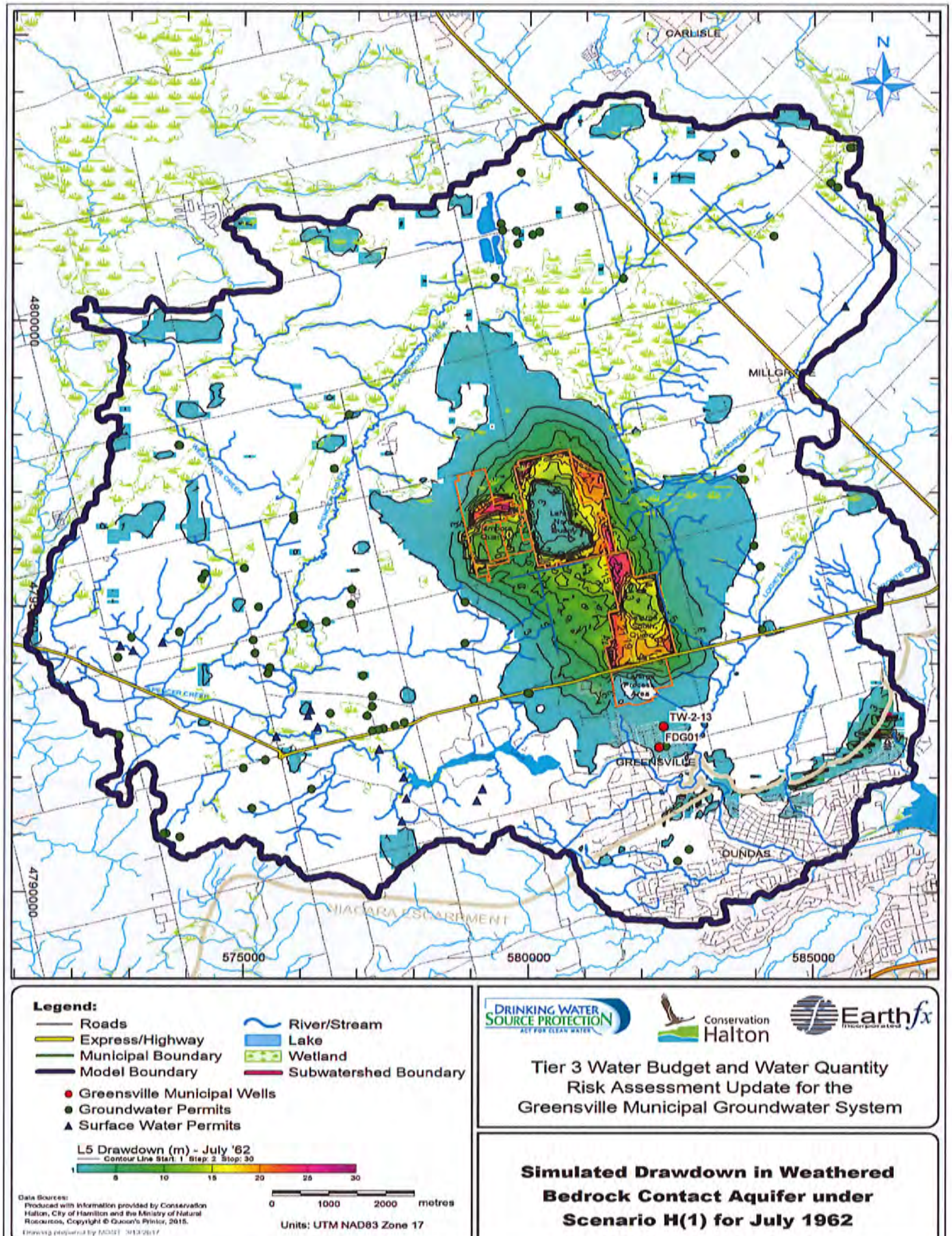


Figure 6.27: Change in head in weathered bedrock contact aquifer between pre-drought and July 1962 – Scenario H(1).

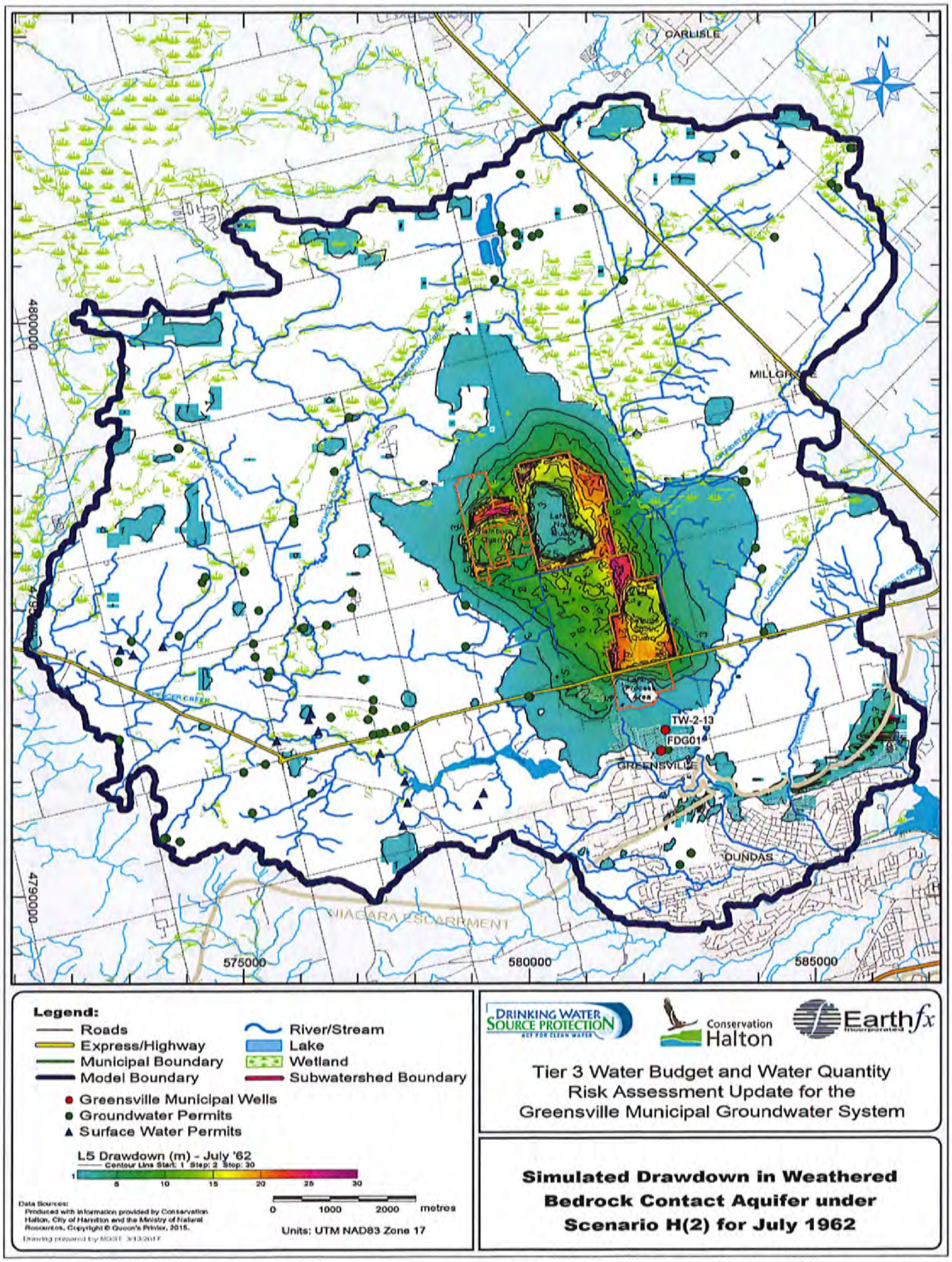


Figure 6.28: Change in head in weathered bedrock contact aquifer between pre-drought and July 1962 – Scenario H(2).

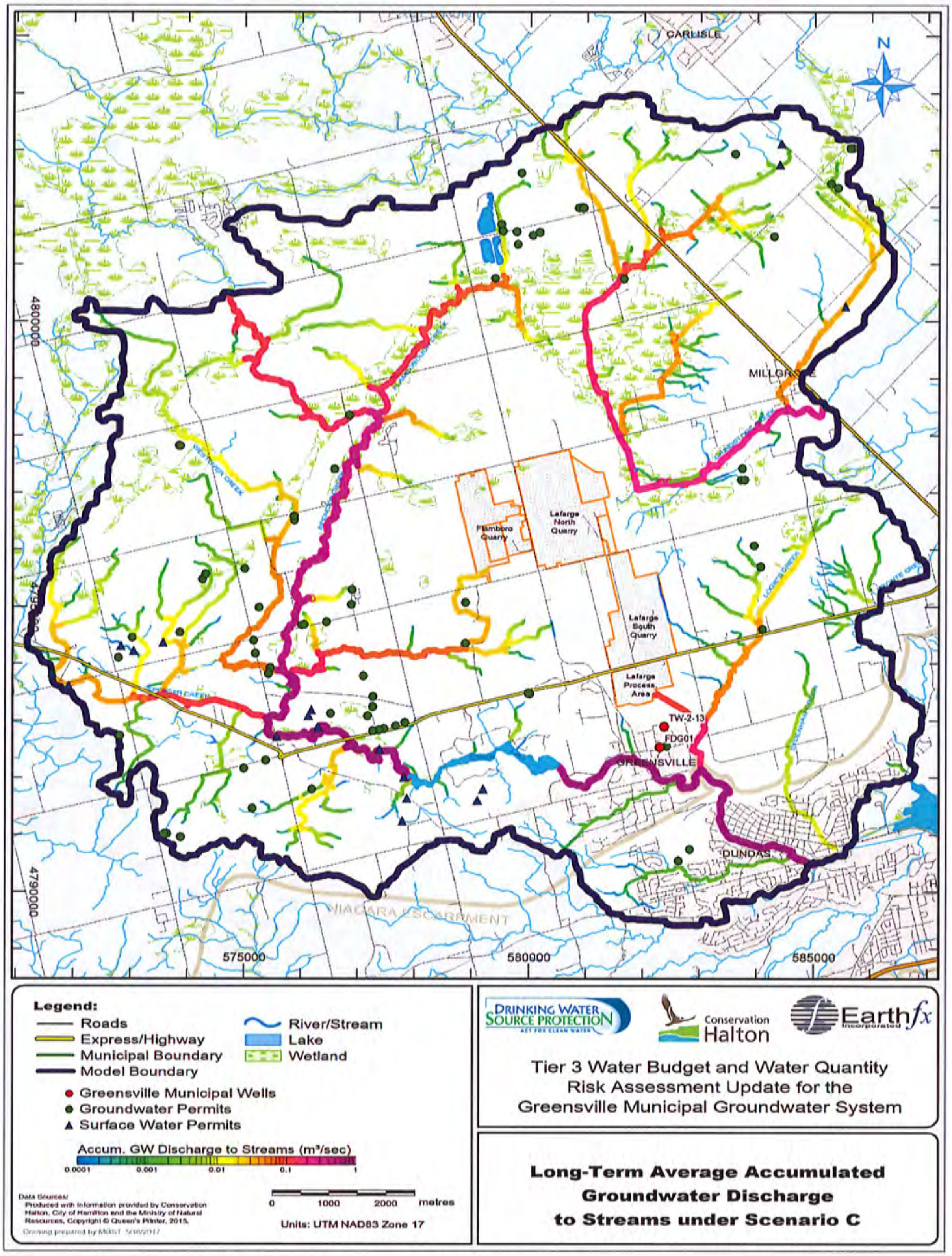


Figure 6.29: Simulated accumulated groundwater discharge to streams (baseflow) under Scenario C.

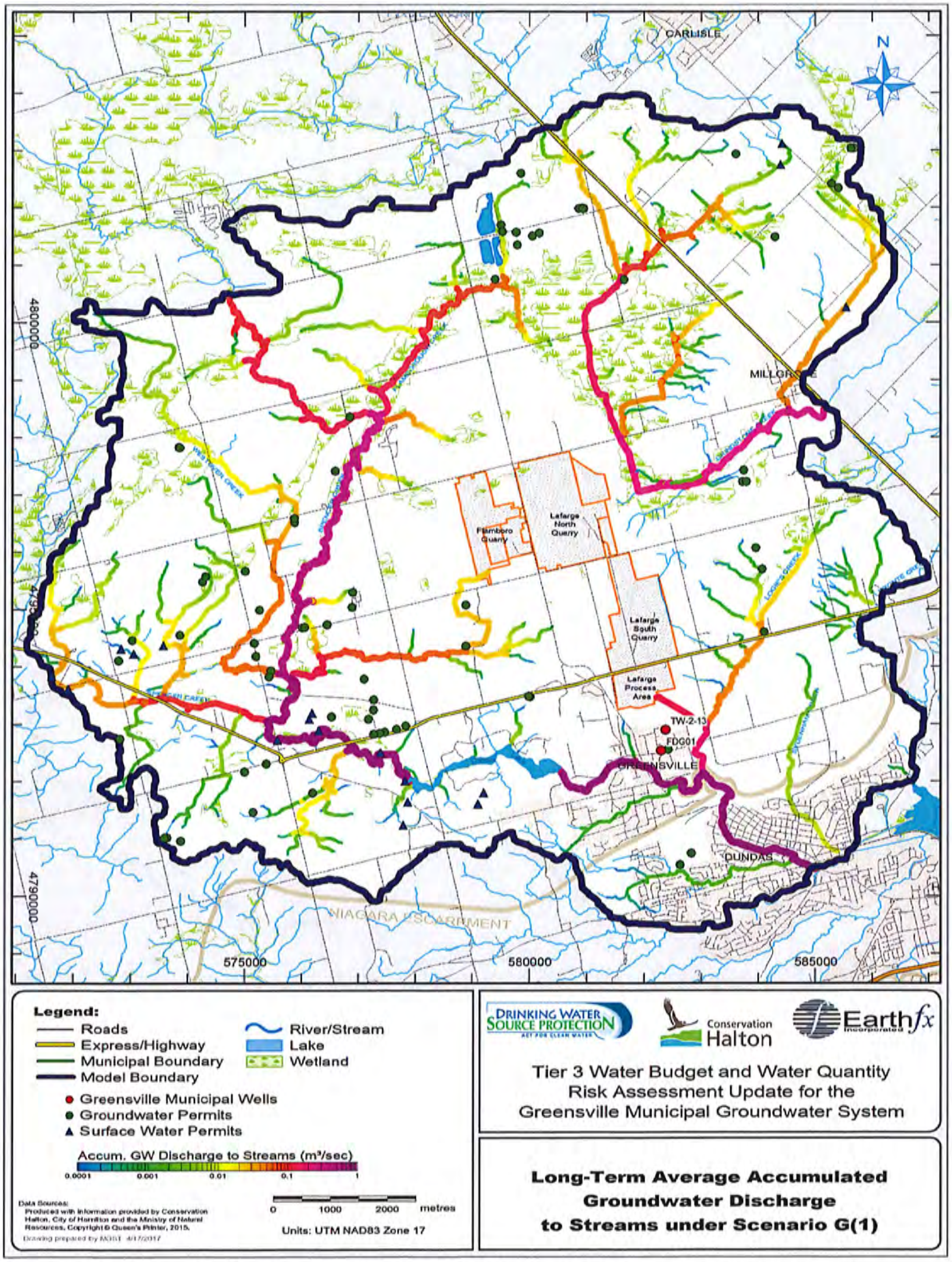


Figure 6.30: Simulated accumulated groundwater discharge to streams (baseflow) under Scenario G(1).

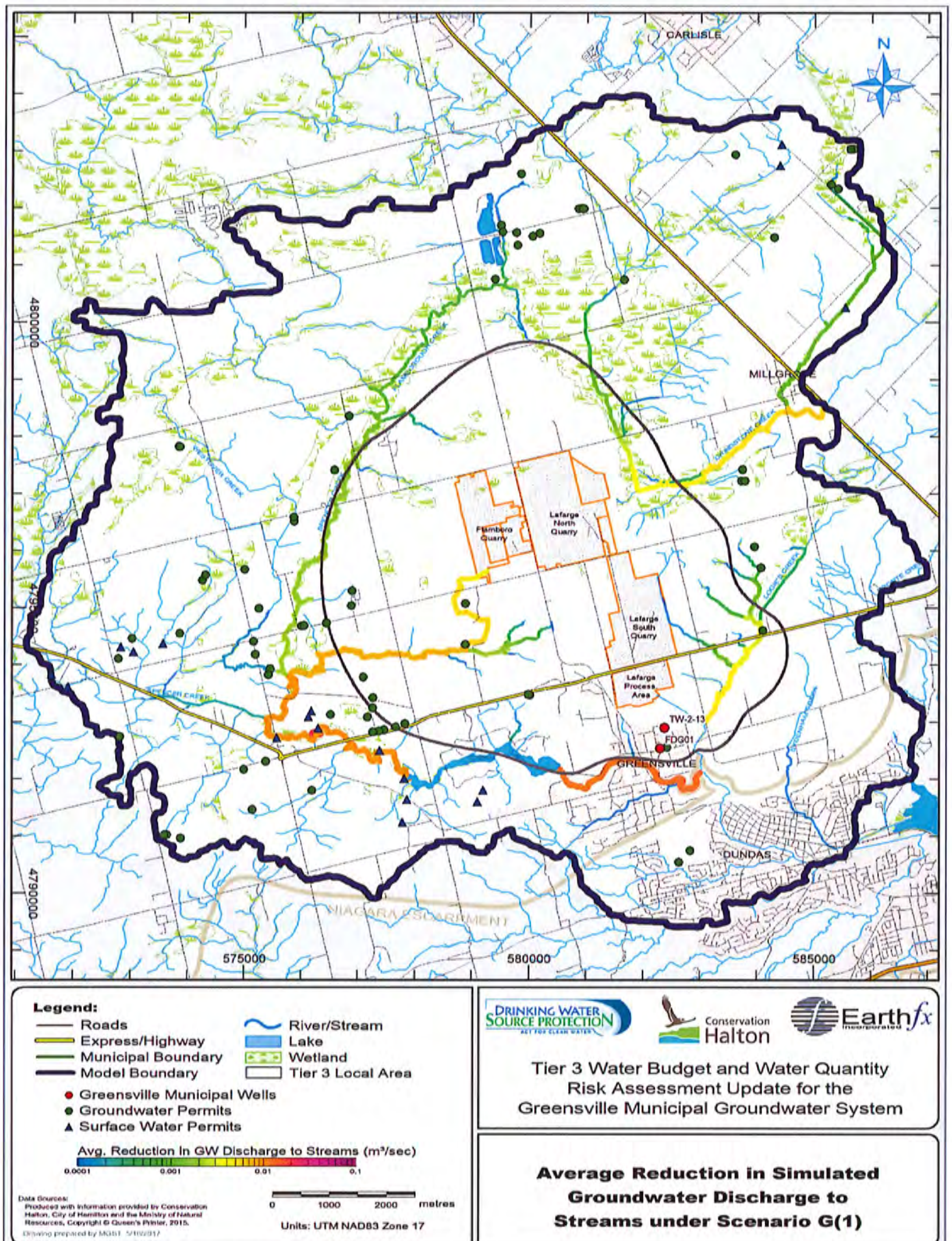


Figure 6.31: Reduction in simulated groundwater discharge to streams between Scenario G(1) and Scenario C.

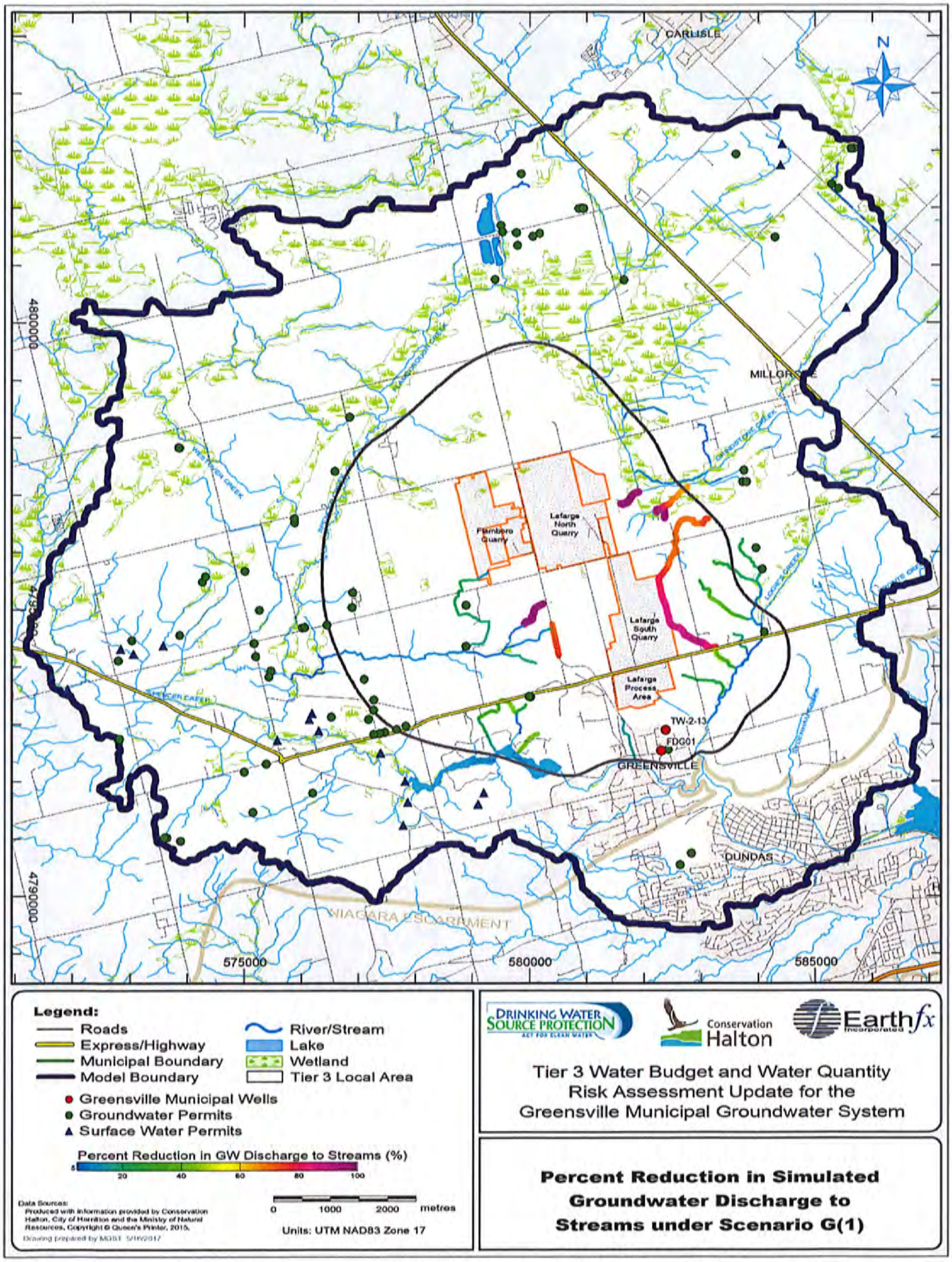


Figure 6.32: Percent reduction in simulated groundwater discharge to streams between Scenario G(1) and Scenario C.

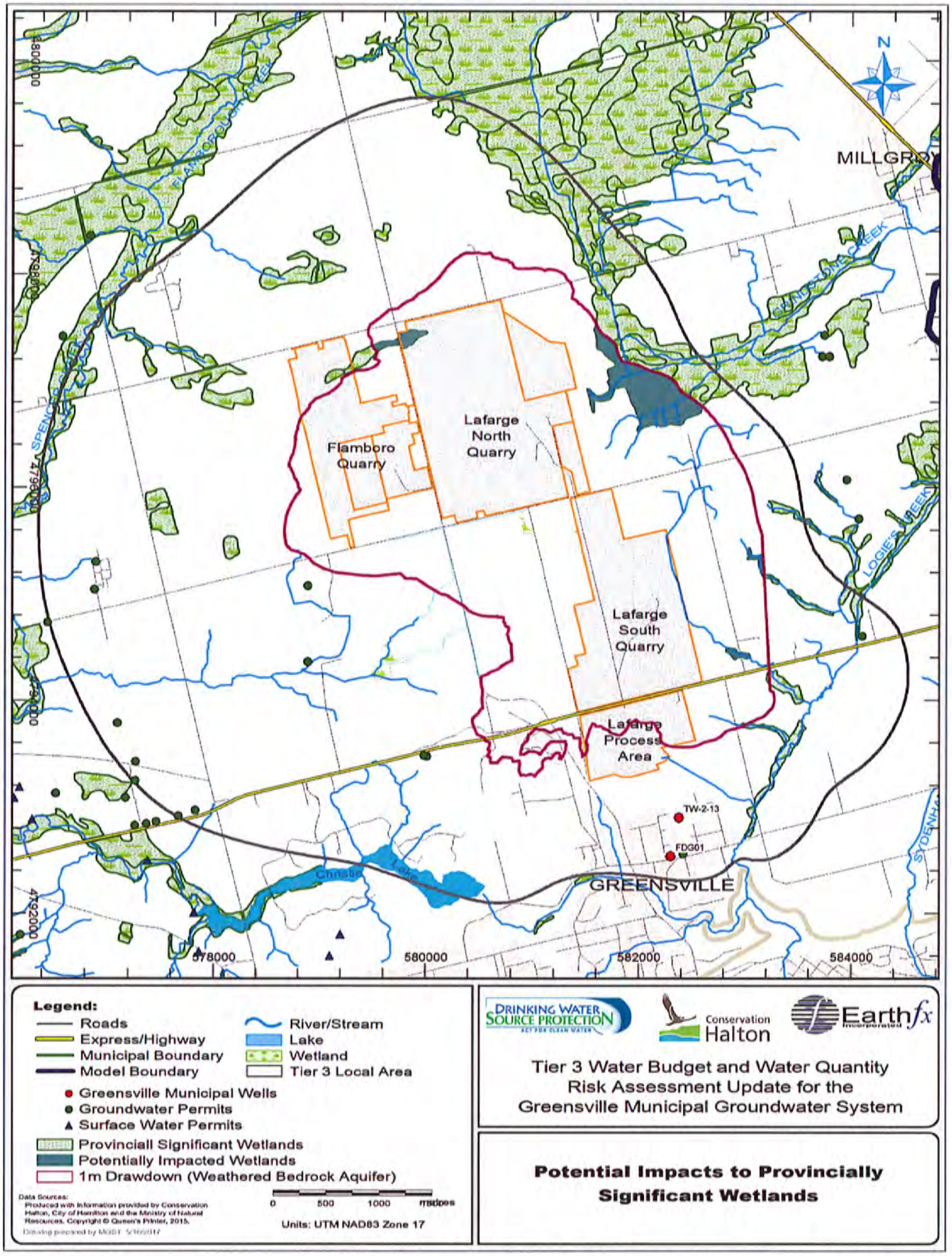


Figure 6.33: Potentially impacted PSWs based on simulated drawdowns in weathered bedrock aquifer.

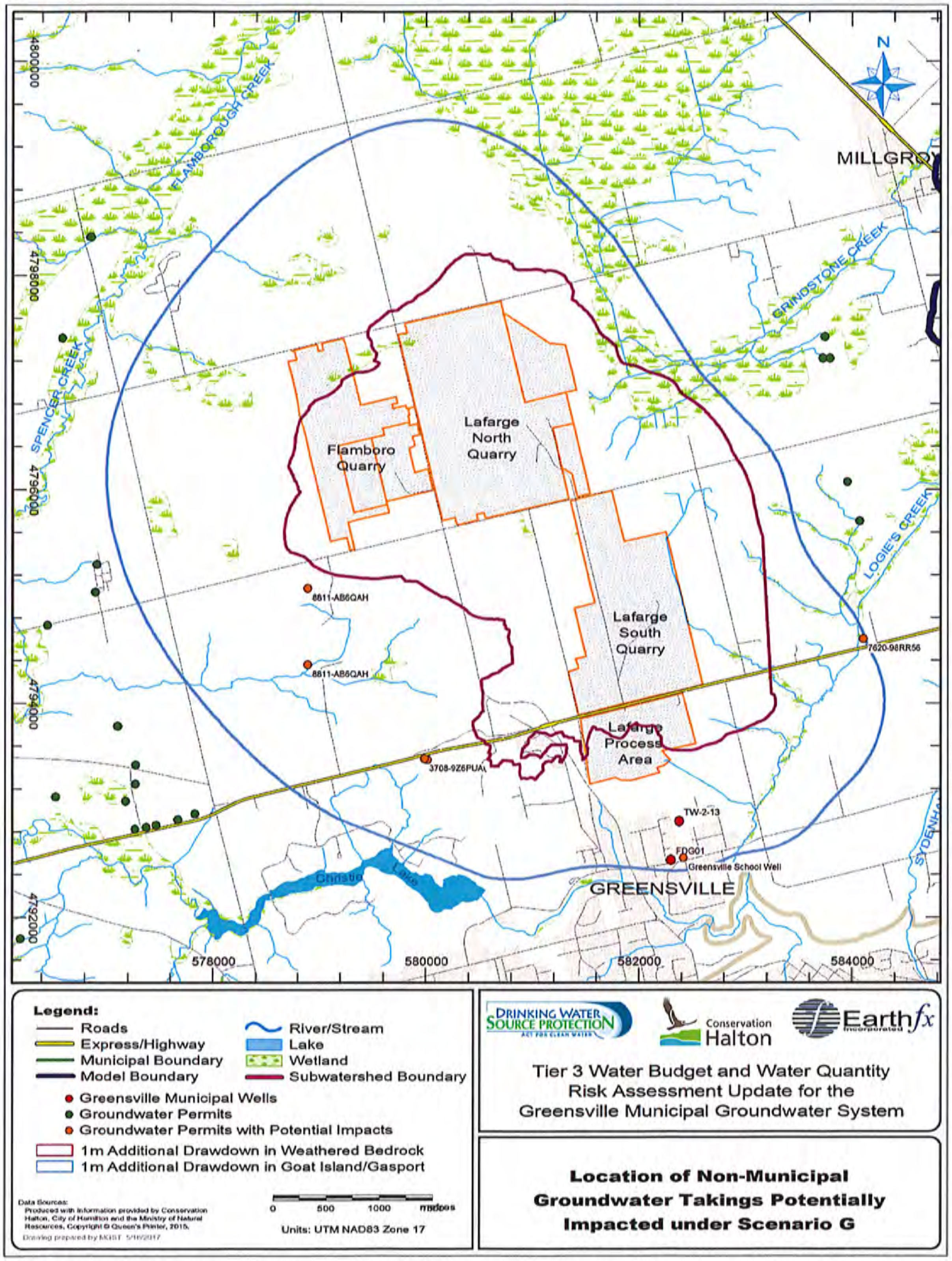


Figure 6.34: Location of non-municipal groundwater takings potentially impacted under Scenario G.

7 Significant Groundwater Recharge Areas

7.1 *Introduction*

Groundwater recharge is defined as the net volume of precipitation that reaches the water table. Note that the term *infiltration* is used to describe the volume of water that enters the soil zone, and *percolation* is defined as the water that passes through the base of the soil zone into the unsaturated zone. Infiltration, percolation and groundwater recharge are different because of the wide range of processes and parameters including, but not limited to, soil characteristics (e.g., hydraulic conductivity, porosity, field capacity, and wilting point), percent impervious cover, vegetation cover type and cover density, evaporative demand (both from the soil zone and the unsaturated zone), local topography, and depth to water table.

To protect sources of high groundwater recharge, the Technical Rules (MOE, 2009) require that significant groundwater recharge areas (SGRAs) be delineated in every source protection area. The Technical Rules for the Assessment Report (MOE, 2009) sets out two alternate methods for delineating SGRAs as follows:

> 44(1): *the area annually recharges water to the underlying aquifer at a rate that is greater than the rate of recharge across the whole of the related groundwater recharge area by a factor of 1.15 or more; or*
>
> *44(2): an area is a significant groundwater recharge area if the area annually recharges a volume of water to the underlying aquifer that is 55% or more of the volume determined by subtracting the annual evapotranspiration for the whole of the related groundwater recharge area from the annual precipitation for the whole of the related groundwater recharge area.*
>
> *45: Despite Rule 44, an area shall not be delineated as a significant groundwater recharge area unless the area has a hydrological connection to a surface water body or aquifer that is a source of drinking water for a drinking water system.*
>
> *46: The areas described in Rule 44 shall be delineated using the models developed for the purposes of Part III of these rules* [i.e. the Tier 3 local area assessment] *and with consideration of the topography, surficial geology, and how land cover affects groundwater and surface water.*

For this assessment, SGRAs in the Middle Spencer Creek subwatershed were delineated using Technical Rule 44(1). The supplemental technical guide (AquaResource, 2012) breaks the method into three general tasks:

1) Determine the threshold for high recharge areas, using one of the methods described in Technical Rule 44;

2) Determine the linkage to drinking water systems, including both municipal water systems and private domestic wells; and

3) Applying professional judgment, which is an allowance for the appropriate modification of the delineated SGRAs, such as to reflect local knowledge or to facilitate policy development.

7.2 *Significant Groundwater Recharge Area Delineation Results*

The Greensville Tier 3 model is a distributed, cell-based model and so estimated infiltration, percolation and groundwater recharge are spatially variable. The PRMS soil zone submodel was developed using a uniform 50 m grid resolution that is different than the underlying variable cell-sized groundwater model.

The uniform high-resolution PRMS grid better simulates runoff and focused infiltration in swales and topographic depressions, and the resulting variation in percolation from the soil zone. Focussed infiltration can result in highly variable estimates of percolation.

Estimates of groundwater recharge were determined using the results of a long-term 25-year GSFLOW simulation as shown in Figure 7.1.

It is understood it will likely be difficult to develop workable policies for the management and protection of small, isolated SGRA zones. Two methods were used to generate a more manageable estimate of groundwater recharge. First, the long term average estimate from the 25 year GSFLOW run was resampled to a uniform 100 m grid, and second, an infilling and smoothing procedure was applied. This cell size was selected to provide SGRA zones no smaller than 1 ha, after applying infilling and smoothing procedures (discussed below).

As per AquaResource (2012), an infilling/smoothing procedure was applied to the raw 100m SGRA grid (Figure 7.2) to remove small holes in the larger contiguous SGRAs and to remove small isolated SGRA patches. The infilling/smoothing approach evaluated whether a given cell was bounded by another of the same classification (i.e., an SGRA or not an SGRA) on at least two of the four sides. In cases where less than two sides were shared with cells having the same classification, the hole/patch was considered to be isolated and removed from the SGRA delineation. This process was repeated until no additional holes/patches were found. The results of the SGRA delineation with infilling and clipping are presented in Figure 7.3. This revised map is considered a more practical and workable solution for planning purposes.

SGRAs are primarily found in the north-western portion of the study area, associated with thin drift and exposed bedrock of the Flamborough Plains. Recharge in these areas is expected to be high due to the limited aquitard cover material and short travel times down to the weathered bedrock aquifer. In addition, the Middle Spencer Creek subwatershed becomes increasingly more rural towards the upper portion, with less residential development. These factors result in the emphasis of the high recharge areas in the northern portion of the subwatershed.

According to Technical Rule 45, the areas identified as SGRAs must be hydrologically-connected to a surface water body or to an aquifer that is a source of drinking water. The Greensville municipal well is located within the Middle Spencer Creek subwatershed in the semi-confined weathered bedrock aquifer. This is the shallowest major aquifer unit in the subwatershed and therefore likely to receive much of the incoming groundwater recharge from the SGRAs. In some areas, overburden aquifers may be present above this weathered bedrock unit, however, they are considered to be discontinuous and are likely in good communication with the underlying weathered bedrock. The delineated SGRAs can therefore be considered to be hydrologically connected to the aquifer, fulfilling the requirements of Technical Rule 45.

7.3 Figures

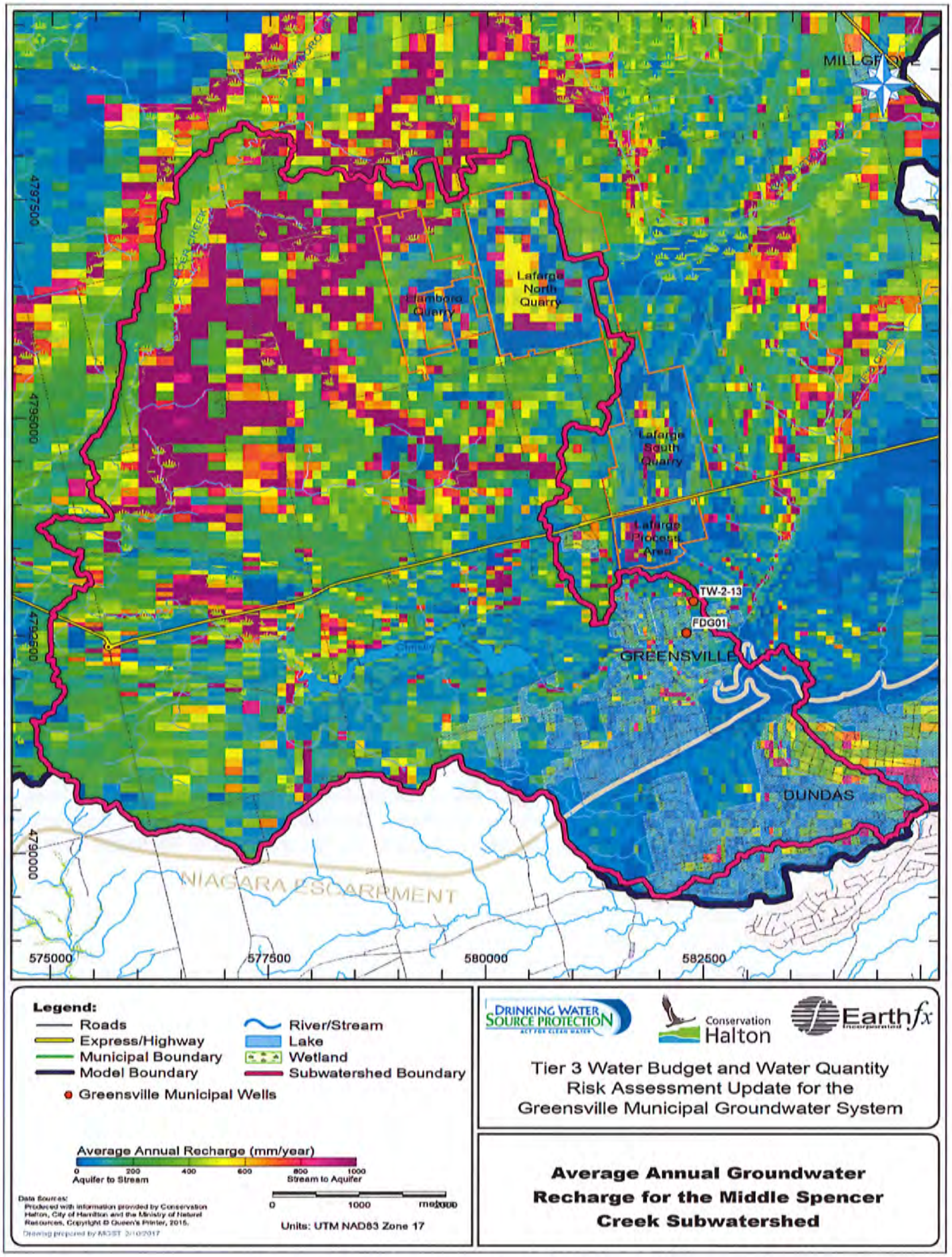


Figure 7.1: Distribution of groundwater recharge.

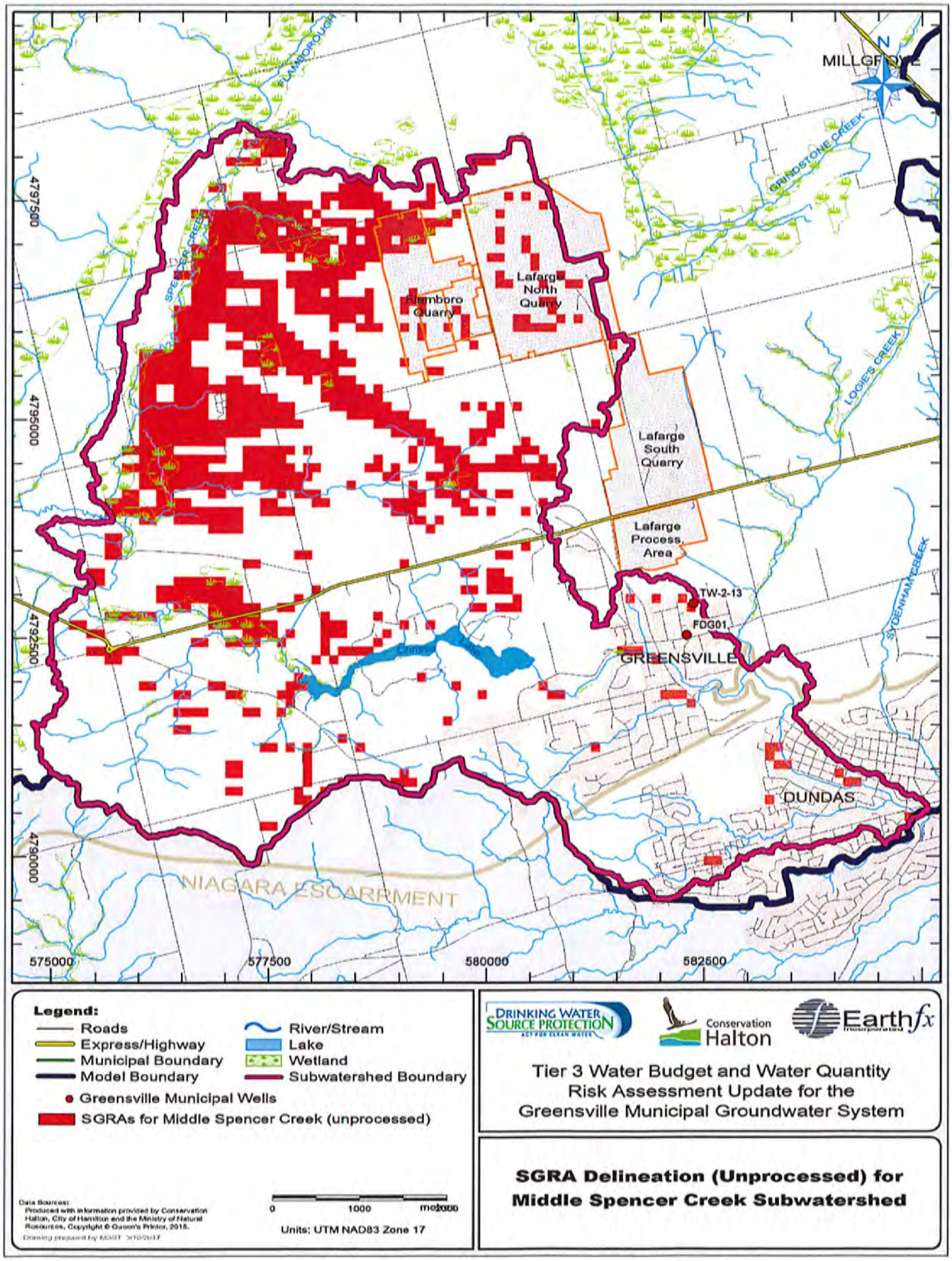


Figure 7.2: SGRA distribution as per Technical Rule 44(1).

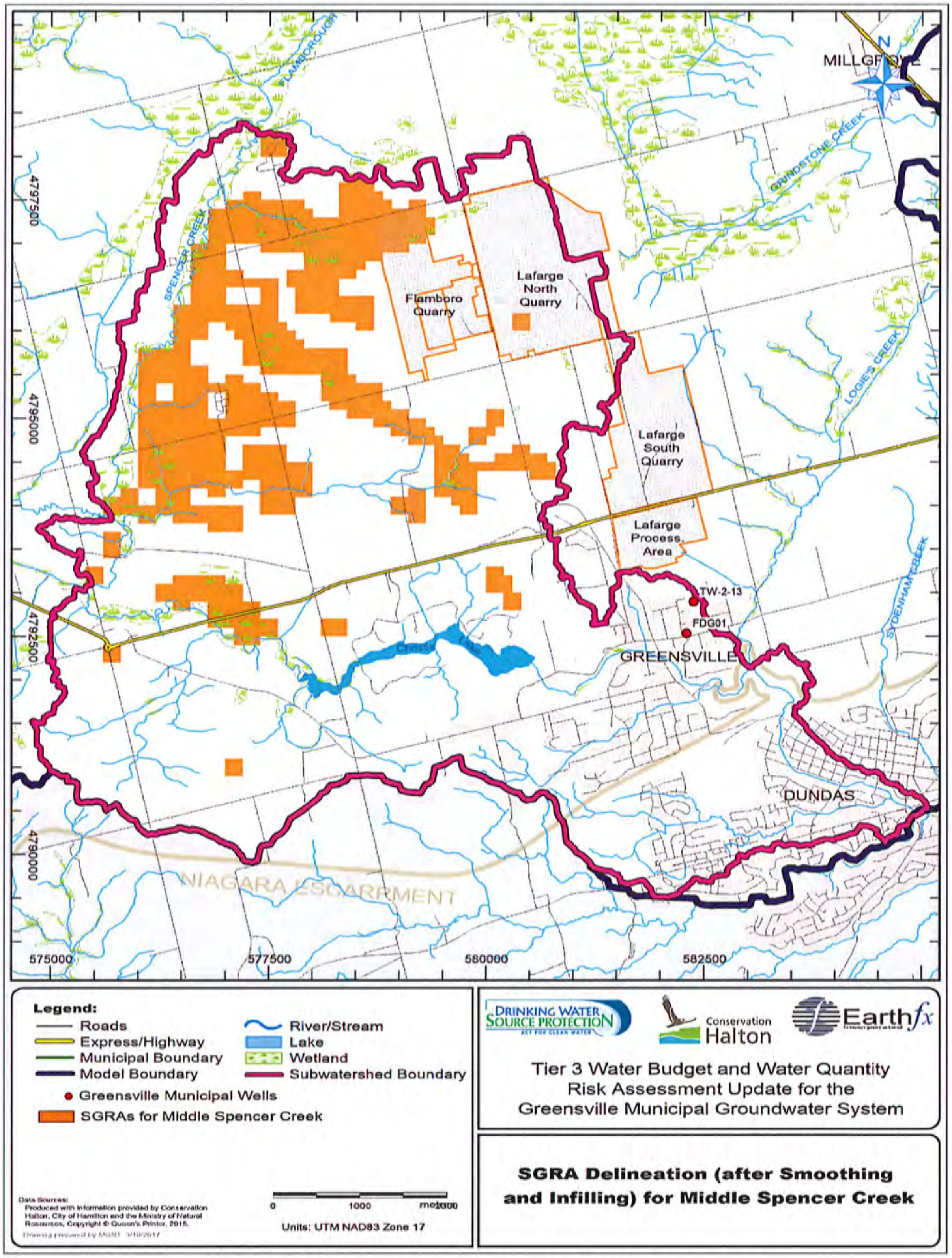


Figure 7.3: SGRA delineation after smoothing and infilling.

8 Summary and Conclusions

8.1 *Previous Work*

The previous Tier 3 Water Budget and Local Area Risk Assessment study of the Greensville municipal water supply was completed in 2015. In that study, the physical setting of the 165 km^2 study area surrounding Middle Spencer Creek and the Greensville wellfield was described (Earthfx, (2014)). Development and calibration of a state-of-the-art integrated groundwater/surface water flow model for the study area, based on the open-source USGS GSFLOW model code, is also described in that report. The integrated model was applied to analyze the water budget of the study area and to conduct the formal risk assessment scenario analyses required for a Tier 3 level assessment.

The original Tier 3 study produced (1) a detailed groundwater and surface water budget; (2) identified significant groundwater recharge areas (SGRA); (3) delineated water quantity wellhead protection areas (WHPA-Q1 and WHPA-Q2); and (4) assessed the capability of the wellfield to meet allocated water demand under current and future conditions under both average climate and extended (10-year) drought conditions. Future conditions considered an increase in non-municipal water takings and an expansion of the two active quarries located north of the Greensville supply well; takings at the municipal well, FDG01, were assumed to remain at the current permitted rate. Results of the study, documented in Earthfx (2015), showed the municipal well to be capable of meeting existing and allocated water demands for current and future land use conditions during both average climate and drought and was therefore assigned a risk level of low. The tolerance of the Greensville well was assessed as high because there was sufficient additional safe drawdown in the wellbore even when pumping was sustained at peak rates. The level of uncertainty in the analysis was assumed to be low because of the model used and the level of data analysis and quality control employed in the study.

8.2 *Analysis of New Data and Model Update*

A new supply well, TW-2-13, located about 370 m northeast of FDG01, is to be integrated into the Greensville municipal water supply system. While no overall increase in pumping is planned, the new well will serve as the primary pumping well, with the existing well serving as backup and pumped an estimated average of one day per week. The addition of the new well and shifting the pumping to a different location necessitated an update of the Tier 3 analyses.

The exploration program for the new well provided new geologic and hydrogeologic data in the wellfield vicinity site. Three test wells (TW-1-13, TW-2-13, and TW-3-13) were drilled 2013 and step tests and a 72-hour aquifer test were conducted.

In 2013, Golder Associates Ltd. issued a Hydrogeology and Hydrology Technical Report for the proposed Lafarge Dundas South Quarry Extension (SQE). The report provided a considerable amount of new hydrogeologic data including the completion of geologic boreholes, installation and monitoring of observation wells, and aquifer testing results. These data were not available during the model construction phase of the previous Greensville Tier 3 (Earthfx, 2014), but were incorporated into this Tier 3 update.

The new wellfield and quarry study data were assembled and entered into the project database. These data were used to update the surfaces that form the conceptual stratigraphic model in the vicinity of wellfield and quarry. Step test and aquifer testing results were analyzed and used to update the hydraulic property values assigned to several hydrostratigraphic units. Streamflow and groundwater level data were entered into the project database. The values served as additional targets for the re-calibration of the updated Tier 3 model. Analysis of groundwater level response in the quarry vicinity also led to revisions in the hydraulic properties assigned to the aquifers and confining units and necessitated a separation of the Guelph Formation and Upper Eramosa Formation into separate units with unique properties in the Conceptual Hydrostratigraphic Model. This was translated into a revision of the original Tier 3 model from 11 to 12 numerical model layers.

Updating the Tier 3 analysis afforded the opportunity to make several other model improvements. Grid resolution was improved (Figure 4.5) with the area of very fine resolution (square cells of 12.5 m to a side) extended to include TW-2-13 (as well as the other two test holes TW-1-13 and TW-3-13). The model grid covering the majority of the quarry footprints was also refined to use cells with a maximum size of 50 m on a side. The updated model grid consists of 232 rows and 175 columns and contains 40,600 grid cells for each of the 12 model layers.

Climate inputs for the integrated model were updated as part of this study to extend the simulation period through $_{WY}$2016. A new unified hourly climate dataset spanning water year $_{WY}$1947 through $_{WY}$2016 was created. This dataset is based on ground stations operated by the City of Hamilton, HCA, and McMaster University and provided a consistent climate product for use in the model.

Water use data for the study area were also updated. Locally, proposed water use for the planned Greensville Elementary School/Library/Community Centre was considered because of the supply well's close proximity to the Greensville municipal wells. Data from newer releases of the MOECC PTTW and WTRS databases were analyzed extensively and used to update annual average and daily pumping rates from non-municipal wells used in the steady-state and transient analyses, respectively. Additional work was done to update the assignment of water takings to the appropriate hydrostratigraphic unit. Current and future domestic pumping was assigned in a manner consistent with the original Tier 3 and the earlier Greensville Rural Settlement Area Groundwater Modelling Assessment (Earthfx, 2010). Consumptive use factors were applied to all takings to account for water not returned to the original source. Quarry water use, which varies in a complex manner based on precipitation, surface water and groundwater inflows into the quarry, were represented internally in the model rather than as specified takings (see Earthfx (2015)).

The objective of the Tier 3 Assessment, as defined in the MNR Water Budget Guide, was to "*estimate the likelihood that a municipality will be able to meet future water quantity requirements*". To complete the Tier 3 Risk Assessment, the integrated groundwater/surface water model developed using the USGS GSFLOW model for the previous analysis was updated. The model covers a 165 km^2 area and the subsurface is represented by 12 hydrostratigraphic layers. The model grid was refined around the municipal well, quarries, and in the shallow subsurface to represent all mapped streams, wetlands, reservoirs and quarry drainage infrastructure.

8.3 *Model Re-calibration*

Changes to the inputs and structure of the integrated surface water and groundwater model necessitated a re-calibration of the model. Model parameters were adjusted to improve the match to observed streamflow at several gauges across the area with a particular emphasis on matching low flows. The groundwater model was calibrated to match both regional groundwater flow levels and patterns across the study area as well as transient water levels at the large number of observation wells in the vicinity of the Greensville municipal well. Overall, the mean error between the observed and predicted water levels has improved.

8.4 *Water Budget Summary*

A new water budget was derived from the results of updated Tier 3 integrated model. These estimates were obtained by aggregating the daily flows produced by the model and averaging them over the simulation period. An overall water budget (Table 5.1) considered all surface water and groundwater fluxes that cross model boundaries including precipitation, ET, pumping, and net overland runoff, streamflow, and lateral groundwater flow. A groundwater budget was also compiled (Table 5.2) and included areal recharge, surface leakage, lateral groundwater flows, stream/lake fluxes in and out of the groundwater system, and well pumping. Quarry water use was not simulated as a direct withdrawal in the model but is included within other components of the overall subwatershed budget.

8.5 *Delineation of Vulnerable Areas*

A WHPA-Q1 was defined by the maximum extent of the 1.0 m drawdown contour around the Greensville municipal well. The drawdowns were determined by comparing simulated groundwater heads under no-pumping conditions with simulated heads at allocated water demand. Simulation of the quarries under the "no-pumping condition" assumed that the quarries would be allowed to fill up to a naturally-controlled level (for example, for the Lafarge Quarry a local ground elevation in the rail cut). Simulations of the pumping conditions included simulating groundwater discharge from the quarries under full build out. The 1.0 m drawdown in the weathered bedrock aquifer was localized within a 0.25 km^2 area around the municipal well and did not intersect the 1.0 m drawdown contour around the quarry. However, the 1.0 m drawdown in the confined Gasport/Goat Island aquifers covered a 43.9 km^2 area and encompassed the quarries and municipal wells. This maximum drawdown was used to define the lateral extent of the WHPA-Q1 (Figure 6.18).

Future changes in land use within the model boundary were simulated to see if the WHPA-Q2 extended beyond the limits of the WHPA-Q1. Areas of future land use change were identified from the Rural Hamilton Official Plan (Figure 3.18). Changes in recharge were primarily due to conversion of agricultural lands to estate residential in the Greensville area. Small localized increases and decreases in recharge were noted but because of limited impact on the municipal wells, the WHPA-Q2 area was determined to be coincident with the WHPA-Q1 area. The Tier 3 Local Area, which is, by definition, synonymous with the WHPA-Q2, is also coincident with the WHPA-Q1.

8.6 *Local Area Risk Assessment*

A series of risk assessment scenarios (Table 6.2) were simulated with the updated Tier 3 Model to assess the impact of increases in water use, drought conditions, and land use change, including quarry build out, on the sustainability of the municipal wells. Results of the future water use and land use change analyses (Scenario G) were presented as maps showing incremental drawdowns from the baseline scenario (Scenario C). Simulated drawdowns within the municipal wells, relative to the safe available drawdown, were presented in Table 6.5. Drawdowns were corrected to account for non-linear head losses and convergent head losses.

Drought impacts were assessed based on a transient simulation (Scenario D) using climate data from $_{WY}$1953 to $_{WY}$1967 (inclusive) and current water use and land use conditions. The first four years of the simulation provided a model "start-up" period during which precipitation was near the historical average. Average water levels in September 1956 served as the baseline for determining drought effects. Maximum decrease in simulated water levels occurred in July 1962 at FDG01 and in November 1963 at TW-2-13 (Table 6.6). Results of Scenario H, using water use and land use (including quarry build out) were also compared against the September 1956 baseline (Table 6.6).

Results of the risk assessment scenarios confirmed the previous Tier 3 analyses and indicate that the Greensville municipal wells are capable of meeting existing water demands for current and projected land use conditions during both average climate and extended drought. Increased dewatering due to *currently approved* quarry buildout, conversion of land use from agriculture to estate residential, and increased pumping at new private wells were considered in the future scenarios but did not cause excessive drawdown at the Greensville well.

The Tier 3 analyses consider the risk to other water uses (e.g., wetlands, aquatic habitat (cold water and warm water streams), other non-municipal water takings, wastewater assimilation, navigation, and recreation) posed by a future increase in municipal water demand. Because no increase in municipal pumping is planned for the Greensville municipal wells, by definition, there is no risk to other water uses. Based on the results of the risk assessment scenarios, the local area around the Greensville municipal wells was assigned a rating of "Low". The risk tolerance of the Greensville wells is considered to be "High" because there is significant additional drawdown in the wells even at sustained peak pumping rates and under drought conditions.

An analysis of uncertainty related to the risk assessment considered the quality and coverage of hydrologic, geologic, and hydrogeologic data, the quality of the model used to evaluate the risk assessment scenarios, the quality of the model calibration, and the level of quality control/quality assurance used in the study. Based on the analysis, it was concluded that the uncertainty was low and there was no reason to increase the assigned risk level.

8.7 *Significant Recharge Areas*

As a final analysis, results of the updated Tier 3 model were used to develop revised maps of significant groundwater recharge (SGRAs). SGRAs are primarily found in the north-western portion of the study area, associated with thin drift and exposed bedrock of the Flamborough Plains (Figure 7.3). Recharge in these areas is high due to the limited aquitard cover material and exposure of the weathered bedrock aquifer. Land use in the area is rural and topography is relatively flat compared to areas closer to the Niagara Escarpment.

8.8 *Conclusion*

This project was undertaken to update the previously completed Greensville Tier 3 study by incorporating the addition of a second municipal water supply well, TW-2-13, to the existing Greensville municipal water supply system. The modelling work completed in support of this study was built upon the previous Tier 3 model; however, benefited from a number of new geologic and hydrogeologic datasets assembled since the previous study.

The updated model was used to develop a water balance for the Middle Spencer Creek subwatershed, and delineate the WHPA-Q1, WHPA-Q2 and Local Area for the Greensville municipal wellfield. The risk assessment confirmed the results of the previous Tier 3 analysis, indicating that the Greensville municipal wells are capable of meeting existing water demands for current and projected land use conditions during both average climate and extended drought, resulting in an assignment of a "low" risk level to the local area.

Report prepared by Earthfx Incorporated (PEO Certificate of Authorization #100072092)

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Appendix A

Previous Greensville Tier 3 Study Reports (Earthfx 2014 and 2015)

The following reports from the previous Tier 3 study of the Greensville Municipal System may be accessed from the Halton-Hamilton Source Protection Region Website from: http://protectingwater.ca/docandmaps.cfm?smocid=1452&parentcatid=837

- Tier 3 Water Budget and Local Area Risk Assessment for the Greensville Groundwater Municipal System – Phase 1 Model Development and Calibration Report (Earthfx, 2014)
- Tier 3 Water Budget and Local Area Risk Assessment for the Greensville Groundwater Municipal System – Phase 2 Risk Assessment Report (Earthfx, 2015)