Water Budget and Stress Assessment

Figures

Tables

C1 TIER 2 WATER BUDGET AND STRESS ASSESSMENT

C1.1 Stratigraphy in the CVSPA

Quaternary aged overburden sediments within the CVSPA provide a detailed record of glacial and interglacial events throughout the most recent glaciation (Wisconsinan glaciation). The last major ice advance began approximately 25,000 years ago, and glacial ice retreated for the last time from the area approximately 10,000 years ago. Sediments deposited during previous periods of glaciation, such as the Illinoian (135,000 years ago) have not formally been identified in the Credit River watershed, however remnants of York Till left behind during the Illinoian, have been identified east of the study area (Karrow, 1989) and may exist at the base of deep buried bedrock valleys in the watershed.

Table C1-1 presents a list of the Quaternary sediments identified in the CVSPA, their distribution, and the general time period in which the deposits were laid down.

Table C1-1: Quaternary Stratigraphy in the CVSPA

	Sub-stage	Glacial Stage	Depositional Environment	Deposit	Description of Deposit in CVC
Pleistocene Epoch	Holocene	Recent (10,000 ybp to present)	Fluvial environment. Gravel, sands, silts, and clays deposited in modern day rivers and creeks and their floodplains.	Alluvium	Gravel, sands, silts and clays
	Late Wisconsinan	Two-Creeks Interstade (12,000 to 11,500 ybp)	Large glacial lake within the Lake Ontario basin, led to deposition of shoreline sands in the CVC area; Glacial ice retreated completely.	Lake Iroquois Sand	Sands
		Port Huron Stade (13,200 to 12,000 ybp)	Ontario ice lobe advanced from Lake Ontario basin northward overriding Escarpment as far as Paris Moraine. Below the Escarpment there were likely high water levels in Lake Ontario basin leading to interbedded tills and shallow water glaciolacustrine sands.	Wentworth Till above Escarpment (Paris Moraine); Halton Till below Escarpment	Wentworth Till- Sandy silt till Halton Till- silty clay till with interbedded sands
		Mackinaw Interstade (14,000 to 13,200 ybp)	Warm interstadial period. Ice retreated out of area; fluvial and outwash sand and gravel deposited; (periglacial and eolian processes dominated).	Oak Ridges Moraine Equivalent below Escarpment	Outwash sands and gravels, tunnel channel deposits
		Port Bruce Stade (15,000 to 14,000 ybp)	Tavistock Till deposited as Georgian Bay lobe moved south into upper CVC area. Areas to the south remained covered with ice (non-deposition).	Port Stanley Till, Tavistock Till, Orangeville Moraine. Guelph Drumlin field above Escarpment.	Silty clay tills NE of Orangeville, Ice-contact drift in Orangeville Moraine

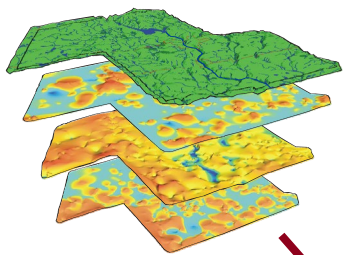
	Sub-stage	Glacial Stage	Depositional Environment	Deposit	Description of Deposit in CVC
		Erie Interstade (18,000 to 15,000 ybp)	Ice sheet remained over much of the study area including CVC and Greater Toronto Area.	None recorded	None recorded
		Nissouri Stade (25,000 to 18,000 ybp)	Ice advanced from north as one till sheet advancing across Ontario into the United States.	Northern Till and Catfish Creek Till	Silty clay till
	Middle Wisconsinan (53,000 to 25,000 ybp)		Glaciolacustrine conditions- water levels in L. Ont. basin remain high. Thorncliffe-delta formed when sediment laden meltwater stream/ river (Laurentian Channel) emptied into L. ON basin.	Thorncliffe Formation Equivalent- below Escarpment only	Sand, silt, and clay, local rhythmites
	Early Wisconsinan (80,000 to 53,000 ybp)		Ice was proximal (in Lake Ontario basin but did not reach land). Sunnybrook was deposited when elevated lake levels (~90 m higher than today) led to quiet water clays deposited with rain out (iceberg) clast debris (forming glaciolacustrine diamicts in Toronto area)	Sunnybrook Drift Equivalent- preserved in topographic lows (buried valleys) below Escarpment	Silts/ clays
			Glacial lake ~ 45m above present day levels filled the Lake Ontario basin. Laurentian Channel fed sediment laden meltwater into the deep basin (in Toronto) causing the formation of a prograding delta (e.g., clays and silt at the base, build into sands and gravels at the top as seen along the Scarborough Bluffs).	Scarborough Sands Equivalent below Escarpment preserved in topographic lows (buried valleys) below Escarpment	Sands
	Sangamonian Interglaciation (80,000-135,000 ybp)		Shallow glaciomarine deposit formed when water levels in the Lake Ontario basin fluctuated in the ice free (warm) interglacial period (deposition of interbedded sands and muds).	Don Formation (near the base of deep buried bedrock valleys)	Sand and mud (silt and clay) beds
	Illinoian Glaciation (135,000- 115,000 ybp)		Ice sheet moved throughout the area depositing a subglacial till that in the CVC area, incorporated the underlying Georgian Bay and Queenston Formation shales.	York Till (at the base of some buried bedrock valleys)	Grey till; abundant shale clasts

“The Big Gap”: (350 Million ybp to 135,000 ybp)		Period of glaciation, characterized by tectonic uplift and erosion leading to a gap in the geologic record across Ontario.	No Record	No Record
Paleozoic Era	**Period**	**Depositional Environment**	**Bedrock Formation**	**Descrip. of Formation**
	Upper Silurian (420- 408 Million ybp)	Shallow high energy shoal to deep basinal. Reefal and interreefal environments.	Amabel Formation Guelph Formation	Dolostone to shaley dolostone
	Lower Silurian (438- 420 Million ybp)	Shallowing upward from offshore basinal to marginal marine environment.	Cabot Head (Cataract Group)	Shale
		Near shore to offshore reefal environment.	Manitoulin (Cataract Group)	Dolostone
		Shallow Marine unit.	Whirlpool (Cataract Grp)	Sandstone
	Ordovician (505- 438 Million ybp)	Deltaic complex with marginal marine and shallow marine environments.	Queenston	Shale
		Shallowing upward shelf succession.	Georgian Bay	Shale and Limestone

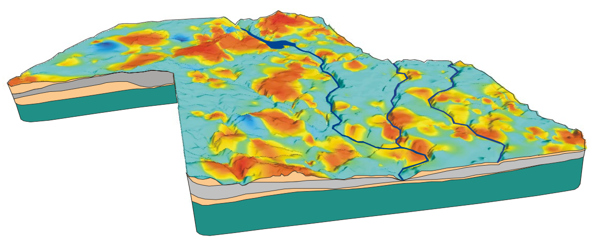
Ybp- years before present. (from Meyer, 2005, (*Personal Communication*); Dates from Barnett, 1992 and Berger and Eyles, 1994; Johnson *et al.*, 1992)

C1.2 Tier 2 Water Budget

1. GIS Layers



2. Conceptual Model



3. Numerical Model

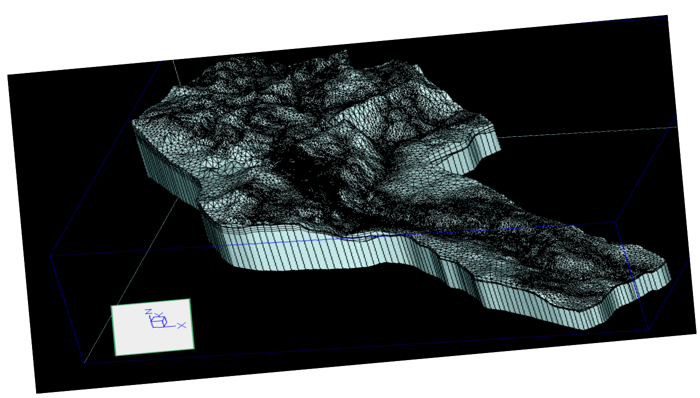


Figure C1-1: Numerical Model Development

C1.2.1 Data Sources

The development and calibration of the numeric flow models relied on numerous datasets, ranging from rainfall, surface water and groundwater data, to physical land parameters (physical geology, soil conditions, land morphology, etc.). Regional and local datasets were provided through provincial and municipal partners (CVC, Provincial Groundwater Monitoring Network (PGMN), MOECC, MNRF, ORMGP, etc.), and have been updated at regular intervals.

Table C1-2 provides a summary of the water budget components and of the modelling capabilities. These inputs were approximated to derive the most accurate representation of precipitation, surface fluxes, recharge, groundwater flows and fluxes, boundary conditions, etc., within each subwatershed. **Table C1-3** provides an overview of the data sets used for the modelling activities.

Table C1-2: Summary of Water Budget Components

Parameter	Source	Description
Precipitation	LBPIA, Orangeville, Guelph Turf Grass Climate Station	Precipitation data (rainfall and snowfall data) collected in locations across the watershed are used as input into the HSP-F model.
Evapo-transpiration	HSP-F	HSP-F estimates actual evapotranspiration using potential evapotranspiration rates and a continuous soil-water balance.
Runoff	HSP-F	HSP-F estimates runoff (including interflow) for each urban and rural land element.
Recharge	HSP-F	HSP-F estimates the amount of groundwater recharge for each urban and rural land element by calculating the amount of infiltration and net evapotranspiration. As discussed in the previous sections, recharge to groundwater from surface water bodies was not considered in this assessment. While it may have local significance, the watershed-based calibration approach does not support a confident assessment and estimation of recharge to groundwater from surface water bodies.
Surface Water Discharge	FEFLOW	The FEFLOW model is able to quantify the amount of surface water that discharges from the groundwater flow system into the surface water features simulated in the model, which may include wetlands, streams, and lakes.
Wells	FEFLOW	The FEFLOW model is also able to simulate the extraction of groundwater from municipal and non-municipal pumping wells.
Inter-Catchment Flow	FEFLOW	The FEFLOW model can quantify the flow of groundwater between subwatersheds within the Credit River watershed. Positive values indicate where the subwatershed is experiencing a net increase of groundwater flow from adjacent subwatersheds. Negative values indicate where the subwatershed is experiencing a net loss of groundwater flow to adjacent subwatersheds.
Inter-Watershed Groundwater Flow	FEFLOW	The FEFLOW model can also be used to examine the groundwater flow through the perimeter boundaries of the groundwater flow model. This flow represents groundwater flow out of or into the Credit River watershed. Negative flows indicate water is leaving the Credit River watershed, while positive flows indicate water is entering the watershed.

Table C1-3: Regional and Local Datasets

<table>
<tr><th colspan="2">Data Source</th><th>Model</th></tr>
<tr><td>Regional and Subwatershed Studies</td><td>Numerous reports were consulted in the development of the conceptual hydrogeologic model of the Credit River watershed. These reports were reviewed to develop a comprehensive understanding of the geologic and hydrogeologic setting of the Credit River watershed. The reports used in the development of the model are outlined in detail in the original conceptual model report (WHI, 2002a). Additional resources consulted since that time is referenced in the reference list located at the back of this report.</td><td>HSP-F and FEFLOW</td></tr>
<tr><td>GIS Mapping</td><td>The CVC maintains various GIS maps that were used in completing this study including:
Ecological Land Use Maps;
Rivers, Streams, Lakes and Classified Wetlands;
Monitoring Station Locations; and
Base maps (roads, political boundaries).
Other mapping products produced by the CVC’s provincial partners include:
Physiology and surface landforms;
Surficial Geology;
Bedrock Geology; and
Digital Elevation Model (DEM).</td><td>HSP-F and FEFLOW</td></tr>
<tr><td>Streamflow Monitoring</td><td>Streamflow data used in support of surface water and groundwater flow model calibration includes Water Survey of Canada HYDAT gauges located across the watershed in addition to spot baseflow measurements completed by the CVC at various locations.</td><td>HSP-F and FEFLOW</td></tr>
<tr><td>Climate Data</td><td>In order to meet all of the climate data input needs of HSP-F, the following parameters are required:
Rate of precipitation (hourly);
Air temperature (hourly);
Wind speed (hourly);
Solar radiation (hourly);
Cloud cover (hourly);
Dew point temperature (daily); and
Potential evapotranspiration (hourly).
Climate input datasets for HSP-F were developed using data collected from a number of stations within and near the watershed, and from Environment Canada’s Atmospheric Environment Service (AES).</td><td>HSP-F</td></tr>
<tr><td>ORMGP Database</td><td>Developed by the Oak Ridges Moraine Groundwater Program (ORMGP), the database contains all water well records and additional exploration and geotechnical borings within the Credit River watershed. These data include updates from the Ministry of the Environment and Climate Change (MOECC) Water Well Record database, municipal supply wells, and geotechnical wells. A summary of the wells and boreholes contained within the groundwater model area is provided below:
Total number of records within model area (includes MOECC domestic wells and geotechnical wells added by ORMGP): 14,041;
Total number of MOECC water well records: 11,369;
Records that do not meet Quality Assurance criteria for location and elevation (Qualified = 9): 1,386; and
Records having static water level observations: 10,365; and
Records having static water level observations and quality assurance criteria < 9: 9,179.</td><td>FEFLOW</td></tr>
</table>

Data Source		Model
Stratigraphic Interpretations	The CVC maintains a database of stratigraphic interpretations (i.e., elevations of interpreted stratigraphic units) at wells located within the watershed. These wells include those from the MOECC Water Well Record Database and those with more reliable borehole logs. This database originated from stratigraphic interpretations completed in 2000 and has evolved with additional studies completed since that time. The database is expanded as wells are interpreted as part of watershed and subwatershed studies. It currently houses all of the stratigraphic picks from cross-sections analyzed for the development of the watershed model and subwatershed scale layer refinements.	FEFLOW
Municipal Groundwater Monitoring	The CVC's municipal partners, including the Town of Orangeville, Township of Mono and the Regional Municipalities of Peel and Halton, provided groundwater level monitoring data to the CVC.	
Groundwater Monitoring - PGMN	Provincial Groundwater Monitoring Network (PGMN). The MOECC and the CVC have developed a network of fourteen wells (in 9 locations) for which a continuous record of water levels has been recorded since 2001. These points were used in the model calibration process to assess the hydraulic head calibration (average head value).	FEFLOW
Permits to Take Water (PTTW)	The MOECC issues Permits to Take Water (PTTW) that allow permit holders to withdraw large volumes of surface water and/or groundwater. These permits are contained within a database that identifies the location, water source, maximum permitted volume and pumping rate, number of days of extraction, and date of expiry. The permits are completed for both surface water and groundwater withdrawals that have a pumping rate of greater than 50,000 litres per day (LPD). The PTTW database contains useful information about the location and maximum permitted pumping rate for all large, permitted water takers in the study area. The CVC has made an effort to verify the location of each PTTW within the Credit River Watershed and to update the database with this information. The CVC collected municipal pumping rates from its partner municipalities as part of the development of its Watershed Characterization Report (CVC, 2007a). The MOECC, in partnership with the CVC, conducted a Water Use Assessment in the Credit River watershed. This assessment included a survey of permit-holders and resulted in refined estimates (or reported measured takings) for a number of permits within the watershed.	FEFLOW

Models Justification

The modelling analysis was an integrated approach, using the following software:

- Hydrological Simulation Program—Fortran (HSP-F) (v.12; Bicknell *et al*., 2001) software: evaluation of surface flow based on precipitation, geology, soils, slopes, land use, demands, etc.; and
- FEFLOW (Finite Element Flow) software: evaluation of subsurface flows and fluxes based on surface recharge (from HSP-F output), geology, boundaries, demands, hydrogeology, hydraulic conductivities, etc.

HSP-F software was first adopted for modelling work for the Water Quality Strategy (WQS) in 2000. It was used for subwatershed-scale hydrologic and water quality assessment and satisfies the modelling requirements both from a practical and logistical viewpoint (i.e., well documented, long-term EPA support, good track record, supporting utilities, and readily available), and the model contains all necessary simulation components required to address a wide variety of watershed issues including those relating to the Tier 2 water budget work.

The FEFLOW software is a numerical finite-element groundwater flow modelling code that has been validated for many applications and benchmark problems (numerical and analytical solutions). FEFLOW utilizes standard GIS formats, which enables integration with other disciplines and tools used by the CVC. It uses the finite-element method to obtain a solution to the groundwater flow boundary value problem by breaking the model domain into a three-dimensional mesh of finite-element cells. The groundwater model for the CVSPA was initially developed in 1998 using FEFLOW. It has been continually updated, and applied to support the CVC's water management activities, as well as subwatershed and watershed-scale studies.

The water budget and flux tools within FEFLOW were used to quantify the volumes or fluxes of water across lines (rivers/aquifer boundaries/political boundaries), polygons (model layers), and boundary conditions (flow in or out of the model). The groundwater flow model provides a significant amount of insight into the flow and flow rates of water through the watershed.

Both the HSP-F and FEFLOW numerical models are regional tools, constructed from regional scale datasets. While the models have been refined locally in some instances, some local complexities may not be represented within the models, and they are not considered to represent all of the local hydrologic conditions within the CVSPA.

The outputs of the HSP-F were introduced into the groundwater model as recharge within the integrative process. FEFLOW, in turn, was used to simulate steady-state groundwater conditions throughout the CVSPA. The area modelled (model domain) is shown **Figure C1-2**.

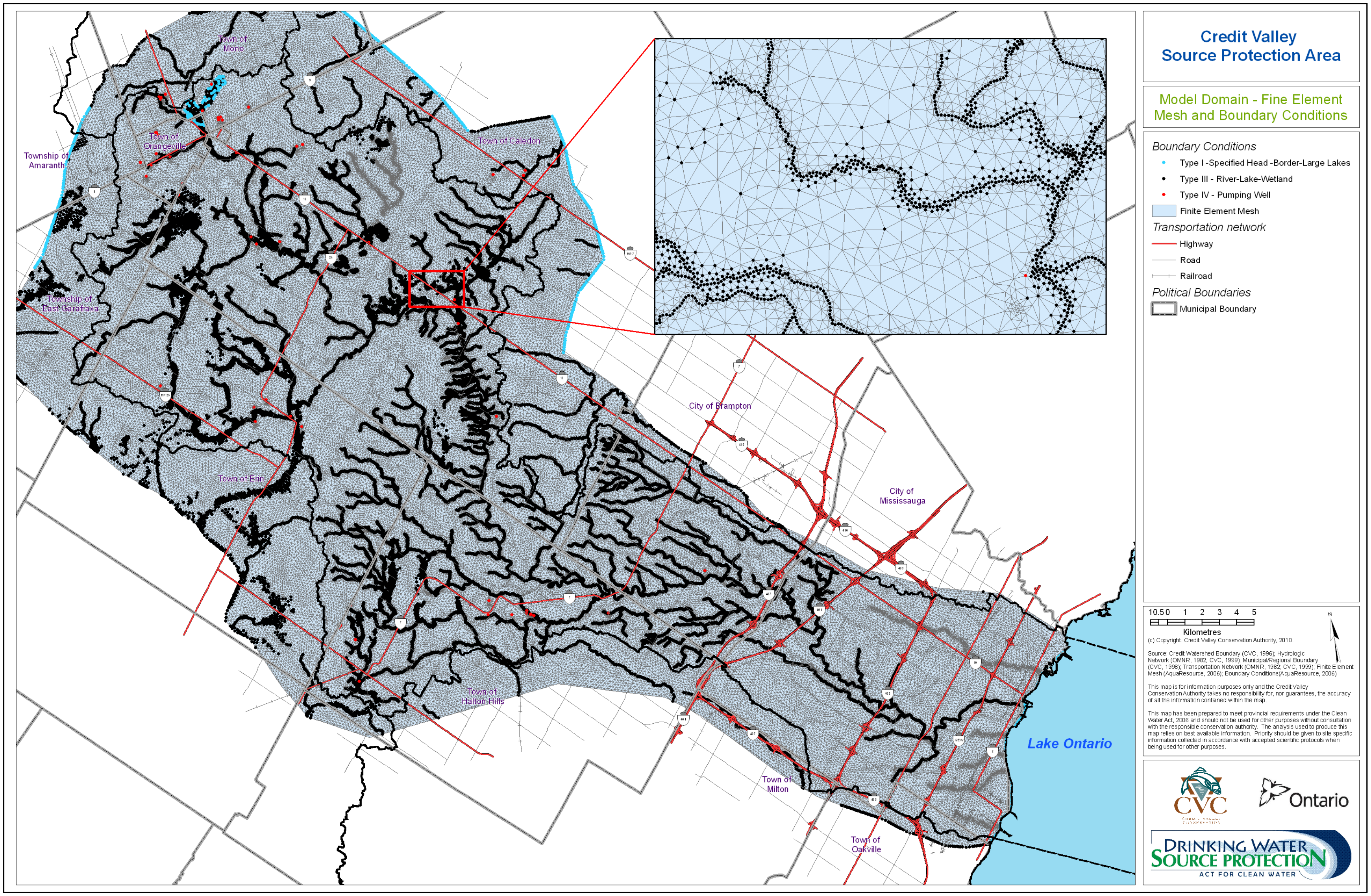


Figure C1-2: Tier 2 Model Domain – Finite Element Mesh and Boundary Conditions (CVSPA)

C1.2.2 Surface Water - HSP-F Model Set-Up

The HSP-F model is structured as follows:

- The CVSPA is represented as a mosaic of urban and rural land elements. Each element has homogeneous characteristics. Several elements together form subcatchments (i.e., discrete sewersheds and small tributary catchments that combine to makeup subwatersheds). Surface runoff discharges from each element into the local watercourse stream/sewer. Subsurface runoff (i.e., interflow and groundwater flow) is generally routed to the nearest local stream channel; however, in some cases subsurface flows are simulated to move to adjacent or downstream reaches to reflect the general groundwater movement patterns observed in the watershed. The watercourse reach may represent a section of a tributary to the main channel, or a section of the main channel of the Credit River.
- Each urban land element is characterized by the land use, topsoil characteristics and topography found within. These characteristics are reflected in the setup and parameterization of the model. Urban areas are characterized to reflect several potential connectivity schemes (i.e., the manner in which pervious and impervious surfaces are connected to each other and the local infrastructure) and land use types. The simulated runoff from each unique urban land type is saved as a unit area surface runoff time series and a corresponding subsurface (i.e., interflow and groundwater) runoff time series. These time series are then brought into the river model and multiplied by a factor to reflect the area of each land type in each subcatchment.
- Rural land element areas are also characterized by the land use, surficial soil types, and topography within. Rural areas are assumed to be primarily pervious and therefore the infrastructure connectivity is not required. Runoff, surface and subsurface, is directed into stream reaches.
- The watercourse network is represented as a series of watercourse reaches. Each of these is characterized using representative stream and valley cross-sections, as well as hydraulic roughness values and channel slopes.

Land Use

The three factors used to classify lands for the HSP-F model include land use cover, surface soil characteristics and topography. Of these, land use cover is the most likely to be affected by human activities and is subject to change in future scenarios. Land use cover for the HSP-F model was derived through two major data sources: CVC's 2004 Ecological Land Classification (ELC) dataset and municipally provided land use data.

The ELC data are a comprehensive dataset for determining land cover for the natural communities, but it is not intended to provide detailed information in urban areas.

Within the ELC dataset, there are two main categories: the existing land use classifications and the ecological land classifications themselves. The data are largely based on the interpretation of the 1996 spring air photo set, at a scale of 1:8000. The existing land use classifications are based on *Credit Watershed Natural Heritage Project Detailed Methodology: Identifying, Mapping and Collecting Field Data at Watershed and Subwatershed Scales, Version 3* (CVC, 1998) and include the following land uses:

- Aggregate (active and inactive);
- Agricultural (intensive and non-intensive, wet meadow); and
- Settlement (manicured open space, rural development, urban, landfill).

Ecological classifications are based on ELC for southern Ontario: First Approximation (Lee *et al.,* 1998). These classifications include:

- Forest (coniferous, deciduous, and mixed forest and plantations);
- Wetlands (coniferous, deciduous, mixed, and thicket swamp, marsh, thicket or treed bog);
- Successional Communities (cultural thicket, savannah, woodland, and meadow); and
- Other (aquatic, open or treed beach/bar, open or shrub bluff).

Surface Soils

Some of the most critical modelling input parameters are related to surface soil types within each catchment. Soil characteristics affect the water balance between surface runoff and subsurface runoff and thus, profoundly affect hydrology. Mapping of topsoil classification according to the Ontario Soil Series system was used in this study. Information on the Hydrologic Soil Group (HSG) for each Ontario Soil Series (per *Ontario Ministry of Transportation Drainage Manual*, 1997) was then used to develop mapping of the HSG over the entire watershed area.

As the watershed is dominated by five HSG classifications (i.e., A, B, C, D and O) a 4-tiered classification system was adopted. The A soils are sands and gravels with high infiltration rates, B soils are fine sands and silts with medium infiltration rates, C soils are silty clays with low infiltration rates, D soils are clays with very low infiltration rates and the O soils are high in organic matter and are assumed to have high infiltration rates. In this model discretization scheme, the HSG classifications were grouped as AB, BC, CD and O.

In order to simplify the growing complexity of the model and avoid unrealistic classifications, a number of assumptions were made and checked with respect to the soils and topography. For instance, the areas of urban land uses with organic soil types were calculated and found to be representative of an insignificant portion of the total land use area, as expected.

The commercial and industrial areas are typically over 90% impervious so the soil type is less relevant than on other urban land use types with higher levels of perviousness. In addition, the pervious portions of these land use types typically are not the native soil, but topsoil or other more permeable fill material. By assigning one soil type to these land use areas, the model is simplified, and results are not expected to be significantly affected.

Topography

Ontario Base Mapping (1:10 000) was used to infer differing slopes of the land within the watershed. About 13,000 spot elevations were plotted from the digital Ontario Base Mapping and a grid was overlaid on the digital contours found on the Ontario Base Mapping. Wherever a grid line intersected a contour, a new spot elevation was created.

To simplify the model with respect to topography, a general hypothesis was made that industrial and commercial lands would not have high slopes due to building constraints and grading.

A very low percentage of urban areas were found to have high slopes (> 3.0%), and it may be that some of these high-sloped urban areas were incorrect due to errors in predicting slopes. High-sloped urban areas were lumped with the moderate sloped (1.51% to 3.00%) urban areas creating two slope ranges for urban areas. The three slope ranges were maintained for the rural land uses as high slopes (> 3%) constitute a significant portion of the rural landscapes.

Unit Response Functions

Urban areas were treated with special detail using a two-step modelling approach. The first step in this approach calculates and stores unit hydrologic response functions (URFs). The second model step uses the URFs as input for instream flow simulation. Rural areas are simulated entirely in the second step using a conventional approach. This URF approach was developed for urbanized portions of the watersheds examined in the Toronto Wet Weather Flow Management Master Plan (TSH, 2003). This approach was chosen for these studies as it allows for more detailed and specific characterization of urban landforms and their connectivity to infrastructure.

URFs are the time series of runoff and heat simulated to discharge from one of several possible generic urban landforms over the simulation period. Each URF area has been constructed using the necessary number and combination of IMPLND (i.e., IMPervious LaND) and PERLND segments. The characteristics of URFs are highly dependent upon their relative perviousness and the connectivity; the flow schematic that describes how surface and subsurface water moves across and through the landform. This flow scheme affects the overall response of the hydrological system and ultimately the associated water quality. The model has been setup to maintain this flexibility.

In industrial, commercial, and institutional land types it is common for all drainage from impervious areas (i.e., roofs, walkways, roadways, and parking lots) to be collected and discharged directly to the storm sewers. Pervious areas (i.e., lawns and gardens) generally constitute only a small portion of the land area and surface runoff generally drains directly to the roadway servicing the buildings. These landform types with high levels of overall imperviousness generate high runoff volumes and very rapid runoff responses during storms and contribute only a small amount of flow to groundwater recharge.

Within residential areas, various infrastructure connectivity schemes are possible. The configurations believed to be the most common in Mississauga and Brampton have roof leaders discharging to the lawns and foundation drains connected to the storm sewers.

Open areas such as parks and hydro corridors have very little impervious area. There may be some parking or walkways associated with these land uses in addition to the adjacent roadways. Conventional URFs for these areas assume that parking areas and roadways are drained directly to the storm sewer.

The relative volume and timing of stormwater runoff from URF areas is highly dependent upon the URF characteristics. At one extreme, low density residential areas with disconnected downspouts and foundation drains discharging to the lawn effectively attenuate stormwater runoff. At the other extreme, high density residential areas with connected roofs and foundation drains generate runoff responses similar to commercial and industrial areas.

To represent all of the conditions within the urbanized portion of the CVSPA, it was necessary to construct a total of 140 unique URFs to represent existing conditions within the urbanized portion of the watershed. The hydrologic response from all of the urban areas was then determined in the second stage of the modelling, by summing up the area-weighted URFs for all the urban area within each subcatchment. The model files contain a table identifying the URFs by code and their characteristics of connectivity, soil type and pervious/impervious area breakdown.

URFs are simulated in a preliminary model run that generates a series of URFs including one for each of the possible connectivities and soil type combinations for standard unit areas of one hectare. The output from this run is expressed as volumes of runoff or heat from surface runoff, collected by the storm sewers, and subsurface from groundwater. Since the model is a lumped parameter model the hydrological characteristics of slope and slope length are common to all URF areas with common surface characteristics. Values applied represent typical values for the slope and land use classes involved. Thus, routing delays for surface and subsurface flows are incorporated in the URFs. Output is stored in a Watershed Data Management (WDM) file using a four-digit numbering system that can accommodate future expansion of landform types.

Instream flow is determined in a second model run that uses the urban URFs as input. The URF for each land type in a catchment is multiplied by its area in hectares to generate the total surface and subsurface flow by land use from each subcatchment.

Watercourse Definition

The watercourse network represented in the HSP-F model consists of a set of watercourse reaches that represent the main stem of the river, as well as selected portions of tributaries. In general, each catchment contains one reach. Inflow to the reach is from local contributions and upstream contributions.

Each watercourse reach is modelled within HSP-F as a RCHRES (i.e., ReaCH/REServoir) segment. The hydraulics of the reach is characterized in the model by supplying a table of hydraulic parameters including outflow and corresponding depth, water surface area and water storage volume for each reach. These data are then used to simulate routing of flows through the reach network.

Representative stream and valley cross-sections for each reach were used to develop the necessary average depth-surface area-reach volume relationships (i.e., flow tables-FTABLES) for each reach. As a first step, the reach cross section shapes and channel roughness were taken from the GAWSER input files (Schroeter and Associates, 2001). The discharge associated with each entry in the table was determined using Manning's equation, where:

$$Q = (A) * (S^{1/2}) * (R^{2/3}) * (n^{-1})$$

In this equation Q is flow rate (m^3/s), A is cross sectional area (m^2), S is water surface slope (%), R is hydraulic radius (m) and n is Manning's roughness coefficient (dimensionless). Stage-discharge relationships for ponds were also taken from GAWSER input files. The stage-discharge relationship for Island Lake was taken from the most recent operations report (CVC, 1986).

In many headwater areas the channel cross sections had not been accurately measured at the time of this study. In these areas, channel shape characteristics (i.e., depth versus width and volume) were taken from other reaches with similar slopes and drainage areas. This form of gap filling is not expected to affect low flow simulation. However, the accuracy of peak flow simulation will be compromised by this method. Also, some areas may have experienced some development and channel modification since the GAWSER study was conducted. Upgrades to the physical model are recommended.

Groundwater Routing

The FEFLOW groundwater model has been used to simulate steady state groundwater conditions for the CVSPA. Using the calibrated model, estimates of groundwater flow transfers between subwatersheds and across watershed boundaries were determined. This was undertaken to determine whether there is significant movement of water into or out of the watershed and across subwatershed boundaries. It was apparent that there is significant groundwater flow from headwater catchments to the central portion of the watershed, and this water bypasses local streams and the headwater portion of the river channel.

While complex subsurface routing is beyond the scope of the surface water model, the most significant shallow groundwater movement patterns were incorporated in a simplistic form. The earliest model runs indicated that simulated streamflow in the headwater subwatersheds (i.e., 13 to 20) was too high, while simulated flows at Norval near Georgetown were approximately correct. This confirmed the groundwater modelling observations about subsurface movements. It was decided that this subsurface flow pattern should be incorporated in the model in order to improve on simulation accuracy in the headwaters.

C1.2.3 FEFLOW Model Set-Up

Model Domain

The location and boundary of the model domain were updated from the boundaries applied in previous CVC modelling studies (WHI, 2002a; WHI 2002b; WHI, 2004) and contains the entire CVSPA.

The model domain was designed to encompass the entire watershed and to extend to the natural boundaries of the groundwater flow system as interpreted from the shallow and deep groundwater level contours. Where natural physical flow boundaries could not be followed, the model domain followed lines of relatively constant equipotential head. These model boundaries were chosen to be beyond the watershed boundary to minimize their effect on model predictions within the watershed. In some areas the proximity of the model boundary to the watershed boundary may influence simulation results. In these areas, interpretation of local groundwater impacts, water budgets, and situations should be identified and handled appropriately.

The finite element mesh was designed to have nodes coincident with streams contained in the updated CVC stream GIS layer. Improvements in computer speed and mesh creation programs enabled the development of a considerably refined mesh. The new finite element mesh was created using GridBuilder™ software. The mesh is refined in areas of the model where it was important to have an enhanced definition of the groundwater level surfaces. A nodal spacing of 25 m was achieved along all streams classified as having a Strahler order of 2 and greater. Nodal spacing is 25 m or less around pumping wells and the maximum nodal spacing is 200 m. The mesh has 12 layers and 13 slices with a total of 2,116,790 nodes and 3,886,656 elements.

Hydrostratigraphy

The vertical and horizontal extent of hydrogeologic units in the subsurface and their connectivity was delineated by interpreting geology reported in boreholes and wells within the watershed

The thickness, distribution, and relative hydraulic conductivity of each of the model hydrostratigraphic units was initially developed as part of the initial CVC Water Budget projects (WHI, 2002a; WHI, 2002b), whereby over 120 cross-sections were drawn and interpreted across the watershed to build the CVC numerical FEFLOW model. Regionally extensive tills plains were used as key marker beds in the development of the model layers as these units are regionally more laterally continuous than aquifer units in the watershed. This initial interpretation has been refined and modified over the course of various projects since 2002 and updates are continually made to a master database of hydrostratigraphic 'picks' or interpretations. The most recent updates to this database were made as part of the Region of Peel Wellhead Protection Area Study (AquaResource, 2007), where the geology in the vicinity of the region's wellfields was re-interpreted and refined.

A surface with an elevation 3 m below the top of bedrock surface was created to represent the elevation of the base of the weathered bedrock (contact) zone. The bottom elevation of the model was specified 50 m below the top of the Georgian Bay Formation throughout the watershed to simulate the groundwater flow into and out of the deeply incised bedrock channels in the southern portions of the watershed (east of the escarpment). At the base of the model, groundwater flow is interpreted to be nearly horizontal.

The bedrock formations are represented in the model by six finite-element layers. While the number of layers used to delineate the Queenston Shale and Georgian Bay Formations could be decreased to increase model efficiency, they have been included to increase the vertical discretization and to aid in the simulation of flow from the bedrock into the bedrock valleys.

The Guelph Formation and Amabel Formation bedrock units have been grouped together into one model layer, as the two dolostone units are considered to have very similar hydrogeologic properties, and in most places, it is difficult to separate the two based on the geologic description in the water well record. Geological mapping of the bedrock surface conducted by the Ontario Geological Survey has mapped the Guelph Formation in the far western reaches of the watershed in the Town of Erin and Township of East Garafraxa, and high-quality boreholes are very rare in this area. In areas to the west, the Eramosa Member of the Amabel Formation has been mapped as a shale aquitard lying between the two dolostone units; however, the lack of high-quality data in the area prevented subdivision of this unit into the separate formations. Similarly, a lack of high-quality characterization data also prevented further identification of a separate production zone within the Amabel Formation.

The Lower Sediments were also grouped together into one model layer as there is insufficient deep borehole information available to adequately characterize the aquifer/ aquitard geometry of the individual Lower Sediment units or their respective hydraulic conductivities. The deep infill of those buried bedrock valleys remains a data gap and a knowledge gap.

The distribution and thickness of the hydrostratigraphic units in the FEFLOW model was found to be consistent with hydrostratigraphic interpretations completed in the adjacent Toronto and Region Conservation Authority (TRCA) watershed (Kassenaar and Wexler, 2006).

Hydraulic Conductivity Distributions

The spatial distribution of hydraulic conductivity information used to calibrate the numerical groundwater flow model was derived from quaternary geology mapping, bedrock geology mapping, and available information from consulting reports corresponding to lithologic descriptions noted in the water well records.

Numerous consulting reports were reviewed to refine the hydraulic conductivity estimates using results from hydraulic testing and any mapping that was completed as part of those studies. The majority of this data was available within various municipal wellfield areas. As part of a recent study (AquaResource, 2007) the CVC FEFLOW model was updated locally around several of the Region of Peel wellfields for the purpose of updating the region's capture zones in Alton, Inglewood, Cheltenham, and Caledon Village (wells 3 and 4). Refinements included the interpretation of approximately thirty additional cross-sections, as well as refinements to hydraulic conductivities.

Results from several pumping test reports, previous modelling studies and other related hydrogeological studies were reviewed, to ensure the model transmissivity estimates were consistent with field-based estimates.

Vertical hydraulic conductivity was estimated to be one tenth of horizontal hydraulic conductivity, to account for horizontal bedding present in most units. Floodplain deposits, glaciolacustrine deposits, and stratified tills are all interpreted to have horizontal stratification that is interpreted to be best estimated by a 10:1 horizontal to vertical anisotropy. This estimate of anisotropy was found to represent observed static water levels in most units, except the highly stratified ice-contact stratified drift deposits associated with the Orangeville Moraine, and the Halton and Newmarket Tills, which in the watershed contain numerous interbeds of sand. In these units the vertical hydraulic conductivity of the stratified drift was calibrated in the groundwater model to be one fiftieth of the horizontal value.

Boundary Conditions

Boundary conditions represented in the groundwater flow model include vertical recharge through the upper layer of the model, rivers, wetlands, and lakes lying at ground surface, and groundwater extraction wells that are screened over specific model layers (hydrostratigraphic units). Boundary conditions were also applied in areas to the outermost elements of the model where groundwater was interpreted to flow into or out of the model domain:

- Recharge: Average recharge rates estimated by the HSP-F model for the 1961-2004 simulation were applied across the watershed for each hydrologic response unit. The recharge map was interpolated to the model mesh within the watershed to apply recharge based on slope, soil (surficial geology), and land use. Outside the watershed, values were applied based on slope and soil type (surficial geology). The HSP-F values were not modified during the model calibration process.

- Lakes: Significant lakes (Island Lake and lakes associated with aggregate extraction) connected to surface water drainage systems were represented as Type III (head-dependent flux) boundaries. Small lakes and ponds that were interpreted to have no regional hydrologic significance and that were not connected to streams or rivers were excluded from the model. Lake Ontario was represented using Type I (specified head) boundaries set equal to 75 m AMSL.

- Wetlands: Larger wetland complexes (i.e., Caledon Lake) were included as Type III (Variable Head) boundaries. Localized small wetlands were not included in the model as they are not interpreted to have an impact on the regional water budget or water levels. The conductance value of the wetlands was used to limit the connection with the groundwater system within the physical limits of the system. A FEFLOW Transfer Coefficient equal to 8.6x10-6 s-1 was applied over elements contained within larger wetlands. Assuming a wetland bed thickness equal to 0.5 m, this transfer coefficient translates to a hydraulic conductivity equal to 4.3x10-6 m/s, which is similar to a silt or silty sand. The actual applicability of these parameters is very uncertain, and they likely vary across the watershed in different types of wetlands. The reference elevation applied for the wetland was assumed to be equal to the elevation represented in the 5 m DEM.
- The boundary conditions applied and presented in this report do not allow water to be recharged from wetlands into the groundwater system. At the regional scale, this approach is justified as many wetlands cannot be assumed to store sufficient water year-round to act as a source of groundwater recharge.
- Streams: All streams classified with a Strahler stream order of 2 or higher were represented within the model as Type III (Variable Head) boundaries.

Most first order streams were not included in the model as they generally are dry, and in the absence of local information they cannot be assumed to be able to supply water to the underlying groundwater flow system. First order streams lying along the face of the escarpment were represented as they were interpreted to be groundwater fed and representative of the water table condition at those locations. The influence of these streams on the overall regional water budget is perceived to be minimal and cannot be reliably tested without additional detailed information on the first order streams in the watershed. Most of the first order streams in the watershed are small, and it is interpreted that this routing to the higher order streams will not impact the groundwater flow into or out of the subwatersheds or impact the water demand calculations outlined in **Section C1.5**.

The stream stage was assigned in the model every 50 m and interpolated along the stream elements using stream stage values estimated from the 5 m DEM.

The stream boundary conditions are constrained within the model so that groundwater cannot flow from the streams into the underlying groundwater flow system if the groundwater level is lower than that of an adjacent stream. At the regional scale applied, this approach is justified as many lower order streams are intermittent or ephemeral and cannot be assumed to contain water year-round, or act as an infinite source of groundwater recharge to the underlying aquifers. While the potential for recharge from higher order streams to the groundwater system was not characterized within this regional study, it is understood that it may be significant within some areas of the watershed, or on a local scale.

A seepage face was added to the model area near Acton to improve the simulation of shallow groundwater heads. Other areas of the model did not require seepage faces along the face of the escarpment, as the high density of streams near the escarpment face adequately simulated the observed groundwater heads and stream discharge.

Pumping Wells

Actively pumping municipal groundwater wells and non-municipal water taking permits were specified in the model as Type IV boundary conditions, as a line element over the open or screened interval of the well. The pumping rates applied were based on feedback obtained from the various municipalities.

FEFLOW distributes the pumping rate through the depth of a well based on the conductivity of layers over which the groundwater is extracted.

C1.3 Model Calibration and Validation

Calibration is the process by which model input parameters and boundary conditions are systematically adjusted within an expected range until the differences between model output and field observations are within selected criteria (i.e., acceptable margin of error) for performance. The model's ability to represent observed conditions is assessed qualitatively to discern trends in water levels and distribution of groundwater discharge, and quantitatively to achieve acceptable statistical measures of calibration.

Surface water flows (HSP-F) were calibrated to surface water monitoring stations, while FEFLOW calibration was focussed on water levels (in PGMN wells, municipal monitoring wells, and domestic wells) and recharge (baseflow—WSC gauges, CVC gauges, CVC spot flow monitoring). In the final analysis, an optimal match is desired, given the constraints of the datasets.

The calibrated HSP-F model was used to simulate precipitation and surface water flow and was calibrated to estimate recharge rates (specified flux) for input into the top layer of the FEFLOW model. HSP-F calibration efforts were focussed on the low flow regime to be able to better constrain the groundwater recharge estimates.

The calibrated FEFLOW model was used to examine groundwater flow directions, groundwater discharge to streams, wetlands, or lakes, and to quantify the flow within the Credit River watershed from one area to another.

While the HSP-F model simulates hourly continuous streamflow, and the FEFLOW model simulates average annual groundwater discharge and baseflow conditions, each model addresses important aspects of the same surface water flow system. As such, the two models can be calibrated to the same streamflow data. The second common aspect shared by the two models is groundwater recharge, which is simulated by the HSP-F model as a model output and used as a model input parameter in the FEFLOW groundwater model.

C1.3.1 HSP-F

The HSP-F model was calibrated to six streamflow gauges with long-term records for the period 1997 to 2000. The calibration process began in the headwater areas and proceeded downstream to the Norval station. Equal emphasis was placed upon calibration of low and high flow rates.

Due to the complexity of the calibration process, involving establishing values for several key model parameters, various periods in the calibration were addressed in a stepwise manner. That is, initially the snow/ice period was calibrated, followed by the spring runoff period and finally the late summer low flow period. In some cases, individual parameters can be calibrated by focusing upon a selected period of time in which that parameter has a significant effect.

A general comparison of the simulated and observed streamflow at six gauged sites is provided in **Table C1-4** for the calibration period. The American Society of Civil Engineers Task Committee on Definition of Criteria for Evaluation of Watershed Models has recommended that continuous hydrologic simulation modelling be evaluated using relatively few criteria in addition to comparative output hydrographs and time series (ASCE, 1993). Recommended criteria include comparisons of runoff volumes and the Nash-Sutcliffe Coefficient (R2).

Table C1-4 compares the simulated and observed streamflow rates by their extremes (i.e., 5 and 95 percentiles), means, and the Nash-Sutcliffe Coefficient, R2. The Nash-Sutcliffe Coefficient is a measure of the goodness of fit of the simulated data to the observed data. A value between 0 and 1 indicates the degree of agreement between the time series with 1.0 being a perfect agreement between the two sets and 0 meaning that the model cannot improve over the mean of the time series as an estimator.

Table C1-4: Comparative Criteria for the Calibration Period at Seven Gauged Sites

Criteria[1]	Cataract	Boston Mills	West Credit	Black Creek	Silver Creek	Norval
5th Percentile Flow Rate, m^3/s	0.54/0.70	1.35/1.40	0.12/0.12	0.10/0.06	0.45/0.30	1.90/2.10
Mean Flow, m^3/s	1.26/1.20	3.17/3.00	0.34/0.33	0.16/0.13	0.72/0.70	4.14/4.00
95th Percentile Flow Rate, m^3/s	3.80/3.60	9.40/9.70	0.95/1.07	0.45/0.63	2.81/3.20	14.70/14.90
Nash-Sutcliffe Coefficient, R^2	0.64	0.39	0.25	0.67	0.58	0.68

[1] Flow rates are presented as Simulated/Observed.

Correspondence between the flow extremes and means are very good at all sites. The poorest correspondence is at Black Creek, especially at the low flow range. Note that some irregular patterns are apparent in the low range of the observed flows at these two sites. This is likely due to some estimated flows or incorrect flows within the time series. The calibration cannot reflect these very low flows as evidenced by the frequency analysis.

The Nash-Sutcliffe Coefficients are mostly at or above 0.60, except at West Credit and Boston Mills. This criterion indicates that the calibration is acceptable at all locations.

The correspondence between simulated and observed flow frequencies confirms the model's ability to simulate instream hydrologic behaviour. Simulated results reflect observed characteristic Credit River peakiness (i.e., maximum flow rate/mean flow rate) and extended low flow periods. The agreement between flow volumes is confirmation of the model's reliability in terms of annual and seasonal water balance.

Once calibration was achieved, the model was subjected to a validity test wherein model simulation results were compared to measured flow rates from a different time period, namely 2001 to 2004. **Table C1-5** contains a summary of streamflow validation results for six gauged sites in terms of total validation period flow volume.

Flow volumes compare favourably at all sites except at Black Creek, wherein the model simulates higher flows and to a lesser extent at the West Credit site, wherein the model simulates lower flows. These sites represent small drainage areas with some uncertainty around surface and subsurface contributing areas as well as local precipitation inputs.

Table C1-5: Seasonal Total Water Volumes at Six Gauged Sites for the Validation Period, 2001-2004

Gauge Location	Drainage Area (km^2)	Observed Flow (dam^3)	Simulated Flow (dam^3)	Relative Difference (%)
Credit at Cataract	194	196,407	195,316	-0.6
Credit at Boston Mills	392	493,919	492,846	-0.2
West Credit	36.1	54,279	50,990	-6.1
Black Creek	24.5	22,761	26,451	16.2
Silver Creek	129	142,272	142,919	0.5
Credit at Norval	618	719,081	740,605	3.0

Over the whole model testing period, 1997-2004, the model was adjusted to achieve a close balance at Norval. Norval is a location of focus because it is the most downstream gauged site during the model testing period and thus is the best integrated measure of whole watershed water balance. Simulated flow volume was about 0.4% higher than observed at this site over eight years. Over that period all simulated streamflow volumes are within 1% of the observed volumes except at the West Credit gauge where a relative difference of -3.8% was achieved and at Black Creek where the difference is 8.7%.

Following the model calibration and validation process, the model was used to simulate hydrologic conditions in response to a long-term climate data set (1960-2005). Average groundwater recharge under these long-term conditions was used as input into the FEFLOW groundwater flow model.

Table C1-6 lists comparative criteria for the validation period for the six gauged sites. Flow extremes and mean as well as the Nash-Sutcliffe Coefficient are provided.

Table C1-6: Comparative Criteria for the Validation Period at Seven Gauged Sites

Criteria	Cataract	Boston Mills	West Credit	Black Creek	Silver Creek	Norval
5th Percentile Flow Rate (m3/s)	0.54/0.60	1.35/1.50	0.12/0.12	0.10/0.06	0.46/0.30	2.05/1.90
Mean Flow (m3/s)	1.26/1.30	3.17/3.20	0.34/0.33	0.15/0.13	0.72/0.80	4.25/4.30
95th Percentile Flow Rate (m3/s)	3.81/3.20	9.39/8.30	0.95/1.07	0.45/0.42	2.81/3.10	14.5/14.20
Nash-Sutcliffe Coefficient (R2)	0.55	0.32	0.22	0.22	0.25	0.46

In general, the correspondence between observed and simulated extreme flows (i.e., 5th and 95th percentiles) and mean flows is good. The Nash-Sutcliffe Coefficients are reasonable, although lower for the calibration.

While validation results are not as good as those for the calibration period, these results, nevertheless, confirm that the model is sufficiently accurate for the Source Protection Water Budget application.

Sensitivity Analysis

A sensitivity analysis was completed for the hydrological portion of the model to help identify the parameters that are the most sensitive. Five input parameters were tested from the snow routine and 25 input parameters were tested from the water routine. Typical values for these input parameters were increased and decreased and the changes in key output parameters were reviewed.

The most sensitive parameters were found to be MGMELT and TSNOW for the input parameters in the SNOW block. MGMELT is the maximum rate of snowmelt by ground heat, in depth of water per day, which applies when the snowpack temperature is at the freezing point. TSNOW is the air temperature below which precipitation will be snow, under saturated conditions. Under non-saturated conditions the temperature was adjusted slightly.

The most sensitive parameters found in the PWATER (i.e., Pervious area WATER) module (those hydrologic parameters relating to pervious land segments only) group include LZSN, INFILT, AGRWRC, INFEXP, UZSN, NSUR, SREXP, CEPSC, and LZETP.

C1.3.2 FEFLOW

The FEFLOW model was calibrated using groundwater recharge rates estimated from HSP-F. It was calibrated to static water levels from various data sources as well as to baseflow data.

Groundwater Level

Table C1-7 summarizes the specific data sources used to evaluate model calibration. These data sources include static water levels reported as well as continuous water levels reported for Orangeville / Mono, the Region of Peel, and PGMN monitoring wells. The Orangeville / Mono, Peel, and PGMN monitoring data are considered to be the most accurate water level calibration targets.

Table C1-7: Summary of Calibration Targets

Target Type	Dataset Name	Number of Points	Description
Water Levels	ORMGP Database	10,075	Static water levels reported in the ORMGP database.
Water Levels	High Quality Wells	248*	Average water levels reported in Orangeville, Peel, Halton, and PGMN monitoring wells (listed in Appendix D). Pumping levels in the municipal supply wells are not included. Halton Region monitoring data was not made available in time for this study.
Baseflow	HYDAT / Water Survey Canada Stream Gauge	9	Minimum and maximum estimates of average annual baseflow at HYDAT Gauges using BFLOW as summarized in Section 2.0.
Baseflow	CVC Spotflow Measurements	23	Minimum and maximum spotflow measurements made by the CVC from 2000 to 2005. Distribution of spot baseflow measurements is skewed towards the summer months, and average annual baseflow is expected to be in the upper range of the spot baseflow measurements.

* High quality data from the Region of Halton was provided following the calibration process; therefore, this data was not considered part of the calibration dataset.

Groundwater discharge calibration targets were determined from two sets of streamflow observations: continuous streamflow records at HYDAT/WSC gauges and spot baseflow observations recorded by the CVC as part of the IWMP. Without having regulated instream reservoirs, baseflow estimates can be assumed to be representative of groundwater discharge contributions after accounting for any inputs from wastewater treatment plants. An estimated baseflow range was calculated at each WSC gauge using a baseflow separation technique. These estimates are considered to be more representative of average annual groundwater discharge rates than the spot baseflow measurements. The CVC's spot baseflow measurements are recorded predominantly during the summer months, and as a result, would be lower than the expected average annual rates.

The calibration targets were used in model calibration with these specific considerations:

- Baseflow estimates at WSC gauges were assumed to accurately reflect groundwater discharge, and they were treated as key calibration targets. Spot baseflow estimates were assumed to be less accurate, but it was desired to have simulated groundwater discharge fall within the minimum and maximum range of observed values;
- Agreement between modelled and observed water levels was desired to ensure that groundwater flow directions were generally consistent with those reflected by observed data;
- Traditional statistical measures of calibration are minimized including mean error of residuals, mean absolute error of residuals, and normalized root mean square; and
- The differences between simulated and observed water levels at monitoring wells were plotted as 'spatial residuals' across the watershed. A key calibration goal was to minimize spatial trends in these residuals to validate the conceptual model. At the end of the calibration process, spatial trends in residuals suggest areas within the watershed where further local characterization and calibration is required.

Calibration results suggest that on average, the model represents observed water level conditions within standardized and accepted statistical measures of calibration. These standard statistical measures of calibration are summarized below:

- **Mean Error:** the Mean Error (ME) is determined by calculating the mean of all calibration residuals. A calibration residual is calculated as the difference between each simulated and observed water level. The Mean Error should be as close to '0' as possible;
- **Mean Absolute Error**: the Mean Absolute Error (MAE) is calculated by taking the mean of the absolute value of all calibration residuals. The MAE should be as close to '0' as possible. This statistic provides an indication of general trends of all water levels being high or low. This may be diagnostic of large scale trends such as globally high or low recharge;
- **Root Mean Squared Error**: the Root Mean Squared (RMS) also referred to as the 'Standard Error' is calculated by taking the square root of the sum of the squares of the residuals divided by the number of points. A basic calibration goal is to minimize the RMS error to the lowest practical number. Uncertainties associated with water level targets, scale of a model, and the implementation of the geological conceptual model will constrain the lowest level of error achievable; and
- **Normalized Root Mean Squared Error**: the Normalized Root Mean Squared (NRMS) error is calculated by dividing the RMS by the difference between the maximum and minimum observed water levels.

The model's ability to represent observed conditions was assessed both qualitatively, to assess trends in water levels and distribution of groundwater discharge, and quantitatively, to achieve acceptable statistical measures of calibration.

The resulting calibration demonstrates an acceptable match between observed and simulated groundwater levels and estimated and simulated baseflow measurements.

The calibrated and verified model represents the regional groundwater flow conditions very well and has been continuously updated since its original development.

When considering the entire water level target dataset, the results show that the calibration statistics appear to be consistent regardless of the location quality code criteria. When considering the ORMGP dataset, the RMS error is approximately 9 m and the normalized RMS error is approximately 2.1%. These calibration statistics are considered acceptable given the variability and uncertainty of the observed static water levels associated with water well record static water levels and the regional hydrogeological characterization. In general, it appears that the model tends to under-predict groundwater levels, with the mean error being negative.

Baseflow

The ability of the regional groundwater model to accurately simulate groundwater discharge is of critical importance given CVC's mandate to understand and manage coldwater fisheries. Two datasets representing baseflow calibration targets were derived including: estimated baseflow at WSC/HYDAT gauges and CVC's spot baseflow measurements taken throughout the watershed.

Table C1-8 summarizes the minimum and maximum estimated average annual baseflow rates and the calibrated groundwater discharge rates for the 1991-2000 period at the WSC/HYDAT gauges. As indicated by this table, and supporting analysis presented in the foundation document, the calibrated groundwater discharge rates agree with the estimated baseflow rates.

In general, the calibrated groundwater discharge rates are consistent with the range of observed values. As the spot baseflow measurements are typically collected during the summer months, they may underestimate the average annual baseflow at the monitoring stations. Therefore, the simulated groundwater discharge rates should fall within the upper range (or above) the observed spot baseflow values. Comparison of spot baseflow values with HYDAT data would help further refine the understanding of the baseflow contributions where the data affords such an assessment.

C1.4 Results

Table C1-9 summarizes precipitation, runoff, evapotranspiration, and groundwater recharge for each subwatershed within the CVSPA. For groundwater, the table summarizes recharge, discharge to surface water features, large, permitted water takings, inter-catchment flow, and inter-watershed flow.

Table C1-8: Summary of Baseflow Calibration at WSC Gauges (1991-2000)

		Flow Estimated (m3/s)			
WSC Station ID	Description	Minimum Baseflow Recession	Maximum Baseflow Recession	Average Baseflow Recession	FEFLOW GW Discharge (Corrected for STP flow)
02HB024	Black Creek	0.14	0.18	0.16	0.19
02HB020	West Credit / Erin	0.30	0.36	0.33	0.34
02HB013	Melville	0.33	0.41	0.37	0.47
02HB019	Shaw's Creek	0.49	0.61	0.55	0.44
02HB008	Silver Creek / Norval	0.69	0.91	0.80	0.88
02HB001	Cataract	1.24	1.48	1.36	1.29
02HB018	Boston Mills	2.97	3.56	3.26	2.94
02HB025	Credit R / Norval	4.16	5.16	4.66	4.35
02HB002	Erindale	5.08	6.71	5.90	4.93

Table C1-9: Integrated Water Budget Summary for the CVSPA

	Surface Water Flow Model (HSP-F)								Model Coupling		Groundwater Flow Model (FEFLOW) (m^3/d)				
Sub-watershed Name	Area (km^2)	Precipitation		Runoff		Evapotranspiration		Recharge		Surface Water		Permit-ted Wells	Inter-Catch-ment Flow	Inter-Water-shed Flow
		m^3/d	Mm/ year	m^3/d	Mm/ year	m^3/d	Mm/ year	m^3/d	Mm/ year	Streams/Wetlands	Lakes			
1. Loyalist Creek	9.8	21,000	778	11,200	415	8,600	318	1,200	45	-800	0	0	-400	-450
2. Carolyn Creek	5.6	11,900	778	5,900	385	5,500	363	500	30	-100	0	0	-400	0
3. Sawmill Creek	16.5	35,100	778	16,700	370	16,100	358	2,300	50	-1,500	0	0	-800	-100
4. Mullett Creek	32.9	70,200	778	35,800	397	29,700	329	4,700	52	-3,700	0	0	-1,000	0
5. Fletcher's Creek	42.5	90,600	778	40,300	346	45,300	389	5,000	43	-4,300	0	0	-700	-350
6. Levi Creek	24.7	52,700	778	19,800	293	27,200	401	5,700	84	-2,050	0	0	-3,650	800
7. Huttonville Creek	15.1	32,200	778	12,600	305	17,400	421	2,200	52	-1,400	0	0	-800	300
8a. Springbrook Tributary	4.8	10,200	778	3,900	295	5,500	421	800	62	-550	0	0	-250	0
8b Churchville Tributary	8.4	18,000	778	7,800	339	9,200	396	1,000	43	-400	0	0	-600	0
9. Norval to Port Credit	72.8	155,200	778	60,900	305	71,100	356	23,300	117	-33,400	-950	0	11,050	0
21. Lake Erie Tributaries	33.0	70,400	778	12,600	140	40,700	450	17,050	188	-7,300	-7,850	0	-1,900	-1,050
22. Lake Erie Tributaries	44.2	94,300	778	33,700	278	48,700	402	11,900	98	-2,400	-8,100	0	-1,400	-1,650
Lower Watershed	**310.5**	**661,800**	**778**	**261,200**	**307**	**325,000**	**382**	**75,650**	**89**	**-57,900**	**-16,900**	**0**	**-850**	**-2,500**
10. Black Creek	79.3	199,800	920	26,600	122	122,900	566	50,300	232	-36,500	0	-8,800	-5,000	3,450
11. Silver Creek	48.8	121,400	909	21,700	162	72,900	546	26,800	201	-25,750	0	-6,500	5,450	-200
12. Chelham /GWilliams	62.1	132,400	779	28,800	170	69,900	411	33,700	198	-35,300	0	0	1,600	850
13. East Credit R.	50.6	124,300	897	29,100	210	60,100	434	35,100	253	-38,550	0	-500	3,950	750
14. Glen Williams/Norval	23.1	49,300	778	21,000	331	25,000	394	3,400	53	-3,200	0	0	-200	-50
20. Credit R. - Forks of the Credit to Cheltenham	46.0	112,800	894	34,200	271	56,000	444	22,600	179	-36,600	0	-550	14,550	2,100
Middle Watershed	**309.9**	**740,000**	**863**	**161,400**	**190**	**406,800**	**444**	**171,900**	**179**	**-175,900**	**0**	**-16,350**	**20,350**	**6,900**
15. West Credit River	105.6	258,700	894	40,200	139	126,300	437	92,100	319	-86,400	0	-6,350	650	1,950
16. Caledon Creek	52.0	127,700	897	19,400	136	64,300	451	44,000	309	-22,100	2,100	-3,700	-20,300	-3,000
17. Shaw's Creek	72.0	176,800	896	31,400	159	86,900	440	58,500	297	-49,350	150	-3,500	-5,800	4,050
18. Melville to Forks of the Credit	39.2	96,100	895	13,500	125	45,800	426	36,800	343	-49,050	0	-1,350	13,600	0
19. Orangeville	59.8	143,600	876	27,400	167	68,500	418	47,700	291	-29,200	500	-7,000	-12,000	-14,350
Upper Watershed	**328.6**	**802,900**	**892**	**131,900**	**147**	**391,800**	**435**	**279,100**	**310**	**-236,100**	**2,750**	**-21,900**	**-23,850**	**-11,350**
Entire Watershed	**948.9**	**2,204,700**	**848**	**554,500**	**213**	**1,123,600**	**432**	**526,650**	**203**	**-469,900**	**-14,150**	**-38,250**	**-4,350**	**-6,950**

C1.5 Demand Estimation

A subwatersheds potential for stress is estimated by comparing the amount of water consumed with the amount of available water. The Percent Water Demand is calculated using the following formula (MOE, 2008):

$$\textit{Percent Water Demand} = \frac{Q_{DEMAND}}{Q_{SUPPLY} - Q_{RESERVE}} \times 100\%$$

where Q_{DEMAND} is equal to the consumptive demand calculated as the estimated rate of locally consumptive takings. Q_{SUPPLY} is the water supply term, calculated for surface water as the monthly median flow for the area to be assessed and for groundwater supplies as the estimated annual recharge rate plus the estimated groundwater inflow into a subwatershed. $Q_{RESERVE}$ is the water reserve, defined as the specified amount of water that does not contribute to the available water supply. For surface water supplies, reserve was estimated using the 90th percentile monthly median flow, at a minimum (i.e., the flow that is equalled or exceeded 90% of the time). Groundwater reserve was calculated as 10% of the total groundwater discharge to streams and wetlands in each subwatershed.

For surface water systems, the equation is carried out using monthly estimates. The maximum Percent Water Demand for all months is then used to categorize the Surface Water Quantity Potential for Stress into one of three levels; high, moderate, or low, as listed on **Table C1-10**.

Table C1-10: Surface Water Potential Stress Thresholds

Surface Water Potential Stress Level Assignment	Maximum Monthly % Water Demand
Significant	> 50%
Moderate	20% - 50%
Low	<20 %

For groundwater systems, the stress assessment calculation is carried out for the average annual demand conditions and for the monthly maximum demand conditions. The stress level is categorized again into three levels (high, moderate, or low) using thresholds listed on **Table C1-11.**

Table C1-11: Groundwater Potential Stress Thresholds

Groundwater Potential Stress Level Assignment	Average Annual	Monthly Maximum
Significant	> 25%	> 50%
Moderate	> 10%	> 25%
Low	0 – 10%	0 – 25%

Subwatersheds are classified as having a ‘significant’ or ‘moderate’ potential for hydrologic stress so the subwatersheds with a higher probability of experiencing water quantity related environmental impacts can be studied in greater detail (Tier 3) than those with a lower probability of impact.

The Tier 3 studies improve the understanding of potential impacts of demands on municipal drinking water sources at the localized level. Subwatersheds identified as having a ‘low’ potential for stress are not likely to be affected by water takings under current water taking, and additional analysis (Tier 3 level) is not required unless increased or additional water takings move the subwatershed into a higher stress class (e.g., ’moderate’ or ’significant’ potential for hydrologic stress).

Subwatersheds classified as ‘moderate’ or ’high’, but where there are no existing or planned municipal water systems, are likewise omitted from further study.

C1.6 Demand Assessment and Consumptivity

Understanding that consumptive use is dependent on the scale of the assessment, different consumptive factors for three different spatial scales were utilized in the analyses:

Consumptive with respect to the source—If water was removed by a source and not returned to the same unit as it was withdrawn, the taking was assumed to be 100% consumptive. Groundwater takings usually fall into this category, where it is common for water to be taken from a deep groundwater aquifer and returned to a surface water feature. If the water was returned to the same source, as well as the same sub-basin, the purpose specific consumptive factor was used; certain surface water takings can fall into this category.

Consumptive with respect to the subwatershed—If water was taken and not returned to a water body within that subwatershed, it was assumed to be 100% consumptive at the scale of the sub-basin. Municipal supply wells drawing water from one particular subwatershed and discharging via wastewater effluent to another would be considered 100% consumptive at this scale. If the water was returned within the same subwatershed, the purpose specific consumptive factor was used. Dewatering operations, which extract groundwater to lower local water levels, then discharge this water to the local surface water system, would be assigned a consumptive factor specific to dewatering operations.

Consumptive with respect to the watershed—If water was removed from the watershed and not returned within the watershed, it was assumed to be 100% consumptive. Water bottling operations would fall into this category. All other types of water taking operations would be assigned consumptive factors specific to their purpose of taking.

C1.6.1 Summary of Permitted Demand in CVSPA

Permits to Take Water

The Ministry of the Environment and Climate Change’s Permit to Take Water (PTTW) Program has been in place since the early 1960’s. The MOECC requires that any person taking more than 50,000 L/day, on any given day in a year, is required to hold an active PTTW. Exceptions are granted for domestic water use, livestock watering and water taken for firefighting purposes. Information such as geographic location of the source, maximum permitted volumes, and the general and specific purposes of the water takings are stored within the PTTW database. Permitted surface water and groundwater takings in the CVSPA are shown in **Figure C1-3**.

Water Use Project 2006

The CVC partnered with the MOECC Central Region to initiate the Credit River Watershed Water Use Assessment Project in 2006 (MOE, 2006). The purpose of the project was to obtain accurate and up-to-date non-municipal water taking values from the Credit River watershed. This information was intended to improve compliance for PTTW purposes; where any non-municipal water taking over 50,000 L/day must be regulated and approved by the MOECC. The collection of these data was also intended to aid the CVC in their water budget and modelling initiatives.

The project involved the identification of potential water use sites within the watershed. Each site was contacted either by telephone, mail, fax or e-mail with the goal of estimating water taking status through the completion of a questionnaire. A total of 40 questionnaires were returned to the project team for water taking activities relating to 27 unique PTTWs.

Municipal Demand

Table C1-12 lists the reported pumping rates (2007) for each permitted municipal drinking water well, except for those in Subwatershed 16, which were updated for 2008. The table also lists the estimated actual pumping rates for several of the wells (Permit To Take Water database).

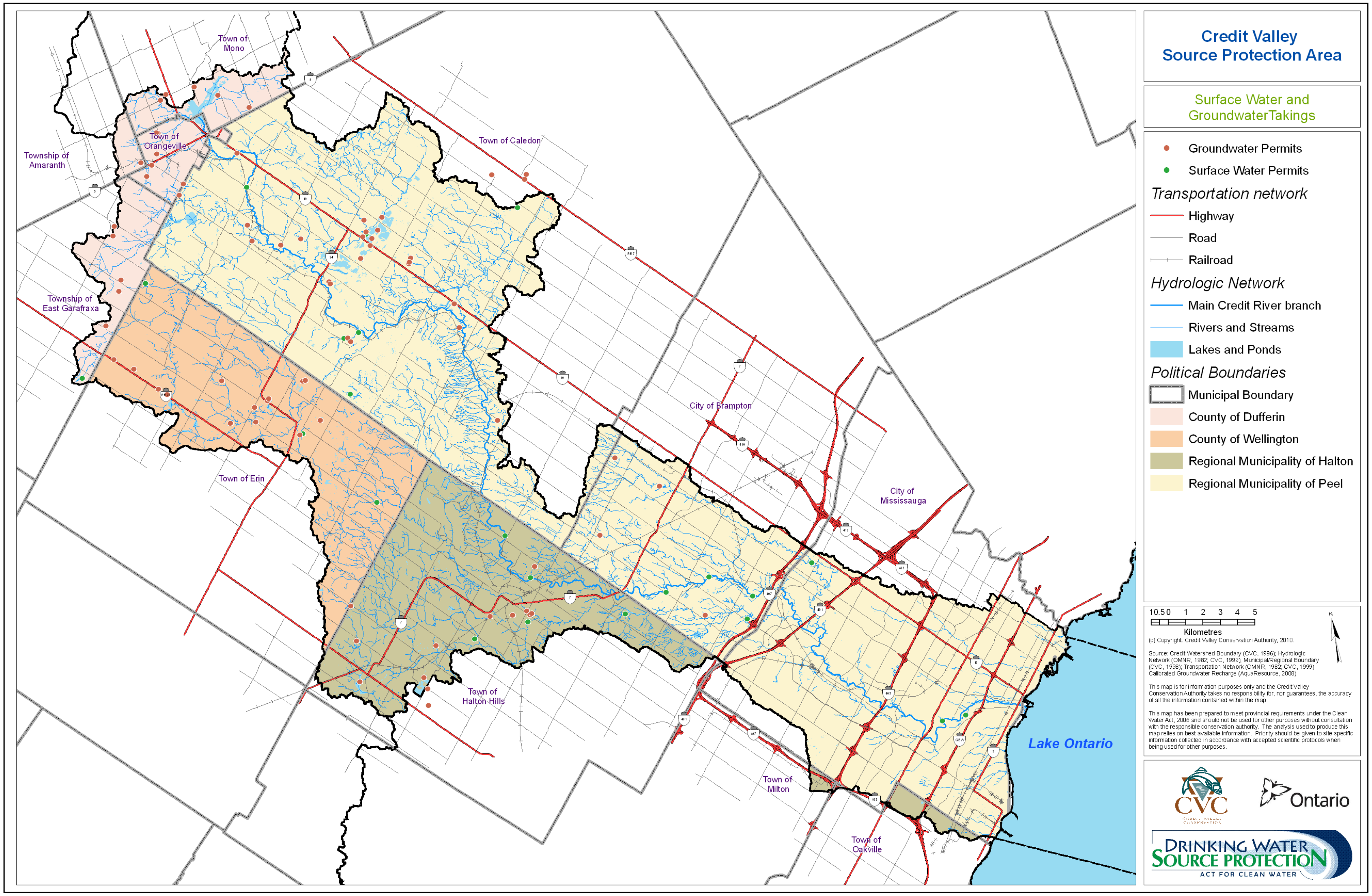


Figure C1-3: Credit Valley Source Protection Area – Permitted Water Takings

Table C1-12: Summary of Municipal Pumping Wells

Well Name	Easting (NAD83)	Northing (NAD 83)	Reported Average Day Pumping (m^3/d)	Permitted Pumping Rate (m^3/d)	Aquifer	Pumping Rate Source	Subwatershed
Region of Peel / Town of Caledon							
Alton Well 3/ 4	575070	4857132	211[1]	1,046	OB[2]	AquaResource, 2007	17
Caledon Village Well 3/3A	581788	4855688	182[4]	1,964	OB[2]	AquaResource, 2007	16
Caledon Village Well 4	576670	4856617	542[4]	3,273	OB[2]	AquaResource, 2007	16
Cheltenham Well PW1/ PW2	587284	4844416	250[1]	1,469	OB[2]	AquaResource, 2007	20
Inglewood Well 2	586043	4850102	73[1]	1295	OB[2]	AquaResource, 2007	20
Inglewood Well 3	585994	4851504	245[1]	1,295	OB[2]	AquaResource, 2007	20
Inglewood Well 4			N/A	1,296	OB[2]	Matrix, 2018	20
Town of Erin							
Erin Well 7	573556	4847599	734	2,586	BR[3]	CVC, 2007a	15
Erin Well 8	573466	4846759	648	2,357	BR[3]	CVC, 2007a	15
Hillsburgh Well H2	568676	4849209	216	982	OB[2]	CVC, 2007a	15
Hillsburgh Well H3	568233	4849607	216	655	OB[2]	CVC, 2007a	15
Region of Halton							
Cedarvale Well 1A	587035	4833232	816	2,618	OB[2]	CVC, 2007a	11
Cedarvale Well 3A	587196	4832986	564	3,931	OB[2]	CVC, 2007a	11
Cedarvale Well 4	587293	4833040	N/A	7,855	OB[2]		11
Cedarvale Well 4A	587293	4833040	3,156	5,891	OB[2]	CVC, 2007a	11
Princess Anne Well 5	586186	4833157	N/A	4,582	OB[2]		11
Princess Anne Well 6	586186	4833157	1,967	13,091	OB[2]	CVC, 2007a	11
Lindsay Court Well	584902	4833304	4,022	6,544	OB[2]	CVC, 2007a	10
4th Line Well A	577038	4835290	1,031	1,309	BR[3]	CVC, 2007a	10
Davidson Well 1	577011	4833241	780	1,250	BR[3]	CVC, 2007a	10
Davidson Well 2			780	1,250			
Prospect Park Well	576804	4830877	1,344	4,546	OB[2]	CVC, 2007a	10
Town of Orangeville / Mono							
Mono Cardinal Woods Well 1	571266	4866092	9	818	BR[3]	CVC, 2007a	19
Mono Cardinal Woods Well 3	571646	4866369	251	1,571	BR[3]	CVC, 2007a	19
Mono Coles Well 1/2	576251	4864785	138	570	OB[2]	CVC, 2007a	19
Mono Island Lake Well 1 - PW1			N/A	1,958	OB[2]		19

Well Name	Easting (NAD83)	Northing (NAD 83)	Reported Average Day Pumping (m^3/d)	Permitted Pumping Rate (m^3/d)	Aquifer	Pumping Rate Source	Subwatershed
Mono Island Lake Well 2 - TW1	574610	4865780	26	821	OB[2]	CVC, 2007a	19
Orangeville Well 2A	570126	4862459	354	1,308	BR[3]	CVC, 2007a	19
Orangeville Wells 5-5A	569536	4862719	2,511	6,000	OB[2]	CVC, 2007a	19
Orangeville Well 6	571383	4860475	1,578	3,600	BR[3]	CVC, 2007a	17
Orangeville Well 7	570508	4863057	867	1,310	BR[3]	CVC, 2007a	19
Orangeville Well 8A	570816	4864119	252	655	BR[3]	CVC, 2007a	19
Orangeville Well 8B	570788	4864232	408	655	BR[3]	CVC, 2007a	19
Orangeville Well 8C	570716	4864269	579	655	BR[3]	CVC, 2007a	19
Orangeville Well 9A-9B	569718	4861869	1,254	2,952	BR[3]	CVC, 2007a	19
Orangeville Well 10	575103	4862304	622	1,453	OB[2]	CVC, 2007a	19
Orangeville Well 11	571726	4861089	711	1,309	BR[3]	CVC, 2007a	17
Total			**27,960**	**95,424**			

1 Estimates cited for the Region of Peel represent the reported average pumping in the respective wells for the year 2006.

2 OB denotes municipal wells screened in an unconsolidated overburden aquifer

3 BR denotes municipal wells screened in a Paleozoic bedrock aquifer

4 Estimates cited for the Region of Peel represent the reported average pumping in the respective wells for the year 2007.

Non-Municipal

Table C1-13 and **Table C1-14** show permitted non-municipal takings (2007) of groundwater and surface water, respectively:

Table C1-13: Non-Municipal Permitted Groundwater Takings

Subwatershed #	Permit	Purpose	Pumping / Consumption Rate (m³/d)				
			Permit	Pumped	Source[1]	Consumption Rate	
						Average	Max
5	5821-6G7V8B	Pits and Quarries	2,455	2,455	Est	143	246
5	96-P-3031	Aggregate Washing	3,273	3,273	Est	191	327
6	1358-6LKMVL / 64-P-249	Agriculture Irrigation	910	392	Rep	161	354
7	98-P-3006	Aggregate Washing	2,864	1,080	Rep	72	108
10	02-P-3087	Pits and Quarries	4,582	n/a	Rep	1,000	1,000
10	2888-6M4HEG	Aggregate Washing	9,819	9,819	Rep	655	982
13	5236-6K3L8S	Golf Course Irrigation	851	851	Est	248	596
13	5236-6K3L8S	Golf Course Irrigation	458	458	Est	134	321
13	5236-6K3L8S	Golf Course Irrigation	458	458	Est	134	321
14	1470-6LBQSN	Golf Course Irrigation	9,083	13	Rep	13	13
15	01-P-2192	Schools	100	100	Est	100	100
15	6046-6GNKQD	Aquaculture	3,273	0	Rep	0	0
15	8351-6MGLYG	Aggregate Washing	7,816	7,816	Est	456	782
15	8357-6MGLYG	Aggregate Washing	10,421	10,421	Rep	1,042	1,042
15	96-P-2024	Groundwater	1,370	685	Est	685	685
15	96-P-2024	Groundwater	1,370	685	Est	684	684
15	97-P-2048	Aggregate Washing	2,288	2,288	Est	229	229
15	97-P-2048	Aggregate Washing	982	982	Est	98	98
15	98-P-2029	Bottled Water	225	225	Est	225	225
15	98-P-2106	Aggregate Washing	2,030	2,030	Rep	1,015	2,030
16	0528-75EPX9	Aggregate Washing	3,273	3,273	Est	251	327
16	02-P-3122	Aggregate Washing	26,186	19432	Est	314	972
16	8248-6E4QWB	Aggregate Washing	3,456	1,855	Rep	701	1,855
16	8248-6E4QWB	Aggregate Washing	21	10	Rep	8	10
16	8248-6E4QWB	Aggregate Washing	19,613	12,065	Rep	0	0
16	8248-6E4QWB	Aggregate Washing	140	132	Rep	16	99
16	8248-6E4QWB	Aggregate Washing	19,613	0	Rep	0	0
16	8248-6E4QWB	Aggregate Washing	22,094	9,549	Rep	804	1,432
16	8248-6E4QWB	Aggregate Washing	8,208	468	Rep	40	70
17	2415-6GWQ44	Aggregate Washing	2,728	2,728	Rep	159	273

Subwatershed #	Permit	Purpose	Pumping / Consumption Rate (m^3/d)				
			Permit	Pumped	Source[1]	Consumption Rate	
						Average	Max
17	7034-5U6H84	Aggregate Washing	4,004	4,004	Est	234	400
17	7134-6LJKT4	Aggregate Washing	73	73	Est	4	7
17	73-P-0480	Other - Agricultural	1,210	1,000	Rep	250	1,000
17	92-P-2054	Aggregate Washing	5,940	5,940	Est	347	594
18	01-P-3035	Snowmaking	114	114	Rep	114	114
18	01-P-3035	Snowmaking	86	86	Rep	86	86
18	02-P-3020	Groundwater	84	84	Est	42	42
18	02-P-3020	Groundwater	84	84	Est	42	42
18	7072-6MASY6	Golf Course Irrigation	1,767	1,767	Est	412	1,237

Notes: [1](Est) – Pumped volumes estimated from known permit information. (Rep) Pumped volumes estimated from MOECC/CVC Water Use Assessment.

Table C1-14: Permitted Surface Water Takings

Subwatershed #	Permit	Purpose	Water Use (m³/d)			
				Estimated Consumptive Use		
			Permitted Rate	Avg.	Max	Source
5	5132-6BPQRC	Golf Course Irrigation	n/a	893	1,532	Rep
6	0146-6LMJBJ	Golf Course Irrigation	1,514	194	582	Rep
6	86-P-3022	Nursery	n/a	306	367	Rep
9	0030-6NAJ36	Golf Course Irrigation	2,180	115	277	Rep
9	0146-6LMJBJ	Golf Course Irrigation	1,514	890	2,671	Rep
9	1422-6KWUBJ	Golf Course Irrigation	4,908	1,145	3,436	Est
9	2813-6KWMGY / 65-P-659	Agriculture Irrigation	3,273	1,374	2,356	Rep
9	8060-6PFRWP	Golf Course Irrigation	2,724	636	1,907	Est
9	8060-6PFRWP	Golf Course Irrigation	n/a	836	1,254	Rep
10	1353-6MYPZJ	Golf Course Irrigation	2,319	284	567	Rep
10	1353-6MYPZJ	Golf Course Irrigation	1,375	4	8	Rep
11	6334-6GDJ4P	Wildlife conservation/ wetlands/ recreational	n/a	0	0	Rep
12	0065-6MYNFT	Nursery production/ Irrigation	n/a	1,221	1,832	Rep
13	00-P-3017	Other - Construction	606	454	454	Est
14	1470-6LBQSN	Golf Course Irrigation	2,592	197	394	Rep
15	1742-6G6MGQ	Wildlife conservation	8,208	0	0	Rep
15	6046-6GNKQD	Aquaculture	10,015	0	0	Rep
16	01-P-3005	Other - Remediation	19,613	0	0	Rep
16	97-P-3030	Wildlife Conservation	22,094	0	0	Rep
17	3025-6GEH98	Wildlife conservation / wetlands/ recreational	n/a	0	0	Rep
17	4727-6G7HQN	Wildlife conservation/ wetlands/ recreational	n/a	0	0	Rep
18	01-P-3035	Snowmaking	2,726	170	681	Rep
18	01-P-3035	Snowmaking	2,726	170	681	Rep
18	01-P-3035	Snowmaking	n/a	0	0	Rep
18	7072-6MASY6	Golf Course Irrigation	6,546	2,182	6,546	Est
18	7072-6MASY6	Golf Course Irrigation	6,546	0	0	Est
18	7072-6MASY6	Golf Course Irrigation	8,510	0	0	Est
18	7072-6MASY6	Golf Course Irrigation	13,092	0	0	Est
19	99-P-3042	Golf Course Irrigation	594	139	416	Est

Notes: [1](Est) – Pumped volumes estimated from known permit information. (Rep)- Pumped volumes estimated from MOECC/CVC Survey.

Unserviced Rural

The unserviced rural (domestic) water demand was also derived by estimating the number of unserviced residences in each subwatershed and then estimating the average water use for each unserviced residence. The number of unserviced residences was estimated using mapping of rural septic systems prepared by the CVC (2007). The location of rural septic systems was determined using detailed orthophotos across the unserviced watersheds; it was assumed in the assessment that a septic system existed at each dwelling located within an unserviced area. The location of the septic system was added to a GIS and mapped spatially across the CVSPA.

Each rural unserviced lot was assumed to have one active domestic well. The total number of active domestic wells was then estimated for each sub-watershed. The number of domestic wells was then multiplied by the average number of persons occupying each rural lot and by the domestic per capita water use.

The Official Plan for the Town of Erin (2007) estimated 2.90 persons per unit in the Town of Erin, while Beatty and Associates (2003) estimated 3.2 persons per unit in the Regional Municipality of Peel. To be conservative, 3.2 persons were assumed to reside in each rural lot. A domestic per capita water use of 335 L/day per person was used. In addition, a consumptive use coefficient equal to 0.2 (20%) represents the fact that most rural domestic drinking water is returned to groundwater through septic systems.

These values produced the unserviced rural water demand per subwatershed as shown in **Table C1-15**.

Table C1-15: Unserviced Domestic Water Demand

Subwatershed		No. Septic Systems	Estimated Unserviced Water Demand	
No.	Name		Total (m^3/d)	Consumptive (m^3/d)
1	Loyalist Creek	5	5	1
2	Carolyn Creek	0	0	0
3	Sawmill Creek	11	12	2
4	Mullett Creek	65	70	14
5	Fletcher's Creek	81	87	17
6	Levi Creek	225	241	48
7	Huttonville Creek	138	148	30
8a	Springbrook Tributary	85	91	18
8b	Churchville Tributary	33	35	7
9	Norval to Port Credit	470	504	101
10	Black Creek	1,048	1,123	225
11	Silver Creek	704	755	151
12	Credit River - Cheltenham to Glen Williams	580	622	124
13	East Credit River	405	434	87
14	Credit River - Glen Williams to Norval	197	211	42
15	West Credit River	1,188	1,274	255
16	Caledon Creek	322	345	69
17	Shaw's Creek	438	470	94
18	Credit River - Melville to Forks of the	297	318	64
19	Orangeville	443	475	95
20	Credit River - Forks of the Credit to Cheltenham	490	525	105
21	Lake Erie Tributaries	0	0	0
22	Lake Erie Tributaries	0	0	0
	Total	**7,225**	**7,745**	**1,549**

The highest unserviced rural water demands are in the western portion of the watershed, namely the West Credit River, Black Creek, Silver Creek and Credit River – Cheltenham to Glen Williams subwatersheds. Norval to Port Credit Subwatershed has a fairly high rural water demand due to the unserviced area in the northern portion of the subwatershed; the middle to lower portions of the subwatershed are serviced by Lake Ontario surface water.

C1.6.2 Consumptivity

Consumptive Use Factors

Table C1-16 provides a list of suggested consumptive use factors (AquaResource, 2005) that were used for water takings where water is returned to the same source from which it is taken. These default values correspond to the 'Specific Purpose' assigned by the MOECC to each permit.

Table C1-16: Subwatershed and Watershed Consumptive Use Factors

Category	Specific Purpose	Consumptive Factor (MOECC, 2007)	Category	Specific Purpose	Consumptive Factor
Agricultural	Field and Pasture Crops	0.80	Institutional	Hospitals	0.25
Agricultural	Fruit Orchards	0.80	Institutional	Other - Institutional	0.25
Agricultural	Market Gardens / Flowers	0.90	Institutional	Schools	0.25
Agricultural	Nursery	0.90	Miscellaneous	Dams and Reservoirs	0.10
Agricultural	Other - Agricultural	0.80	Miscellaneous	Heat Pumps	0.10
Agricultural	Sod Farm	0.90	Miscellaneous	Other-Miscellaneous	1
Agricultural	Tender Fruit	0.80	Miscellaneous	Pumping Test	0.10
Agricultural	Tobacco	0.90	Miscellaneous	Wildlife Conservation	0.10
Commercial	Aquaculture	0.10	Recreational	Aesthetics	0.25
Commercial	Bottled Water	1	Industrial	Manufacturing	0.25
Commercial	Golf Course Irrigation	0.70	Industrial	Other - Industrial	0.25
Commercial	Mall / Business	0.25	Industrial	Pipeline Testing	0.25
Commercial	Other - Commercial	1	Industrial	Power Production	0.10
Commercial	Snowmaking	0.50	Recreational	Fish Ponds	0.25
Construction	Other - Construction	0.75	Recreational	Other - Recreational	0.10
Construction	Road Building	0.75	Recreational	Wetlands	0.10
Dewatering	Construction	0.25	Remediation	Groundwater	0.50
Dewatering	Other - Dewatering	0.25	Remediation	Other - Remediation	0.25
Dewatering	Pits and Quarries	0.25	Water Supply	Campgrounds	0.20
Industrial	Aggregate Washing	0.25	Water Supply	Communal	0.20
Industrial	Brewing and Soft Drinks	1	Water Supply	Municipal Wells	0.20
Industrial	Cooling Water	0.25	Water Supply	Other - Water Supply	0.20
Industrial	Food Processing	1			

Monthly Usage Factors

Monthly estimates of water use and supply are required to evaluate the transient stress level within a subwatershed. Comparisons of annual water use and supply have been found to mask areas of potential water quantity issues since many of those occur during the low flow season. With the lack of temporal characterization within the PTTW database, additional information is required to arrive at monthly estimates.

These estimates can be used to help improve the estimation of the monthly water use in a watershed as this approach recognizes that many water taking facilities operate during specific periods of the year (e.g., snow making is generally active in the winter, while golf course irrigation is active in the summer months).

Table C1-17 outlines the months of the year that particular permits are expected to extract ground or surface water. The demand adjustments outlined in the table were taken from pre- empting Module 7 (Default Monthly Demand Adjustments) (MOE, 2007), and remains valid in respect of the *Technical Rules* (MOE, 2008).

Table C1-17: Monthly Demand Adjustments

General Purpose	Specific Purpose	Jan	Feb		Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Agricultural	Field and Pasture Crops	0	0		0	0	0	0	1	1	0	0	0	0
Agricultural	Fruit Orchards	0	0		0	0	0	0	1	1	0	0	0	0
Agricultural	Market Gardens/ Flowers	0	0		0	0	0	0	1	1	0	0	0	0
Agricultural	Nursery	0	0		0	0	0	0	1	1	0	0	0	0
Agricultural	Other-Agricultural	0	0		0	0	0	0	1	1	0	0	0	0
Agricultural	Sod Farm	0	0		0	0	0	1	1	1	1	0	0	0
Agricultural	Tender Fruit	0	0		0	0	0	0	1	1	0	0	0	0
Agricultural	Tobacco	0	0		0	0	0	0	1	1	0	0	0	0
Commercial	Aquaculture	1	1		1	1	1	1	1	1	1	1	1	1
Commercial	Bottled Water	1	1		1	1	1	1	1	1	1	1	1	1
Commercial	Golf Course Irrigation	0	0		0	0	0	1	1	1	1	0	0	0
Commercial	Mall/ Business	1	1		1	1	1	1	1	1	1	1	1	1
Commercial	Other-Commercial	1	1		1	1	1	1	1	1	1	1	1	1
Commercial	Snowmaking	1	1		0	0	0	0	0	0	0	0	0	1
Construction	Other-Construction	1	1		1	1	1	1	1	1	1	1	1	1
Construction	Road Building	1	1		1	1	1	1	1	1	1	1	1	1
Dewatering	Construction	1	1		1	1	1	1	1	1	1	1	1	1
Dewatering	Other-Dewatering	1	1		1	1	1	1	1	1	1	1	1	1
Dewatering	Pits and Quarries	1	1		1	1	1	1	1	1	1	1	1	1
Industrial	Aggregate Washing	0	0		0	0	1	1	1	1	1	1	1	0
Industrial	Cooling Water	1	1		1	1	1	1	1	1	1	1	1	1
Industrial	Food Processing	1	1		1	1	1	1	1	1	1	1	1	1
Industrial	Manufacturing	1	1		1	1	1	1	1	1	1	1	1	1
Industrial	Other-Dewatering	1	1		1	1	1	1	1	1	1	1	1	1
Industrial	Other-Industrial	1	1		1	1	1	1	1	1	1	1	1	1
Industrial	Pipeline Testing	1	1		1	1	1	1	1	1	1	1	1	1
Institutional	Other-Institutional	1	1		1	1	1	1	1	1	1	1	1	1
Institutional	Schools	1	1		1	1	1	1	0	0	1	1	1	1
Miscellaneous	Dams and Reservoirs	1	1		1	1	1	1	1	1	1	1	1	1
Miscellaneous	Heat Pumps	1	1		1	1	1	1	1	1	1	1	1	1
Miscellaneous	Other-Miscellaneous	1	1		1	1	1	1	1	1	1	1	1	1
Miscellaneous	Pumping Test	1	1		1	1	1	1	1	1	1	1	1	1
Miscellaneous	Wildlife Conservation	1	1		1	1	1	1	1	1	1	1	1	1
Missing	Missing	1	1		1	1	1	1	1	1	1	1	1	1
Recreational	Other-Recreational	1	1		1	1	1	1	1	1	1	1	1	1
Recreational	Wetlands	1	1		1	1	1	1	1	1	1	1	1	1
Remediation	Groundwater	1	1		1	1	1	1	1	1	1	1	1	1
Remediation	Other-Remediation	1	1		1	1	1	1	1	1	1	1	1	1
Water Supply	Campgrounds	0	0		0	0	1	1	1	1	1	0	0	0
Water Supply	Communal	1	1		1	1	1	1	1	1	1	1	1	1
Water Supply	Municipal	1	1		1	1	1	1	1	1	1	1	1	1
Water Supply	Other-Water Supply	1	1		1	1	1	1	1	1	1	1	1	1

For all types of water taking operations, other than agricultural, this table indicates when the taking is assumed to be active. During these times, it is assumed that water is being extracted every day within that month.

For agricultural permits a slight variation to this method was applied. Rather than assuming the taking was active every day during the particular month, the study team estimated the number of days per month that an irrigation permit would be actively taking water. The length of time the average irrigator requires to apply an irrigation event to the full crop is poorly understood. OMAFRA staff indicated that a farmer may take four to seven days to fully irrigate their crop (Rebecca Shortt, pers comm., 2006). Based on this, the study team conservatively assumed that for each irrigation event, the water taking is active for seven days. With four irrigation events occurring during the average year, this translates into 28 days in which the water taking would be active, over the two-month period of July and August.

Non-permitted agricultural demand was not assessed. It was assumed that non-permitted agricultural demand in the CVSPA would be quite small. Also, the size of the Credit subwatersheds would make it difficult to convert the larger-scale results using the methods of de Loë (2001, 2005).

C1.6.3 Stress Assessment - Current Scenario

Surface Water – Percent Water Demand and Potential Water Quantity Stress

The calculated monthly Percent Water Demand estimates (surface water) are presented in **Table C1-22**. These values were calculated using the water supply and reserve terms located in **Table C1-19** and **Table C1-20** and the consumptive demand estimates found in **Table C1-18**.

The monthly maximum Percent Water Demand is used to determine the potential stress classification, as per the thresholds presented in **Table C1-10**.

Table C1-18: Surface Water Unit Consumptive Demands (m^3/d)

Subwatershed		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	Loyalist Creek	-	-	-	-	-	-	-	-	-	-	-	-
2	Carolyn Creek	-	-	-	-	-	-	-	-	-	-	-	-
3	Sawmill Creek	-	-	-	-	-	-	-	-	-	-	-	-
4	Mullett Creek	-	-	-	-	-	-	-	-	-	-	-	-
5	Fletcher's Creek	-	-	-	1,532	1,532	1,532	1,532	1,532	1,532	1,532	-	-
6	Levi Creek	-	-	367	367	367	948	948	948	948	367	367	-
7	Huttonville Creek	-	-	-	-	-	-	-	-	-	-	-	-
8a	Springbrook Tributary	-	-	-	-	-	-	-	-	-	-	-	-
8b	Churchville Tributary	-	-	-	-	-	-	-	-	-	-	-	-
9	Norval to Port Credit	-	-	1,254	3,610	3,887	11,901	11,901	11,901	11,901	3,610	-	-
10	Black Creek	-	-	-	575	575	575	575	575	575	-	-	-
11	Silver Creek	-	-	-	-	-	-	-	-	-	-	-	-
12	Credit River - Cheltenham to Glen Williams	-	-	-	1,832	1,832	1,832	1,832	1,832	1,832	1,832	1,832	-
13	East Credit River	454	454	454	454	454	454	454	454	454	454	454	454
14	Credit River - Glen Williams to Norval	-	-	-	-	394	394	394	394	394	394	-	-
15	West Credit River	-	-	-	-	-	-	-	-	-	-	-	-
16	Caledon Creek	-	-	-	-	-	-	-	-	-	-	-	-
17	Shaw's Creek	-	-	-	-	-	-	-	-	-	-	-	-
18	Credit River - Melville to Forks of the Credit	1,363	1,363	-	-	-	6,546	6,546	6,546	6,546	-	-	1,363
19	Orangeville	-	-	-	-	-	416	416	416	416	-	-	-
20	Credit River - Forks of the Credit to Cheltenham	-	-	-	-	-	-	-	-	-	-	-	-

Table C1-19: Surface Water Supply (m^3/d)

Subwatershed		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	Loyalist Creek	7,120	7,680	8,740	9,240	6,980	5,010	3,960	3,370	2,740	3,190	5,420	5,870
2	Carolyn Creek	4,060	4,320	4,960	5,100	4,010	2,840	2,310	2,000	1,580	1,840	2,960	3,300
3	Sawmill Creek	12,400	13,200	14,700	15,600	12,400	8,710	7,180	6,210	5,000	5,800	9,250	10,100
4	Mullett Creek	22,400	24,900	28,400	28,800	22,000	16,200	12,900	10,700	8,450	10,200	18,000	18,900
5	Fletcher's Creek	32,400	31,800	34,400	37,900	30,500	21,900	17,800	15,700	12,700	13,700	20,300	24,300
6	Levi Creek	32,400	31,800	34,400	37,900	30,500	21,900	17,800	15,700	12,700	13,700	20,300	24,300
7	Huttonville Creek	9,650	8,510	9,660	10,800	8,660	5,900	4,680	4,390	3,600	3,580	4,760	6,630
8a	Springbrook Tributary	3,750	3,380	3,720	4,260	3,480	2,410	1,880	1,820	1,480	1,440	1,900	2,610
8b	Churchville Tributary	6,240	6,440	7,270	7,610	6,090	4,320	3,470	3,010	2,470	2,570	4,100	4,660
9	Norval to Port Credit	810,000	790,000	826,000	949,000	867,000	631,000	518,000	493,000	406,000	394,000	515,000	596,000
10	Black Creek	45,800	47,100	49,200	57,800	54,100	39,400	32,800	32,300	26,600	25,700	31,800	36,500
11	Silver Creek	101,000	103,000	112,000	132,000	119,000	95,800	81,800	76,500	67,600	65,500	78,900	85,000
12	Cheltenham to Glen Williams	469,000	449,000	473,000	570,000	529,000	373,000	289,000	303,000	243,000	240,000	287,000	351,000
13	East Credit River	57,100	53,400	57,300	69,100	63,400	44,800	34,000	35,700	28,200	27,600	34,100	43,200
14	Glen Williams to Norval	587,000	583,000	603,000	718,000	667,000	476,000	384,000	385,000	316,000	307,000	377,000	450,000
15	West Credit River	115,000	111,000	121,000	150,000	146,000	105,000	78,500	84,200	64,700	62,900	76,700	94,100
16	Caledon Creek	30,800	31,400	33,100	41,000	39,200	27,400	20,600	21,900	17,400	17,200	21,000	25,400
17	Shaw's Creek	54,200	52,700	55,800	67,400	67,200	47,700	36,000	38,300	29,300	28,700	34,500	43,800
18	Melville to Forks of the Credit	158,000	167,000	173,000	212,000	197,000	137,000	108,000	111,000	89,400	87,800	108,000	125,000
19	Orangeville	41,400	40,400	45,000	53,300	44,100	32,000	27,500	25,500	23,200	25,100	29,600	30,600
20	Forks of the Credit to Cheltenham	407,000	392,000	414,000	498,000	466,000	330,000	256,000	269,000	215,000	212,000	254,000	304,000

Table C1-20: Surface Water Reserve (m^3/d)

Subwatershed		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	Loyalist Creek	2,600	3,330	3,660	3,750	4,040	3,180	2,340	1,750	1,090	1,380	1,600	2,610
2	Carolyn Creek	1,580	1,900	2,150	2,140	2,430	1,900	1,420	1,060	722	880	1,010	1,580
3	Sawmill Creek	4,880	5,910	6,560	6,520	7,530	5,910	4,360	3,220	2,200	2,740	3,180	4,890
4	Mullett Creek	8,310	10,500	11,200	11,000	13,100	10,200	7,350	5,430	3,460	4,530	5,160	8,000
5	Fletcher's Creek	12,700	14,600	15,700	15,700	18,200	15,100	10,800	8,270	6,210	7,520	8,570	12,700
6	Levi Creek	12,700	14,600	15,700	15,700	18,200	15,100	10,800	8,270	6,210	7,520	8,570	12,700
7	Huttonville Creek	3,740	3,520	4,310	4,000	4,350	4,340	2,620	2,390	1,980	2,270	2,440	3,630
8a	Springbrook Tributary	1,460	1,420	1,740	1,560	1,620	1,720	1,020	929	759	896	919	1,440
8b	Churchville Tributary	2,450	2,850	3,070	3,090	3,640	2,960	2,150	1,610	1,190	1,430	1,640	2,380
9	Norval to Port Credit	355,000	367,000	455,000	409,000	511,000	444,000	325,000	258,000	211,000	264,000	282,000	382,000
10	Black Creek	22,800	22,200	29,000	25,100	31,300	27,900	20,700	18,100	15,600	18,000	18,800	24,900
11	Silver Creek	57,600	57,200	68,600	66,500	76,400	65,100	58,700	57,800	51,800	50,400	50,200	62,500
12	Cheltenham to Glen Williams	215,000	200,000	272,000	231,000	244,000	256,000	155,000	140,000	119,000	143,000	139,000	227,000
13	East Credit River	24,600	23,400	31,200	28,100	27,900	26,400	16,700	14,100	11,900	14,100	13,200	25,900
14	Glen Williams to Norval	282,000	261,000	354,000	303,000	338,000	337,000	222,000	202,000	177,000	200,000	223,000	296,000
15	West Credit River	55,200	51,000	70,700	60,700	64,800	60,100	38,200	31,000	26,000	31,300	29,400	54,300
16	Caledon Creek	14,800	13,700	19,100	16,500	17,100	17,200	11,000	7,480	6,580	9,290	9,090	15,500
17	Shaw's Creek	26,100	25,300	33,700	28,400	28,200	25,800	17,000	12,800	11,300	13,500	13,200	22,500
18	Melville to Forks of the Credit	79,500	77,400	103,000	88,300	106,000	96,900	65,700	55,600	49,100	57,100	66,400	89,800
19	Orangeville	20,200	20,300	21,800	28,600	27,100	20,800	18,000	17,200	15,600	16,300	18,300	21,700
20	Forks of the Credit to Cheltenham	191,000	178,000	240,000	203,000	217,000	231,000	140,000	126,000	108,000	127,000	131,000	201,000

Table C1-21: Surface Water Supply Minus Reserve (m^3/d)

Subwatershed		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	Loyalist Creek	4,520	4,350	5,080	5,490	2,940	1,830	1,620	1,620	1,650	1,810	3,820	3,260
2	Carolyn Creek	2,480	2,420	2,810	2,960	1,580	940	890	940	858	960	1,950	1,720
3	Sawmill Creek	7,520	7,290	8,140	9,080	4,870	2,800	2,820	2,990	2,800	3,060	6,070	5,210
4	Mullett Creek	14,090	14,400	17,200	17,800	8,900	6,000	5,550	5,270	4,990	5,670	12,840	10,900
5	Fletcher's Creek	19,700	17,200	18,700	22,200	12,300	6,800	7,000	7,430	6,490	6,180	11,730	11,600
6	Levi Creek	19,700	17,200	18,700	22,200	12,300	6,800	7,000	7,430	6,490	6,180	11,730	11,600
7	Huttonville Creek	5,910	4,990	5,350	6,800	4,310	1,560	2,060	2,000	1,620	1,310	2,320	3,000
8a	Springbrook Tributary	2,290	1,960	1,980	2,700	1,860	690	860	891	721	544	981	1,170
8b	Churchville Tributary	3,790	3,590	4,200	4,520	2,450	1,360	1,320	1,400	1,280	1,140	2,460	2,280
9	Norval to Port Credit	455,000	423,000	371,000	540,000	356,000	187,000	193,000	235,000	195,000	130,000	233,000	214,000
10	Black Creek	23,000	24,900	20,200	32,700	22,800	11,500	12,100	14,200	11,000	7,700	13,000	11,600
11	Silver Creek	43,400	45,800	43,400	65,500	42,600	30,700	23,100	18,700	15,800	15,100	28,700	22,500
12	Credit River - Cheltenham to Glen Williams	254,000	249,000	201,000	339,000	285,000	117,000	134,000	163,000	124,000	97,000	148,000	124,000
13	East Credit River	32,500	30,000	26,100	41,000	35,500	18,400	17,300	21,600	16,300	13,500	20,900	17,300
14	Credit River - Glen Williams to Norval	305,000	322,000	249,000	415,000	329,000	139,000	162,000	183,000	139,000	107,000	154,000	154,000
15	West Credit River	59,800	60,000	50,300	89,300	81,200	44,900	40,300	53,200	38,700	31,600	47,300	39,800
16	Caledon Creek	16,000	17,700	14,000	24,500	22,100	10,200	9,600	14,420	10,820	7,910	11,910	9,900
17	Shaw's Creek	28,100	27,400	22,100	39,000	39,000	21,900	19,000	25,500	18,000	15,200	21,300	21,300
18	Credit River - Melville to Forks of the Credit	78,500	89,600	70,000	123,700	91,000	40,100	42,300	55,400	40,300	30,700	41,600	35,200
19	Orangeville	21,200	20,100	23,200	24,700	17,000	11,200	9,500	8,300	7,600	8,800	11,300	8,900
20	Credit River - Forks of the Credit to Cheltenham	216,000	214,000	174,000	295,000	249,000	99,000	116,000	143,000	107,000	85,000	123,000	103,000

Table C1-22: Percent Water Demand Estimate (Surface Water)

	Subwatershed	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Max % Demand
1	Loyalist Creek	-	-	-	-	-	-	-	-	-	-	-	-	
2	Carolyn Creek	-	-	-	-	-	-	-	-	-	-	-	-	
3	Sawmill Creek	-	-	-	-	-	-	-	-	-	-	-	-	
4	Mullett Creek	-	-	-	-	-	-	-	-	-	-	-	-	
5	Fletcher's Creek	-	-	-	7%	12%	23%	22%	21%	24%	25%	-	-	**25%**
6	Levi Creek	-	-	2%	2%	3%	14%	14%	13%	15%	6%	3%	-	15%
7	Huttonville Creek	-	-	-	-	-	-	-	-	-	-	-	-	
8	Springbrook Tributary	-	-	-	-	-	-	-	-	-	-	-	-	
8b	Churchville Tributary	-	-	-	-	-	-	-	-	-	-	-	-	
9	Norval to Port Credit	-	-	0%	1%	1%	6%	6%	5%	6%	3%	-	-	6%
10	Black Creek	-	-	-	2%	3%	5%	5%	4%	5%	-	-	-	5%
11	Silver Creek	-	-	-	-	-	-	-	-	-	-	-	-	
12	Credit River - Cheltenham to Glen Williams	-	-	-	1%	1%	2%	1%	1%	1%	2%	1%	-	2%
13	East Credit River	1%	2%	2%	1%	1%	2%	3%	2%	3%	3%	2%	3%	3%
14	Credit River - Glen Williams to Norval	-	-	-	-	0%	0%	0%	0%	0%	0%	-	-	
15	West Credit River	-	-	-	-	-	-	-	-	-	-	-	-	
16	Caledon Creek	-	-	-	-	-	-	-	-	-	-	-	-	
17	Shaw's Creek	-	-	-	-	-	-	-	-	-	-	-	-	
18	Credit River - Melville to Forks of the Credit	2%	2%	-	-	-	16%	15%	12%	16%	-	-	4%	16%
19	Orangeville	-	-	-	-	-	4%	4%	5%	5%	-	-	-	5%
20	Credit River - Forks of the Credit to Cheltenham	-	-	-	-	-	-	-	-	-	-	-	-	

Notes:

Potential Hydrologic Stress greater than the Moderate Threshold (20%)

Groundwater – Percent Water Demand and Potential Water Quantity Stress

The groundwater stress assessment uses average annual groundwater supply and reserve for monthly stress calculations, and therefore, monthly values are not provided. Monthly values are then equivalent to average annual values expressed as a rate (m^3/day).

The *Technical Rules* requires that the maximum monthly stress be computed. This is calculated using the maximum monthly water demand and the average annual water supply and reserve. For the assessment, monthly water demand was calculated for each specific permit. However, the maximum monthly water demand for a subwatershed was conservatively calculated as the sum of all the maximum monthly consumptive demands for each of the permits in a subwatershed. This approach is conservative and consistent with the requirements of the *Technical Rules.*

Monthly water demand estimates were not available for all permitted wells. Only average annual and maximum monthly values were provided by permit holders, but these values were considered representative of actual pumping rates. Since monthly estimates were not available for all wells, the maximum monthly water demand for a subwatershed was calculated as the sum of all the maximum consumptive demands for each of the permits in a subwatershed. This approach is conservative in that all of the maximum monthly permitted rates for each subwatershed are not likely to occur within the same month and is consistent with the requirements of the *Technical Rules* because ultimately, they require that Percent Water Demand be calculated on an average and maximum and monthly basis.

C1.7 Stress Assessment - Drought Scenarios

The outputs of the Part A current drought scenario are shown in **Table C1-23**. The table shows the model predicted drawdown at each municipal well at the water table and the upper bedrock contact zone.

The predicted decrease in groundwater heads for the water table and upper bedrock surface (contact zone) are presented in **Figure C1-4** and **Figure C1-5**, respectively. The most significant drawdown resulting from the two-year simulated drought is observed in groundwater recharge areas. Drawdown decreases closer to wetlands and streams.

The contact zone drawdown contours are smoother than those predicted for the water table, and this is a result of the larger sensitivity to surface water in the shallow water table zone.

The *Technical Rules, 2009*—Table 1, Scenario E—specify that the drought scenarios be undertaken for both the current and future periods. However, municipal pump rates could not be procured for the future (Scenario E), and, as such, the drought scenarios were not undertaken for that period.

Table C1-23: Part A Drought Scenarios

Well Name	Easting	Northing	Aquifer[1]	Drawdown (m)	
				Water Table	Contact Zone
Caledon Village Well 3/3A	581788	4855688	OB	1.4	1.4
Caledon Village Well 4	576670	4856617	OB	2.2	0.8
Caledon East Well 2 (in TRCA)	591269	4857898	OB	1.6	1.7
Caledon East Well 3 (in TRCA)	591402	4858178	OB	1.6	1.5
Caledon East Well 4 (in TRCA)	589439	4858503	OB	1.4	1.4
Cheltenham Well 1	587284	4844416	OB	0.2	0.2
Cheltenham Well 2	587284	4844416	OB	0.2	0.2
Inglewood Well 2	586043	4850102	OB	0.4	0.3
Inglewood Well 3	585994	4851504	OB	0.2	0.2
Erin Well 7	573556	4847599	BR	1.6	1.6
Erin Well 8	573466	4846759	BR	0.8	0.8
Hillsburgh Well H2	568676	4849209	OB	0.9	0.9
Hillsburgh Well H3	568233	4849607	OB	2.1	2.0
Halton Cedarvale Well 1A	587035	4833232	OB	0.6	0.6
Halton Cedarvale Well 3	587196	4832986	OB	0.5	0.5
Halton Cedarvale Well 4	587293	4833040	OB	0.5	0.5
Halton Cedarvale Well 4A	587293	4833040	OB	0.5	0.5
Halton Princess Anne Well 5	586186	4833157	OB	0.0	2.1
Halton Princess Anne Well 6	586186	4833157	OB	0.0	2.1
Halton Lindsay Court Well	584902	4833304	OB	2.4	2.2
Halton 4th Line Well A	577038	4835290	BR	2.4	2.1
Halton Davidson Well 1	577011	4833241	BR	1.7	1.7
Halton Davidson Well 2	577011	4833241	BR	1.7	1.7
Halton Prospect Park Well	576804	4830877	OB	0.3	0.3
Mono Cardinal Woods Well 1	571266	4866092	BR	1.0	1.0
Mono Cardinal Woods Well 3	571646	4866369	BR	2.6	1.5
Mono Coles Well 1	576251	4864785	OB	2.3	2.3
Mono Coles Well 2	576251	4864785	OB	2.3	2.3
Mono Island Lake Well 2-TW1	574610	4865780	OB	0.1	0.2
Observation Well 2A	570126	4862459	BR	1.0	1.1
Orangeville Wells 5-5A	569536	4862719	OB	3.6	3.5
Orangeville Well 6	571383	4860475	BR	1.5	1.4
Orangeville Well 7 – Passmore	570508	4863057	BR	1.3	1.3
Orangeville Well 8A	570816	4864119	BR	1.8	1.8
Orangeville Well 8B	570788	4864232	BR	1.8	1.8
Orangeville Well 8C	570716	4864269	BR	1.9	1.9
Orangeville Well 9A-9B	569718	4861869	BR	3.4	2.6
Orangeville Well 10	575103	4862304	OB	0.2	0.2
Orangeville Well 11	571726	4861089	BR	3.6	2.2

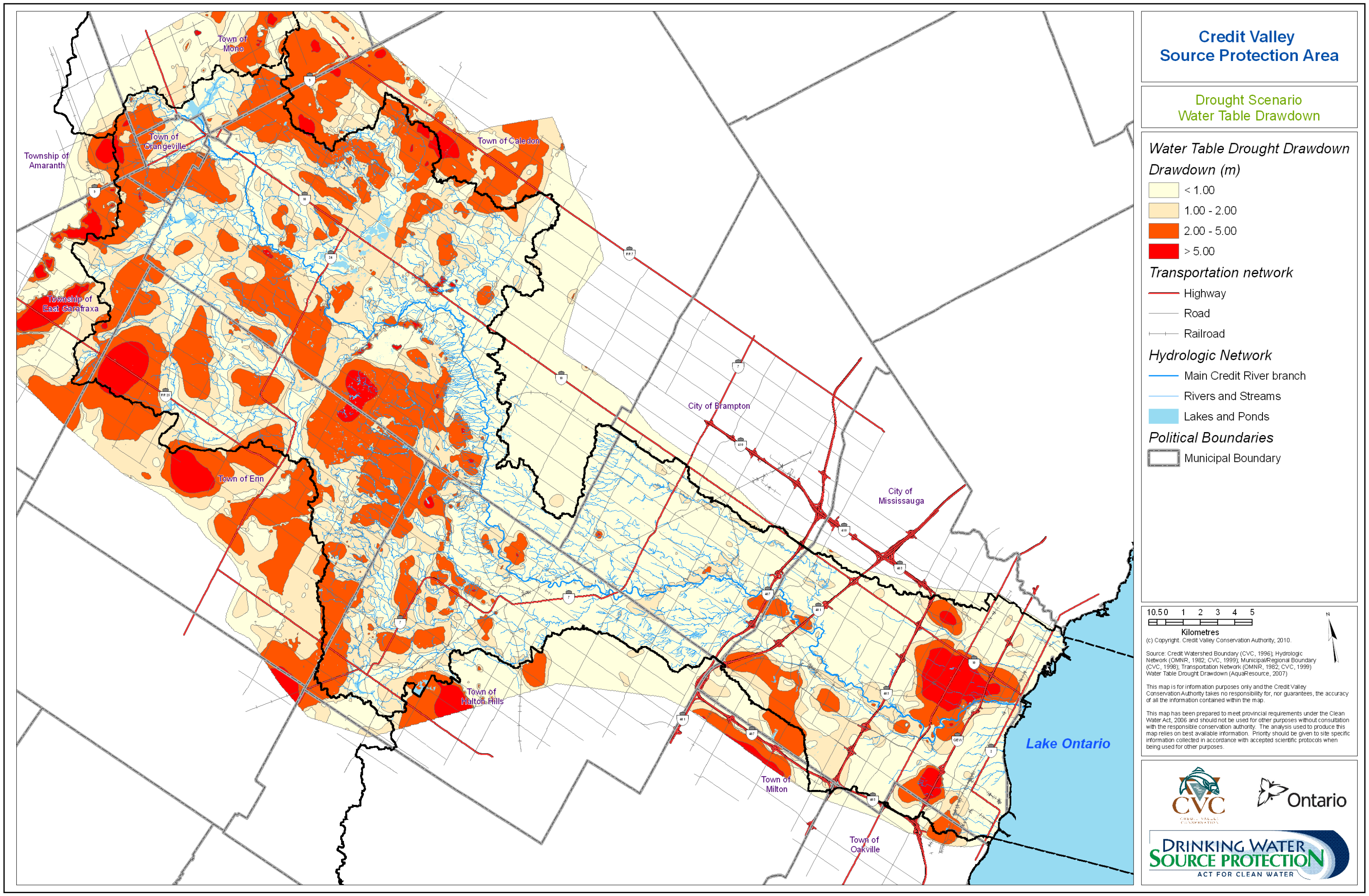


Figure C1-4: CVSPA –Drought Scenario – Water Table Drawdown

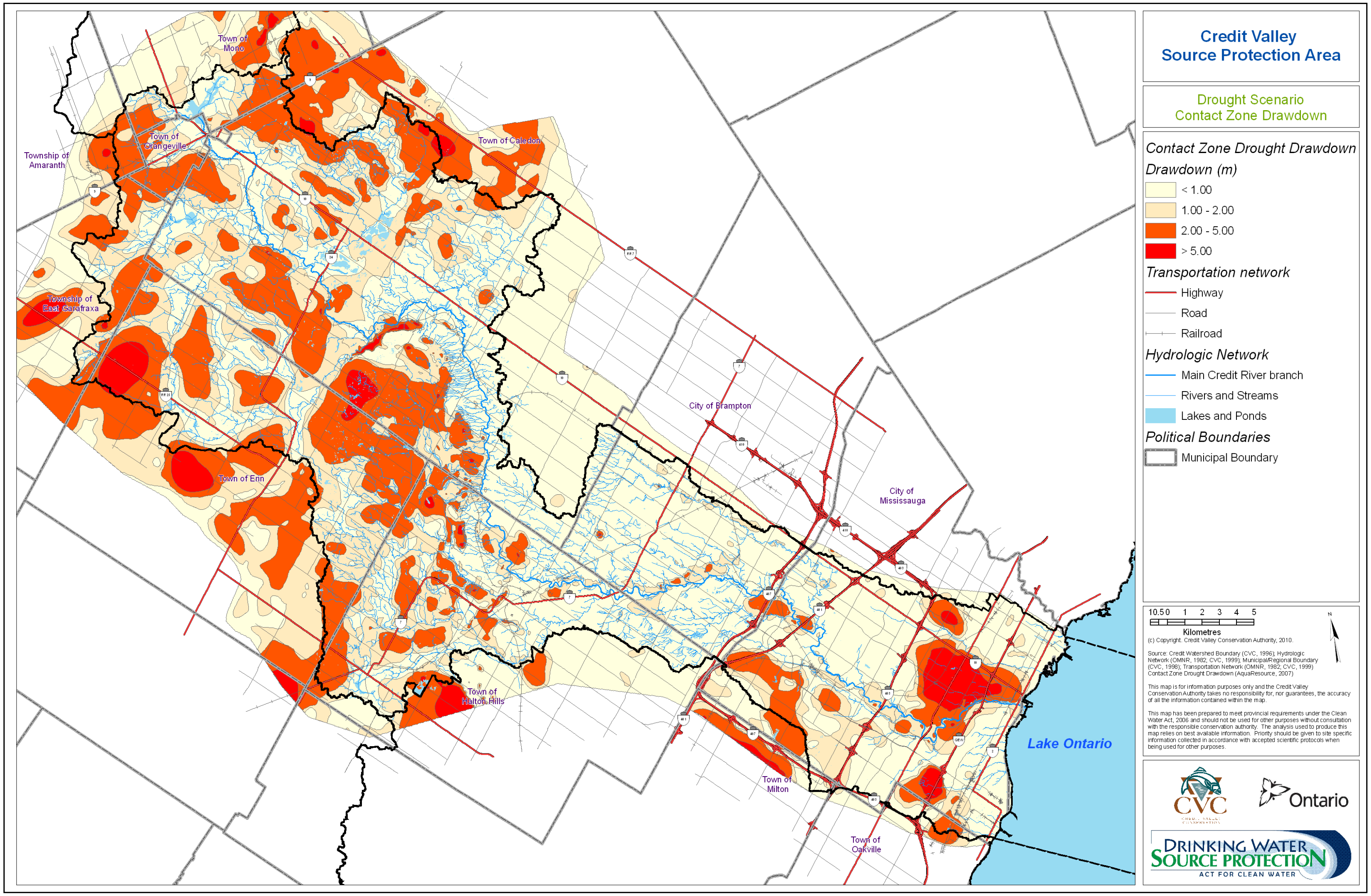


Figure C1-5: CVSPA –Drought Scenario – Contact Zone Drawdown

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C2 TIER 3 WATER BUDGET AND LOCAL RISK ASSESSMENT - ORANGEVILLE

C2.1 Water Budget Modelling Process

Tier 3 Water Budget requires a finer level of detail than that typically undertaken for the Tier 2 assessment. The Tier 3 hydrologic model improves upon the Tier 2 Water Budget model in terms of the model simulation and representation of the movement of groundwater between and across subwatershed boundaries.

A major deliverable is an improved estimate of the water budget components included in the hydrologic cycle within the study area. The surface water and groundwater flow models developed for the Tier 3 Assessment were used to estimate average annual values for the various components of the hydrologic cycle.

While CVC has an existing surface water hydrology model developed for Subwatershed 19, additional rural and urban spatial characteristic detail was warranted for this Tier 3 assessment to represent all land use patterns, tributaries, and the seepage and outlet configuration of Island Lake South Dam. Improved spatial detail allows for assessment of conditions at key locations within the subwatershed, specifically around wellheads and within major tributaries.

One specification of the Tier 3 surface water model was the need to have continuous simulation capability, reflecting drought and flooding conditions within the simulation time series. Representing a wide variety of climate conditions is necessary in order to predict the ability for the muncipalities' water sources to reliably meet water demand under this range of climate conditions.

Although the HSP-F model simulated hourly continuous streamflow, and the MODFLOW model simulated average annual groundwater discharge and baseflow conditions, each of the models estimates important aspects of the same surface water flow system. As such, the two models were calibrated to the same streamflow data. The second common aspect shared by the two models is groundwater recharge, which is a parameter that is simulated by the HSP-F model as a model output and used as a model input parameter in the MODFLOW groundwater model.

The HSP-F model and MODFLOW model were separate and independent models, but the modelling was linked through the groundwater recharge and groundwater interflow components.

C2.1.1 Model Domain

The model domain is presented in **Figure C2-1**. The grid encompasses the entire headwaters subwatershed (Subwatershed 19) within CVC, as well as portions of the Humber River watershed, Nottawasaga River watershed and the Grand River watershed. The model domain is approximately 17 km in a north-south direction, and 18 km wide (east-west), resulting in a model area of approximately 306 km^2.

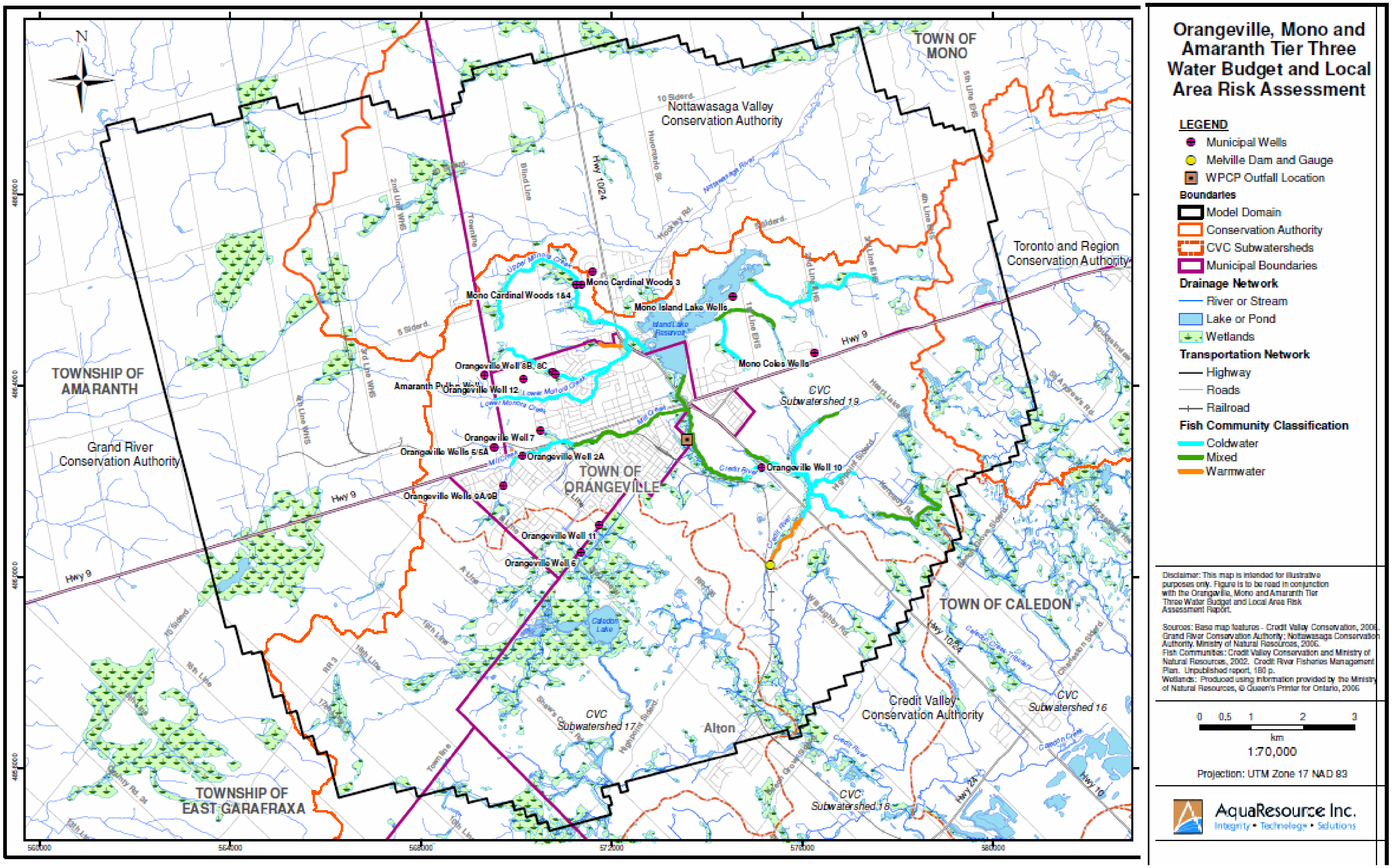

Figure C2-1: Tier 3 Water Budget Model Domain

The Hydrologic Model: HSP-F

The HSP-F model used for this study was refined from that used in the Tier 2 assessment, and updates include a highly refined drainage system and Island Lake outfall configuration. Also, the model provides valuable information regarding groundwater recharge that can be used to guide the estimation of recharge in groundwater models such as that used in the MODFLOW model for this study. Finally, with the unique HRU modelling scheme, future developments with LID features and/or BMPs can be modelled in detail. This capability was required for the impact assessment of future conditions in the subwatershed with alternative development approaches.

Modelling Objectives

The Tier 3 assessment requires models that simulate all relevant water budget components in a spatially detailed and temporally dynamic manner. The surface water model developed provided recharge estimates to the groundwater model and it also incorporated groundwater model information to improve upon the surface water model setup.

The hydrology model examines and quantifies the impacts and/or benefits of watershed scale and local scale activities as well as future conditions that may prevail due to climate change and urban development. These activities include various forms of urbanization, rural developments, waste management, land management, road management and stormwater management (SWM). The model may also find application in addressing local scale issues such as spills, construction impacts, urban infrastructure impacts (i.e., cross connects and inflow/infiltration to sewers), instream aquatic plant growth and instream thermal regime management.

In terms of water balance, the surface water model simulates the entire hydrologic cycle. This includes precipitation, snowpack accumulation and melt, surface runoff, unsaturated soil moisture and evapotranspiration (ET). It must be capable of estimating the short- and long-term supply of water to groundwater and the discharge rate of groundwater to streams.

The model explicitly represents effects of urbanization on local hydrology, including such processes and effects as changes in net ET and soil water infiltration caused by impervious surfaces, rapid surface runoff from impervious surfaces, and transfer of runoff from impervious surfaces onto pervious surfaces and vice versa, reflecting the complex nature of connectivity in the urban environment.

The model represents runoff processes at sufficient spatial resolution to allow for continuous simulation of the streamflow regime in local areas of concern and throughout the watershed. Also, the model is able to simulate the streamflow regime at an appropriate temporal scale (e.g., hourly) to simulate the rapid runoff associated with urban stormwater drainage on the tributaries and the main channel of the Credit River. The water budget simulation also is able to determine the risk associated with under supply during times of peak water demand. These are short term events requiring dynamic modelling.

The model provides time-domain simulation over multi-year continuous periods, to assess the impacts of control measures and strategies over an appropriate range of meteorological conditions. The model explicitly represents and simulates the effects of various urban control measures such as pollutant source control, runoff reduction measures, runoff treatment systems, and other stormwater management (SWM) practices. Similarly, the model explicitly represents and simulates the impacts of future urban growth and intensification within the subwatershed and explicitly represents and simulates the effects of various rural land management practices.

The HSP-F model developed was structured as follows:

- The subwatershed is represented as a mosaic of urban and rural land elements. Each element has homogeneous characteristics. Several elements together form catchments (i.e., discrete sewersheds and small tributary catchments that combine to make up the subwatershed). Surface runoff discharges from each element into the local watercourse stream/sewer. Subsurface runoff (i.e., interflow and groundwater flow) is generally routed to the nearest local stream channel, however in some cases subsurface flows are simulated to move to adjacent or downstream reaches to reflect the general groundwater movement patterns observed in the study area. The watercourse reach may represent a section of a tributary to the main channel, a pond, or a section of the main channel of the Credit River.
- Each urban land element is characterized by the land use, topsoil characteristics and topography found within. These characteristics are reflected in the setup and parameterization of the model. Urban areas are characterized to reflect several potential connectivity schemes (i.e., the manner in which pervious and impervious surfaces are connected to each other and to the local infrastructure) and land use types. The simulated runoff from each unique urban land type is saved as a unit area surface runoff time series and a corresponding subsurface (i.e., interflow and groundwater) runoff time series. These time series are input to the river model and are multiplied by a factor to reflect the area of each land type in each subcatchment.
- Rural land element areas are also characterized by the land use, surficial soil types, and topography within. Rural areas are assumed to be primarily pervious and therefore the infrastructure connectivity is not required. Runoff, surface and subsurface, is directed into stream reaches.
- The watercourse network is represented as a series of watercourse reaches. Each of these is characterized using representative stream and valley cross-section dimensions, as well as hydraulic roughness values and channel slopes.

C2.1.2 Subcatchments

Table C2-1 lists the catchments, their drainage areas and identifies the local stream reach. The average catchment size is approximately 155 hectares. The model was setup to produce and store streamflow output at the downstream limit of all catchments.

Table C2-1: Subwatershed 19 Catchments and Drainage Areas

Catchment #	Drainage Area (ha)	Stream/Tributary Name	Catchment #	Drainage Area (ha)	Stream/Tributary Name
1901	274.1	Upper Monora Creek	1919	23.1	Tributary to Island Lake
1902	199.7	Upper Monora Creek	1920	170.4	Tributary to Island Lake
1903	73.8	Middle Monora Creek	1921	91.6	Tributary to Island Lake
1904	14.4	Middle Monora Creek	1922	59.4	
1905a	169.5	Lower Monora Ck. North	1923	190.3	
1905b	73.2	Lower Monora Ck. North	1924	145.0	Credit River
1906	14.8	Lower Monora Ck. North	1925	327.4	Credit River
1907a	150.2	Lower Monora Ck. South	1926	303.8	Caledon Tributary
1907b	61.3	Lower Monora Ck. South	1927	429.5	Caledon Tributary
1908	91.2	Lower Monora Ck. South	1928	69.2	Caledon Tributary
1909	27.5	Lower Monora Ck. South	1929	165.6	Caledon Tributary
1910	29.5	Lower Monora Creek	1930	111.3	Caledon Tributary
1911	3.1	Lower Monora Creek	1931	68.7	Caledon Tributary
1912	14.7	Monora Creek	1932	395.0	Credit River
1913	328.3	Mill Creek	1933	9.6	Credit River
1914	183.8	Mill Creek	1934	14.4	Credit River
1915	280.4	Mill Creek	1935	176.5	Credit River
1916	74.6	Mill Creek	1936	53.4	Caledon Tributary
1917	752.1	Orangeville Tributary			
1918	220.6	Tributary to Island Lake			

Land Use

The three factors used to classify land elements for the HSP-F model setup include land use cover, surface soil characteristics and topography. These three factors describe the physical characteristics of the landscape and characterize the factors that influence hydrologic response and non-point source water quality at the land element scale. Of these three factors, land use cover is the most likely to be affected by human activities and is expected to significantly change in the subwatershed as the Town of Orangeville is built-out to its limits.

For the purposes of model development, the aggregated land uses were designated as either urban or rural. This step is critical as urban and rural lands are treated differently in modelling. Further explanation of this treatment is provided below.

All urban land use classifications were sorted and lumped into ten major land use designations while the rural land uses were sorted and lumped into nine major land use designations, as presented in **Table C2-2**.

Sorting and lumping land uses into these designations was conducted in a manner that would group land use types according to their likely hydrologic and water quality characteristics. For example, several vegetation types are grouped as Forest and several commercial land use type categories are lumped as Strip Mall Commercial. This lumping is necessary to reduce the overall complexity of the model and recognize the similarity in hydrologic and other characteristics.

In these land use designations, all local roads are included with the prevalent land use in each area. In this way all roads are accounted for, and their imperviousness is incorporated into that of the adjacent land use type. Since HSP-F considers the lot level configuration for each land use type this scheme ensures that the roads and their drainage are included.

Table C2-2: Urban and Rural Land Use Designations

Land Use Category	Land Use Modelled with HSP-F	Code	Examples
Urban	Residential, low density	RLD	Single, detached homes
	Residential, medium density	RMD	Semi-detached and duplexes
	Residential, high density	RHD	Town homes and multiple link homes
	Parks, Manicured Open Space	OPL	City Parks
	Commercial, big box	CBB	Big box stores, large central malls, large office complexes
	Commercial, strip mall	CSM	Strip malls, stand-alone commercial/other businesses
	Industrial, big box	IBB	Factories, plants, big box industrial
	Industrial, prestige	IPR	Industrial offices, mixed industrial and offices
	Educational and Institutional	EIS	Schools, universities, churches, recreational centres
	Main Transportation Corridors	THC	Provincial Highways, Expressways (Highway 10)
Rural	Aggregate, active	AGG	Open pit mining
	Agricultural, intensive	AGT	Tilled land
	Agricultural, non-intensive	AGP	Pasture land
	Open space, unmanaged	OPU	Fields, meadows, savannah,
	Open space, manicured	OPM	Municipal parks, sport fields, golf etc.
	Forest	FOR	Deciduous and coniferous forests and plantations
	Wetlands	WET	Swamps, bogs, marshes
	Residential, rural estates	RES	Rural estate development, unserviced
	Residential, un-serviced	RLDU	Rural villages

Table C2-3 summarizes land use in the subwatershed in terms of the land use designations used in modelling, and as this table shows Island Lake was included in the rural/wetlands category. More than three quarters of the subwatershed has been designated as rural with about 42% of that area in agriculture. Forest, wetlands, and open space constitutes most of the remaining rural area. Almost one quarter of the subwatershed was designated as urban, with the residential areas constituting about 56% of the urban area. Commercial areas, highway corridors and parklands constitute the remainder.

Table C2-3: Summary of Current Subwatershed Land Use Areas Composition

Land Use Group	HSP-F Land Use Classification	Relative Proportion of Subwatershed (%)
Rural	All Types	77.0
	Agriculture (Intensive and Non –Intensive)	32.6
	Aggregate Extraction	1.2
	Residential	0.9
	Forest/Wetland/Open	42.3
Urban	All Types	23.0
	Residential	12.8
	Commercial	5.7
	Open/Park	3.7
	Highway Corridors	0.9

C2.1.3 Surface Soils

Beyond land-use, some of the most critical hydrologic modelling input parameters are related to soil characteristics within each subcatchment. Characteristics of soils in the upper topsoil or vadose zone affect the water balance at the smallest element scale, thus, profoundly affecting overall water quality and hydrology. Soil characteristics relating to hydrologic behaviour are assigned based upon surficial geology in this study.

Four soil type classifications were employed in this model setup. The soil classifications correspond to soils with high infiltration and water holding characteristics (i.e., sands and gravels), medium infiltration characteristics (i.e., fine sands and silts), low infiltration characteristics (i.e., silts and clays) and organic soils with high water holding capabilities. For the purposes of identification, the four soil types are termed AB, BC, CD, and O, respectively; corresponding to their approximate hydrologic soil groupings (HSG), an agricultural classification system often used in modelling. **Table C2-4** summarizes the occurrence of the four soil types in the subwatershed. Finer soil types tend to lie in the central and southern portion of the subwatershed. Pervious soil types are dominant across much of the subwatershed, especially in the headwater areas.

Table C2-4: Abundance of the Four Soil Classifications Used in Modelling Subwatershed 19

Soil Type	Portion of Subwatershed, %
AB, Sands and Gravel	64.7
BC, Fine sands and silt	31.9
CD, silts and clays	2.2
O, organic	1.2

Organic soils in urban areas occur rarely in Subwatershed 19, as these soils are generally replaced with soils that are more structurally competent or desirable from a horticultural viewpoint. Therefore, urban area organic soil has been replaced in the study area map with one of the nearest other soil types. This replacement of organic soils affects less than six hectares of urban area, or about 0.4% of the urban area and is expected to have no significant impact on the study results.

As commercial and industrial areas are highly impervious in makeup, it was decided to simplify these schemes by using only the BC soil type with these land uses. The four land use types involved are assumed to be up to 80% impervious so the soil type is less relevant than on other urban land use types with higher levels of exposed soils. In addition, the pervious portions of these land use types typically are not the native soil, but topsoil or other more permeable fill material. By assigning one soil type to these land use areas, the model is simplified, and results are not expected to be significantly affected

C2.1.4 Ground Surface Topography

Three slope classes were used for rural areas. These classes correspond to relatively flat areas with slopes <2%, moderately sloped areas ranging from 2 to 5% grades and steeply sloping areas with slopes >5%. In urban areas, wherein steeply sloping areas are rare, only two slope classes are used. These correspond to flatter areas with slopes less than 2% and higher sloped areas with slopes >2%.

C2.1.5 Hydrologic Response Units

Urban areas are treated with special detail in the Subwatershed 19 hydrologic model using a two-step modelling approach. The first step in this approach involves the simulation of runoff from discrete generic hydrologic response units (HRUs). HRUs are unique landforms as defined by their land use, soil type and slope class within a catchment. Simulations were run for all urban landforms resulting in an output for each consisting of surface and subsurface runoff time series. The second model step uses the HRUs as input for simulating instream flow. Rural areas are simulated entirely in the second step using a conventional approach. This HRU approach was developed for urbanized portions of the watersheds examined in the Toronto Wet Weather Flow Management Master Plan (TSH, 2003).

HRUs are the time series of runoff that are simulated to discharge from one of several possible generic urban landforms over the simulation period. Each HRU area has been constructed using the necessary number and combination of IMPLND (i.e., IMPervious LaND) and PERLND segments. The characteristics of HRUs are highly dependent upon their relative perviousness and their connectivity; the scheme that describes how surface and subsurface water moves across and through the landform and discharges to the groundwater, sewers or other landforms. This flow scheme affects the overall response of the hydrological system and ultimately the associated water quality. The model has been setup to maintain this flexibility.

In industrial, commercial, and institutional land types it is common for all drainage from impervious areas (i.e., roofs, walkways, roadways, and parking lots) to be collected and discharged directly to the storm sewers. Pervious areas (i.e., lawns and gardens) generally constitute only a small portion of the land area and surface runoff generally drains directly to the roadway servicing the buildings. These landform types with high levels of overall imperviousness generate high runoff volumes and very rapid runoff responses during storms and contribute only a small amount of flow to groundwater recharge.

Within residential areas various infrastructure connectivity schemes are possible for roof downspouts and foundation drains. Orangeville has a variety of downspout configurations. However, within residential areas the overwhelming majority of downspouts drain to the lawns or driveways and are therefore indirectly connected to infrastructure.

Residential foundation drains in Orangeville are also configured in a variety of ways. There are areas with no foundation drains, areas with sump pumps discharging to lawns, drains connected to sanitary sewers and drains connected to storm sewers (pers. comm. D. Jones, Town of Orangeville). The majority of residential areas have foundation drains that discharge to sanitary or storm sewers. These two configurations have been used to model connectivity in Orangeville. Catchments 15, 24 and 25 in the southwest portion of the town have been setup with foundation drains connected to sanitary sewers as this is the current configuration over most of this area. In this configuration the drainage is included in the WPCP inflow. All other residential areas are modelled with foundation drains connected to storm sewers, as indicated by town staff (pers. comm., D. Jones). Note that all component parts of the typical lot are represented by pervious or impervious land segments.

Open areas such as parks and hydro corridors typically have very little impervious area. There may be some parking areas or walkways associated with these land uses in addition to the adjacent roadways. Conventional HRUs for these areas assume that parking areas and roadways are drained directly to the storm sewer.

The relative volume and timing of stormwater runoff from HRU areas is highly dependent upon the HRU characteristics. At one extreme, low density residential areas with roof downspouts and foundation drains directed to lawns, effectively attenuate stormwater runoff. At the other extreme, high density residential areas with roof downspouts and foundation drains connected to sewers generate runoff responses similar to relatively impervious commercial and industrial areas.

To represent all of the conditions within the urbanized portion of the watershed, it was necessary to construct a total of 62 unique HRUs. This includes 36 residential HRUs at three densities, on three soil types with two connectivity schemes and two slope classes. There are 26 non-residential HRUs. This includes two slope classes, three soil types and seven different land uses. The hydrologic response from all of the urban areas was then determined in the second stage of the modelling, by summing up the area-weighted HRUs for all the urban area within each subcatchment.

The characteristic levels of perviousness and lot makeup for each land use type have been taken from previous modelling undertaken in the Credit watershed. These values were estimated by sampling and measuring several urban areas in Mississauga at random (CVC and EbnFlow, 2008). **Table C2-5** lists the average land use type breakdowns used in modelling urban areas in Subwatershed 19.

Table C2-5: Typical Urban Land Uses and Breakdown by Components

Land Use Type	% Pervious	Roofs	Road	Lawn/ Open	Parking/ Drive	Patio	% Impervious
Low Density Residential	70	13	9	70	7	1	30
Med. Density Residential	50	24	13	50	10	3	50
High Density Residential	35	32	17	35	11	5	65
Big Box Commercial	20	29	12	4	55	0	80
Strip Mall Commercial	20	17	19	4	60	0	80
Education/Institutional	68	9	9	68	14	0	32
Parks/Cemetery	90	5	5	90	0	0	10
Highways/Corridors	40	0	40	40	0	0	60
Prestige Industrial	20	30	7	20	43	0	80
Big Box Industrial	7	45	6	7	42	0	93

The flow schemes for each land use type are simulated in the MASS LINK BLOCK of HSP-F. In this block, surface water (SURO), interflow (IFWO) and groundwater flow (AGWO) pathways are specified, allowing for the routing of water from one portion of the lot to another as a lateral inflow (i.e., SURLI) and eventually to the storm sewer RCHRES (i.e., ReaCH/REServoir) element or the groundwater RCHRES element. Surface and subsurface RCHRESs have been simulated for each land use type so that the flow streams from surface and subsurface pathways can be maintained separately and stored as a time series of flow. This scheme is necessary for water quality simulation as well as facilitating analysis of the results. In order to simulate various stormwater control schemes, it is possible to alter the connectivity in this block to reflect changes in roof leader and foundation drain connections, to change the relative perviousness, and to insert lot level storage (i.e., rain barrels, storm ponds etc.) into the landform. This scheme maintains a high level of flexibility for the simulation of possible future Low Impact Development (LID) and Beneficial Management Practices (BMP).

As the model is a lumped parameter model the hydrological characteristics of slope and slope length are common to all HRU areas with common surface characteristics. Values applied represent typical values for the slope and land use classes involved. Thus, routing delays for surface and subsurface flows are incorporated in the HRUs. Output is stored in a Watershed Data Management (WDM) file using a four-digit numbering system that can accommodate future expansion of landform types.

C2.1.6 Watercourse Network Definition

The subwatershed's watercourse network represented in the HSP-F model consists of a set of reaches that represent the main stem of the Credit River, as well as selected portions of tributaries. In general, each catchment contains one reach. Inflow to the reach is from local contributions and upstream contributions.

Each watercourse reach is modelled within HSP-F as a RCHRES segment. The hydraulics of the reach is characterized in the model by supplying a table of hydraulic parameters including outflow and corresponding depth, water surface area and water storage volume for each reach. These data are then used to simulate routing of flows and water quality parameters through the reach network.

Representative stream and valley cross-sections for each reach were used to develop the necessary average depth-surface area-reach volume relationships (i.e., flow tables-FTABLES) for each reach. HEC-2 (U.S. Army Corps of Engineers) input files provided by CVC were used to determine representative cross sections for all reaches.

The stage-depth and stage-weir flow relationships for Island Lake were taken from the GAWSER modelling (Schroeter and Associates, 2001). Operating permits issued by the Ontario MOECC to CVC specify dam outflow rates on a monthly basis, as determined for flow augmentation downstream. CVC regulates outflow using a manually operated gate valve at the south dam. A time series of requested outflows was developed for this site based on the operating permit. No continuous long-term record of outflow rates is available to aid calibration, so the mandated flows have been used as a starting point in calibration.

Major tributaries include Upper and Lower Monora Creek in the northwest and Mill Creek in the southwest. Unnamed tributaries in the east side of the subwatershed also contribute significantly in terms of drainage areas.

Groundwater Routing

The MODFLOW groundwater flow model developed for this study and calibrated to steady state groundwater conditions was used as input into the groundwater routing portion of the HSP-F model.

The groundwater flow model was used to provide information on the steady state transfer of groundwater between catchments and across subwatershed boundaries. There is significant movement of water into and out of the subwatershed and across some catchment boundaries. This transfer of water across the subwatershed boundaries was accounted for in establishing a subwatershed water balance. These transfers are based upon long term steady state simulations. Pumping rates are based on average annual estimates discussed in **Table 3.1.4** of this Assessment Report.

Transfers of groundwater across catchment boundaries are less important as they do not directly affect the subwatershed water balance. However, calibration at smaller scales is improved when these transfers are recognized. Since the model is not calibrated at the catchment scale no attempt has been made to incorporate the transcatchment boundary flows estimated from groundwater model runs. Calibration at the catchment scale requires continuous flow recording at the catchment outlets and is beyond the scope of this study.

The surface water model has been setup to account for the net effects of transboundary groundwater flow and groundwater pumping. This is accomplished by “bleeding off” (losses) or increasing (gains) the groundwater flows proportional to the amounts simulated without transfers. For example, in catchments 1901 and 1902 the net groundwater transfer including pumping is a net outflow of 2950 m^3/d. The model for these catchments was set up to lose an equivalent amount of groundwater, over the year, from the catchments. Catchment 1907a has been setup to gain a proportional amount of groundwater to equal the estimated 1,600 m^3/d of inflow from the Upper Grand River watershed. In all, 16 catchments have been set up to account for groundwater transfers and pumping, as listed in **Table C2-6**.

Table C2-6: Groundwater Rerouting in Headwaters of the Subwatershed

Source Catchment	Catchment Area (ha)	Portion of Groundwater Directed away (negative) or gained (positive)(%)
1901	274	-100
1902	200	-100
1903	74	-75
1905a	170	-50
1907a	150	+145
1917-1920	1166	-100
1923	190	-100
1926	304	-100
1927	429	-10
1928	69	-90
1929	166	-100
1932	395	+25
1935	176	+110

C2.2 Model Calibration and Validation

A general comparison of the simulated and observed streamflow at Melville is provided in **Table C2-7** for the calibration and validation periods. It compares the simulated and observed streamflow rates by their extremes and by total flow volumes and average or mean flow rates. The Nash-Sutcliffe Model Coefficient is a measure of the goodness of fit of the simulated data to the observed data. A value between 0 and 1 indicates the degree of agreement between the time series with 1.0 being a perfect agreement between the two sets and 0.0 meaning that the model cannot improve over the mean of the time series as an estimator.

Table C2-7: Calibration (2001-04) and Validation (2005-07/2007) Period Comparison Criteria

Comparison Criteria[1]	Observed Calibration Period	Simulated Calibration Period	Observed Validation Period	Simulated Validation Period
Maximum Streamflow Rate, m^3/s	4.850	3.670	6.760	3.780
Minimum Streamflow Rate, m^3/s	0.138	0.199	0.247	0.276
Median Streamflow Rate, m^3/s	0.402	0.411	0.510	0.500
Annual Average Flow Volume, dam^3	15815	16323	20718	20372
Average Winter Flow Volume, dam^3	4134	4112	6176	6638
Average Spring Flow Volume, dam^3	5264	5410	5115	5652
Average Summer Flow Volume, dam^3	2999	3010	3291	3106
Average Fall Flow Volume, dam^3	3418	3791	4859	5075
Nash-Sutcliffe Coefficient, R^2	N/A	0.430	N/A	0.400

1. Winter: Jan.-Mar., Spring: Apr.-Jun., Summer: Jul.-Sep., Fall: Oct.-Dec.
2. N/A: not applicable

All years closely agree except 2002, where observed streamflow data has periods where flows drop suddenly and rebound to higher levels a short time later. This erratic behaviour could be due to ice or debris interference at the gauge. These episodes result in underestimated flow rates, periodically in the record. Therefore, the 2002 estimates of total annual streamflow are believed to be underestimates. Accordingly, the simulated total annual streamflow for this site exceeds the observed values by about 13%. Differences could also be due, in part, to an overestimate of Island Lake outflow rates and dam seepage rates in that year. Total streamflow volumes are about 3.2% above the reported observed flow volumes for the whole calibration period. In general, these compare favourably. The months with the largest discrepancies are in the winter and fall. Snow accumulation and melt processes complicate these periods, accounting for much of the difference noted. Simulated streamflow tends to reflect observed rates and trends in a reasonable manner. Significant differences exist for individual events, especially during winter, but maximum and minimum flows tend to be in reasonable agreement.

The correspondence between simulated and observed flow frequencies confirms the model's ability to simulate subwatershed scale hydrologic response. Simulated results reflect observed storm or melt event response and extended low or baseflow periods. The overall mean flows compare closely.

Good agreement in these terms indicates that the model calibration reflects the long-term averages as well as the occurrence of minimum and maximum flow regimes within the time series.

The agreement between annual, seasonal, and monthly flow volumes confirms the model's reliability in terms of annual and seasonal processes and water balance.

C2.2.1 Model Validation

Once calibration was achieved for the 2001-2004 period, the model was subjected to a validity test wherein model simulation results were compared to measured flow rates from a different time period, namely 2005 to mid-2007. **Table C2-7** displays some comparative statistics for the simulated and observed streamflow time series for this period.

The Hydrogeologic Model: MODFLOW

The computer code MODFLOW was selected to develop the numerical groundwater flow model for the Tier 3 Water Budget Pilot Project. MODFLOW is a three-dimensional, saturated, finite difference groundwater modeling code that was first developed by the United States Geological Survey. This code has been used extensively for water budgeting and various other uses worldwide. The groundwater flow model was developed with the aid of Visual MODFLOW, a graphical user interface developed by Waterloo Hydrogeologic Inc. (WHI, 2005).

The model area of a MODFLOW model is divided horizontally and vertically into a discrete set of square or rectangular blocks or cells, with each cell representing a discrete horizontal and vertical unit of porous media. Each model cell has specified hydraulic properties (hydraulic conductivities, storage parameters, etc.) that are assigned and remain constant throughout the model simulation. The initial selection of model properties are based on the conceptual understanding of the geology and hydrogeology of the study area and refined through the model calibration process.

MODFLOW directly simulates the hydraulic head (i.e., groundwater level) within each cell. Groundwater flow velocities and rates can be calculated from these heads using the hydraulic properties. The hydraulic head and flow through each cell within the model can be calculated in either steady state, which is used to simulating equilibrium conditions, or transiently, to simulate the systems response to changing stresses that may occur over a given period(s) of time.

Modeling Process

The numerical modeling process for the Tier 3 Water Budget Study consists of three steps or stages, which were as follows:

a) **Calibration and Verification**.
b) **Steady State Calibration**. During the steady state calibration, the model input parameters were adjusted in order to obtain a reasonable fit to the modern day (2006) head and baseflow

values. Average groundwater recharge rates, calculated based on the calibrated HSP-F model, range from zero on groundwater discharge areas to 317 mm/yr on the Orangeville Moraine, were used as input into this simulation. Average hydraulic head measurements in the Town of Orangeville observation wells were matched to the range of observed heads wherever possible; and

c) **Transient Calibration**. The model input parameters from the steady state calibration were used as initial conditions to match the transient data from a 44-day pumping test conducted within the Town of Orangeville. Input parameters (hydraulic conductivities and storage estimates) were modified to achieve a reasonable match.

Long Term Model Verification: Following calibration to both steady state and transient calibration, the groundwater flow model simulated hydraulic head and baseflow fluctuations between 1990 and 2008. Average monthly recharge rates (HSP-F) and pumping rates reported by the town were applied.

Scenarios: Once the model was adequately calibrated and verified, it was used to assess the potential changes in the hydraulic heads and groundwater discharge to streams across the study area in response to changes in land use (recharge) and groundwater extraction.

Sensitivity Analysis: An assessment of the sensitivity of the model input parameters was conducted in order to provide a basis for a discussion on the uncertainty associated with the modeling and the model results.

C2.2.2 Model Domain

The model domain is presented in **Figure C2-1**. The grid encompasses the entire headwaters subwatershed (Subwatershed 19) within CVC, as well as portions of the Humber River watershed, Nottawasaga River watershed and the Grand River watershed. The model domain is approximately 17 km in a north-south direction, and 18 km wide (east-west), resulting in a model area of approximately 306 km^2.

The model domain was designed to encompass the entire upper subwatershed area of the Credit River, and to extend to the natural boundaries of the groundwater flow system as interpreted from the shallow and deep groundwater level contour maps. In the west, the model was extended beyond the Credit River and Grand River watershed divide as previous modeling efforts (WHI, 2005) suggested that the groundwater divide was located west of the subwatershed surface water divide. The model domain lies largely outside the subwatershed boundary to minimize potential boundary effects on model predictions within the study area.

C2.2.3 Model Grid

The model domain was rotated such that the primary direction of groundwater flow would be aligned with the x and y coordinate axes of the model. The smallest grid cells (12.5 m by 12.5 m) were defined at the municipal pumping wells, and the largest grid cells (200 m by 200 m) at the periphery of the model. The model grid was refined around the pumping wells to better represent drawdown near the wells.

The grid has a total of 415 rows and 405 columns within the model area for a total of 168,075 cells. The model subdivides the subsurface vertically into nine layers; each layer generally represents a hydrostratigraphic unit such as a sand aquifer, or a till aquitard. The top and bottom elevation of each layer are assigned in MODFLOW to each of the model cells. The hydrostratigraphic layers are discussed in more detail below.

C2.2.4 Hydrostratigraphic Layer Structure

The numerical model was subdivided into nine hydrostratigraphic layers (**Table C2-8**) based on the 11 hydrostratigraphic units. The two uppermost hydrostratigraphic units included thin surficial layers of sand and gravel, or clay and till beds. These units were relatively thin in most areas and were underlain by coarse-grained sediments, and therefore were predominately unsaturated within the study area. As MODFLOW simulates only saturated groundwater flow, these upper unsaturated layers could be excluded from the numerical flow model without causing any negative impact on the underlying groundwater flow system. Therefore, the two uppermost hydrostratigraphic layers (coarse-grained outwash and alluvium, and surficial silts/clays) were excluded from the numerical groundwater flow model. The underlying nine hydrostratigraphic layers (assumed to be saturated) are outlined in **Table C2-8** below.

Table C2-8: Model Representation of Hydrostratigraphic Units

Model Layer	Hydrostratigraphic Unit	General Lithology
1	Upper Aquitard and Intermediate Aquifer	Coarse-grained outwash sand deposits associated with the Orangeville Moraine; Newmarket Till, glaciolacustrine clays, Singhampton Moraine
2	Lower Aquitard	Tavistock Till, Port Stanley Till, Catfish Creek Till (/ Northern Till)
3	Lower Aquifer	Sand and gravel overlying fractured bedrock
4	Bedrock Aquifer	Contact zone aquifer
5	Bedrock Aquifer	Guelph Formation
6	Bedrock Aquitard	Eramosa Member of the Amabel Formation
7	Bedrock Aquifer	Amabel Formation (Colpoy Member)
8	Bedrock Aquitard	Clinton- Cataract Group
9	Bedrock Aquitard	Queenston Formation

C2.2.5 Model Properties

Hydrogeologic properties assigned within the MODFLOW model included hydraulic conductivities and storage parameters (specific storage and specific yield). Hydraulic conductivity plays a significant role in the calculated hydraulic head distribution within the model domain (based on boundary condition values). Storage parameters are not used in a steady state simulation. However, under transient or time varying conditions, specific yield and specific storage control the timing and response of the groundwater system to external stresses.

Hydraulic Conductivities

Hydraulic conductivity values were assigned to the hydrostratigraphic units prior to model calibration, and then adjusted during the model calibration process. The calibrated hydraulic conductivity estimates are consistent with geologic descriptions within high quality borehole logs, and with independent estimates resulting from pumping test analysis conducted in the study area. In addition, local level knowledge and conceptualization, information from previous modeling efforts, field base studies and literature values (Freeze and Cherry, 1979; Anderson and Woosner, 2002) were also consulted to ensure the calibrated conductivities were reasonable and defensible.

Four hydraulic conductivity zones were applied in the uppermost model layer. The zones were generalized versions of the surficial geology mapping of the area and correspond to; various surficial tills and fine-grained clays/ silts; coarse-grained outwash sands and gravels; sands and gravels of the Orangeville Moraine (western limb); and gravel outwash channel associated with Orangeville Wells 5/5A. The ratio of horizontal (Kxy) to vertical (Kz) hydraulic conductivity was assigned to be 10:1 (to account for horizontal bedding) in all zones except the zone representing the Orangeville Moraine. The 10:1 ratio is somewhat arbitrary and the sensitivity of the model to changes in anisotropy was examined in the sensitivity analysis. The conductivity zone applied to represent the Orangeville Moraine, west of the town's municipal wells was assigned a ratio of 400:1. This value was applied to account for the presence of till and clay interbeds within the moraine in this area. The geometric mean of the vertical hydraulic conductivity of the moraine sediments was calculated at several points along this portion of the moraine using conductivity estimates and thicknesses reported in high quality borehole logs. The vertical hydraulic conductivity applied in the model, which is greatly influenced by the lower hydraulic conductivity material, was calculated to be within the range calculated (range of geometric means). The model calibration is highly sensitive to changes in the anisotropy ratio within the Orangeville Moraine, and therefore the anisotropy ratio of the overburden layers should be considered in the sensitivity analysis.

Three hydraulic conductivity zones were assigned in Layer 2 of the model. One zone represented the Tavistock, Port Stanley, and Catfish Creek Tills, and the other two represented the sands beneath the Orangeville Moraine (where the till layer has been eroded; one represented the sand below the western limb of the moraine and the other the eastern limb). As above, the ratio of horizontal to vertical anisotropy was 10:1 for the till zone and the sands beneath the eastern limb of the moraine, while a ratio of 100:1 was used on the sands beneath the western limb of the Orangeville Moraine where interbeds of fine-grained material are common.

Layer 3 is the lowest overburden layer in the model, and it consisted of two conductivity zones that represented a basal aquifer layer, and a lower till (Catfish Creek) layer. Both of these zones were assigned a 10:1 ratio of horizontal to vertical anisotropy.

The remaining model layers consisted of one or two hydraulic conductivity zones that represented the various competent and weathered bedrock units. The ratio of horizontal to vertical anisotropy in the bedrock was 10:1 to account for horizontal bedding within the units. This anisotropy ratio is consistent with other groundwater models containing the Amabel Formation (Golder Associates, 2006 and AquaResource, 2008).

Burnside (2005) completed a geophysical study to examine the Amabel Formation bedrock and potential bedrock lineaments in the vicinity of Wells 5/5A in 2004-2005. The bedrock lineaments identified in this study were not represented in the current conceptual model as the depth and spatial extent of the lineaments remains poorly understood. The hydraulic conductivity within the Amabel Formation is unlikely to be constant across the study area, and that fossiliferous reefal structures and similar features are likely to exist leading to zones of localized hydraulic conductivity variations. However, there is currently insufficient information available to delineate the extent of these localized conductivity zones in the subsurface and therefore, a uniform hydraulic conductivity was applied.

Storage Parameters

Specific yield and specific storage values were assigned to zones coincident with the hydraulic conductivity zones and the values were manipulated during the transient model calibration process. The final calibrated values are listed in **Table C2-9** below.

Table C2-9: Porosity Estimates Applied in the Groundwater Model

	Geologic Description	Specific Yield	Specific Storage
Overburden	Sand and Gravel	0.25	2e-5
	Silt	0.18	4e-6
	Clay	0.05	1e-6
	Till	0.18	3e-6
Bedrock	Limestone/ dolostone	0.15	1e-5
	Shales	0.04	1e-7

C2.2.6 Model Boundary Conditions

Boundary conditions applied in the groundwater flow model were chosen to approximate the regional groundwater flow patterns and to approximate the major groundwater fluxes within the study area. Boundary conditions applied in the model consisted of three types:

- Dirichlet boundary conditions are boundaries where the value of the head is assigned to specific cells within the model, and the amount of discharge into our out of the model cell fluctuates to satisfy the specified head condition. Physically, these boundary conditions (constant heads) are commonly used to simulate areas where aquifer potentials are expected to remain at a constant level. This is commonly flow to and from large rivers, or lakes, or areas where water enters or exits the model domain.
- Neumann boundary conditions are boundary conditions for which a flux value is assigned to specific model cells. The hydraulic head within the cell is allowed to fluctuate to meet that flux condition. These boundary conditions are also called constant flux boundaries and are used to represent groundwater extraction or injection wells, or recharge to the groundwater system. No-flow boundaries are one type of specified flux boundary where the rate of lateral flow across the boundary is assumed to be negligible or equal to zero. In general, no flow boundaries are applied to simulate groundwater divides or impermeable geologic units.
- Cauchy boundary conditions are boundaries where a flux across a boundary is calculated given a value of head assigned to specific model cells. The flux value is dependent on the difference between a specified head and the calculated heads in the surrounding model cells. These head dependent flow boundary conditions are often used to represent flow into a drain or into or out of a river.

Boundary conditions applied in the model include groundwater recharge (provided from the HSP-F model), flow into and out of surface water features (streams, rivers, lakes), groundwater pumping wells, and flow into and out of the model along its perimeter.

Recharge

The HSP-F model was calibrated to estimate recharge rates (specified flux) for input into the top layer of the MODFLOW model. HSP-F calibration efforts were focused on the low flow regime to be able to better constrain the groundwater recharge estimates. The resulting calibrated recharge estimates ranged from a low of 0 mm/yr. where groundwater discharges to some wetlands to a high of

317 mm/yr. on the Orangeville Moraine. As the HSP-F model was calibrated to gauge data for low flow conditions, additional confidence is placed on the overall average rate and total volume of recharge entering the subwatershed and leaving and groundwater discharge.

Rivers, Streams, Ponds, Lakes and Wetlands

The MODFLOW drain and river packages were used to simulate the interaction between the groundwater flow system and the surface water features (rivers, streams, wetlands, and lakes). The river package was used to simulate the major lakes in the model including Caledon Lake and Island Lake and many of the perennial rivers and streams in the model, including the Credit River.

The drain package was used to simulate rivers, streams, or wetlands where portions of the feature are observed to be dry or losing water to the groundwater system at least part of the year. A reference elevation for the drain is applied, and when the head in the cell is simulated to go beneath the reference elevation, the drain is simulated to go dry, and there is no flow of water from the drain into the groundwater system. When the model simulated head is above the reference elevation, there is flow from the groundwater system into the drain (boundary condition) and removed from the model. In many instances the lower order (first and second order) streams were simulated using drains, as drains allow water to be removed from the groundwater system, but they will not supply water to the groundwater system. These boundary conditions were selected as they ensure that an unrealistically large amount of water cannot be supplied by a stream to an underlying pumped aquifer.

Specification of stream and river boundaries involves applying a rivers stage (elevation) as well as the degree of “conductance” between the stream and the underlying groundwater system. The river stage was estimated from the 5 m Digital Elevation Model (DEM), and the stream conductance, was assigned based on our understanding of the width of the stream, the vertical hydraulic conductivity of the riverbed, and the thickness of the riverbed. The riverbed conductivity was assumed to be influenced by the surrounding surficial geology (ranged from $1x10^{-4}$ to $1x10^{-5}$ m/s along the moraine, and $1x10^{-7}$ m/s along the till plains). The width of the river was varied from 0.5 m for smaller swales and first order streams, to over 3 m for portions of the Credit and Nottawasaga rivers. The conductance of the rivers was modified during the model calibration process.

Perimeter Boundaries

To determine appropriate boundary conditions for the perimeter of the model, water levels (shallow and deep) were reviewed in the surrounding area. Where water level trends suggested that natural flow boundaries exist (groundwater divides), a no flow boundary was applied. In other areas where water level trends indicated cross-boundary flow, constant head boundary conditions equivalent to the equipotential heads in those layers were applied. In the eastern reaches of the model a few constant head cells were applied at depth (lower aquifer; model layer 3) to account for the eastward flow of water out of the model area along a narrow-buried bedrock valley. Similarly, in the southwest portion of the model area, a river boundary condition was applied in layer 1 to simulate flow into and out of Shaw’s Creek, and in the underlying aquifer layers, a constant head was applied to allow groundwater to flow into or out of the model domain as Shaw’s Creek (at this location) was not conceptualized to be a regional groundwater flow divide.

Pumping Wells

All municipal pumping wells that lie within the model area were simulated in the model using specified flux boundary conditions. The pumping rates simulated in the model varied depending on the model simulation (e.g., long term steady state, transient pumping test simulation, model verification). **Table C2-10** below outlines the municipal wells simulated in the steady state and model verification models. They are also shown on **Figure C2-2**.

Table C2-10: Municipal Large Permitted Water Takers

Well Name	Easting (NAD83)	Northing (NAD83)	Screen Top Elevation (masl)	Screen Bottom Elevation (masl)	Aquifer
Cardinal Woods Well1	571266	4866092	418.16	376.40	Bedrock
Cardinal Woods Well 3	571646	4866369	412.70	367.95	Bedrock
Coles Well 1	576251	4864785	374.00	392.00	Overburden
Coles Well 2	576251	4864785	395.00	379.00	Overburden
Island Lake Well TW1	574610	4865780	368.80	360.26	Overburden
Purple Hill 1	574637	4864328	440.61	417.75	Bedrock
Purple Hill 2	574426	4864393	439.79	412.35	Bedrock
Purple Hill 3	574603	4864321	440.81	440.81	Bedrock
Purple Hill 4	574568	4864504	435.61	435.61	Bedrock
Purple Hill 5	574426	4864513	436.59	436.59	Bedrock
Orangeville Well 2	570126	4862464	462.19	462.19	Bedrock
Orangeville Well2A	570126	4862459	447.70	424.74	Bedrock
Orangeville Well 4	571291	4862749	446.12	442.54	Bedrock
Orangeville Well 5/5A	569536	4862719	457.80	451.68	Overburden
Orangeville Well 6	571383	4860475	426.07	395.28	Bedrock
Orangeville Well 7	570508	4863057	431.55	407.85	Bedrock
Orangeville Well 8A	570816	4864119	434.84	370.54	Bedrock
Orangeville Well 8B	570788	4864232	433.16	366.46	Bedrock
Orangeville Well 8C	570716	4864269	435.68	371.38	Bedrock
Orangeville Well 9A/ 9B	569718	4861869	448.88	410.50	Bedrock
Orangeville Well 10	575103	4862304	358.00	371.00	Overburden
Orangeville Well 11	571726	4861089	428.25	396.46	Bedrock
Orangeville Well 12 –Transmetro	570285	4864350	432.86	411.76	Bedrock
Orangeville Well 13 - Pullen	569335	4864208	418.52	373.32	Bedrock

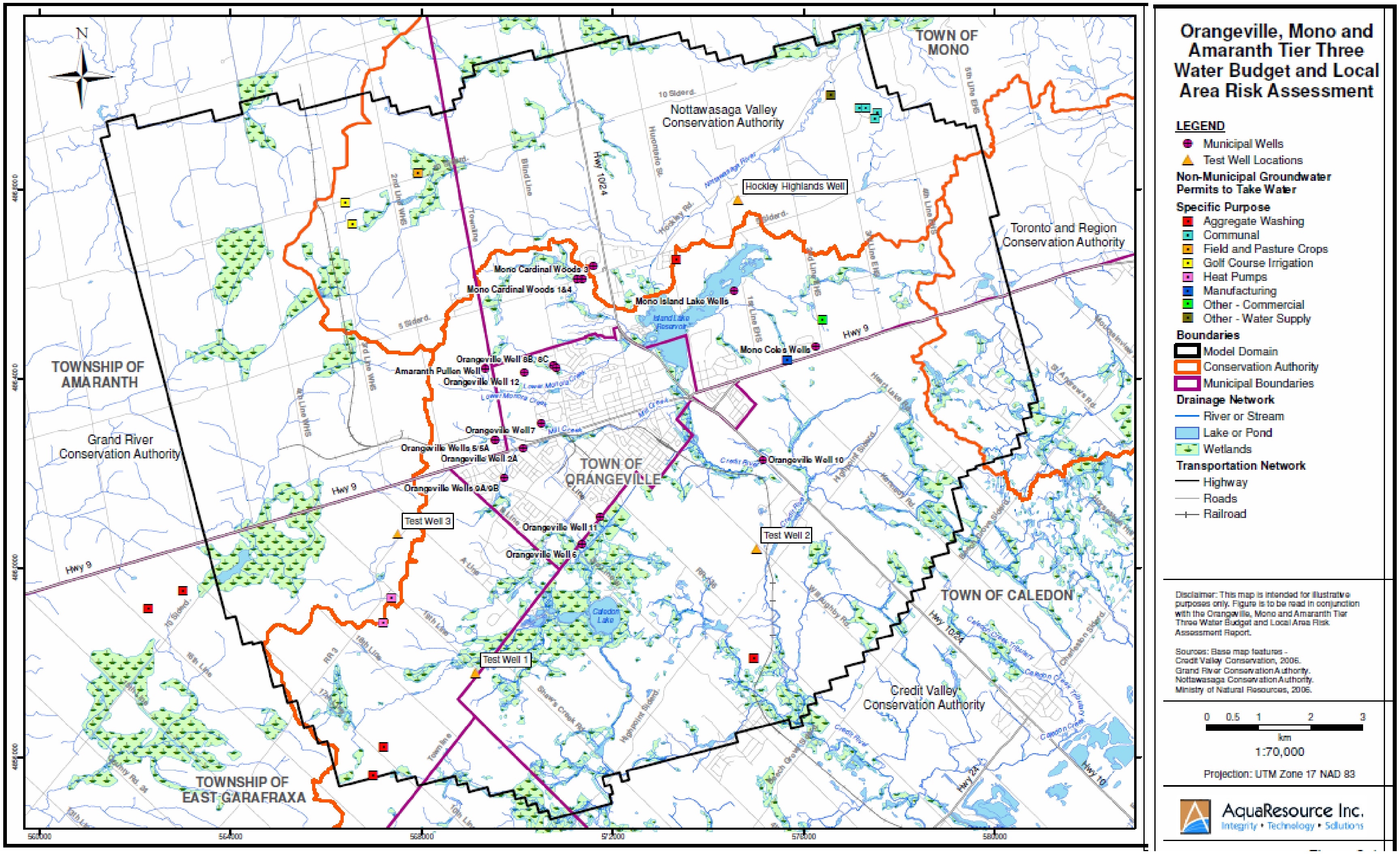

Figure C2-2: Municipal and Non-Municipal Demands

Large, permitted groundwater takers were also simulated in the numerical model. **Table C2-11** below identifies these wells and the average rates that were simulated. They are also shown on **Figure C2-2**.

Table C2-11: Non-Municipal Large Permitted Water Takers

Permit Number	Modeled Average Rate (m^3/day)	General Purpose	Specific Purpose	Expiry Date
03-P-2357	246	Industrial	Aggregate Washing	31-Oct-13
03-P-2393	75	Agricultural	Field and Pasture Crops	31-Aug-14
2415-6GWQ44	166	Industrial	Aggregate Washing	
2867-6UJKLT	192	Commercial	Golf Course Irrigation	31-Dec-08
5474-758JDY	170	Industrial	Aggregate Washing	31-Aug-15
7034-5U6H84	183	Industrial	Aggregate Washing	15-Nov-13
7134-6LJKT4	2	Industrial	Aggregate Washing	31-Aug-15
7722-6VCMHB	144	Industrial	Manufacturing	31-Aug-16
87-P-2018	282	Miscellaneous	Heat Pumps/ livestock	30-Nov-08
90-P-2007	262	Water Supply	Other - Water Supply	31-Mar-10
92-P-2054	223	Industrial	Aggregate Washing	30-Oct-11
97-P-2037	759	Water Supply	Communal	31-Oct-07
99-P-2019	16	Commercial	Golf Course Irrigation	31-Mar-09
99-P-2028	56	Water Supply	Communal	31-Mar-09

As the table shows, the water takings are relatively low when compared to the municipal groundwater takings. Some permits have multiple sources (i.e., wells) and the modeled rate is the sum of the average rates calculated for each well. In some instances, the wells are only pumping a portion of the year (e.g., agricultural irrigation), and the average daily pumping rate simulated in the groundwater flow model was calculated by determining the number of days of the year that the well is pumping, divided by 365 days in the year.

C2.2.7 Transient Model Setup

44-Day Pumping Test

As outlined above, the steady state model was calibrated to average annual conditions. The model calibrated steady state heads were used as the initial heads for the transient 44-day pumping test model simulation.

Computational time intervals in MODFLOW simulations are referred to as 'stress periods', and time-varying (transient) stresses such as changes in groundwater pumping or recharge over time must be constant throughout each stress period. The 44-day pumping test simulation was divided into 15 stress periods to simulate the groundwater extraction and the recovery period, as seen in **Table C2-12**. Groundwater withdrawals are considered constant for the duration of the stress period, and this assumption is consistent with the actual reported pumping rates for the test (Burnside, 2004).

Table C2-12: Stress Periods and Pumping Rates used in Pumping Test Simulation

Stress Period	Start Time (day)	Stop Time (day)	Pumping Rate (m³/day)		
			Pullen Well	Transmetro Well	Dudgeon (8C) Well
1	0	1	1309	0	1165
2	1	1.	1309	0	1165
3	1.	5	1309	982	1165
4	5	6.2	1636	982	1165
5	6.2	8.2	2455	982	1165
6	8.2	12.1	2455	1473	1165
7	12.1	13.2	3273	1473	1165
8	13.2	19.1	3273	1473	1165
9	19.1	20.2	3600	1964	1165
10	20.2	27.1	0	1964	1165
11	27.1	31	0	1964	1165
12	31	34.2	0	1472	1165
13	34.2	39	0	982	1165
14	39	43	0	0	1165
15	43	51	0	0	0

Long Term Model Verification

Validation is another step in the model calibration process whereby the calibrated model output is compared to a different set of field observations than those observations used to calibrate the model. In this assessment, a second transient simulation was undertaken to examine the model predicted and observed responses to municipal pumping within the study area. This step aimed to simulate the head response resulting from groundwater extraction from all wellfields within the Town of Orangeville between 1990 and 2006. The model was set up with monthly varying recharge (output from the calibrated HSP-F model) and average monthly groundwater extraction from each of the Town of Orangeville municipal wells. The 17-year simulation was divided into a total of 204 monthly stress periods, and as noted above, the groundwater pumping and recharge were considered constant throughout each of the stress periods.

C2.3 Groundwater Flow Model Calibration

C2.3.1 Calibration Approach

Numerical groundwater flow models are typically calibrated by systematically adjusting the model input parameters and boundary conditions to determine the optimum match (within an acceptable margin of error) between the model predicted results and field observations. The model's ability to represent observed conditions is assessed qualitatively to assess trends in water levels and distribution of groundwater discharge, and quantitatively to achieve acceptable statistical measures of calibration. The model calibration process in this study included calibration to steady state conditions, a transient calibration to a 44-day pumping test, and it was subsequently verified using average monthly reported municipal pumping rates and recharge data from 1990-2006. The calibration process is iterative and calibrating to two different data sets helped refine the understanding and decrease the uncertainty with the model input parameters.

C2.3.2 Calibration Targets – Long Term Model Verification

In general, a model is considered to be well calibrated qualitatively if there is a good fit between the observed head contours and the model predicted contours, and it may also be considered well calibrated from a quantitative perspective if the model predicted heads and groundwater discharge estimates fall within the range of reported values. The general philosophy followed during this calibration exercise was to achieve calibration results 'as good as possible' using reasonable parameter estimates. Local adjustments to hydraulic conductivity zones were not made without having reliable geology data to support modifications to the conceptual model.

The groundwater flow model is considered to be well calibrated for the following reasons:

- Aquifer and aquitard property estimates are within expected ranges of values, and similar to those used in the CVC watershed scale groundwater flow model, the Town of Orangeville MODFLOW model, and the CVC Island Lake Water Budget model;
- The hydraulic conductivity and recharge estimates produced simulated groundwater levels in the high-quality monitoring and observation wells that are largely within the range of measured values, and the observed groundwater flow gradients are similar to the measured gradient;
- Statistical measures of water level calibration accuracy, including Mean Error, Root Mean Squared Error, and Normalized Root Mean Squared Error are within the acceptable range of typical state of practice 'rules of thumb';
- Simulated groundwater discharge is consistently within the range and distribution of observed values, indicating that the overall groundwater recharge rate is appropriate;
- Simulated groundwater levels with time during the 44-day pumping test are very similar to the measured hydraulic head versus time plots for monitoring wells in the western portions of the Town of Orangeville, suggesting the model is well calibrated to transient conditions; and
- Model predicted groundwater levels over the 17-year period (1990-2007) are very similar to the measured hydraulic heads in monitoring wells over the same period in the western portions of the Town of Orangeville suggest the model is well calibrated to transient conditions.

C2.4 Tier 3 Water Budget Modelling Results

The combined results of the two water budget models produce an improved understanding of the hydrologic and hydrogeologic flow systems. The following sections quantify and outline the water budget components within Subwatershed 19 (headwaters subwatershed) of the Credit River watershed. Each of the components presented were calculated assuming no net change in stored water occurs over the time period 1961 to 2006 and were based on the limitations and assumptions of the long-term climate dataset.

C2.4.1 Groundwater Recharge

Figure C2-3 shows groundwater recharge simulated in the calibrated groundwater flow model. Groundwater recharge is greatest (320 mm/yr.) on the Orangeville Moraine and areas where deposits of sand and gravel are mapped at surface, and recharge is the least (0 mm/yr.) where surface water is mapped at Island Lake. Recharge is also lower within the urban areas where there is a greater percent imperviousness associated with roads, parking lots and buildings.

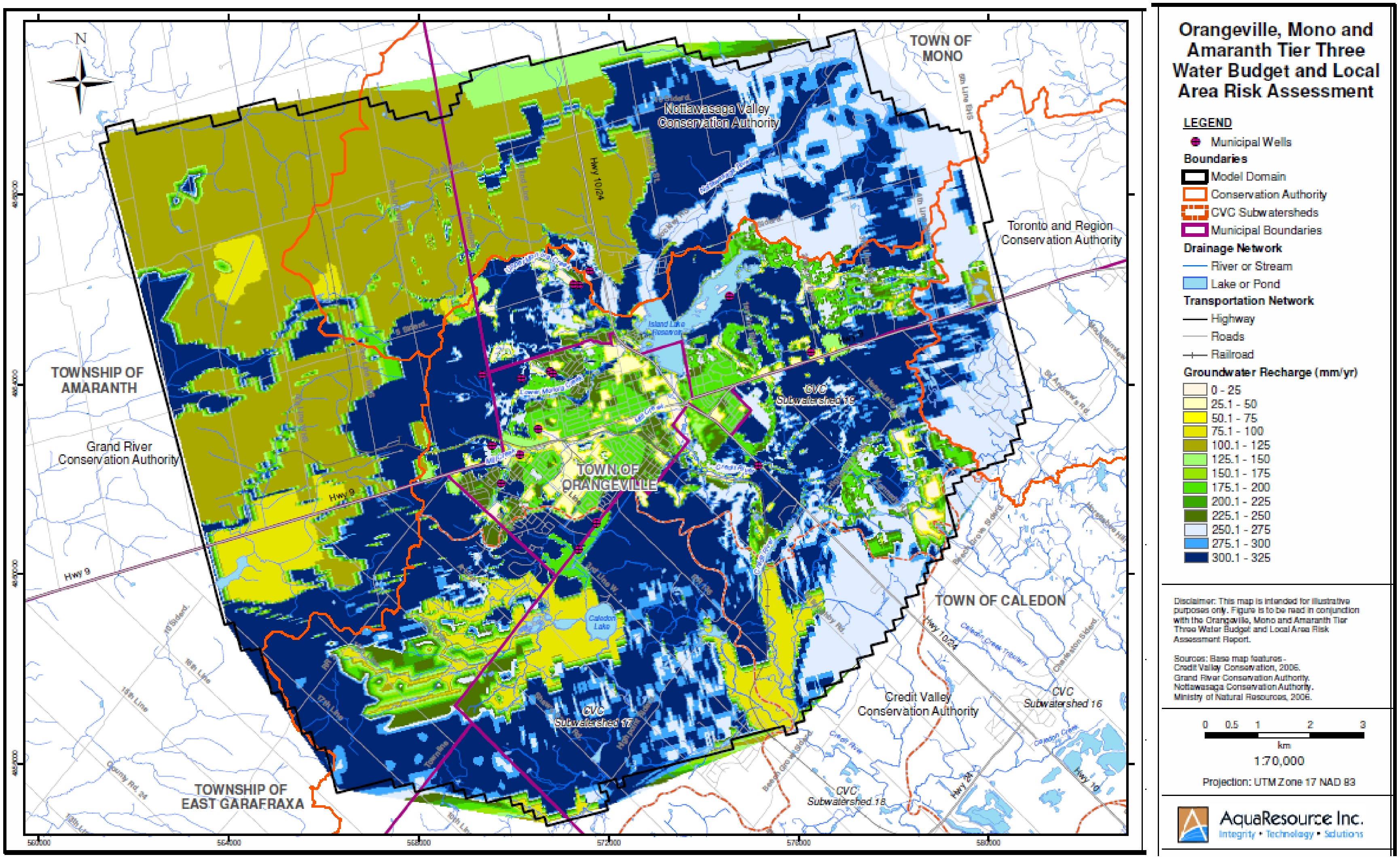

Figure C2-3: Groundwater Recharge

C2.4.2 Water Table Contours

Figure C2-4 illustrates the model predicted water table contours produced in the steady state groundwater flow model. As illustrated, water table contours generally mimic the ground surface topography, and flow converges towards the higher order streams and wetlands. The groundwater elevation contours generally compare well with the observed water level contours.

The largest horizontal gradients (tightly spaced contours) are observed at regional discharge locations, which include the Nottawasaga and Credit rivers.

C2.4.3 Bedrock Water Level Contours

Figure C2-5 illustrates the model predicted bedrock (Amabel Formation) potentiometric surface (i.e., water level elevation) contours within the study area. The water level contours are similar to the overburden water levels; however, the bedrock water levels exhibit more subdued hydraulic gradients, influenced by groundwater recharge and discharge areas. Flow in the bedrock is heavily influenced by the Credit River and the Caledon Lake wetland complex in Subwatershed 17 where the overburden is coarse-grained and the streams are in good hydraulic connection with the underlying bedrock flow system.

Figure C2-5 also illustrates cross-boundary groundwater flow between Subwatershed 19 and the adjacent watersheds (and subwatersheds) as simulated in the calibrated groundwater flow model. These major cross-boundary flow terms are summarized in **Table C2-13**.

Table C2-13: Summary of Cross-Boundary Groundwater Flow

Boundary	Cross Boundary Flow (m^3/d)
Grand River Watershed Into Subwatershed 19.	+5,000
Subwatershed 17 into Subwatershed 19 (West)	+3,200
Subwatershed 17 into Subwatershed 19 (East)	+800
Nottawasaga River Watershed into Subwatershed 19 (West)	+1,900
Subwatershed 19 into Nottawasaga River Watershed (East).	-8,400
TRCA Watershed Into Subwatershed 19	+200
Net Cross-Boundary Groundwater Flow	**+2,700**

As listed above, cross-boundary flows into Subwatershed 19 are significant along the boundaries with the Grand River watershed to the west and Subwatersheds 17 and 18 to the south. These flows are interpreted to be induced by hydraulic gradients resulting from municipal pumping. Cross-boundary flow out of Subwatershed 19 to the north is significant along the eastern boundary with the Nottawasaga watershed (east of Island Lake). These flows are induced by the steep hydraulic gradients into the deeply incised Nottawasaga River valley located north of the Island Lake Reservoir.

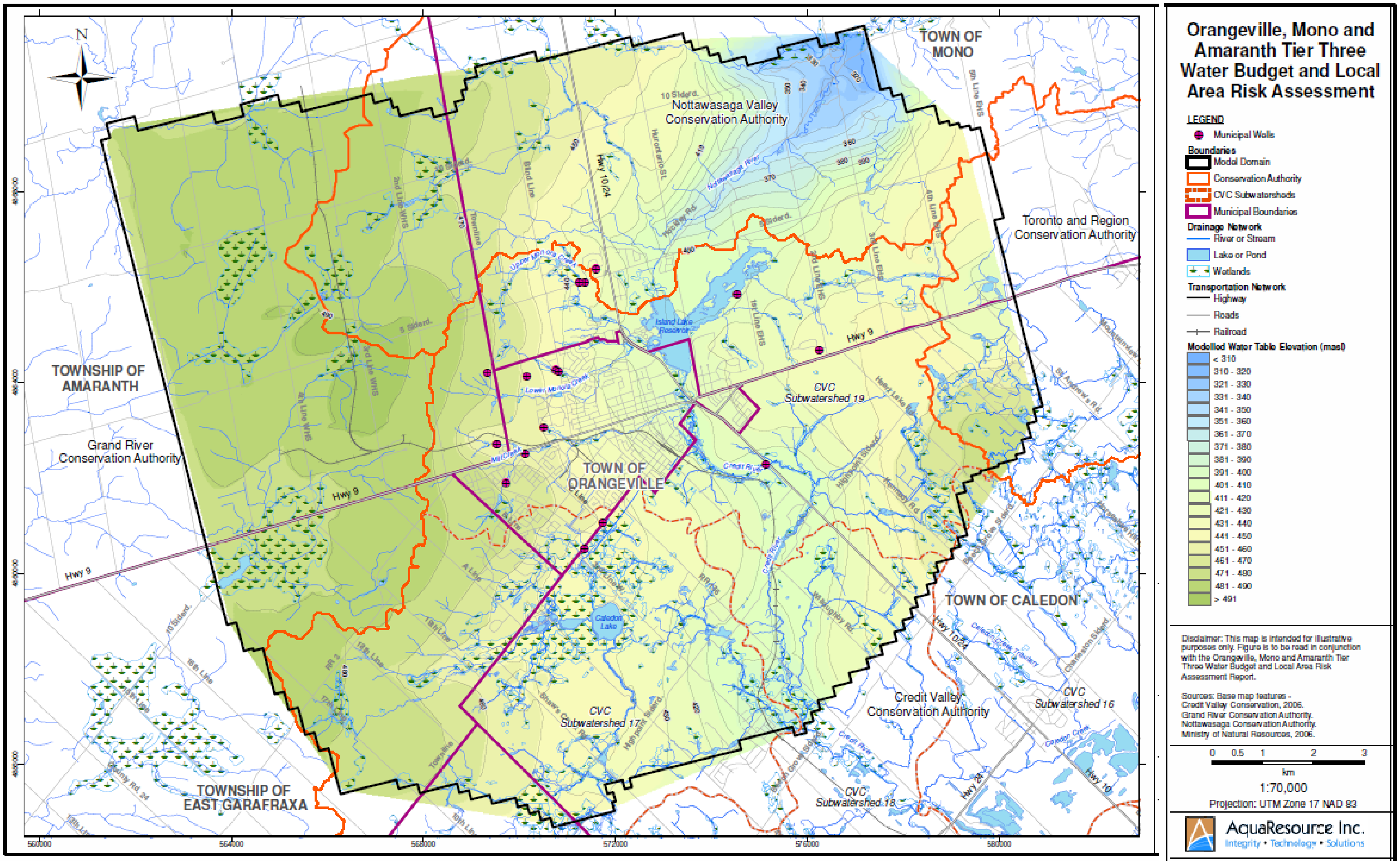

Figure C2-4: Model Predicted Water Table Elevation

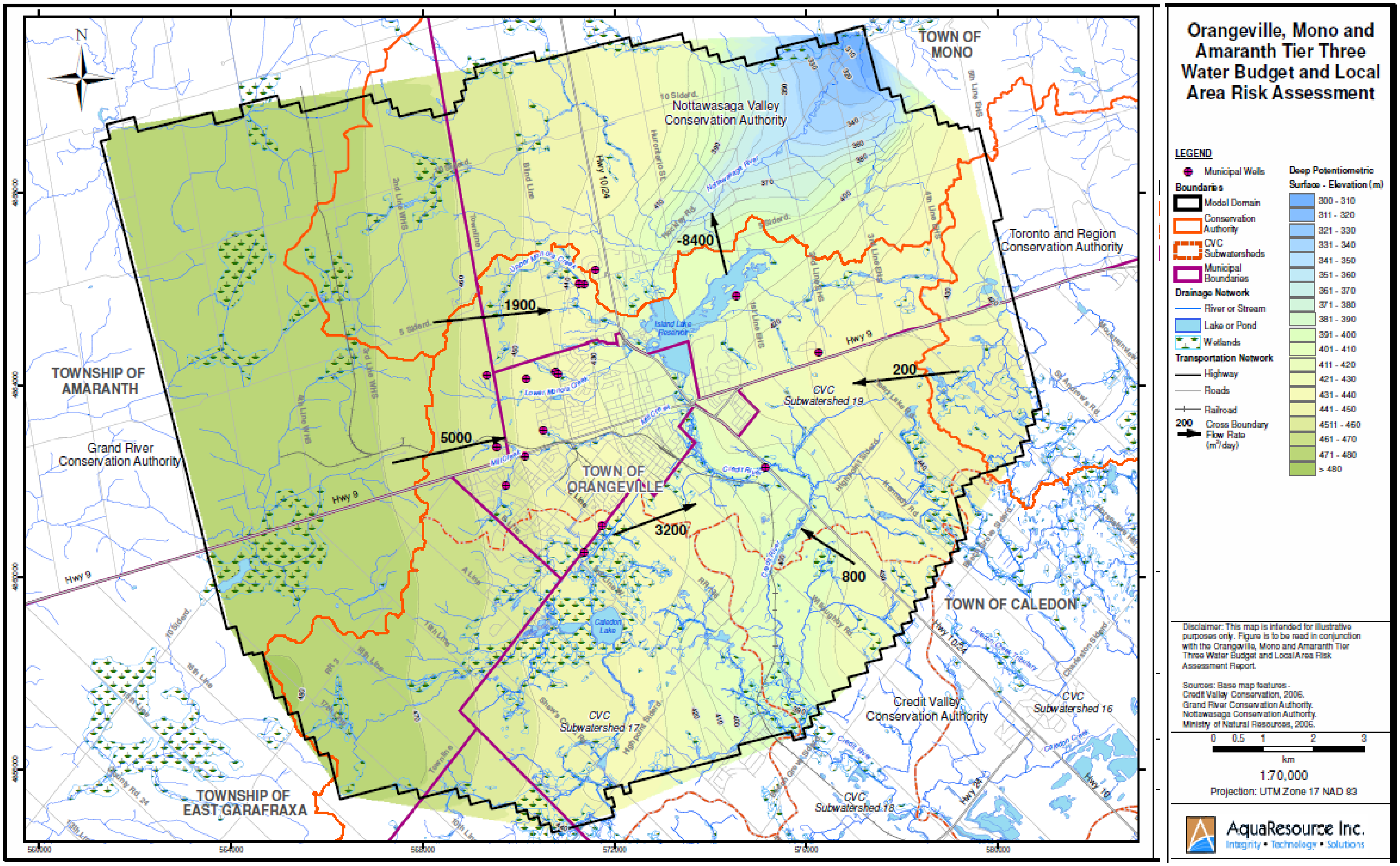

Figure C2-5: Model Predicted Potentiometric Surface (Amabel, with Cross-boundary Flow)

C2.4.4 Vertical Hydraulic Head Difference

Figure C2-6 illustrates the simulated vertical hydraulic head difference across the study area, calculated as the difference in head between the water table and bedrock (Amabel Formation) potentiometric surface. The map is shaded to show where groundwater heads are directed upwards (green) or downwards (blue). Areas where the water table elevation is lower than the bedrock potentiometric surface are predicted along the Credit River and its tributaries, and some wetland complexes; a reflection of groundwater discharge to those areas. Large areas of groundwater discharge are predicted in the Caledon Creek Wetland Complex, along Monora Creek and the Credit River; these are consistent with observed groundwater discharge into the rivers, creeks, and wetlands. The greatest downwards potential between water table and the deep potentiometric surface is present along the crest of the Orangeville Moraine where shallow overburden groundwater recharges the underlying bedrock aquifers.

C2.5 Subwatershed 19 Water Budget

As part of the water budget process, estimates of the water budget component fluxes were used to better understand the processes contributing to the water budget in the area.

Table C2-14 summarizes the estimated overall water budget fluxes for Subwatershed 19. The table summarizes watershed inflows including precipitation, wastewater influent, and groundwater. Outflows include Evapotranspiration, Streamflow (Credit River), Groundwater Pumping, and Groundwater Flow. The water budget parameters are calculated based on information derived from both the surface water and groundwater flow models and are presented in units of m^3/d, mm/year, and as a percentage of precipitation.

Table C2-14: Overall Water Balance Table (Subwatershed 19)

	Flow (m^3/d)	Flow (mm/yr)	Percent of Precipitation
Inflows			
Total Precipitation	148,500	891	100%
Groundwater Flow In			
Flow from GRCA into Sub 19	5,000	30	3%
Flow from Subs 17 and 18 into Sub19	4,000	24	3%
Flow from NVCA into Sub 19	1,900	11	1%
Flow from TRCA into Sub 19	200	1	0%
Total Inflow	**159,600**	**958**	**108%**
Outflows			
Evapotranspiration	93,200	560	63%
Streamflow (Melville)	58,000	348	39%
Groundwater Flow Out			
Flow from Sub 19 into NVCA	8,400	50	6%
Total Outflow	**159,600**	**958**	**108%**

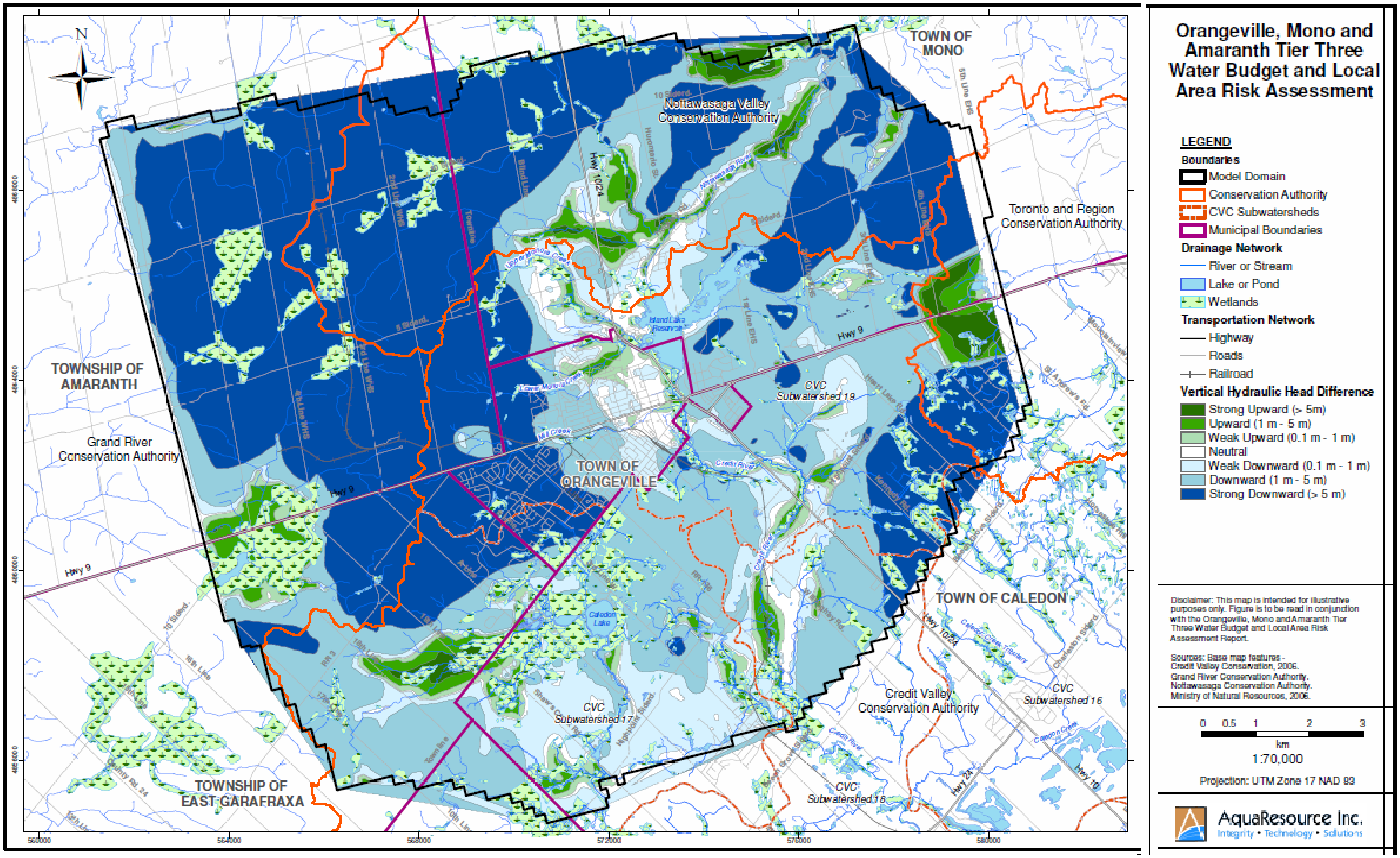

Figure C2-6: Vertical Hydraulic Head Difference

Table C2-15 shows the average annual precipitation in Subwatershed 19 is 891 mm/yr. as measured at the MOECC Orangeville climate station. This translates to a rate of 148,500 m^3/d. Groundwater modelling results indicate that a fairly significant amount of groundwater flows into Subwatershed 19 across the subwatershed boundaries. Approximately 5,000 m^3/d flows into the subwatershed from the Grand River watershed, and an additional 4,000 m^3/d flows into the Subwatershed 18 from Subwatershed 17 to the south. Much of the cross-boundary flow from the Grand River is interpreted to be in response to municipal pumping.

Outflows from Subwatershed 19 include evapotranspiration, streamflow, groundwater pumping, and groundwater flow. Average annual evapotranspiration is approximately 560 mm/yr. Average annual streamflow, as measured at Water Survey of Canada Melville Gauge is 58,000 m^3/d, or 348 mm/yr across the subwatershed. Approximately 8,400 m^3/d of groundwater flows to the north out of Subwatershed 19 into the Nottawasaga River watershed along the eastern boundary of the subwatershed. This flow to the north is driven by the steep hydraulic gradient into the valley north of the Island Lake Reservoir.

Table C2-15 contains the water balance for groundwater within Subwatershed 19. The water budget models predict an average annual groundwater recharge rate of 237 mm/yr, or 39,500 m^3/d into Subwatershed 19. As shown in **Table C2-15**, groundwater inflows into Subwatershed 19 are approximately 11,100 m^3/day, representing approximately 7% of the total recharge.

Table C2-15: Water Balance, Groundwater (Subwatershed 19)

	Flow (m^3/d)	Flow (mm/yr)	Percent of Precipitation
Inflows			
Groundwater Recharge	39,500	237	100%
Flow from Sub 17 into Sub 19	4,000	24	10%
Flow from NVCA into Sub 19	1,900	30	5%
Flow from TRCA into Sub 19	200	1	1%
Flow from GRCA into CVC	5,000	30	13%
Total Groundwater Inflow	**50,600**	**304**	**128%**
Outflows			
Surface Water Discharge	34,800	208	88%
Permitted Wells	7,400	44	19%
Flow out of Sub 19 into NVCA	8,400	50	21%
Total Groundwater Outflow	**50,600**	**304**	**128%**

Groundwater outflows include discharge to surface water (streams and wetlands), groundwater wells, and groundwater flow out of the watershed. Total groundwater discharge to surface water is approximately 34,800 m^3/d or 208 mm/yr. Groundwater pumping is 7,400 m^3/d, or approximately 19% of the total recharge into the subwatershed. Groundwater flow into the Nottawasaga River watershed from Subwatershed 19 is 8,400 m^3/d or 21% of the total recharge into the subwatershed.

C2.6 Water Demand

C2.6.1 Municipal Water Demand

The Town of Orangeville, Town of Mono and Township of Amaranth have municipal water supplies within the study area. Each relies entirely on groundwater for their municipal drinking water needs.

Efforts to collect and confirm water demand estimates include:

- Review of the MOECC's Water Taking Reporting System (WTRS) to incorporate actual pumping rates for permit holders;
- Review of monitoring reports and discussions with permit holders to ensure that site conditions and operating practices are incorporated into the consumptive demand estimate, if possible; and
- Site visits if warranted to better estimate consumptive water use.

C2.6.2 Town of Orangeville

The Town of Orangeville relies solely on groundwater for its municipal water supply demands, and it obtains its water from twelve municipal wells located in nine wellfields. The town's wells are illustrated on **Figure C2-2** and listed below in **Table C2-16**.

Table C2-16: Town of Orangeville Water Supply Wells

Wellfield	Well Name	Alternate Name	Permitted Capacity (m^3/d)	Aquifer Type
Well 2A	Well 2A		1,308	Bedrock
Well 5	Well 5		6,000	Overburden
	Well 5A			
Well 6	Well 6		3,600	Bedrock
Well 7	Well 7	Passmore Well	1,310	Bedrock
Well 8	Well 8B	Dudgeon Wells	655	Bedrock
	Well 8C		655	Bedrock
Well 9	Well 9A	Montgomery Village Wells	878	Bedrock
	Well 9B			Bedrock
Well 10	Well 10		1,452	Overburden
Well 11	Well 11		1,308	Bedrock
Well 12	Well 12	Transmetro Well	1,308	Bedrock

Wells 2A, 7 and 9A/9B are located close to each other in the southwestern portion of the Town of Orangeville. These wellfields pump water from the Guelph and Amabel Formation bedrock aquifers.

Wells 5/5A are located in the Township of Amaranth, west of the Orangeville-Amaranth Townline. The wells are screened 11.6 to 17.6 m below ground surface in an unconfined sand and gravel aquifer.

Wells 6 and 11 lie in the southern reaches of the Town of Orangeville along the Orangeville-Caledon Townline Road. Well 6 is an open-hole bedrock well that extends to 30 m below ground surface, and Well 11 extends approximately 55 m below ground surface in the Guelph-Amabel Formation bedrock.

Wells 8B, 8C and 12 are bedrock wells located in the northwestern reaches of the Town of Orangeville. The wells are located near the North Arm of Lower Monora Creek and are completed in the Amabel Formation bedrock aquifer.

Well 10 is an overburden well located adjacent to Highway 10 and the Credit River, in the Town of Caledon (Region of Peel). The well is screened from approximately 55 to 61 m below ground surface in a buried bedrock valley.

C2.6.3 Town of Mono

The wells that comprise the municipal supply system in the Town of Mono are listed below in **Table C2-17** and illustrated on **Figure C2-2**.

Table C2-17: Town of Mono Water Supply Wells

Wellfield	Well Name	Permitted Capacity (m^3/d)	Aquifer Type
Cardinal Woods	Cardinal Woods Well 1 (MW1)	817	Bedrock
	Cardinal Woods Well 3 (MW3)	1,571	Bedrock
	Cardinal Woods Well 4 (MW4)	753	Bedrock
Coles	Coles Subdivision 1 (Coles 1)	570	Overburden
	Coles Subdivision 2 (Coles 2)	570	Overburden
Island Lake	Island Lake PW1	1,958	Overburden
	Island Lake TW1	820	Overburden
	Island Lake PW2/06	Inactive	Overburden

The Cardinal Woods Wellfield is located north of the Town of Orangeville, on the west side of Highway 10 near 5th Sideroad. The three wells in the wellfield pump water from the Amabel Formation bedrock aquifer, with Well 3 (MW3) operating as the main duty well, and wells 1 and 4 (MW1 and MW4) operating as backup wells.

The Coles Wellfield is located southeast of the Island Lake Reservoir along Highway 9. The two wells (Coles 1 and Coles 2) are screened in an overburden aquifer from 61 to 82 m below ground surface in a fine-grained sand aquifer.

The Island Lake Wellfield consists of two active wells located southeast of Island Lake near First Line E. The wells are screened from approximately 52 to 59 m below ground surface in a sand and gravel aquifer.

C2.6.4 Township of Amaranth

The Township of Amaranth plans to use the Pullen Well to service a rural residential subdivision located west of the Town of Orangeville (**Figure C2-2**). The well is completed within the Amabel Formation bedrock aquifer.

C2.6.5 Safe Additional Drawdown

Safe additional drawdown is defined as the additional depth that the water level within a pumping well could fall while maintaining that well's allocated pumping rate. To establish the safe additional drawdown for each municipal pumping well within the study area, the following components need to be evaluated or calculated for each municipal well:

1. Safe water level elevations. The lowermost elevation within the municipal pumping well that an Operator can pump a well: this elevation may be related to the well screen elevation, pump intake elevation or similar operational limitations;
2. Existing (2008) water level elevations in the pumping wells. The elevation of the observed

average annual pumped water level within each municipal well;

3. Estimated non-linear well losses at each well. Drawdown within the well in response to well inefficiencies (e.g., entrance losses, turbulent flow around pump fittings) created during groundwater extraction; and
4. Convergent head losses at each well. MODFLOW does not specifically simulate the water level at the location of a well located within a grid cell. Additional water level drawdown is referred to as convergent head loss and can be quantified to properly predict the pumped water level in a well.

Each of the above terms is discussed in detail below.

The safe water level elevation at each municipal water supply well within Orangeville was supplied by Town of Orangeville Public Works staff. Their safe water level elevations were developed based on the elevation at the top of the well screen, the elevation of the pump intake and other pump settings (i.e., low level lockout elevation), which included a measure of safety to account for seasonal water level fluctuations and well losses that may not be accounted for in the groundwater flow model. The safe water level elevations for the Town of Mono wells and the Pullen well were based solely on the pump intake elevations as operational considerations were not available. **Table C2-18** lists the safe water level elevations for the municipal wells within the study area, while **Figure C2-7** shows the derivation of safe additional drawdown at Orangeville Well 8C.

The pumped water level elevation in the production aquifer in each municipal well in 2008 (existing conditions year) was obtained by examining water level hydrographs derived from data loggers within each municipal pumping well. As noted above, the difference between the safe water level elevation and the existing pumped water level elevation is referred to as the safe available drawdown for the well.

As an important additional step in evaluating the safe additional drawdown, an analysis of losses at each wellhead was undertaken. Well losses include convergent head losses, and non-linear in-well losses were also undertaken.

C2.6.6 Convergent Head Losses

The MODFLOW groundwater flow model calculated the average water level in the grid block that contains the well. Since a well is relatively small compared a typical grid block, MODFLOW underestimates the drawdown at a pumping well. Therefore, the model results need to be adjusted to compare simulated drawdown at a well against safe available drawdown. The additional head losses that occur between the average water level in a grid block and the pumping well are referred to as the convergent head losses. For each municipal production well, the additional drawdown due to convergent head losses can be approximated using the following equation (Peaceman, 1983):

$$\Delta s_{well-block} = \frac{\Delta Q}{2\pi T} \ln\left\{\frac{0.208\ \Delta x}{r_w}\right\}$$

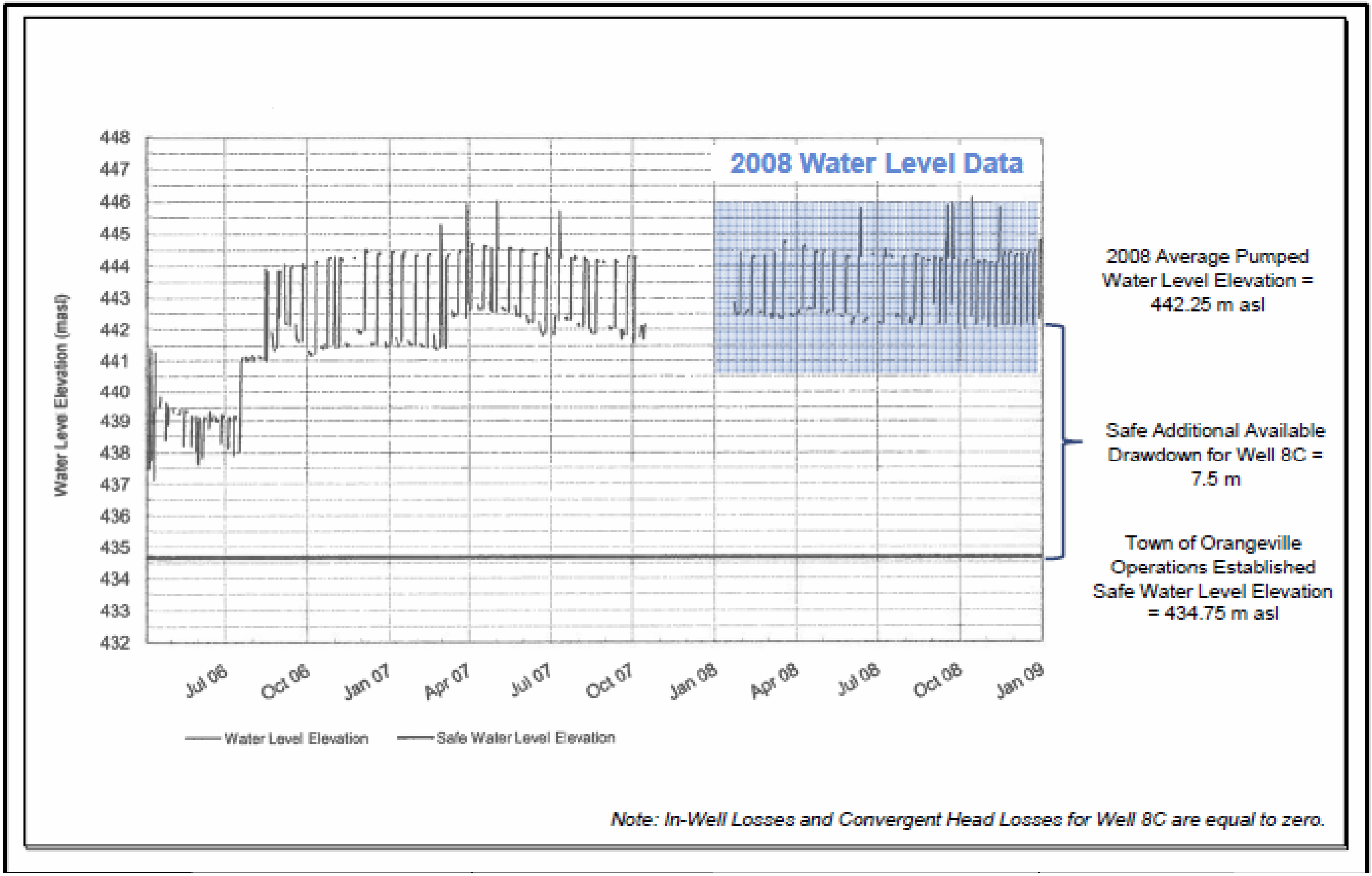

Figure C2-7: Safe Additional Drawdown Calculation – Orangeville Well 8 C

where ΔQ is the increase in the pumping rate with respect to 2008 conditions, T is the cumulative transmissivity of the model layers across which the well is screened, Δx is the MODFLOW grid spacing, and r_w is the radius of the pumping well.

Table C2-18: Safe Additional Drawdown (2008 Conditions)

Well Name	Top of Screen (overburden)	Top of Open Hole (Bedrock)	Pump Intake	Safe Water Level	Existing Pumped Water Level (2008)	Safe Additional In-Well Drawdown (m; 2008)
	Elevation (masl)					
Orangeville 2A	-	441.17	n/a	442.00	447.50[2]	5.5
Orangeville 5/ 5A	457.47	-	454.50	458.05	461.25	3.2
Orangeville 6	-	426.42	n/a	432.00	435.70	3.7
Orangeville 7	-	431.55	n/a	435.40	445.50	10.1
Orangeville 8B	-	432.52	432.13	433.80	441.50	7.7
Orangeville 8C	-	436.47	432.13	434.75	442.25	7.5
Orangeville 9A/ 9B	-	453.76	455.00	456.00	460.75[3]	4.8
Orangeville 10	358.00	-	355.26	365.26	402.00	36.7
Orangeville 11	-	427.20	n/a	427.12	434.50	7.4
Orangeville 12	-	436.63	435.55	436.55[1]	450.25	13.7
Mono Cardinal Woods 1	-	418.16	417.80	418.80[1]	423.63	4.8
Mono Cardinal Woods 3	-	412.70	411.44	412.44[1]	415.41	3.0
Mono Coles 1 and 2	395.00	-	385.00	386.00[1]	420.72[4]	34.7
Mono Island Lake Wells	368.80	-	368.47	369.47[1]	391.60	22.1
Pullen Well	-	418.52	445.93	446.93[1]	478.00[5]	31.1

Notes:
n/a- not available
[1] Safe water level elevation based on pump intake elevation plus 1 m
[2] Transducer not working in 2008. Average in fall 2007 was 447.5 masl
[3] Problems with SCADA data (2008); used 2009 average pumped water level elevation
[4] Pumped water levels in Coles 1/2 were unavailable; static water level from well log is cited instead.
[5] Well not pumping in 2008; used static water level elevation from well log.

Table C2-18 shows the convergent head losses, calculated for each municipal well, where there is an increase in pumping from the current to the planned system rates. (Note: in the MODFLOW model, each municipal pumping well lies within a 12 m x 12 m grid cell).

C2.6.7 Non-Linear In-Well Losses

Well losses refer to the difference between the theoretical drawdown in a well and the observed drawdown and are due to factors such as turbulence in the well itself as water flows into the pump. These well losses need to be considered in the Tier 3 assessment, as the additional available drawdown refers specifically to the water level in the well and not the average water level in the aquifer in the vicinity of the well. The in-well losses are calculated as the additional drawdown that is expected within the pumping well due to the incremental increase from the existing to the allocated rates.

The two components of observed drawdown in a given pumping well are described in the following equation (Jacob, 1947; Hantush, 1964; Bierschenk, 1963);

$$\Delta s_{in\text{-}well} = BQ + CQ^2$$

Where s is drawdown, Q is the pumping rate, B is the aquifer loss coefficient, which increases with time (Theis, 1935), and C is the well loss coefficient, which is constant for a given pumping rate. The first term of the equation (BQ) describes the linear component of the drawdown (i.e., doubling the pumping rate leads to a doubling of the drawdown). This term accounts for the head losses in the formation in the vicinity of the well. The second term of the equation (CQ^2) describes the non-linear well-loss component of drawdown (Jacob, 1947) in the well itself: this is the additional component that was quantified in this assessment.

Non-linear in-well losses are estimated using step test results. Step tests are hydraulic tests where a pumping well is pumped at a series of pumping rates and the drawdown throughout the test is recorded. Non-linear well loss coefficients were estimated using the step test results presented in Burnside (2005) and Burnside and Gartner Lee (2004).

Well loss coefficients were calculated by plotting graphs of specific drawdown (drawdown divided by pumping rate) against time for the individual step tests. Plotting the specific drawdown against time (time since the start of the test) will yield observation points that lie along a straight line as drawdown increases with increasing time. The slope of the line fit to the data points is equal to the well loss coefficient (C). The results of the step tests were plotted using this process to estimate the well loss coefficient, which was then used in the following equation (Jacob, 1947) to calculate the drawdown due to in-well losses for the increased pumping from existing to the allocated rates:

$$\Delta s_{in-well} = C\left[\left(Q_{2008} + \Delta Q\right)^2 - Q_{2008}^2\right]$$

Where Q_{2008} is the existing (2008) pumping rate, and ΔQ represents the increase in pumping from existing to the allocated rates. Based on this analysis, the in-well losses were calculated for each well in the study area and are listed in **Table C2-18.**

C2.6.8 Total Safe Additional Drawdown

The MODFLOW model predicts water levels in the aquifer at a pumping well, and not within the pumping wells themselves. As such, the in-well losses and convergent head losses had to be summed together and added to the drawdown predicted by MODFLOW for each scenario and this value is then compared to the safe additional in-well drawdown (far right-hand column of **Table C2-14**) to assess whether the pumping well can sustain pumping at the elevated rates.

For simplicity, the in-well losses and convergent head losses were summed together and subtracted from the safe additional in-well drawdown at each well to derive one estimate of total safe additional aquifer drawdown at each pumping well, as shown in **Table C2-19**. This value was then compared to the modelled drawdown in each of the Risk Assessment scenarios to assess whether the proposed land use development and/or increased pumping to the allocated rates are sustainable, or if the water level in the well is predicted to fall below the safe additional available drawdown level cited in the table.

Table C2-19: Total Safe Additional Drawdown (m)

Well Name	Drawdown (m)			
	Safe Additional In-Well Drawdown	Well Losses	Convergent Head Loss	Safe Additional Aquifer Drawdown[1]
Orangeville 2A	5.5	0.58	0.25	4.7
Orangeville 5/5A	3.2	0.02	0.02	3.2
Orangeville 6	3.7	0.18	0.58	2.9
Orangeville 7	10.1	0.24	1.16	8.7
Orangeville 8B	7.7	0.13	0.17	7.4
Orangeville 8C	7.5	0.00	0.00	7.5
Orangeville 9A/9B	4.8	0.00	0.52	4.3
Orangeville 10	36.7	0.00	0.79	35.9
Orangeville 11	7.4	0.16	1.01	6.2
Orangeville 12	13.7	1.49	0.74	11.5
Pullen Well	31.1	0.01	0.65	30.4
Mono Cardinal Woods 1	4.8	0.00	0.00	4.8
Mono Cardinal Woods 3	3	0.02	0.00	3.0
Mono Island Lake Wells	22.1	0.03	0.07	22.0
Mono Coles 1 and 2	34.7	0.00	0.02	34.7

[1] Calculated by subtracting the sum of the well losses and convergent head losses from the safe additional in-well drawdown.

C2.6.9 Historical Water Demand and Conservation Measures

Over the past few decades, population growth within the Town of Orangeville resulted in increasing demands placed on the groundwater system. The town experienced peak demands on the groundwater system between 2000 and 2002, which led them to proactively initiate a series of water conservation measures to reduce groundwater demands. The resulting reduction in water demand from the instituted water conservation measures was significant; between 2003 and 2005, the average maximum daily demand decreased by approximately 18%.

Table C2-20 outlines the water conservation measures implemented by the town since the year 2000 to reduce the maximum daily and average annual water demands.

Table C2-20: Water Conservation Measures

Initiative	Description	Implementation Date
Water Use Audit	Home audits offered as part of the WaterCare program.	2000 and 2001
Universal Metering	Universal metering program began for residents within the Town.	2002; metering started in Jan 2003
Water Efficient Fixtures	Water efficient fixtures were offered and distributed through the WaterCare and Universal Metering programs.	
Toilet Replacement Program	Town reimbursed residential, commercial, industrial and institutional water users $50 for each >13 L/flush toilet that replaced a Town-approved 6 litre per flush low-flow toilet.	November 2005
Leak Detection	Watermain leak detection program.	2000- present
Public Information, Education and Outreach	Posted water conservation tips on the website, and made copies of Canada Mortgage and Housing Corporation's "Household Guide to Water Efficiency" booklet available to residents.	2000- present

C2.6.10 Population Growth and Demand

For the Tier 3 assessment, the hydrologic and hydrogeologic response to the increase in municipal pumping associated with committed and planned demands needs to be assessed.

Town of Orangeville

In November 2001, Orangeville initiated a Long-term Servicing Strategy (LTSS) (Burnside, 2004) to develop a sustainable strategy for the town's future water supply and sewage treatment servicing. The LTSS strategy considered future environmental, social, economic, technical, and political implications for each of the servicing options. The key recommendations from the LTSS were to provide future water services via additional groundwater wells and future sewage servicing by expanding and upgrading the existing Water Pollution Control Plant (WPCP). Technical studies are currently underway as part of a Class Environmental Assessment examining upgrades to the WPCP.

The Official Plan for the Town of Orangeville (2007) specifies the location and type of land use development that will take place within the town's municipal boundary. Using the Official Plan, and existing Town of Orangeville housing densities, population projections within the Town of Orangeville (2008) were estimated. Consideration was given at the onset of the study to incorporate information from the Places to Grow Act, which was considered by the town to encourage a greater population in the town than the municipal water and sewage services could support. As such, the build-out of the town using population projections specified in the Places to Grow Act was not considered in this study.

The Town of Orangeville's estimated build-out population was estimated to range from 33,000 to 34,000 people, based on the 2007 Official Plan and the current Orangeville housing density. A population of 34,000 was used as the basis for the Tier Three assessment. The estimated population in the town (2008) is 31,119, and as such, the population increase to reach the build-out population was estimated to be approximately 2,881. Population growth to this level is possible in the near future (e.g., less than 10 years).

The Town of Orangeville estimated the average annual water demand needed to support the 34,000 build-out population is 11,529 m^3/d. This population projection and associated water demand estimate do not reflect any changes to the Official Plan in response to the Province's Places to Grow Legislation. However, they reflect population and per capita demand increases, as well as increases in industrial, commercial, and institutional water demands for specific developments in the town.

The Town of Orangeville typically relies on a maximum day factor of 1.5, which is the ratio of the maximum daily water demand to the average day water demand, when considering the capacity of their water supply system. Dividing the town's total permitted rate of 17,333 m^3/d by the maximum day factor results in 11,555 m^3/d, which can be considered the maximum average day pumping rate that the Orangeville system could sustain providing that all pumping wells are operating as intended. From these results, it can be concluded that the currently permitted pumping wells could meet the town's build-out requirements providing that none of the wells were taken off-line and all were operating as intended. Obviously, this scenario is not practical for the town, and backup wells are necessary in case of contamination or maintenance problems. However, the maximum average day pumping rate is suitable for the Tier 3 assessment to represent future requirements from the system.

Table C-39 contains existing plus committed plus planned rates for the town's pumping wells. These values were initially estimated as the total permitted rate divided by 1.5. The final values were obtained based on feedback from the town based on estimated maximum yields from each well and also running the model iteratively until the model indicated that the assigned pumping rate could be sustained over the long-term. The existing plus committed plus planned pumping rates are the outcome of this exercise, and they represent average annual rates that could be sustained by the Town of Orangeville supply wells. It is recognized that higher daily pumping rates may be temporarily experienced by the wells; however, these elevated rates cannot be sustained over the long-term due to operational limitations.

Town of Mono

The Town of Mono contains a number of small communities and the majority of the residents in these areas are serviced by individual private groundwater wells. There are four municipally-serviced subdivisions in the Town of Mono: Coles, Island Lake, Purple Hill and Cardinal Woods subdivisions. Current legislation, including the *Oak Ridges Moraine Conservation Plan*, *Greenbelt Act,* and the Niagara Escarpment Commission (NEC) limit development to the existing approved developments within the Town of Mono.

The Town of Mono expects to increase pumping at several wells in the future to service existing development plus two new residential developments that have been approved but are not yet been constructed. As such, these municipal demands are classified as existing plus committed demands. The developments are proposed to have 175 and 300 units each. The estimated demand for these developments was calculated as 0.75 m^3/d per unit, which is consistent with the nearby Town of Orangeville's water demand calculation for single family dwellings. Town of Mono staff estimated the water for the 175-unit residential subdivision would come from the Cardinal Woods Wellfield, and the 300 unit subdivision in the Purple Hill area would be supplied by the Island Lake Wellfield. This equated to long-term planned demand rates of 392 m^3/d for the Cardinal Woods Wellfield, 116 m^3/d from the Coles Wellfield, and 347 m^3/d from the Island Lake Wellfield. These projected water demands may not be required for a few years, depending on construction and occupancy. While the estimated water demand for with these developments is not represented within the Existing Conditions scenario, these demands are considered part of the committed demand for the Town of Mono.

The Town of Mono is identified within as a part of Ontario's Greenbelt, and the *Greenbelt Plan* (2005) prohibits urbanization in an attempt to provide permanent protection to the agricultural lands and the ecological features and functions of the landscape. Consequently, no additional development can occur within the Town of Mono beyond the approved subdivisions noted above.

Township of Amaranth

The Pullen Well is not currently pumping as it is expected to service a rural residential subdivision in the coming years. As such, this is considered part of the Planned Demands. The permit for the Pullen Well recently expired and a new permit to take water has been requested and is being reviewed by the Ministry of the Environment and Climate Change. Based on the number of dwellings proposed, and the estimated demand per dwelling, the Planned Demand rate for the well was estimated by the Town of Amaranth to be 220 m^3/d.

C2.6.11 Non Municipal Water Demand

In addition to the municipal water takers, there are also other large, permitted water takers within the study area. **Figure C2-2** shows the locations of these takers, while **Table C2-21** outlines permit number, as well as the general and specific purpose for the permit. There are 13 non-municipal permits to take water and they are issued for a variety of purposes.

Table C2-21: Non-Municipal Groundwater Permits within the Study Area

Permit Number	Easting	Northing	Maximum Permitted Taking (m^3/d)	General Purpose
03-P-2393	567915	4868343	982	Agricultural / Field and Pasture Crops Irrigation
5474-758JDY	573320	4866520	4,080	Aggregate Washing/ Processing
8074-77WH6J	567195	4856211	7,920	Aggregate Washing
8225-72DKX3	574950	4858090	5,460	Aggregate Washing/ Processing
8543-7LQKPC[1]	576384	4865231	356	Commercial/ Golf Course Irrigation
90-P-2007	576558	4869977	262	Communal / Water Supply
92-P-2054	566972	4855604	5,940	Aggregate Washing/ Processing
99-P-2019	566397	4867719	7	Golf Course Irrigation
99-P-2019	566547	4867267	41	Commercial
99-P-2028	577161	4869708	55	Communal
99-P-2028	577531	4869620	55	Communal
99-P-2028	577478	4869501	55	Communal
99-P-2028	577290	4869720	55	Communal
87-P-2018[2]	567192	4858827	n/a	Heat Pumps
87-P-2018[2]	567364	4859360	n/a	Heat Pumps
97-P-2037[2]	574511	4866604	n/a	Communal
2867-6UJKLT[2]	576314	4866693	n/a	Golf Course Irrigation

[1] Maximum permitted taking reported in the MOECC Permit to Take Water, spring 2009.
Reported in 2009 permit to take water database, but not simulated in groundwater flow model. Permit located southwest of Island Lake Reservoir, on 2nd Line EHS (north of the Coles Wellfield).
[2] Simulated in the groundwater flow model, but permit was not listed in the 2009 permit database. Permit may have expired or new permit may be pending renewal.

C2.7 Land Use

C2.7.1 Existing Conditions Land Use

Existing land use within the study area is shown on **Figure C2-8** and was compiled using a variety of data sources (below).

Ecological and Urban Land Use Mapping

The CVC developed accurate urban land use maps across SPA as part of the *Credit River Water Management Strategy Update* (CVC *et al.*, 2007). These maps were created to capture forecasted changes in land use (specifically, percentage imperviousness) in specific catchment areas during the 5-year monitoring period. Orthophoto images from 1999 (< 1 m resolution), IRS satellite scenes (5 m resolution), and SPOT satellite images (10 m resolution) for 2004 were used to complete the analysis.

Urban land use mapping was merged with watershed-scale Ecological Land Classification (ELC) mapping to create watershed-scale land use mapping, representative of 2004 land use. ELC maps were created using aerial photograph interpretation (1993, 1996, 1999), and later updated using 50 cm orthophotographs, and some field verification undertaken between 1999 and 2002. The ELC mapping classifies all land use types within the watershed using a standardized, hierarchical system that provides a means of identifying, describing, naming and mapping ecological communities. The urban land use within Subwatershed 19 was updated in June 2007 as part of the Subwatershed 19 study. Field work was undertaken to update the urban areas including the classification of the density of residential developments.

In addition to the ELC and urban land use mapping, the Official Plans for the various towns and townships surrounding the Credit River watershed were also consulted. The Official Plans for the Town of Mono (Town of Mono, 2005), Town of Amaranth (Town of Mono, 2004), and Township of East Garafraxa (Township of East Garafraxa, 2006) were consulted to update land use outside the CVSPA. Residential and commercial developments located in these areas were reviewed to represent current land use as interpreted from aerial photographs.

The combination of CVC urban and ecological land use mapping and modified Official Plan mapping produced the current conditions representation of land use within the study area.

C2.7.2 Official Plan Land Use

The study area contains portions of the Towns of Caledon (Region of Peel), Orangeville, and Mono, and the Townships of East Garafraxa and Amaranth. Each town or township has an Official Plan that describes the how land within their jurisdiction should be developed to ensure that future growth meets the needs of the communities. The Official Plans designate where new residential, commercial, and industrial areas will be developed, and also defines environmental protection areas, including wellhead protection areas.

Future land use within the study area, represented by the various Official Plans, is illustrated on **Figure C2-9** and was created by merging the Official Plans from the Towns of Caledon, Mono, and Townships of East Garafraxa and Amaranth, as well as the Official Plan Land Use mapping compiled by the CVC in their Subwatershed 19 Study (CVC, 2009). In consultation with the Town of Orangeville's planners, CVC modified the Town of Orangeville's Official Plan mapping to take into consideration lands which could not be developed due to environmental or infrastructure constraints. The mapping compiled by the CVC was undertaken across the subwatershed and included the Official Plan mapping for the Town of Orangeville. The spatial datasets were combined, and similar Official Plan classifications were grouped to create a single map for the study area.

Note: **Figure C2-9** illustrates official plan maps for the study area prior to any responses to the Province's *Places to Grow Act*. At the time of this study, the Towns of Orangeville, Mono, and Amaranth had not made any amendments to their Official Plans in response to the new legislation.

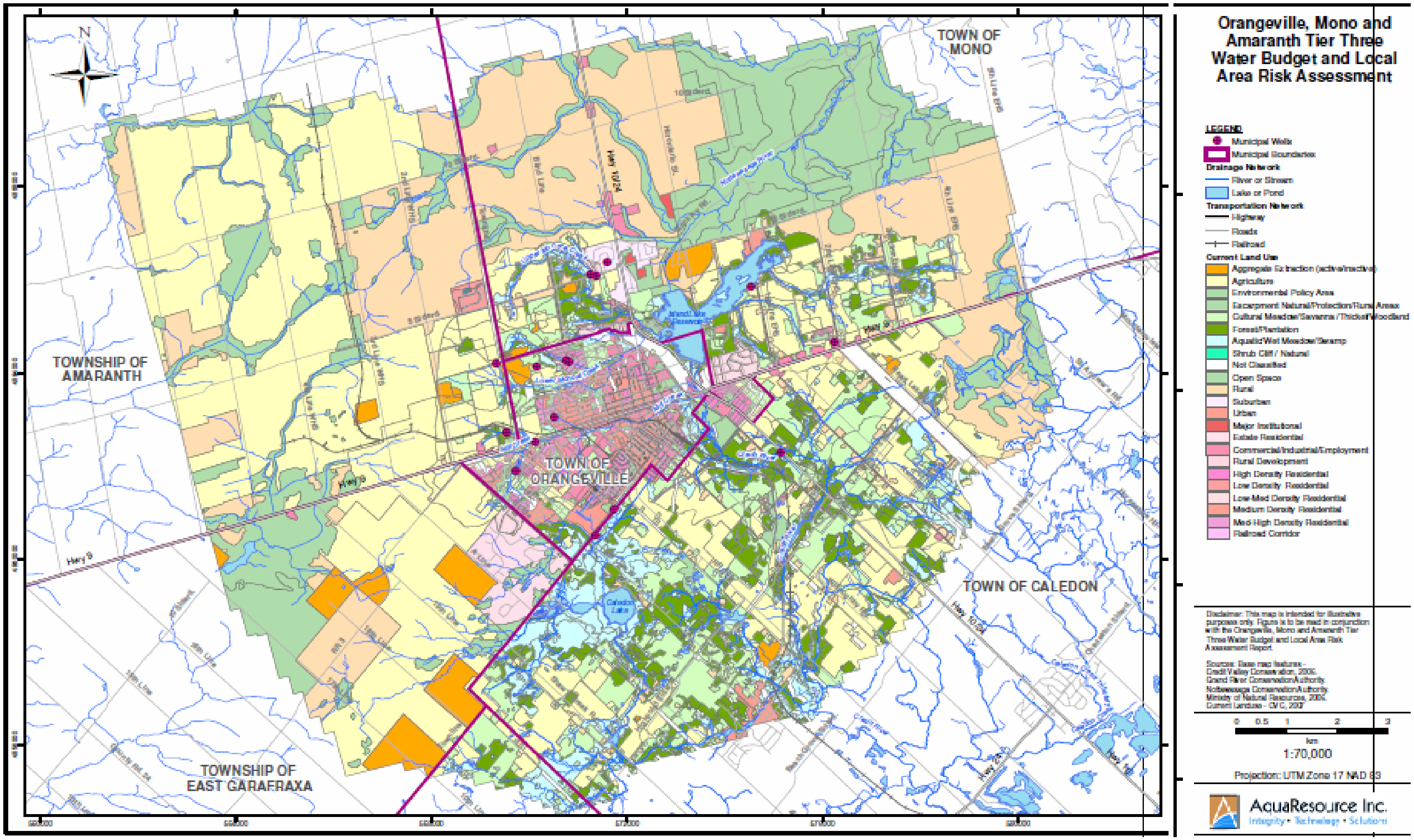

Figure C2-8: Current Conditions Land Use

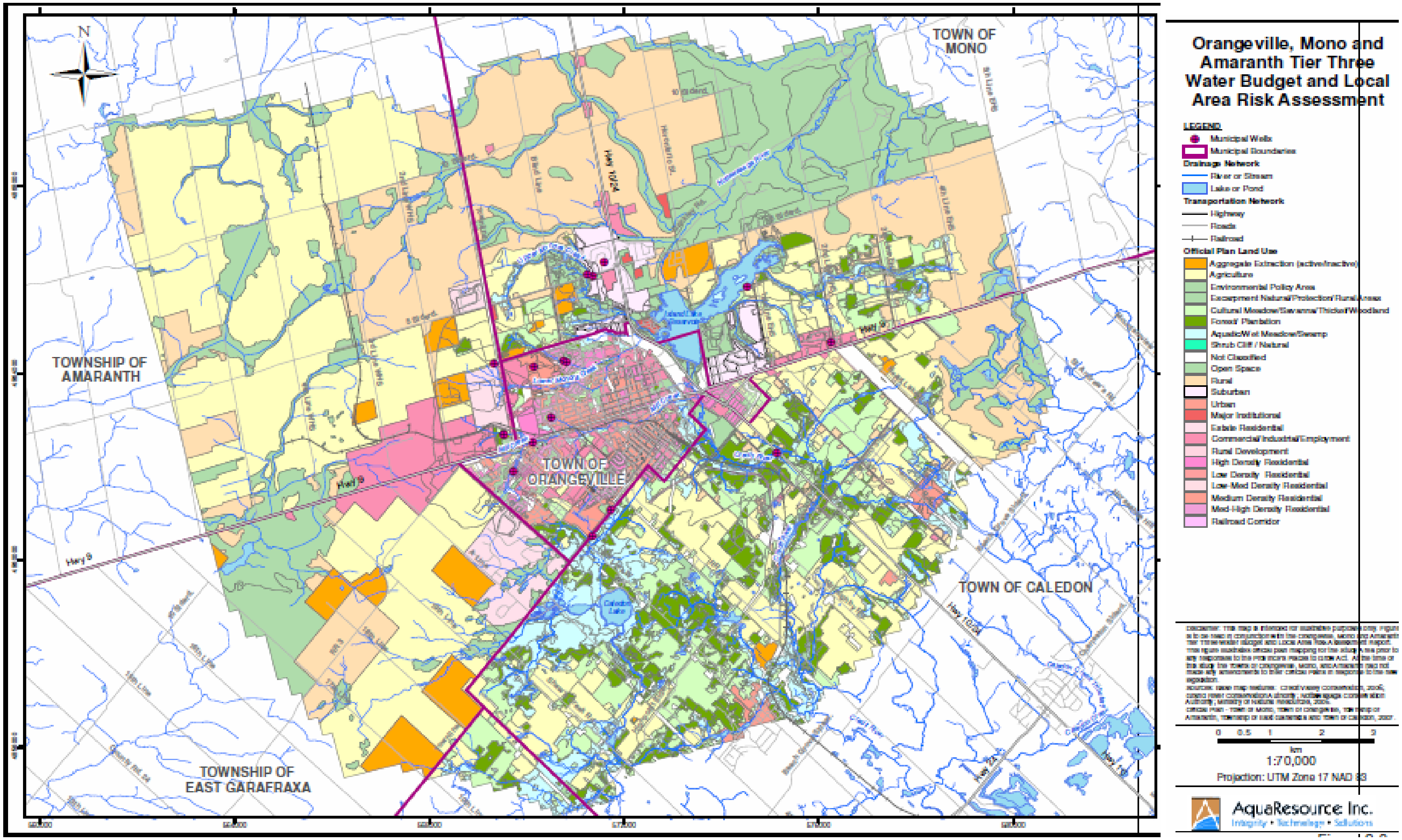

Figure C2-9: Official Plan Land Use

C2.8 Local Area Risk Assessment

Vulnerable Area Development

WHPA-Q1

WHPA-Q1-A underlies much of the Town of Orangeville, and extends west into the Township of Amaranth, north into the Town of Mono, east towards the Credit River and the Island Lake Reservoir, and south beneath Caledon Lake and its associated wetlands. WHPA-Q1-B underlies the Cardinal Woods Wells and the subdivision area located in the northwestern portion of the study area. Drawdown associated with the Island Lake and Coles Wells was less than 0.5 m and as such, no 1 m drawdown cone exists for either of these wells. Similarly, drawdown associated with Well 10 was less than 1.5 m, and the 1 m drawdown cone was restricted to a very small area immediately surrounding the well (< 100 m). Given the small size and limited drawdown associated with these wells, a 100 m buffer area was drawn around each of the municipal wells to delineate the WHPA-Q1-C, D and E (**Figure 3.23**).

WHPA-Q2

Several land use development areas occur within the WHPA-Q1-A area, including several large commercial (employment area lands) west of Orangeville Wells 2A and 9A/9B, and in the vicinity of Well 12. Residential lands are proposed south of the Pullen Well, and in the area around Wells 8B, 8C and 7. One potential land use development straddles the WHPA-Q1-A boundary and this area warranted additional examination (**Figure 3.24**). This small proposed residential subdivision is located approximately 800 m east of Wells 8B and 8C along the North Arm of Lower Monora. To assess the impact of this residential development on the water quantity for the Well 8 Wellfield, the model was run with existing land use and existing pumping, and the head at the municipal wells (in the production aquifer) was noted. The model then updated to simulate the 50% reduction in recharge for the residential lands (**Table 3.15**), and the model was re-run. The reduction in hydraulic head due to the development of residential lands was predicted to be less than 1 cm in the aquifer at the municipal wells (Wells 8B, 8C). Using professional judgment, and consideration that the available drawdown at the Well 8B, 8C wells is greater than 7 m, the reduction in recharge was not considered to have a measurable impact on the wells. As such, the residential lands that lie outside the WHPA-Q1-A area, were not included in the WHPA-Q2-A area, however the portion of the subdivision that lies within the WHPA-Q1-A is still included in the WHPA-Q2-A area.

Within the Town of Mono, undifferentiated suburban development is proposed in the vicinity of the Cardinal Woods Wells in the WHPA-Q1-B area, north and south of the wellfield (**Figure 3.24**). In addition, commercial lands are proposed to be developed southeast of Cardinal Woods 3. To assess the impact of these developments on the water quantity, the model was run with the existing land use and existing pumping and the head in the production aquifer at the municipal wells was recorded. The model was then updated to simulate the reduction in recharge, and the model was re-run. A 15 cm reduction in head was predicted to occur due to the change in land use. Using professional judgement and giving consideration to the available drawdown at Cardinal Woods Well 3 (3.1 m), the reduction in head at the municipal well due to reduction in recharge was considered to have a measurable impact on the well(s). As such, the above noted lands were considered part of the WHPA-Q2-B area, as illustrated on **Figure 3.24**.

The WHPA-Q2-D area surrounds the Coles Wells and the proposed land use development areas surrounding the wells. The WHPA-Q2-D area was expanded beyond the area of the WHPA-Q1-C area as the land use developments are so closely spaced that they cumulatively may have a measurable impact on the cone of influence of the municipal wells.

The WHPA-Q2-C and WHPA-Q2-E areas surrounding the Island Lake Wells and Orangeville Well 10 (respectively) are coincident with the WHPA-Q2-C and WHPA-Q1-E areas, as there is no change in land use proposed within those WHPA-Q1 areas.

Risk Scenario Development

Information required to prepare the models for each risk assessment scenario was compiled as follows.

C2.8.1 Scenario C – Existing Conditions, Average Climate

Scenario C evaluates the ability for existing municipal water supply wells to maintain existing average annual pumping rates under average climate conditions. This scenario was simulated in steady-state in the MODFLOW model using 2008 (existing) pumping rates (**Table 3.14**), the average annual groundwater recharge distribution from the calibrated HSP-F model (1960 to 2006 simulation).

The groundwater flow model was constructed and calibrated to predict groundwater levels in the aquifer at the municipal pumping wells, and to predict groundwater levels and/or groundwater discharge rates associated with other uses (to the degree the calibration was supported by available field data).

C2.8.2 Scenario D – Existing Conditions, Drought

Scenario D evaluates the ability of each municipal well to pump at existing rates during a drought period. This scenario was simulated using the calibrated groundwater flow model in continuous transient mode (1960 to 2006). Average monthly recharge rates from the HSP-F model were applied in the groundwater flow model throughout the duration of the simulation (1960 to 2006), which included several drought periods.

The impacts of municipal pumping on other uses were not considered in this drought scenario. As a result, the main output parameters for this scenario are water levels at each of the municipal wells.

C2.8.3 Scenario G – Existing Plus Committed Plus Planned Demand, Future Land Development, Average Climate

Scenario G evaluates the ability for existing and planned wells to maintain existing plus committed plus planned pumping rates under average climate conditions and reductions in recharge. This scenario was simulated using the calibrated groundwater flow model in steady-state conditions using recharge rates that reflect long-term average climate conditions.

Scenario G was subdivided into three scenarios (G(1), G(2), and G(3)). In order to isolate the impacts of municipal pumping from land developments, **only the scenario representing increased municipal pumping is considered when evaluating the impact of the scenarios on wetlands and coldwater streams.**

- Scenario G(1) - evaluated the cumulative impact of increased municipal pumping rates (existing plus committed plus planned rates) and reductions in recharge (due to increases in imperviousness) due to future land use changes defined in the Official Plans, on the municipal wells, and other uses. **Table 3.14** lists the existing plus committed plus planned water demands applied to evaluate this scenario. **Figure 3.25** shows the land areas where recharge was reduced in the models. Recharge reductions were assigned to these land areas according to **Table 3.15**.

- Scenario G(2) - evaluated the impact of increased municipal pumping rates (existing plus committed plus planned rates) on the municipal wells, on baseflow to wetlands and to coldwater fisheries. This is the only scenario considered when evaluating the impact of municipal pumping on the environment.

  Baseflow reductions arising from land use development are independent of those resulting from increased groundwater pumping.

- Scenario G(3) - evaluated the impact of reductions in recharge (due to increases in imperviousness) due to future land use changes defined in the Official Plans, on the municipal wells and other water uses. Existing municipal pumping rates were used in this scenario to isolate the influence of land development from that of existing plus committed plus planned demand, on groundwater recharge.

C2.8.4 Scenario H - Existing Plus Committed Plus Planned Demand, Future Land Development, Drought Conditions

Scenario H evaluates the ability for existing wells to maintain allocated municipal pumping rates (existing plus committed plus planned) through a drought period. The groundwater flow model was run transiently to examine the combined impact of drought conditions, land use development, and additional municipal pumping on water levels at the municipal wells. Impacts to other water uses are not considered in Scenario H.

This scenario was also subdivided into Scenario H(1), H(2) and H(3) to evaluate the relative contribution of municipal water takings and land use development at each municipal well under drought conditions.

- Scenario H(1) - evaluates the cumulative impact of increased municipal pumping rates (existing plus committed plus planned rates), reductions in recharge (due to increases in imperviousness) due to future land use developments, and drought conditions on the municipal wells,
- Scenario H(2) - evaluates the impact of increased municipal pumping rates (existing plus committed plus planned rates) on the municipal wells during a drought period. The existing conditions land use was simulated in this scenario; and
- Scenario H(3) - evaluates the impact of reductions in recharge (due to increases in imperviousness) due to future land use developments defined in the Official Plans, and drought conditions on the municipal wells. The existing pumping rates were simulated in this scenario.

C2.9 Sensitivity Analysis of Scenarios

The representation of the groundwater flow system was manually calibrated to available hydraulic head data and spot baseflow measurements using one set of parameters (e.g., recharge and hydraulic conductivity) that reflect the understanding of the conceptual model. However, this set of parameters is non-unique; other parameter sets may produce an equally well-calibrated model. This section presents a sensitivity analysis which completes the following:

1. Using the PEST parameter estimation software (Dougherty, 2004), estimate the 95% confidence interval for all model parameters were possible;
2. Create multiple sets of model input files, each containing different combinations of suitable model parameters that are considered to be acceptably calibrated; and

3. Complete a process referred to as a 'Null Space Monte Carlo' technique which can evaluate a scenario for each of the model input files and predict the result in terms of water level drawdown and baseflow reduction. From these results, it is possible to estimate the probability that water level or baseflow reduction criteria will be violated (or satisfied) by the model.

C2.9.1 Parameter Estimation Software

PEST (Dougherty, 2004) conducts a series of model runs where each model parameter (e.g., hydraulic conductivity zone) is adjusted individually to determine the sensitivity of the model calibration to an incremental change in parameter value. The calibration sensitivity gives insight on the parameterization of the model and identifies:

- The parameter values that are well-supported by field observations;
- The parameters that can be estimated using automated parameter estimation routines (e.g., PEST) to optimize model calibration;
- The relative influence of each parameter in model calibration; and
- The potential for new observations to improve the estimation of a parameter.

It was determined that the most sensitive parameters were the recharge applied on the Orangeville Moraine, and the coarse-grained sands and gravels within and surrounding the moraine, the horizontal and the vertical hydraulic conductivity of the Eramosa Formation, Catfish Creek Till, lower sand and gravel aquifer, Amabel Formation and Orangeville Moraine. The least sensitive parameters included the vertical hydraulic conductivity of the lower bedrock units (Whirlpool, Clinton-Cataract, Cabot Head Formation units), the outwash around Wells 5/5A, and the Guelph Formation.

C2.9.2 Sensitivity Analysis Results

The base case model is one realization of a set of parameters that produced a calibrated model. The model and the input parameters are a generalized representation of a complex hydrogeological system, and the assumptions used to generalize the model have associated uncertainty. The results of the sensitivity and identifiability analysis noted several input parameters that when changed, had little impact on the model calibration. These parameters included the horizontal and vertical hydraulic conductivities of the lower bedrock units (Whirlpool, Cabot Head Formation units) and the Guelph Formation and the vertical hydraulic conductivity of the outwash sands around Wells 5/5A. The model calibration changes very little with incremental changes in these vertical hydraulic conductivity values, so a much larger range of plausible values will produce a calibrated model. These parameters have a higher degree of uncertainty (when calibrating the model to higher quality well data) and their impact on the model prediction was tested and examined.

C2.9.3 Calibrated Model Development Using the Null Space Monte Carlo Technique

The Null Space Monte Carlo technique is used to generate multiple sets of model input parameters that are based on a calibrated model. Each of the sets of model input parameters are geologically reasonable, consistent with the conceptual model of the area, and each model is calibrated according to the statistical measures used. PEST is able to create these models as parameters that are identifiable (part of the solution space) will have a range of acceptable values due to uncertainty and noise in the data. PEST determines how far within this range the parameter can move without significantly reducing the quality of the model calibration. Similarly, for those unidentifiable parameters (the null space), a wide range of values could be applied without negatively impacting the model calibration. PEST uses these two components of parameter space (solution space and null space) to create parameter fields that yield a well calibrated model. The advantage of using the Null Space Monte Carlo is that once a groundwater model is calibrated, PEST can be used to complete the numerical burden of re-calibrating multiple models with unique parameter fields.

In this study, the Null Space Monte Carlo technique was used to create one hundred calibrated models based on the initial model calibration, which simulated the existing conditions land use, and municipal pumping (2008 rates). The first step in the Null Space Monte Carlo technique was to estimate a high and low range for PEST to modify each of the model input parameters. Constraining the upper and lower bounds of the input parameters, allows PEST to vary the model input parameters within ranges that are consistent with the modeller's understanding and conceptualization of the study area. For example, in this assessment, PEST was allowed to vary the horizontal hydraulic conductivity of the Orangeville Moraine hydraulic conductivity zone from 1x10-3 m/s to 1x10-6 m/s in each of the model simulations, until a suitable conductivity (together with other conductivity zones) produced a calibrated model. Each of the model input parameters is updated in this fashion, with PEST working to end up with a set of parameters that will yield a calibrated model. Producing each calibrated model may take hundreds of runs, so the highest number of model iterations per model run was restricted to 50. Limiting the number of iterations allowed more parameter sets to be assessed in a more efficient manner.

The aim of the manual calibration and the PEST calibration was to update the hydraulic conductivities and the recharge values (within 10% of those specified in the base case) until the model predicted hydraulic head and baseflow values closely matched observed head and baseflow values. Of the 100 calibrated models, four models were found to fall just outside the statistical range of what we would consider an acceptable model calibration, and as such they were removed from the analysis.

Model Predictions- Hydraulic Heads

The municipal pumping rates in each of the one hundred equally calibrated models were then updated to the existing plus committed plus planned rates (**Table 3-14**) to produce 96 models that were representative of the conditions in Scenario G(2). Each of the 96 models was run and the output from each of the models was compiled to provide insight into how the uncertainty associated with the model input parameters may impact the model predictions. The initial objective was to identify the number of calibrated conditions where the hydraulic head in the aquifer at the municipal well violated the safe available drawdown at the well. **Table C2-22** outlines the number of simulations where drawdown exceeding the safe available drawdown in the 96 equally well calibrated models for Scenario G(1) and G(2).

As outlined on **Table C2-22** the model predicted drawdown in Wells 5/5A was greater than the safe available drawdown for all 96 models. The safe available drawdown in Wells 5/5A is 3.2 m, and 2% of the models (2/96) had a predicted drawdown of 3.2 to 4 m, 76% of the models (73/96) predicted drawdown of 4 to 5 m, and 19% (18/96) predicted drawdown in the aquifer at the well would be greater than 6 m. These results are consistent with the results of the base case model scenario where the model simulated drawdown exceeded the safe available drawdown (**Table C2-22**).

Table C2-22: Results of Base Case and Null Space Monte Carlo Head Change Model Predictions

Well	Base Case: Was Drawdown Greater than Safe Available Drawdown?		Probability of Drawdown Exceeding Safe Available Drawdown?	
	Scenario G(1)	Scenario G(2)	Scenario G(1)	Scenario G(2)
Orangeville 2A	No	No	0% (0/96)	0% (0/96)
Orangeville 5/ 5A	Yes	No	100% (96/96)	1% (1/96)
Orangeville 6	No	No	0% (0/96)	0% (0/96)
Orangeville 7	No	No	0% (0/96)	0% (0/96)
Orangeville 8B	No	No	0% (0/96)	0% (0/96)
Orangeville 8C	No	No	0% (0/96)	0% (0/96)
Orangeville 9A/ 9B	No	No	0% (0/96)	0% (0/96)
Orangeville 10	No	No	0% (0/96)	0% (0/96)
Orangeville 11	No	No	0% (0/96)	0% (0/96)
Orangeville 12	No	No	0% (0/96)	0% (0/96)
Mono Cardinal Woods 1	No	No	0% (0/96)	0% (0/96)
Mono Cardinal Woods 3	No	No	1% (1/96)	0% (0/96)
Mono Coles 1 and 2	No	No	0% (0/96)	0% (0/96)
Mono Island Lake Wells	No	No	0% (0/96)	0% (0/96)
Pullen Well	No	No	0% (0/96)	0% (0/96)

In addition to Well 5/5A, the safe available drawdown was predicted to be exceeded at Cardinal Woods Well 3 in 1% of the models (1/96) under Scenario G(1) (**Table C2-11**). The base case model did not predict the drawdown in the aquifer at the well would exceed the safe available drawdown.

During the base case calibration there were no predicted exceedances of the safe available drawdown under Scenario G(2); however, one model out of 96 was reported to exceed the safe available drawdown at Well 5/5A.

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C3 TIER 3 WATER BUDGET AND LOCAL RISK ASSESSMENT – ACTON AND GEORGETOWN

C3.1 Water Budget Modelling Process

Tier 3 Water Budget requires a finer level of detail than that typically undertaken for the Tier 2 assessment. The Tier 3 hydrologic model improves upon the Tier 2 Water Budget model in terms of the model simulation and representation of the movement of groundwater between and across subwatershed boundaries.

A major deliverable is an improved estimate of the water budget components included in the hydrologic cycle within the study area. As part of the Tier 3 assessments, surface water and groundwater models were developed and calibrated to simulate the existing drawdown at municipal pumping wells and stream interactions. The calibrated models were then used to assess the impact of changes in pumping and land use on surface water and other users. The numerical models were developed to collectively:

- Represent regional and local three-dimensional hydrostratigraphy;
- Estimate spatially variable groundwater recharge rates by comparing simulated streamflow to observed streamflow hydrographs (annual, monthly and daily flows);
- Represent existing and future drawdown at wells;
- Simulate groundwater discharge to surface water features;
- Simulate transient (time varying) conditions to evaluate cumulative impacts of climatic variability, including drought; and
- Assess the level of uncertainty in simulated existing and future conditions specific to the local area.

C3.2 Model Domain

The model domain, presented in **Figure C3-1** is approximately 745 km^2 in area, encompassing the Town of Halton Hills, including Acton and Georgetown. The area was identified and evaluated as part of the Tier 2 groundwater modelling effort, during which it was found to be of sufficient areal extent to encompass the expected Local Area for the Georgetown and Acton systems (AquaResource Inc., 2009). Boundaries of the study area coincide with regional groundwater divides or beyond the hydraulic influence of to the Acton and Georgetown municipal supply wells. As such, the study area overlaps portions of the Town of Milton, the Township of Guelph/Eramosa, the Town of Erin, the Town of Caledon, the Town of Brampton, and the City of Mississauga, in addition to encompassing the Town of Halton Hills. It also contains watersheds and subwatersheds that fall under the jurisdiction of the Credit Valley Conservation (CVC), Conservation Halton, the Grand River Conservation Authority (GRCA) and the Toronto and Region Conservation Authority (TRCA).

The municipal supply systems for Acton and Georgetown are located in subwatershed 10 and 11 of the Credit River watershed, which falls under the jurisdiction of the CVC. Each system is comprised of three active municipal wellfields, with both communities being entirely reliant on groundwater for all their drinking water needs. Active wellfields include the Fourth Line, Davidson, and Prospect Park wellfields in Acton; and the Lindsay Court, Princess Anne and Cedarvale wellfields in Georgetown. Municipal wellfield locations are illustrated on **Figure C3-1**, with municipal well details summarized in **Table C3-13**. Land use surrounding the municipal wellfields is dominated by agricultural and urban activities (CVC *et al.*, 2002; CVC *et al.*, 2011), with other land uses in the larger study area including aggregate extraction and natural heritage features, e.g., wetlands and/or forest communities.

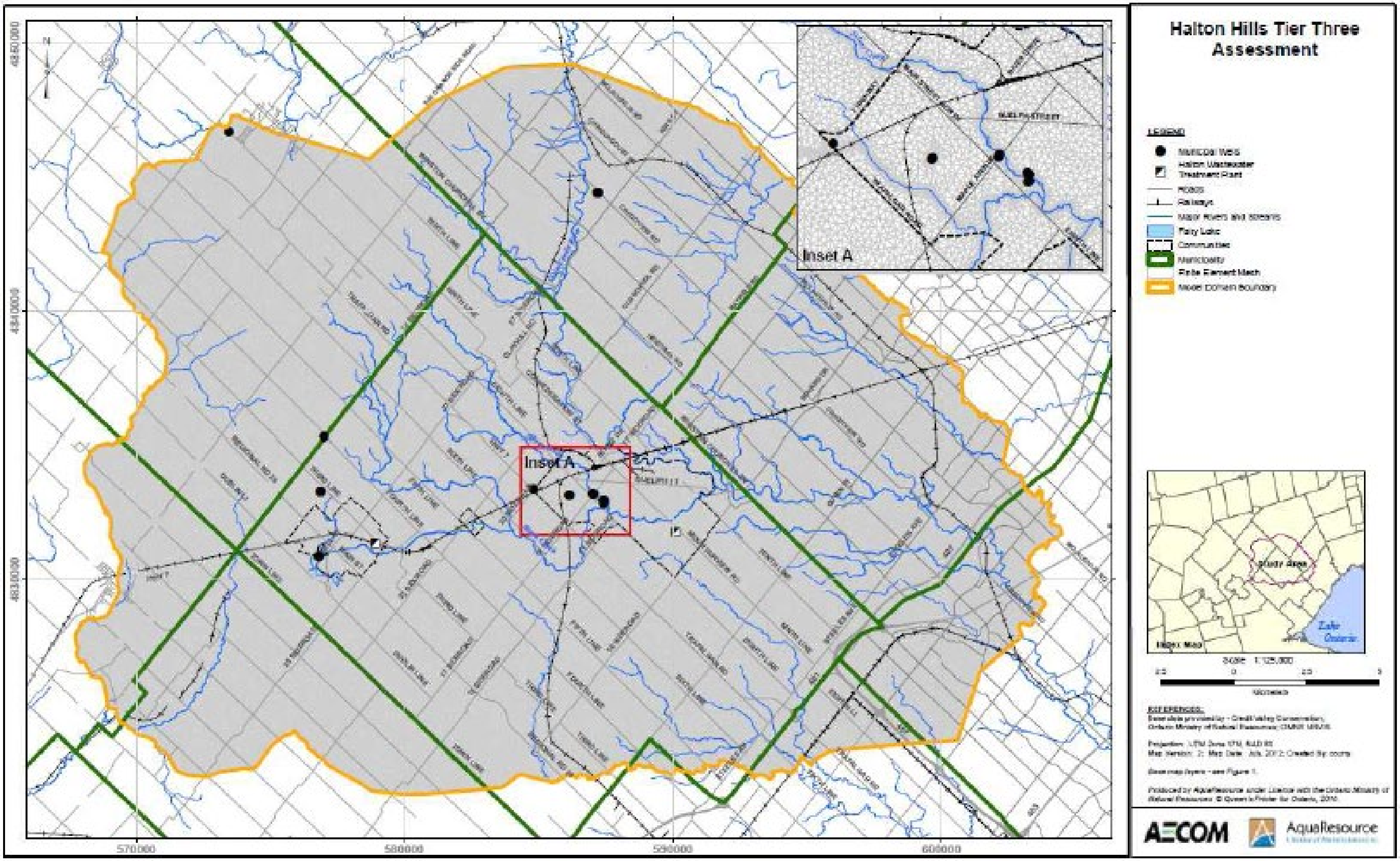

Figure C3-1 : Tier 3 Water Budget Model Domain

C3.3 The Hydrologic Model: MIKE SHE

A detailed three-dimensional, integrated hydrologic model was constructed and calibrated for the Halton Hills Tier 3 assessment area using the software program MIKE SHE. The model was calibrated to available streamflow data for the period 1991 to 2005. Additional calibration targets included historically observed evapotranspiration rates, high quality, and medium quality groundwater level observations as well as groundwater spot streamflow observations. The model was verified for the period between 1981 and 1990. The calibration and verification process resulted in a reasonable match between simulated and observed data for the region, which provides confidence that the groundwater recharge estimates of the model are appropriate for use in the FEFLOW groundwater flow model.

The integrated hydrologic model provided fully distributed estimates of groundwater recharge used for the steady state groundwater flow model. These estimates capture the spatial variability of recharge within the study area, which is influenced primarily by surficial geology, topography, and imperviousness. Local processes also influence groundwater recharge such as groundwater discharge in wetland areas, closed depressions capturing overland runoff, and overland flow from impervious materials infiltrating on adjacent pervious materials. Transient recharge estimates were generated by selecting representative cells for each surficial geology type to capture the temporal variability of recharge for the transient calibration of the FEFLOW model.

Modelling Objectives

The Tier 3 assessment requires models that simulate all relevant water budget components in a spatially detailed and temporally dynamic manner. The surface water model developed provided recharge estimates to the groundwater model and it also incorporated groundwater model information to improve upon the surface water model setup.

The hydrology model examines and quantifies the impacts and/or benefits of watershed scale and local scale activities as well as future conditions that may prevail due to climate change and urban development. These activities include various forms of urbanization, rural developments, waste management, land management, road management and stormwater management. The model may also find application in addressing local scale issues such as spills, construction impacts, urban infrastructure impacts (i.e., cross connects and inflow/infiltration to sewers), instream aquatic plant growth and instream thermal regime management.

In terms of water balance, the surface water model simulates the entire hydrologic cycle. This includes precipitation, snowpack accumulation and melt, surface runoff, unsaturated soil moisture and evapotranspiration (ET). It must be capable of estimating the short- and long-term supply of water to groundwater and the discharge rate of groundwater to streams.

The model explicitly represents effects of urbanization on local hydrology, including such processes and effects as changes in net ET and soil water infiltration caused by impervious surfaces, rapid surface runoff from impervious surfaces, and transfer of runoff from impervious surfaces onto pervious surfaces and vice versa, reflecting the complex nature of connectivity in the urban environment.

The model represents runoff processes at sufficient spatial resolution to allow for continuous simulation of the streamflow regime in local areas of concern and throughout the watershed. Also, the model is able to simulate the streamflow regime at an appropriate temporal scale (e.g., hourly) to simulate the rapid runoff associated with urban stormwater drainage on the tributaries and the main channel of the Credit River. The water budget simulation also is able to determine the risk associated with under supply during times of peak water demand. These are short term events requiring dynamic modelling.

The model provides time-domain simulation over multi-year continuous periods, to assess the impacts of control measures and strategies over an appropriate range of meteorological conditions. The model explicitly represents and simulates the effects of various urban control measures such as pollutant source control, runoff reduction measures, runoff treatment systems, and other stormwater management practices. Similarly, the model explicitly represents and simulates the impacts of future urban growth and intensification within the subwatershed and explicitly represents and simulates the effects of various rural land management practices.

MIKE SHE has multiple algorithms to represent hydrologic processes. This allows for the flexibility of being able to tailor the process representations to the model objective. For the purposes of this assessment, the MIKE SHE model was constructed using process approximations appropriate for recharge estimation. The MIKE SHE model was structured as follows:

- Precipitation was characterized with temporal distributions (time series data) from climate stations within the study area. Input precipitation is spatially distributed according to the Thiessen polygon regions created by the set of input climate stations.
- An input map of vegetation was provided to the model to describe the spatial distribution of vegetation in the watershed. Vegetation was characterized through leaf area indexes and rooting depths. Evapotranspiration was approximated using a two-layer water balance model that considers interception, ponding and evapotranspiration. Actual evapotranspiration was computed considering the vegetation parameters and specifying a potential evapotranspiration rate. In the Halton Hills MIKE SHE model, the leaf area index defines canopy interception of precipitation, and the rooting depth defines the depth to which plants may draw moisture from the subsurface for transpiration. MIKE SHE attempts to meet the potential evapotranspiration rate through consideration of water availability in the various phases of the hydrologic cycle in the following order:
    - Accumulated Snow (if present, through evaporation or sublimation);
    - Canopy Interception (through evaporation);
    - Ponded Water (through evaporation);
    - Unsaturated Zone (through transpiration); and
    - Saturated Zone (through transpiration).

  Once all water content in a storage element is evaporated, no further evaporation occurs from that storage element until it is replenished by a precipitation event, overland runoff or though groundwater flow.
- Snow melt and accumulation is controlled using a degree-day process, which primarily relies on air temperature. The daily temperature variation of the subwatershed is provided using a temperature time series. Freezing or melting of water occurs when the temperature is above or below a threshold temperature (0°C). The rate at which snow melt occurs is controlled by a degree-day coefficient (units: mm snow/ day * °C). This coefficient is often used as a calibration parameter to calibrate the snow melt volumes and timing to observed spring runoff. The wet and dry portions of the snowpack also regulate snow melt. Liquid water is released from the snowpack only when the fraction of wet snow within the snow pack exceeds a threshold value. As with the degree day coefficient, this parameter is adjusted to calibrate to observed snow melt runoff.

- Topography of the subwatershed is characterized by a digital elevation model (DEM). In the Halton Hills MIKE SHE model, a 5 m and 10 m DEM were used. Overland flow is simulated though a diffusive wave approximation of the St. Venant equations (Chin, 2006). Numerically, this method is implemented through a two-dimensional finite difference method. Additional overland considerations:
    - Accumulated Snow (if present, through evaporation or sublimation);
    - Spatially variable surface roughness, characterized through a Manning's number;
    - Spatially variable depression storage, characterized by a depth of storage; and
    - Spatially variable imperviousness, characterized by the fraction of flow immediately directed to river systems.

- Channel flow is simulated using a link to the MIKE-11 modelling system. Channel location and geometry are defined using a drainage network and topography from the available DEM. Channel flow is implemented using a simplified one-dimensional hydrologic routing method.
- One-dimensional (vertical) unsaturated flow is considered using a two-layer water balance approach. This considers an upper layer of the unsaturated zone that extends from the ground surface to the top of the capillary fringe and a lower layer that extends from evapotranspiration extinction depth (the maximum root depth + capillary fringe thickness) to the water table. In areas where the water table is above the evapotranspiration extinction depth, there is only one layer (maximum root depth + capillary fringe).

  Water that is accessible for evapotranspiration is defined by the amount of soil-water content contained within the rooting zone. The soils of the unsaturated zone are described with a spatial distribution, based on surficial geology, and are characterized by a hydraulic conductivity parameter, soil-water parameters (wilting point, field capacity, saturation point) and suction head. Infiltration to the unsaturated zone is calculated using the Green and Ampt method. Limiting factors for infiltration are the soil hydraulic conductivity and the suction head. Soil-water content of the unsaturated zone is maintained on a mass balance basis. When the soil-water content of the unsaturated zone exceeds field capacity, water drains to the saturated zone (percolation). When soil-water content is below field capacity, percolation ceases with further reductions in soil-water content only occurring through evapotranspiration. The Green and Ampt infiltration equation modifies the infiltration rate to account for changes in soil moisture, and when net precipitation falls at a rate faster than the infiltration rate, overland runoff is generated.
- Interflow, or subsurface storm flow, is simulated through a head-dependent boundary condition in the saturated zone. The routing of interflow is defined using a detailed subwatershed delineation. Interflow generated within each subwatershed is routed to streams within the given subwatershed. Interflow volume is calculated as the difference in head between the drain level and the water table, multiplied by the drain time constant. If the water table is below the drain level, no interflow occurs. The drain time constant and depth are calibration parameters for interflow and were adjusted to minimize the differences between the recession portion of the simulated and observed hydrographs.
- Three-dimensional saturated Darcy flow is simulated in MIKE SHE using a finite difference approximation, similar to that of a MODFLOW model (Harbaugh, 2005).

C3.3.1 Model Set up and Input Data

Table C3-1 summarizes the numerical representation of the major hydrologic processes represented in the MIKE SHE model and the applied time steps used for these components.

Table C3-1: Hydrologic processes and Time Step Applied in Hydrologic Model

Hydrologic Process	Process Approximation	Time Step Applied
Overland Flow	Two-Dimensional (2D) - diffusive wave approximation of St. Venant equations of flow.	30 minutes
Channel Flow	One-dimensional (1D) hydrologic routing – Simple mass balance model.	1 hour
Evapotranspiration	Two layer water balance model, which applies a simple mass balance approach to predicting ET.	
Unsaturated Zone	One-dimensional, two layer water balance model. Infiltration based on soilwater content parameters as well as soil conductivity and suction head. Infiltration based on the Green and Ampt method.	1 hour
Saturated Zone	Three-dimensional finite difference implementation of Darcy's equation.	12 hours

Simulation Period

MIKE SHE is a continuous hydrologic model, able to simulate transient hydrologic conditions over time. The time period able to be simulated is tied to the available time period of input data sets required by the model (e.g., climate). The available period of record for the required input data sets is 1960-2010. This time period investigates the impact of climatic variability throughout that period, including droughts in the 1960s and the late 1990s.

Calibration is the process of adjusting model parameters to minimize differences between observed and simulated conditions. For calibration purposes, a time period that corresponds to the land use data coverage was selected (1991-2005). In addition to the calibration period, a verification period was also used (1981-1990) to test the calibrated model parameters to a different input data set.

C3.3.2 Climate Data

Environment Canada climate stations from the MNRF infilled climate data set (Land Information Ontario (LIO), 2008) within or near the study area were selected to characterize the climate of the study area. The data set was infilled to address data gaps and errors by Schroeter and Associates (2007) and includes the 1950-2005 time period. Data selected included daily maximum and minimum temperature, rainfall, snowfall, and precipitation, as well as hourly rainfall. The climate stations utilized as well as their percentage of infilled data are shown in **Table C3-2**. Note that while the percentage of infilled data is high for certain stations, the source of the climate data used to fill the observational gaps is very close to the station location.

An hourly precipitation time series data set was derived by combining hourly rainfall estimates with daily snowfall estimates. Daily snowfall rates were converted to hourly rates assuming a uniform distribution of snowfall over the day.

Table C3-2: Selected Climate Stations

AES ID	Station Name	Latitude	Longitude	Elevation (m)	Infilled Data (%)
6142400	Fergus Shand Dam	43.73	-80.33	418	2
6143090	Guelph Turfgrass CS	43.55	-80.22	325	83
6150916	Brampton MOECC	43.67	-79.7	183	43
6152695	Georgetown WWTP	43.64	-79.88	221	23
6153410	Heart Lake	43.73	-79.78	259	61
6153552	Hornby Trafalgar	43.53	-79.73	183	77
6155187	Milton Kelso	43.5	-79.95	244	62

To calibrate the groundwater flow model against the Cedarvale well field shut down, the climate period was required to be extended to include 2010. To do so, all available hourly and daily climate data was obtained from Environment Canada. As most climate data sets have gaps in the record due to equipment malfunction, a data infill exercise, similar to the process undertaken by Schroeter and Associates, was performed on the data obtained from Environment Canada.

Due to a backlog in Environment Canada quality assurance and control procedures, hourly rainfall data was only available for the Georgetown wastewater treatment plant (WWTP) climate station. These data were infilled and assumed to be representative for the entire Halton Hills model domain, extending the simulation period from December 31, 2005 to December 31, 2010.

Solar Radiation

Daily solar radiation data were collected for a number of different Environment Canada climate stations for the model, as shown in **Table C3-3**. These data were supplemented with an additional solar radiation data set to extend the period of coverage. Solar radiation data for the Orangeville region from 1960 to 2000, prepared for the CVC Tier 2 assessment (AquaResource Inc., 2009) was also used. A continuous daily radiation data set was constructed for 1960-2002.

Table C3-3: Daily Solar Radiation Data

AES ID	Name	Period of Record	Description
611KBE0	Egbert	1988-2003	Environment Canada Research Station
6158350	Toronto	1956-2001	Environment Canada Climate Station
6142285	Elora	1970-2003	Environment Canada Climate Station
6158740	Toronto MET	1967-1988	Environment Canada Climate Station

To simulate the period of 2003 to 2010 the solar radiation data set was extended by comparing years of solar radiation data that exhibit similar yearly temperature characteristics. The mean monthly temperatures of the years without radiation data were compared to the mean monthly temperature of the years without temperature. The best fitting year was found the year with the smallest difference between mean monthly temperatures. The values of daily radiation for the similar year were then used to extend the 1960 to 2002 period to 2010.

These daily radiation data are used in the estimation of daily potential evapotranspiration rates.

Temperature

An hourly temperature time series was derived from daily maximum and minimum temperature values for each climate station. A sinusoidal temperature pattern was generated assuming that maximum and minimum temperatures occur at 3:00 pm and 3:00 am, respectively. This pattern is generally typical of temperature fluctuations within most days but may not be representative of extremes experienced during a time in which a climatic frontal system moves into the area. This would primarily impact the timing of snowmelt events.

Potential Evapotranspiration

Daily potential evapotranspiration rates were generated using WDM Util software, which is distributed with the HSPF (U.S. EPA, 1997) model, for all considered climate stations. This utility was used to generate reference evapotranspiration rates using the Jensen method (Jensen and Haise, 1963). This method considers daily temperature and solar radiation to compute a reference evapotranspiration rate.

Spatial Distribution of Climate Data

Climate and climate related inputs such as precipitation, temperature and evapotranspiration rates are assigned a spatial distribution within MIKE SHE. The initial spatial distribution of climate input used was based on a Theissen polygon generated from the climate stations. The spatial distribution was subsequently modified so that Georgetown WWTP climate data were used only for regions below the Niagara Escarpment and the Guelph Turfgrass climate data were used to cover the area above the escarpment previously covered by the Georgetown climate station.

C3.3.3 Topography

A 5 m DEM produced by the CVC and a 10 m DEM produced by the GRCA were used to construct a continuous DEM coverage of the entire model domain (see Figure 2-4 Conceptual Model Report, AECOM and AquaResource Inc. (2011)). The information captured in the 5 m DEM was preferentially chosen over the 10 m DEM.

C3.3.4 Land Cover

Land cover data came from the Land Information Ontario (LIO) Southern Ontario Land Resource Information System (SOLRIS) data set version 1.2 (April 2008). Land classifications were simplified into nine generalized land use classes (**Table C3-4)**. Vegetation characteristics (e.g., leaf are indices and root depths) were assigned based on the vegetation class associated with the land cover classification. Many of the parameters can be varied temporally to represent seasonal changes associated with the vegetation growth, dormancy and dieback that occur between spring and fall months. Initial values for rooting depth and leaf area index were assigned to vegetation types based on literature values and adjusted during calibration (Canadell *et al.*, 1996; Scurlock *et al.*, 2001).

Table C3-4: Land Use Classes and Parameters

Land Use Class	Water	Recreation	Urban	Agriculture	Forest	Con Forest	Dec Forest	Wetlands	Pits / Quarries
Vegetation Class	N/A	Manicured Open Space	Urban	Intensive Ag.	Mixed Woods	Con Woods	Dec Woods	Wetlands	N/A
Leaf Area Index - Max	0	1	1.45	4.5	6	5	6	3.5	0
Leaf Area Index - Min	0	1	0.8	0.5	0.5	5	0.5	3.5	0
Rooting Depth - Max (mm)	N/A	100	600	1000	3000	3000	3000	2000	100
Rooting Depth - Min (mm)	N/A	100	600	100	100	3000	100	100	100
Surface Roughness (Manning's n)	0.053	0.2	0.083	0.3	0.3	0.3	0.3	0.3	0.012
Depression Storage (mm)	15	8	8	8	25	25	25	40	1
Imperviousness	0	0	0.25	0	0	0	0	0	0

Surface roughness values were assigned to the land use classes based on literature values (Chin, 2006) and adjusted during the calibration process (**Table C3-4**). Similarly, depression storage values were assigned to the land use classes based on literature values (National Research Council of Canada, 1989) and adjusted during the calibration phase (**Table C3-4**).

To represent increased runoff from urban areas, a paved runoff fraction was assigned to the urbanized land class. This fraction represents the portion of precipitation that is directly conveyed to receiving watercourses through storm sewers or other urban drainage systems. The final calibrated values are listed in **Table C3-4** and were initially based on literature values (Sullivan *et al.*, 1978).

C3.3.5 Surficial Geology

The surficial geology mapping of the Ontario Geology Survey (OGS, 2003) was used to represent soil variability within the model domain. Surficial geology classes were aggregated into hydrologically representative soil types based on similar soil properties. Soil classes were parameterized in terms of soil water content (wilting point, field capacity, and saturation point) and infiltration rate. Infiltration rates for soil represent the maximum rate at which water may infiltrate into the unsaturated zone assuming storage available in the unsaturated zone and the groundwater gradient is not limiting. Initial values were sourced from the Hydrology of Floods in Canada (National Research Council of Canada, 1989), and adjusted during calibration. Final calibrated values are shown in **Table C3-5**.

Table C3-5: Surficial Geology Parameters

Generalized Soil Class	OGS Classes	Infiltration Rate (m/s)	Saturation Point	Field Capacity	Wilting Point
Gravel	Gravel	4.2E-06	0.3	0.2	0.04
Sand	Sand	3.5E-06	0.46	0.23	0.07
Wentworth Till	Wentworth Till	6.0E-08	0.56	0.46	0.23
Middle Till	Middle Till	1.5E-07	0.56	0.46	0.27
Halton Till	Halton Till	2.0E-08	0.56	0.46	0.27
Clay	Organic Deposits, Modern Alluvium, Silt and clay	1.9E-08	0.56	0.46	0.27
Bedrock	Guelph Formation, Amabel Formation, Dolostone	5.E-07	0.3	0.2	0.04
Bedrock Shale	Queenston Formation	1.E-08	0.3	0.2	0.04

C3.3.6 Watercourses

The rivers simulated in the hydrologic model were selected based on the river network constructed for the Halton Hills Tier 3 groundwater model. For the groundwater model stream network all streams Strahler Class 2 or greater as well as any rivers with spot flow estimates and identified coldwater streams were utilized. For surface water modelling purposes these rivers were simplified to reduce the complexity of the river network and minimize runtime. The following criteria were used to identify watercourses to be included:

- All watercourses greater than or equal to Strahler Class 3;
- All rivers greater than 500 m in length;
- Given parallel watercourses closer than 300 m to one another, the shorter of the two watercourses were removed; and
- All watercourses with baseflow observations.

In areas immediately surrounding municipal wells, additional watercourses were included to better represent possible groundwater/surface water interactions. The watercourses selected for representation are illustrated in **Figure C3-2**. The selected stream network provides sufficiently detailed representation of the stream networks within the study area to properly represent streamflow.

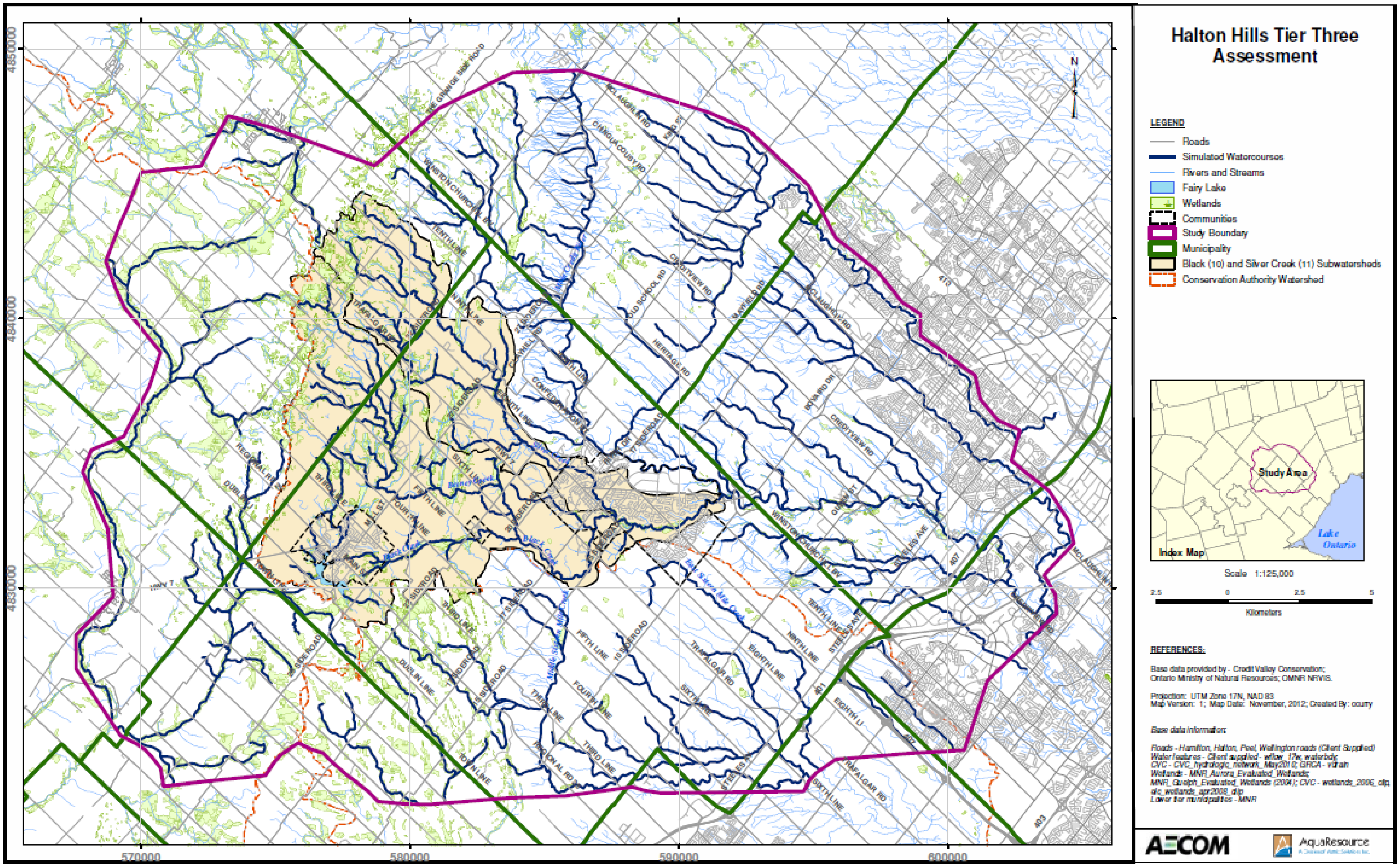

Figure C3-2: Watercourses in MIKE SHE

Watercourse Cross Sections

Cross-sections are necessary to provide channel geometry for routing streamflow and calculating surface water elevations in channels. Cross-section data were generated for the stream using the consolidated DEM for the model domain. A low flow channel was specified by dropping the lowest elevation of the cross-section by 1 m. Cross-sections were generally utilized at 1000 m intervals; however, in areas of rapid topographic change (e.g., the Niagara Escarpment) additional cross-sections were generated to improve the representation of the topographic variability. Simulated stream conditions are for each reach.

Boundary Conditions

A number of streamflow boundary conditions were utilized within the MIKE SHE model. Boundary conditions used included the following:

- The elevation associated with the river outflow boundary condition was set to the elevation of the DEM at the outlet of the river;
- The upstream boundary condition of the Credit River was set equal to the observed discharge at the WSC Boston Mills (02HB018) stream gauge station. WSC archived streamflow data for this station were available up to 2009; this dataset was supplemented by the real time records available online for 2010 from the WSC;
- Acton Quarry discharge to Black Creek was simulated using a point source boundary condition on Black Creek, at the quarry discharge point. Discharge records were available as daily rates for the period of 1982-2007 and monthly rates for 2008-2009; and
- Wastewater treatment plant discharges were incorporated for Acton and Georgetown as point source boundary conditions based on observed discharge values (EarthFX, 2009). Annual average discharge rates were used to characterize the discharge conditions. Some variation in the treatment plant discharge occurs throughout the year, with summer discharges being the smallest. This may result in some minor over estimation of discharge. However, the seasonal variation in discharge over the course of the year is relatively small at less than 15% over the course of the year in terms of mean monthly discharge.

  Acton Annual Average Flow = 0.043 m^3/s

  Georgetown Average Annual Flow = 0.170 m^3/s

C3.3.7 Drainage Depth

The drain depth establishes the elevation in the subsurface, above which the drainage or interflow will occur. This drain depth was established by comparison of the average simulated water table depth produced by FEFLOW to the ground surface elevation. The depth chosen was selected to represent the transient nature of subsurface stormflow or interflow. The drainage depth was set at 0.5 m above the elevation of the steady state FEFLOW groundwater table. In this configuration, as the water table rises after significant rainfall events, the water table elevation may exceed the drain depth and generate interflow.

C3.3.8 Subwatersheds

A consolidated map of subwatershed boundaries was developed by amalgamating subwatershed mapping from the CVC, GRCA, TRCA and Halton Region Conservation Authority (HRCA). Subwatershed boundaries are required to determine which watercourses receive interflow.

C3.3.9 Saturated Zone

The saturated parameters of the MIKE SHE model are the same as the FEFLOW model. Layer elevations, hydraulic conductivity and specific storage values were imported directly from the FEFLOW model to the MIKE SHE model. The first layer of the FEFLOW model was not explicitly simulated as it represents the surficial geology layer, which is represented by the unsaturated zone in MIKE SHE. The layer elevations and model parameters form the geologic model within MIKE SHE. Numerical layers for the subsurface were generated using a 2 m minimum thickness. The parameterization of the numerical layers occurs through the intersection of the geologic layers and the numerical layers. In this way, numerical layers which are thicker than geologic layers are parameterized based on the parameters of the geologic layers they intersect. This allows the thicker numerical layers to retain representative properties of the geologic model.

Pumping well locations and rates, in addition to the Acton Quarry boundary condition used to represent the dewatering operation, were also taken from the FEFLOW model. These are outlined in the hydrostratigraphy description of the Conceptual Model Report (AECOM and AquaResource Inc., 2012a).

C3.4 Model Calibration and Verification

C3.4.1 Calibration

Streamflow calibration was initially focused on matching simulated and observed mean annual flow values. Annual streamflow volumes provide a long-term evaluation of the water balance of the model. The appropriate partitioning of precipitation to evapotranspiration, streamflow and groundwater recharge should produce mean annual flows that approximate observed streamflow values. Observed and simulated mean annual flow for the Water Survey Canada gages at Black Creek (02HB024), Silver Creek (O2HB008) and Blue Springs Creek (02GA031) are presented in **Figure C3-3** through **Figure C3-5**. A good approximation of observed mean annual flow was achieved for Black Creek, Silver Creek and Blue Springs Creek. The mean simulated and observed flows as well as the percentage difference between these values are summarized in **Table C3-6**.

Table C3-6: Average Annual Discharge Calibration Statistics

	Black Creek	**Silver Creek**	**Blue Springs Creek**
Mean Observed Flow (m^3/s)	0.22	1.24	0.52
Mean Simulated Flow(m^3/s)	0.22	1.34	0.48
Difference (%)	**0**	**8**	**-8**
Mean Simulated Flow (mm/year)	368	333	341
Mean Observed Flow (mm/year)	369	307	368
Difference (mm/year)	**-1**	**26**	**-27**

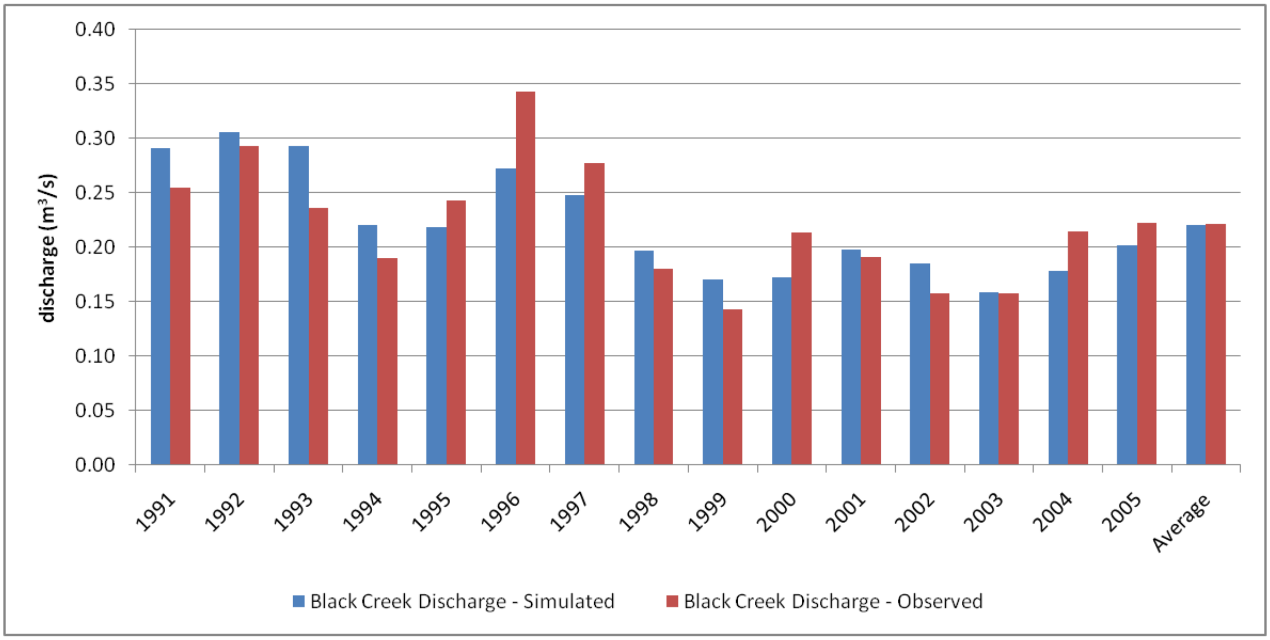

Figure C3-3: Average Annual Discharge - Black Creek (1991-2005)

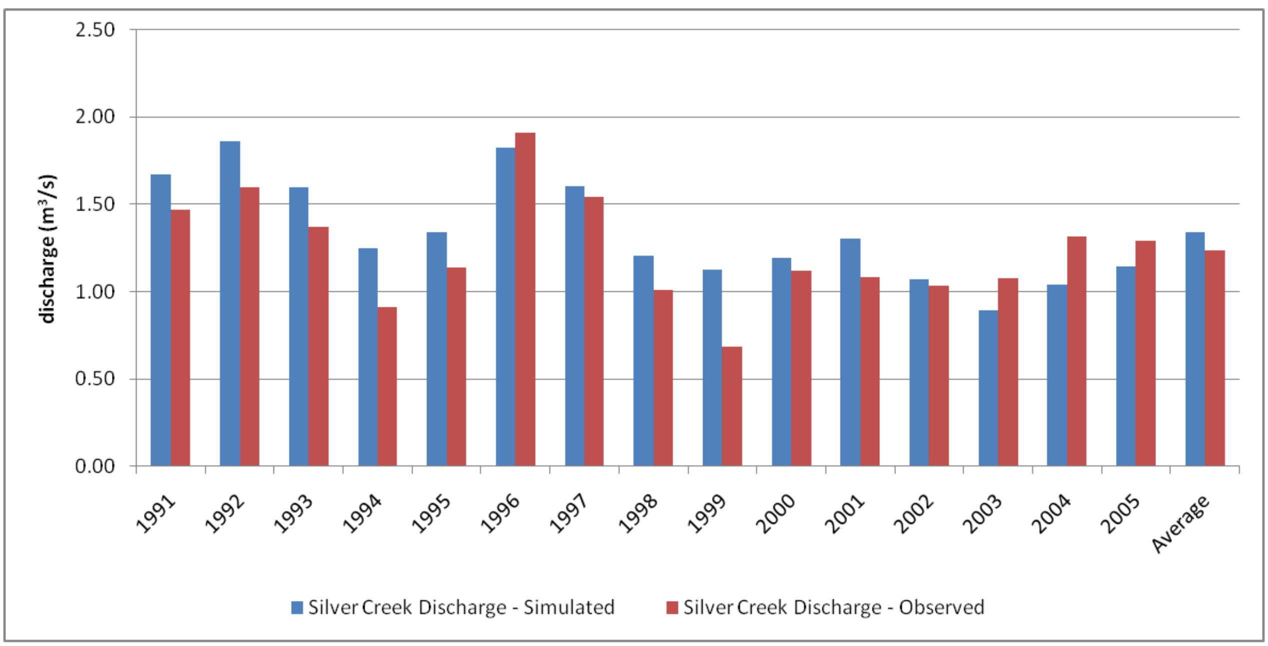

Figure C3-4: Average Annual Discharge - Silver Creek (1991-2005)

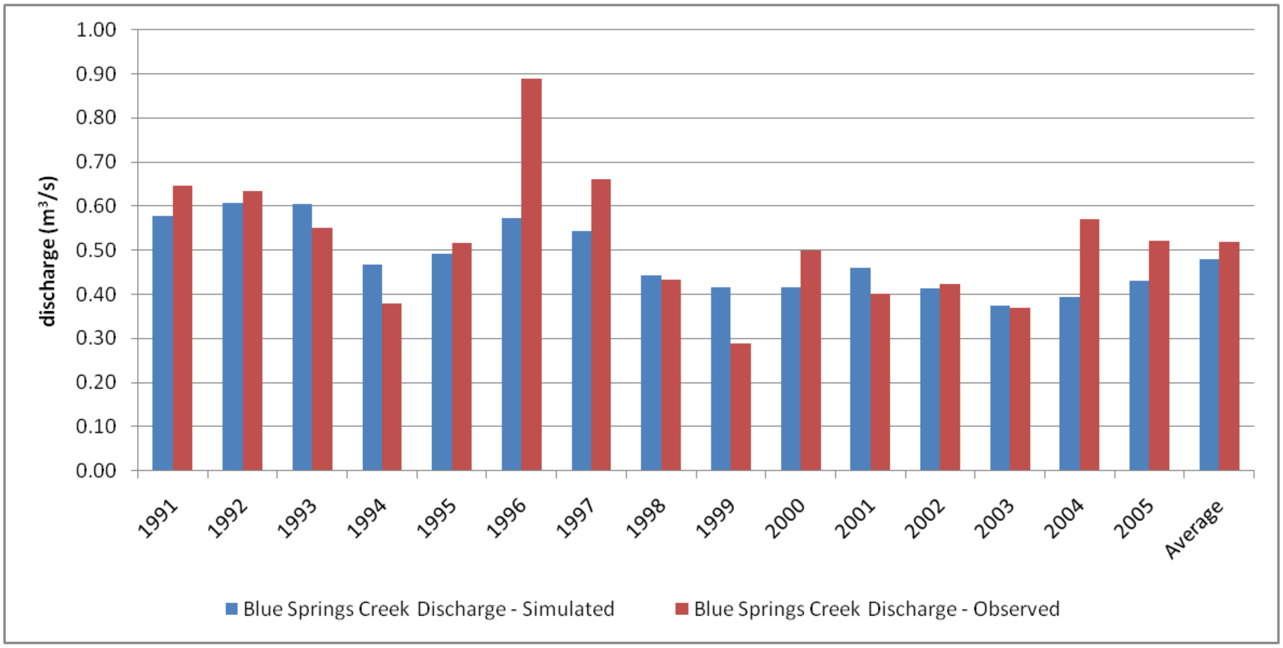

Figure C3-5: Average Annual Discharge – Blue Springs Creek (1991-2005)

Table C3-6 further represents the annual average flow normalized to the drainage area of each subwatershed. The representation of average annual streamflow in Black Creek, Silver Creek and Blue Springs creek can be considered good given that the standard error in streamflow estimates is typically anywhere from 5-15% of the actual streamflow value (Winter, 1981). Examining the annual flows normalized to the drainage area we observe that the difference in simulated and observed flows is well within the accepted range of error for stream gauges.

A comparison of the simulated to observed mean monthly flow values provide an assessment of how well the model represents the seasonal behaviour of the watershed. In general, a good approximation of mean monthly streamflow was achieved in Black Creek with a slight over estimation of flow during the summer. In the case of Silver Creek, a good approximation to spring flows was achieved, as well as flows during the fall and winter; however, summer flow predictions for Silver Creek are generally overestimated. Flows during this period are primarily baseflow derived, and parameter adjustments to evaporative, unsaturated, snowmelt and overland flow processes were not able to address this issue. The over-prediction of Silver Creek baseflow was also an issue for the Cedarvale groundwater model as described in the Conceptual Geology Report and may be simulation of groundwater flow along the escarpment (AECOM and AquaResource Inc., 2012a). Mean monthly flow estimates for Blue Springs Creek are significantly under-represented for spring snowmelt months. This may be an indication of an interflow process not being well represented within Blue Springs Creek, or a connection to Silver Creek/Black Creek was not being considered. An increased level of effort to match the flows in Blue Springs Creek was not completed as this area lies far from the Georgetown and Acton municipal well field areas.

A comparison of the simulated to observed mean monthly flow values provide an assessment of how well the model represents the seasonal behaviour of the watershed. In general, a good approximation of mean monthly streamflow was achieved in Black Creek with a slight over estimation of flow during the summer. In the case of Silver Creek, a good approximation to spring flows was achieved, as well as flows during the fall and winter; however, summer flow predictions for Silver Creek are generally

overestimated. Flows during this period are primarily baseflow derived, and parameter adjustments to evaporative, unsaturated, snowmelt and overland flow processes were not able to address this issue. The over-prediction of Silver Creek baseflow was also an issue for the Cedarvale groundwater model as described in the Conceptual Geology Report and may be simulation of groundwater flow along the escarpment (AECOM and AquaResource Inc., 2012a). Mean monthly flow estimates for Blue Springs Creek are significantly under-represented for spring snowmelt months. This may be an indication of an interflow process not being well represented within Blue Springs Creek, or a connection to Silver Creek/Black Creek was not being considered. An increased level of effort to match the flows in Blue Springs Creek was not completed as this area lies far from the Georgetown and Acton municipal well field areas.

Streamflow calibration statistics for mean monthly streamflow for Black Creek, Silver Creek and Blue Springs Creek for calibration period are included in **Table C3-7**.

Table C3-7: Flow Calibration Statistics

	Black Creek	Silver Creek	Blue Springs Creek
Mean Observed Flow (m^3/s)	0.22	1.24	0.52
Mean Error (m^3/s)	0.00	0.10	-0.04
Mean Absolute Error (m^3/s)	0.03	0.29	0.14
RMSE (m^3/s)	0.03	0.33	0.18
R	0.96	0.92	0.82
NSE	0.82	0.76	0.43

C3.4.2 Verification

The verification process provides an evaluation of the robustness of the calibration parameter set by using alternate sets of input data (climate) and observation data (observed flow). The verification period examined for the model was 1981-1990. Black Creek streamflow observations were only collected from 1987 to 1990 during this period, with 1987 flows only being partially observed. As such, streamflow at Black Creek was only compared to observation data for the years of 1988 to 1990.

Mean annual flow for Black Creek, Silver Creek and Blue Springs Creek are presented in **Figure C3-6**, **Figure C3-7**, and **Figure C3-8**, respectively. In general, a good approximation of observed mean annual flow was achieved during the verification period. As a result, it is concluded that a reasonable partitioning of precipitation into evapotranspiration, streamflow and groundwater recharge is occurring.

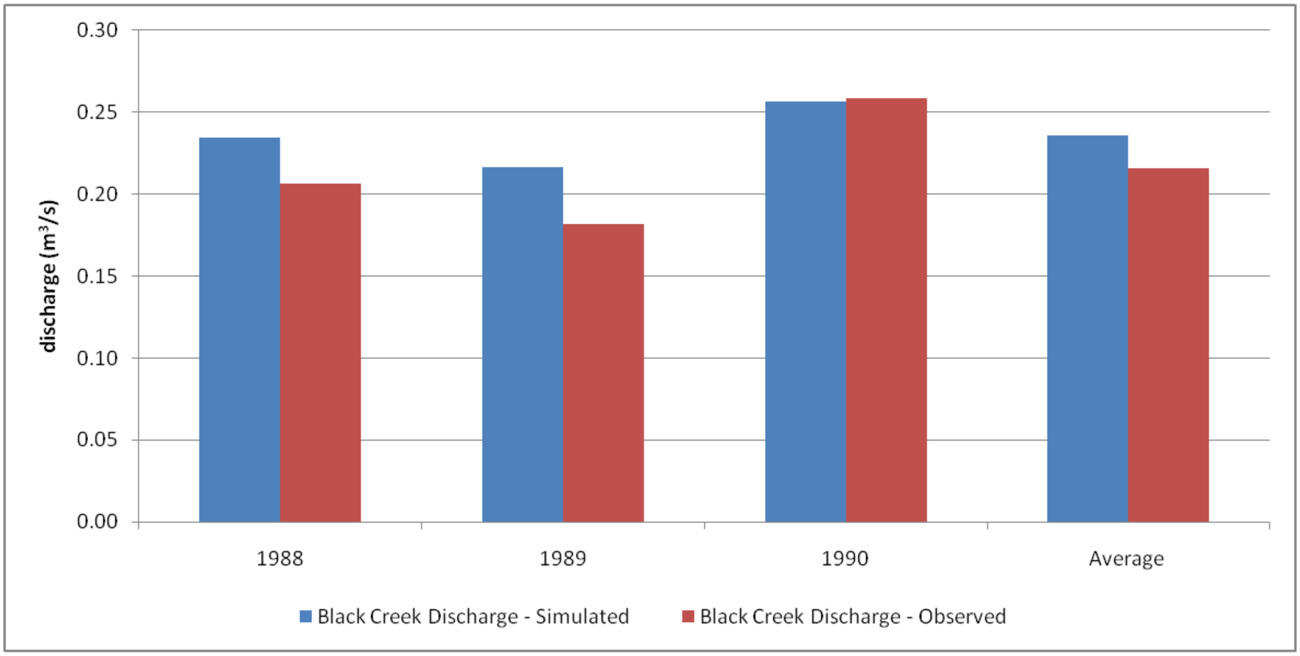

Figure C3-6: Mean Annual Flow – Black Creek – Verification Period (1988-1990)

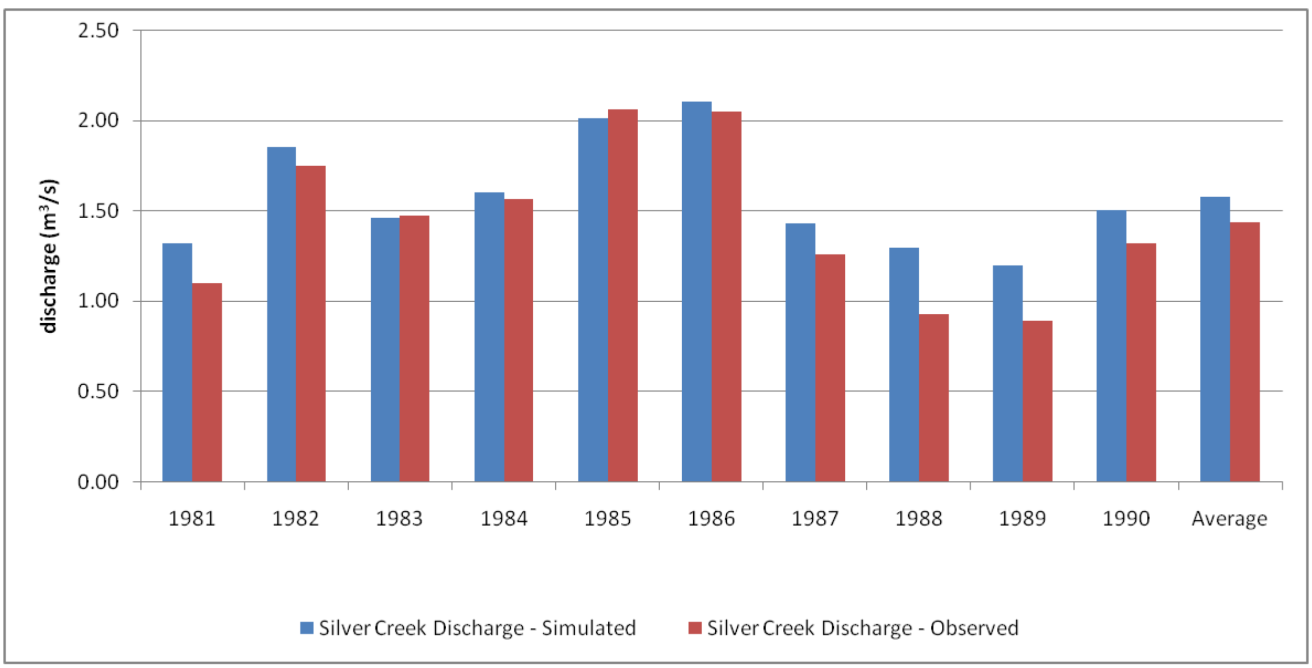

Figure C3-7: Mean Annual Flow – Silver Creek – Verification Period (1981-1990)

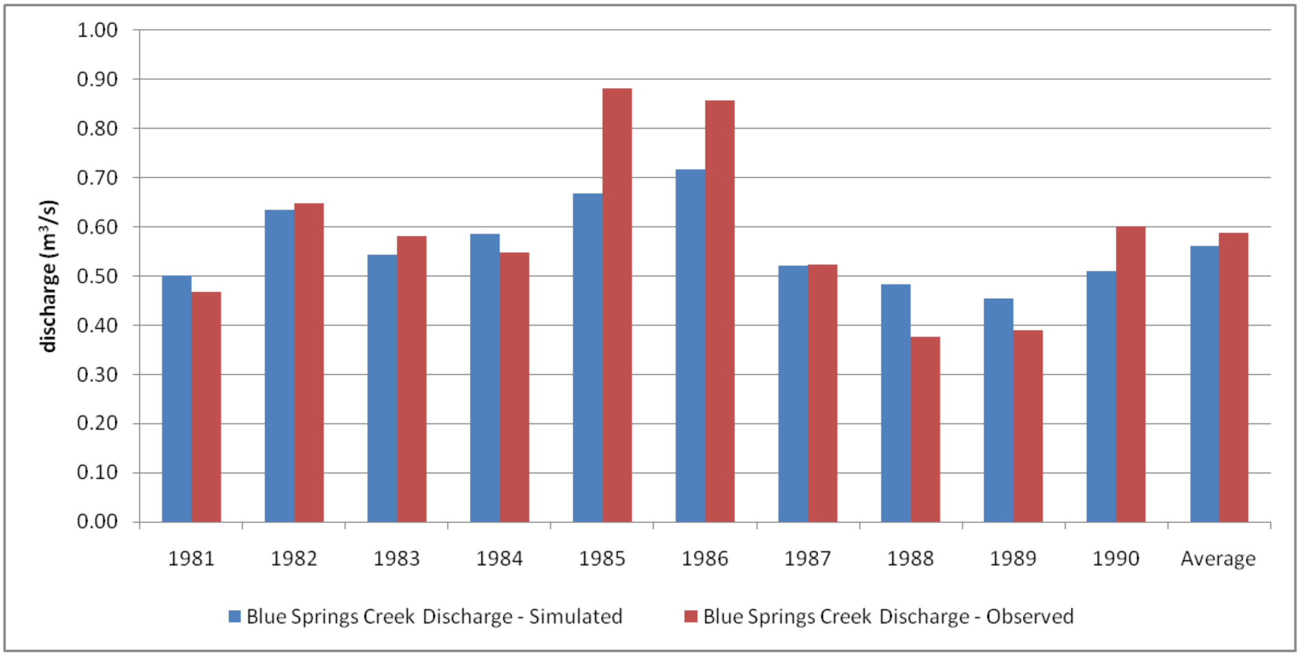

Figure C3-8: Mean Annual Flow – Blue Springs Creek – Verification Period (1981-1990)

Analyses of the mean monthly flow for Black Creek, Silver Creek, and Blue Springs Creek were also undertaken, and together with the mean annual flow data, they show that in general, the MIKE SHE model is performing similarly within the verification period as it did for the calibration period.

A reasonable approximation of monthly flow was achieved for Black Creek. Seasonal spring flows are well represented, while the baseflow dominated summer and early fall are slightly overestimated. In Silver Creek, a good approximation of mean monthly flows was achieved. Spring flows were well matched within Silver Creek while summer flow is moderately overestimated. In Blue Springs Creek, simulated spring and summer flows are not well matched. As previously stated in the calibration section, significant effort was expended trying to minimize the differences in the observed and simulated streamflow within Blue Springs Creek. As was the case with the calibration period, it was observed that no realistic adjustments to the model parameters could significantly affect the snowmelt portion of the hydrograph associated with Blue Springs Creek. The difficulties in simulating streamflow in this subwatershed are likely due to hydrologic complexities associated with seasonal interflow processes not captured in the model or the hydrologic effects of the wetlands in this region.

Daily hydrographs for the year of 1990 are presented for Black Creek, Silver Creek and Blue Springs Creek were also analyzed. In general, the response of Black Creek to precipitation events is well matched in terms of timing. The magnitude of response is reasonably approximated with some events in the summer periods being overrepresented. The recession of the hydrograph following the precipitation events is generally well represented.

Simulated flow for Silver Creek shows a similar level of performance as Black Creek. The snowmelt, magnitude of response, timing, and recession components of the hydrograph are well represented. As with the calibration period, summer baseflows are overestimated.

Daily streamflow values in Blue Springs Creek are reasonably represented in terms of timing; however, similar to the calibration period, recession during the snowmelt and spring period is significantly underestimated. Snowmelt related flows are simulated later than observed flows in certain years. The magnitude of observed flows at Blue Springs Creek is poorly approximated during this period as are the hydrograph recessions during this period.

The calibration statistics for the simulated flows during the verification period are summarized in **Table C3-8**. As expected, streamflow statistics show worse model performance for Black Creek and Silver Creek. Conversely streamflow statistics show an improvement in model performance for Blue Springs Creek.

Table C3-8: Verification Period – Calibration Statistics

	Black Creek	Silver Creek	Blue Springs Creek
Mean Observed Flow	0.22	1.44	0.59
Mean Error	0.02	0.14	-0.03
Mean Absolute Error	0.04	0.33	0.13
RMSE	0.05	0.38	0.17
R	0.91	0.88	0.87
Log NSE	0.53	0.60	0.56

Streamflow rank duration curves for Black Creek, Silver Creek and Blue Springs Creek were also assessed. Rank duration plots for the validation period are very similar to those of the calibration period. Generally, high flow periods are well represented in Black Creek and Silver Creek. Whereas low and moderate flow periods tend to be overestimated in these subwatersheds. In Blue Springs Creek, high flow periods are generally underestimated, and low flow periods are overestimated.

C3.5 Groundwater Heads

As the full groundwater system is simulated in MIKE SHE, simulated and observed groundwater heads can be compared to determine how well the model replicates groundwater flow conditions. As the saturated zone parameters were only calibrated within the FEFLOW model, groundwater elevations were not considered a calibration data set, but rather a verification data set.

The performance of the MIKE SHE model in replicating groundwater elevations was assessed against local high quality observation wells within the study area. Performance metrics for the groundwater calibration are summarized within **Table C3-9**.

Table C3-9: Groundwater Calibration Statistics

Number Observations	127
Mean Error (m)	2.17
Mean Absolute Error (m)	3.75
Root Mean Squared Error (m)	5.07
Normalized Root Mean Squared Error (m)	2.5%
Minimum Observed Head (m AMSL)	214.25
Maximum Observed Head (m AMSL)	417.09

A normalized root mean squared (NRMS) error of 2.5% was achieved for local high quality and regional high-quality wells. This percentage value allows the goodness of fit in one model to be compared with another, regardless of the scale. Typically, an NRMS of less than ten percent is considered representative (Spitz and Moreno, 1996; Lutz *et al.*, 2007; Gallardo *et al*., 2005); however, NRMS error is dependent on the range of observed water levels. In this study area, the range of observed water levels is approximately 200 m. As such, an error band of 10 m represents an NRMS of approximately 5%.

A scatter plot of simulated versus observed heads for high quality and medium quality observation data are presented in **Figure C3-9**. Approximately 70 percent of simulated heads fall within 5 m of observed heads. The cluster of wells simulated more than 10 m below the observed water level correspond to the shallow observation wells around the Princess Anne municipal wells. The calibration metrics and plot suggest that the groundwater flow system is well represented by MIKE SHE.

C3.6 Sensitivity Analysis

Sensitivities of model parameters were qualitatively assessed through the calibration process. Sensitivity of model output to variations in the following model parameters were tested.

Input Climate Data Sets

As the primary input of water into the model, model predictions can be extremely sensitive to the climate data sets utilized. Discussions with the Peer Review Committee identified the possibility that climate data collected at the Georgetown climate station may not be representative due to operational and/or station siting issues.

To test the sensitivity of model output, data for the Georgetown WTTP climate station was replaced with data from the Guelph Turf Grass climate station. While the daily hydrographs were modified for certain events, this variation did not significantly change the monthly mean predicted streamflow at the Black Creek, Silver Creek or Blue Springs stream gauges.

Evapotranspiration

As evapotranspiration is responsible for approximately two thirds of an area's water budget, parameters related to this process can have a significant impact on predicted streamflow and groundwater recharge rates. While evapotranspiration rates for a specific land cover/soil type can be highly uncertain, regional estimates of evapotranspiration are relatively well established, and have been previously documented to be in the range of 500 to 600 mm/yr. for the study area (MNR, 1984). Such estimates provide a useful constraint on MIKE SHE estimates of evapotranspiration.

Model parameters related to evapotranspiration include: potential evapotranspiration rates; vegetation rooting depth; soil water holding capacities; and vegetation leaf area index. Modifications in these parameters can result in significant variations in predicted evapotranspiration and as a result, changes in streamflow and groundwater recharge. The model results show excess streamflow during the summer months in certain watercourses (Silver Creek). Among other challenges, this can be evidence of insufficient evapotranspiration. During the calibration process, adjustments to model parameters related to evapotranspiration were attempted to reduce this excess summer streamflow; however, these adjustments resulted in the model domain evapotranspiration rate to be far in excess of 600 mm/yr., which is not supportable given the current understanding of evapotranspiration in southern Ontario.

Interception storage is related to evaporation and is the amount of water captured by vegetation prior to rainfall reaching the ground surface. This water is subsequently evaporated following the precipitation event. While the incremental values associated with interception storage are small (~ 1 mm), on an annual basis they can be significant (~5% of annual precipitation was captured as interception storage within MIKE SHE). Variations in this parameter did not significantly impact streamflow or groundwater recharge predictions.

Unsaturated Zone

Unsaturated zone processes, and their related parameters, are predominantly responsible for generating groundwater recharge. Due to the majority of evapotranspiration occurring within the unsaturated zone, there is significant overlap between parameters related to evapotranspiration and unsaturated zone processes. Major parameters influencing unsaturated zone processes are as follows: infiltration rate; soil water holding capacities (wilting point, field capacity and saturation); and vegetation rooting depth. These values exhibited a significant influence on the streamflow and recharge predictions of the model, although there was limited freedom in modifying these values, while keeping evapotranspiration rates within published ranges. Peak daily flows were particularly sensitive to the specified infiltration rates. Adjustments to the unsaturated parameters were made to minimize differences between observed and simulated streamflow; however, there was insufficient freedom in the parameters to address the excess summer flow simulated within Silver Creek, and insufficient spring flow within Blue Springs Creek.

Overland Flow

Overland flow processes are primarily responsible for routing runoff from grid cells to receiving water courses. Typically, these processes primarily affect hydrograph timing and sub-daily streamflow values. As MIKE SHE is a gridded model that allows for the infiltration of water that flows onto a particular grid cell from an upgradient grid cell, overland flow processes can affect infiltration, which in turn affects the unsaturated zone and groundwater recharge. The primary determinant in overland flow processes is the land surface topography as specified by the DEM, which was treated as a static input data set and not modified during calibration. The Manning's roughness coefficient is also an important parameter for determining overland flow and is specified based on land cover. Increasing the Manning's coefficient will reduce the velocity of overland flow, which will reduce peak streamflow, and will allow additional opportunity for overland flow to infiltrate prior to reaching the surface water network. Adjustments to the Manning's coefficient were made during the calibration process. While these adjustments resulted in changes to certain events within the simulated hydrograph, they did not significantly impact long term average streamflow.

Depression storage is another value that affects overland runoff but is also related to unsaturated zone and evapotranspiration processes. Depression storage represents the storage within micro-depressions found on the landscape (e.g., furrows left in agricultural fields after plowing). When liquid precipitation falls at a rate faster than the rate of infiltration for a grid cell, overland flow is generated. Prior to overland flow leaving a particular grid cell, available depression storage must be exhausted. Water that is held in depression storage is subsequently infiltrated or evaporated. Increasing depression storage has the impact of reducing overland runoff, increasing infiltration, and increasing evaporation. Groundwater recharge can also be affected by variations in this parameter, but is typically a minor influence, as depression storage has the largest impact during the summer months, a time in which groundwater recharge is typically zero.

Snow Melt

Snow melt processes govern the accumulation and depletion of the snowpack. The melting of accumulated snowpack during thaws (e.g., spring melt) can generate significant streamflow. The overland flow, interflow and baseflow components which comprise streamflow are all affected significantly by snow melt. Major parameters that influence the snow melt process are the threshold melting temperature and the degree day coefficient (mm snow /day/°C). These values predominantly impact the timing of the snowmelt, but typically do not affect the volume of water released by the snowmelt. One area of uncertainty is the density of newly fallen snow. Following Environment Canada's standard assumption, newly fallen snow was assumed to have a non-variable (temporally or spatially) 10% water content. Given the insufficient snowmelt volume contained within the Blue Springs Creek simulated hydrograph, it is possible that snowfall occurring above the escarpment has a higher density than the 10% assumed. The value was not adjusted due to a desire to maintain the snowmelt processes for Black Creek, which also drains land area above the escarpment.

Channel Flow

Channel flow processes relate to the conveyance and translation of a river hydrograph as it moves downstream. Channel cross-section geometries, channel slope and Manning's roughness all affect channel flow processes. Channel geometries and slope were products of the DEM and were not varied within the calibration process. Variations in Manning's coefficients were not investigated due to channel routing not being a key objective of this modelling study.

Drainage

Drainage flow processes control the interflow or subsurface storm flow. The major parameters that affect drainage are drainage depth and the drain time constant, and adjustments to these parameters affect the recession timing and volume of water contained within the recession component of a hydrograph. Consequently, the drainage flow processes have a significant effect on streamflow. Generally, as interflow volume decreases, less water is contained within the recession component of the hydrograph and baseflow volume increases. Adjustments were made to the drain level and drain time constant during calibration to replicate recession components of the observed hydrograph.

Saturated Zone

As saturated zone properties and layers were directly imported from the FEFLOW model, no variations in the saturated zone properties were tested within MIKE SHE. It was noted that the largest changes in simulated baseflow were observed when significant updates to the saturated zone properties were incorporated into the MIKE SHE model.

Watercourse Leakage

Streambed conductance greatly influences the amount of exchange between the saturated zone and the watercourses represented in MIKE SHE. Adjustments to this parameter can cause significant losses from the watercourse to the saturated zone, or vice versa. Streambed conductance was adjusted in those areas where spot streamflow measurements indicated leakage, with the objective to replicate observed leakage rates.

C3.7 Output for Groundwater Flow Model

The spatial distribution of average annual groundwater recharge for the 1991-2005 time period is presented in **Figure C3-10**. The recharge distribution is influenced primarily by the distribution of the various surficial geology types within the study area. The portion of the study area east of the escarpment is largely characterized by low recharge associated with Halton Till; however, a number of isolated areas with coarse-grained sands and gravels near surface and resulting high recharge rates occur within this portion of the model. The low permeability of Halton Till generates overland flow, which travels downslope and infiltrates into the adjacent sand units resulting in increased groundwater recharge. West of the escarpment, relatively higher recharge rates are predicted for the Wentworth Till and associated gravel sediments, which are common in this area.

Secondary features that influence the spatial distribution of recharge are imperviousness and areas of groundwater discharge. The urbanized regions of Georgetown, Acton and Brampton have reduced recharge rates relative to the regions surrounding them as a result of the paved runoff abstraction applied in these areas. This represents runoff associated with the directly connected impervious areas in these urban areas. The wetland regions associated with the Eramosa-Blue Springs Creek Wetland Complex are predicted by the MIKE SHE model as areas of significant groundwater discharge. Consequently, the upward groundwater gradients in this area limit recharge from occurring in this area.

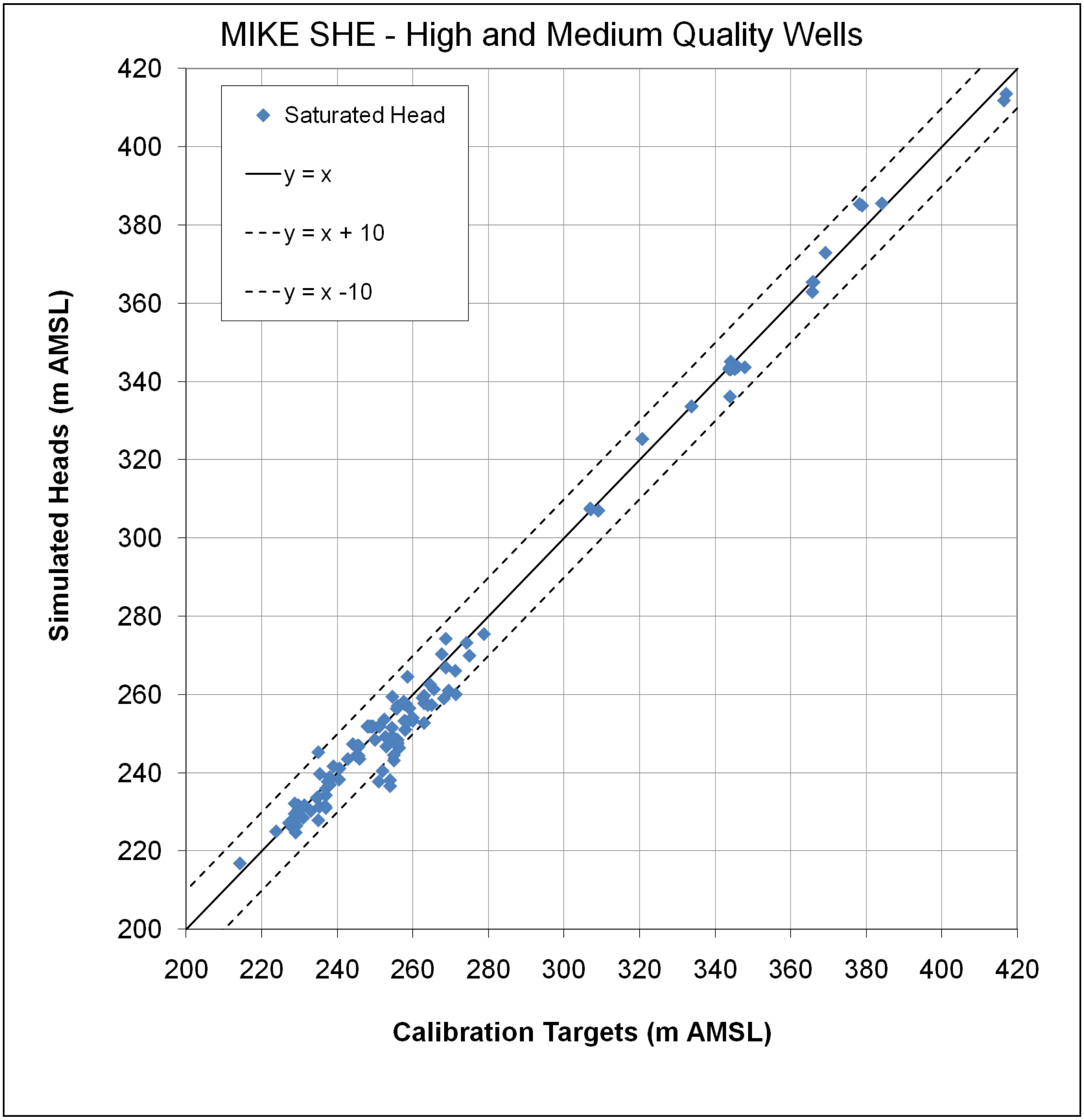

Figure C3-9: Simulated versus Observed Water Levels – Scatter Plot

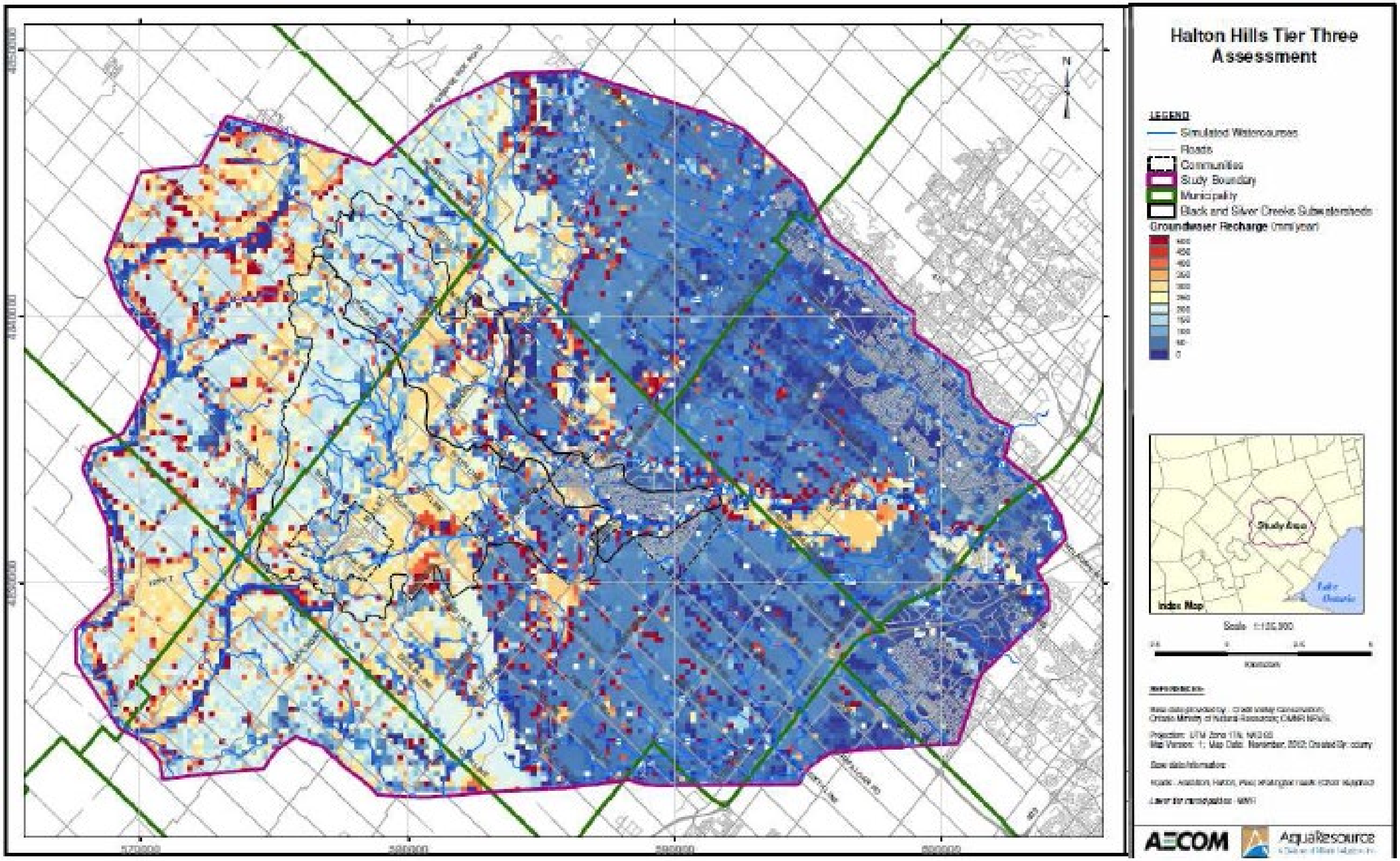

Figure C3-10: Spatial Distribution of Average Annual Recharge (1991-2005)

The median and arithmetic mean recharge rates for the various soil classes are summarized in **Table C3-10**. The median recharge rate may be considered more representative than the mean because of the presence of localized features. The localized features whose recharge rates deviate significantly from the median are typically areas with significant hydraulic controls (e.g., ground surface depression, upward groundwater gradient), which overwhelm typical groundwater recharge mechanisms (e.g., infiltration, soil water holding capacities). These localized features are infrequent within the model domain but skew the mean value of recharge for specific soil types. If a specific soil class (e.g., sand) is located in areas with a higher density of these localized features than a more pervious soil class (e.g., gravel), then the mean recharge rate for the sand may be higher than for the more pervious gravel.

Table C3-10: Recharge Estimate (1991-2005)

Soil Class	Median Recharge (mm/year)	Mean Recharge (mm/year)	Area (km^2)	Percentage of Study Area
Global	111	139	765	100
Halton Till	60	61	301	39
Wentworth Till	184	198	156	20
Clay	48	13	97	13
Gravel	300	288	91	12
Bedrock	228	205	87	11
Sand	309	329	34	4

C3.7.1 Transient Groundwater Recharge Estimate

To allow the groundwater flow model to be run in transient mode and calibrated against water level hydrographs, transient recharge is required. To facilitate this, time series of groundwater recharge rates were generated from the MIKE SHE simulations.

Transient recharge rates were extracted from six representative grid cells, each representing the mean conditions for the six major soil classes. Care was taken to ensure that recharge rates for the selected grid cells were not overly influenced by wetlands, streams, or regions of significant topographic variability.

Transient recharge estimates for representative cells of Halton Till and gravel for the period of 1995-2005 are presented **Figure C3-11** and **Figure C3-12**, respectively. A general seasonal trend of significant groundwater recharge occurs during the snowmelt in the late winter and spring while the summer and fall period are generally simulated as having relatively little or no recharge. A twelve-month moving average is also plotted on **Figure C3-11** and **Figure C3-12**, which allows longer term trends to be more easily identified. The drought period experienced in southwestern Ontario during 1998 and 1999 is clearly evident in the Halton Till transient recharge estimate. This drought period is also evident in the gravel transient recharge estimate though to a lesser extent.

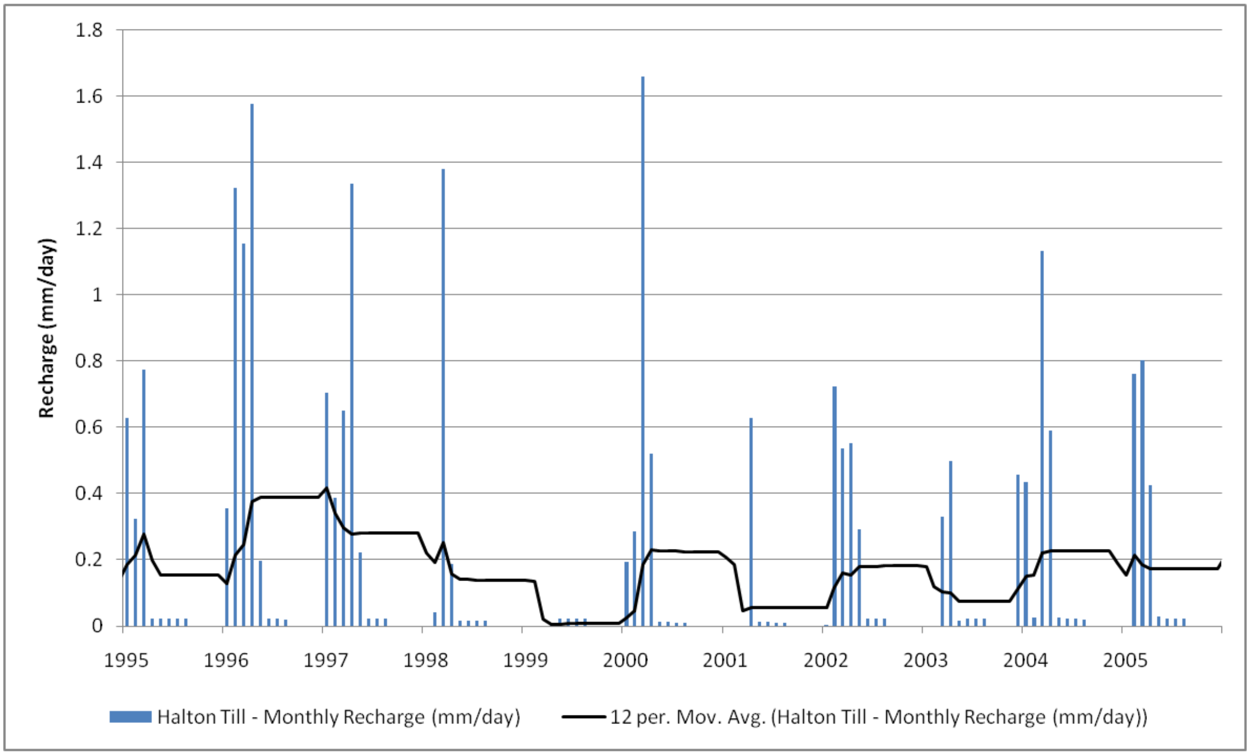

Figure C3-11: Halton Till - Transient Recharge Estimate

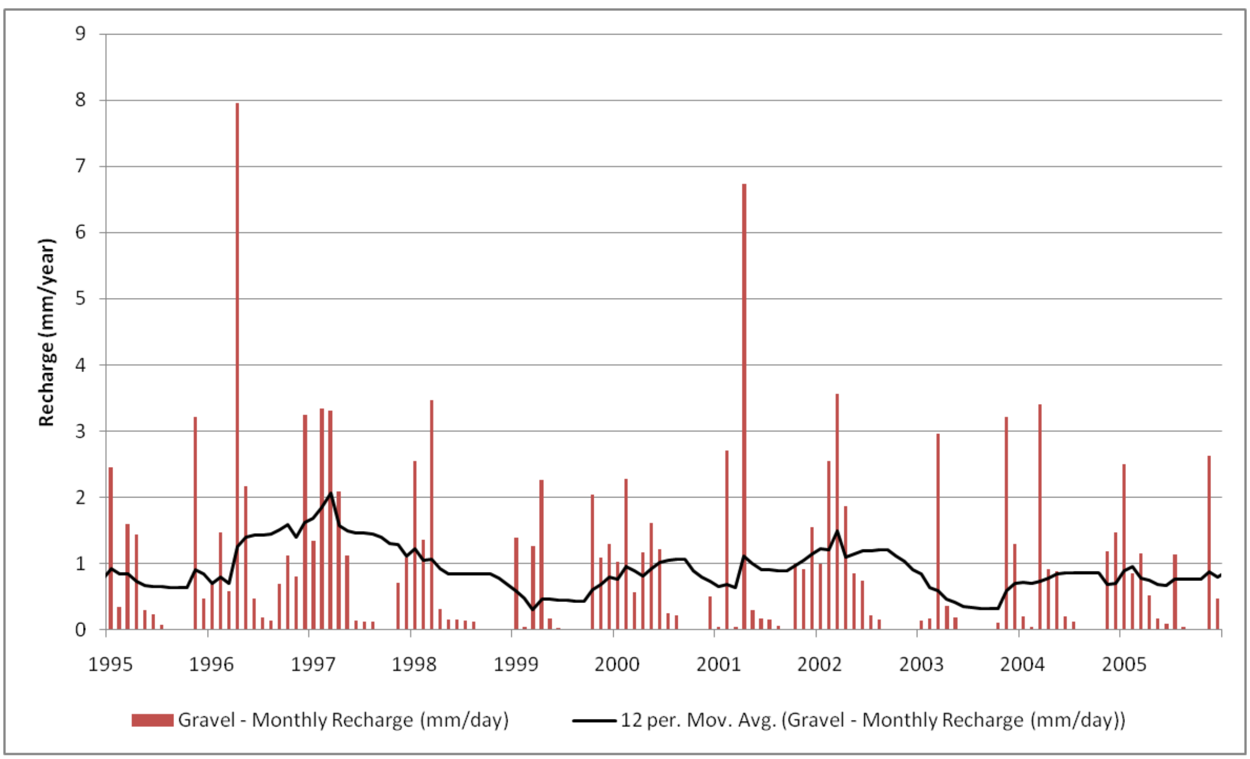

Figure C3-12: Gravel- Transient Recharge Estimate

C3.7.2 Groundwater-Surface Water Interaction

The MIKE SHE simulation results illustrate groundwater exchange between the saturated zone and overland features. The discharge zones correspond with the regions of zero recharge in the spatial distribution of recharge (**Figure C3-10**). Generally, discharge features (flow of groundwater into streams) dominate the groundwater surface water exchange within the study area. Note that groundwater exchange shows the general seepage locations adjacent to streams. Discharge conditions to streams are shown in **Figure C3-13**.

Regions of significant discharge predicted by the MIKE SHE model correspond largely with observed wetlands and topographic features. The significant groundwater discharge areas simulated within Blue Springs Creek and the Eramosa River correspond with mapped wetland complexes in those regions. Additionally, the simulated groundwater discharge areas compare favourably to cold-water fisheries mapping. A number of significant discharge areas also correspond to regions of steep slope in the study area, with discharge occurring at various points along the face of the escarpment. The zone of discharge present below the confluence of Silver Creek and the Credit River is likely a product of a rapid reduction in overburden thickness on the south side of the of the Credit River where discharge occurs. Similarly, the zone of discharge along the Credit River upstream of Georgetown is likely a product of the thinning of the overburden within that area. Finally, a number of isolated areas simulated as recharge features are most likely due to local closed depressions represented on the study DEM.

Stream Leakage

A critical aspect of this modelling effort was determining stream losses to the groundwater system in areas immediately surrounding the municipal wellfields. **Figure C3-13** illustrates the simulated baseflow throughout the study area.

Spot streamflow measurements collected within subwatershed 10 and 11 provide insight into losing and gaining conditions within those watercourses and are summarized on **Figure C3-14**. The simulated baseflow trends of Beeney Creek, as well as the Hospital Tributary and Silver Creek through the Cedarvale wellfield were examined in detail. **Figure C3-15** illustrates the simulated baseflow conditions within the hydrologic model for Beeney Creek, Hospital Tributary and Silver Creek (through Cedarvale wellfield). The reach on Black Creek between SW41 and SW38 is observed to have leakage, which is not represented in the model, but does show a decrease in baseflow compared to upstream sections.

Streamflow observations (AECOM and AquaResource Inc., 2012c) indicate that within the Cedarvale wellfield area, Silver Creek leakage is intermittent throughout the year and is small with respect to total streamflow (less than one percent). Spot streamflow observations in low flow periods in 2009 (AECOM and AquaResource Inc., 2012c) for Beeney Creek indicate that significant stream leakage occurs through spot streamflow station SW10 and SW17. Up to 5,000 m^3/d of stream leakage is observed in this reach of Beeney Creek during spot streamflow monitoring. As spot streamflow observations are conducted during low flow conditions it is expected that seasonal variations will cause this leakage to increase or decrease during the year as stream stage and to a lesser extent, groundwater heads, vary throughout the year. The baseflow characteristics of Hospital Tributary were studied in an environmental report (Gartner Lee Limited, 2007), and they concluded that the baseflow contributions increase moving down the stream.

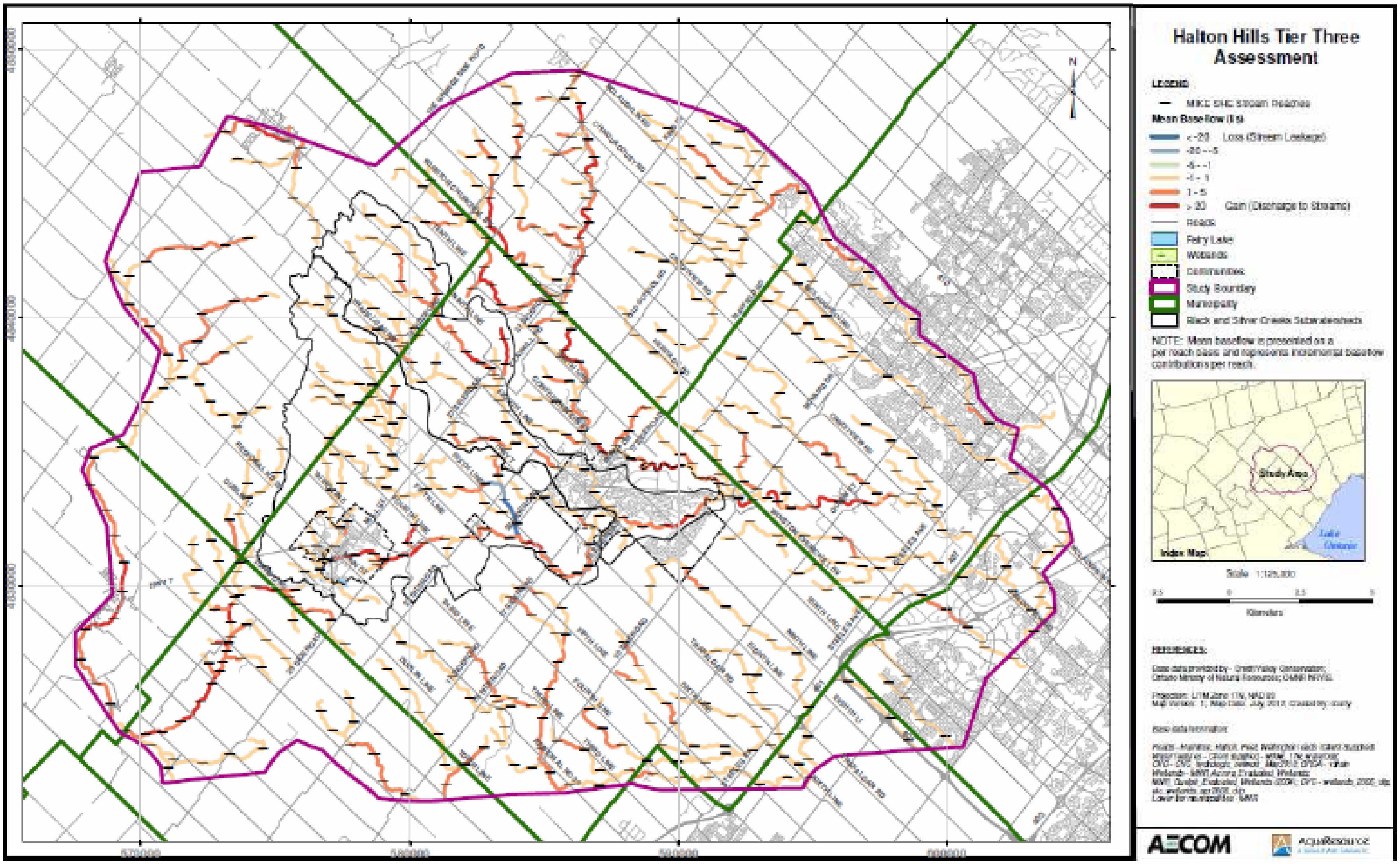

Figure C3-13: Simulated Baseflow in Hydrologic Model (1991-2005)

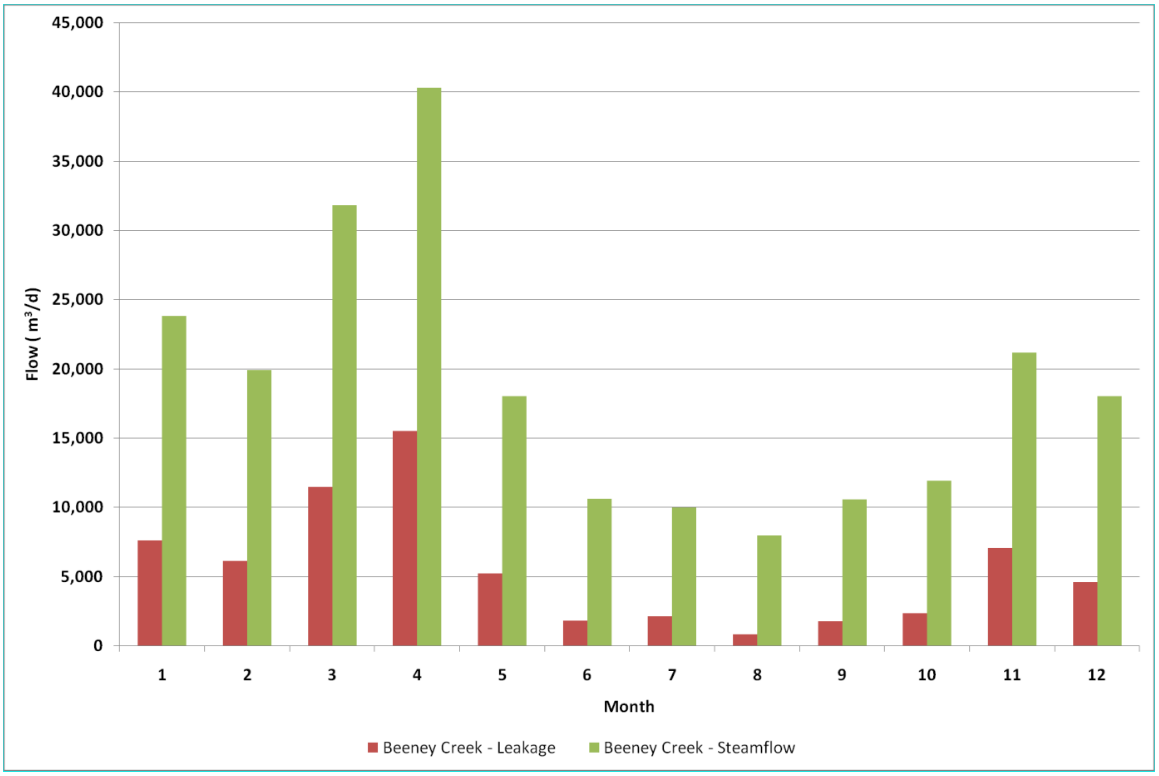

Figure C3-14: Simulated Beeney Creek Leakage and Streamflow (1991-2005)

The simulated losses from a section of Silver Creek through the Cedarvale wellfield are consistent with the observed data (AECOM and AquaResource Inc., 2012c). Hospital Tributary is simulated as having intermittent baseflow in its headwaters with the only sustained baseflow occurring as it approaches the confluence with Black Creek. This pattern is consistent with observed data. The streambed conductance of Beeney Creek was adjusted to approximate observed losses observed through this reach. **Figure C3-14** shows the mean monthly leakage simulated in Beeney Creek though SW10 to SW17 alongside mean monthly streamflow simulated in Beeney Creek.

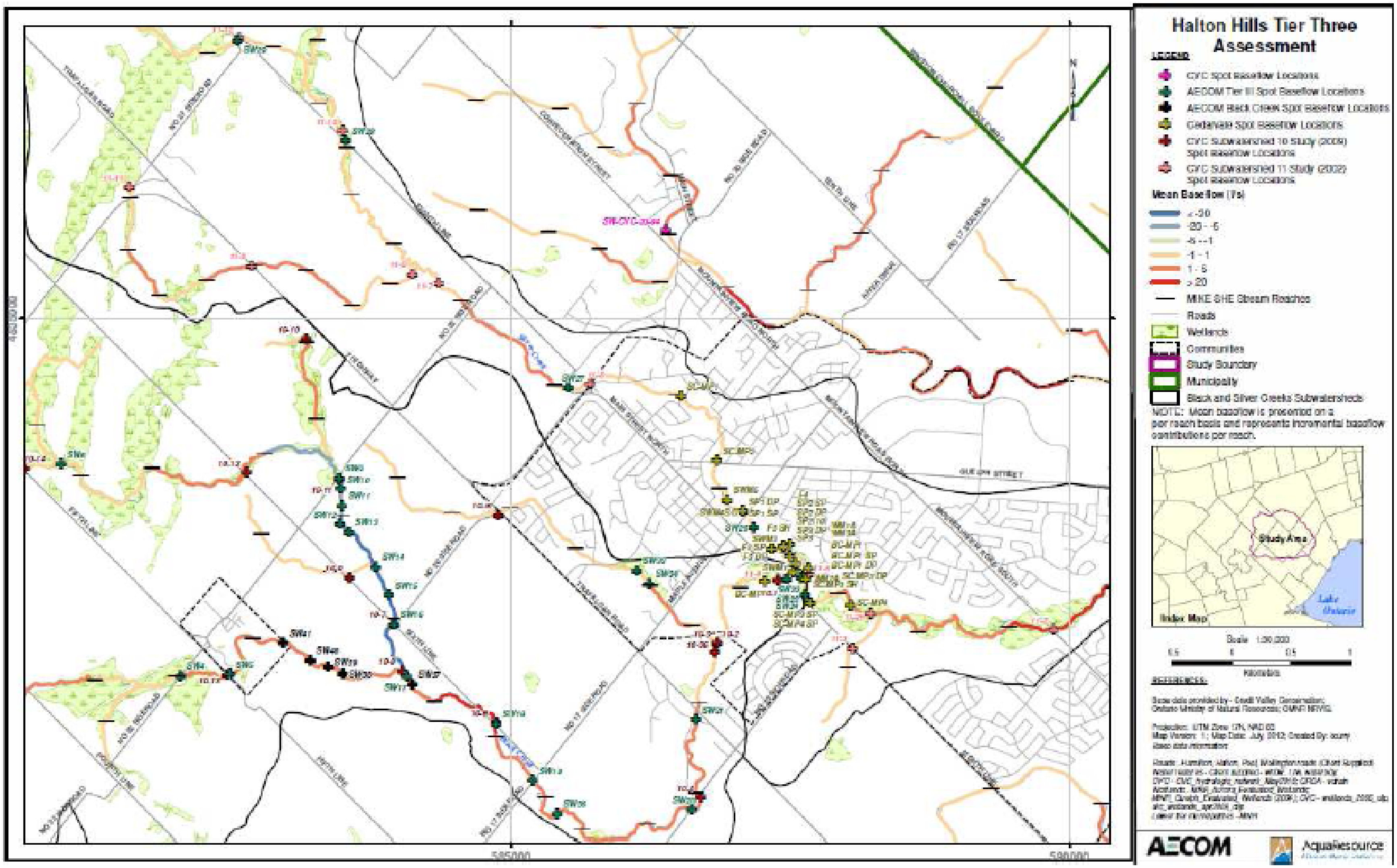

Figure C3-15: Losing Reaches in Subwatershed 10, 11 (1991-2005)

C3.8 The Hydrogeologic Model: FeFlow

The groundwater flow model for the study area was developed using FEFLOW, a commercially available finite element groundwater modelling code developed by DHI-WASY (2011).

FEFLOW was selected for use in this assessment for the following reasons:

- Ability to discretize the mesh around specific areas of interest such as pumping wells or rivers, to more accurately simulate observed physical features and follow naturally complex boundary conditions such as the steep slopes of the Niagara Escarpment;
- Efficiency of localized mesh discretization;
- Ability of the elements to conform to the pronounced vertical variation of aquifer/aquitard layers (e.g., Niagara Escarpment);
- Advanced boundary conditions to avoid potential impacts of non-physical boundary conditions on the simulation results; and
- Stable water table simulation that facilitates and decreases numerical issues.

Groundwater flow is simulated by applying boundary conditions at the locations where water enters or leaves the groundwater flow system (e.g., recharge, discharge to streams, pumping wells) and is subjected to a variable hydraulic conductivity value, storage value, and porosity field. For the purpose of a Tier 3 assessment, the FEFLOW model was built in variably saturated mode to allow the benefit of water table approximation.

Modelling Process

The groundwater modelling tool was developed to represent the groundwater flow system in Acton and Georgetown, as described in Conceptual Model Report (AECOM and AquaResource Inc., 2012a), and to complete a risk assessment of the sustainability of existing municipal wells and impacts to other users under existing and planned groundwater demand. Through steady-state and transient calibration, long-term verification, and sensitivity analysis the conceptual model was refined, and the representativeness of the model was demonstrated by comparison of simulated and observed groundwater levels, and stream flow. The objectives and details of each of the model tool development steps are described below.

Steady-State Calibration

During the steady-state calibration, the initial model input parameters and boundary conditions developed from the conceptual model were adjusted to obtain a reasonable fit to the range of average groundwater levels (heads) and streamflow (baseflow) values. The steady-state time period selected was 2005 through 2009, as it provided the most complete observation dataset, closely represented existing pumping conditions and the year-to-year pumping rates for the pumping wells showed minimal variability. Average pumping rates for 2005 through 2009 were specified for each of the Acton and Georgetown pumping wells as shown in the foundation report and average groundwater recharge was applied directly from the hydrologic model cells based on the 1991 to 2005 calibration simulation. At the time of calibration more recent climate data was not available, but the average conditions from 1991 through 2005 were considered representative for 2005-2009 average conditions based on available precipitation data.

Transient Calibration

Transient calibration was undertaken to further refine the conceptual model and improve the representativeness of the modeling tool by evaluating hydraulic connections within the groundwater system (e.g., along the buried valley) and between the surface water and groundwater systems. These additional refinements were based on a single transient simulation from 2008 through 2010 using daily pumping data. This time period was selected as transient water level monitoring data are available for many high-quality monitoring wells for all or part of this time period and it includes the Prospect Park long-term aquifer pumping test and shutdown of the Cedarvale wellfield for maintenance for more than four months. Both of these events imposed a significant change in pumping stress to the aquifer system for a period of time exceeding the duration of previous stresses from aquifer tests or well shutdowns. The observed dataset from this time is the best available for evaluating hydraulic connections and refining parameter distributions and boundary conditions. In addition, the transient calibration provides a means of assessing the variability of recharge, its influence on groundwater levels, and testing the transient recharge output from the hydrologic model which was used to generate transient recharge for evaluating well sustainability under drought conditions. Changes to input parameters made during the transient calibration were incorporated back into the steady-state model to ensure the same input parameters were represented and supported under both conditions.

Long Term Model Verification

A transient simulation using monthly pumping and recharge was completed for the 1995 through 2005 time period following calibration to steady-state and transient conditions. This transient simulation was undertaken to further evaluate the representativeness of the refined conceptual model. A set of observed transient groundwater levels available for a few wells between Acton (Beeney Creek) and the Cedarvale wellfield (Georgetown) were compared against the simulated conditions to further demonstrate the suitability of the model representation of central area of the model following model calibration. A key feature represented in the model verification was the seasonality of recharge, and the recharge pulse lag effect observed in wells east of the escarpment along the Acton-Georgetown Bedrock Valley.

Sensitivity Analysis

An assessment of the sensitivity of the model input parameters was conducted to provide a basis for a discussion on the uncertainty associated with the modelling and the model results. Additional sensitivity analysis scenarios were calibrated as part of the Updated Vulnerability Analysis documented in the foundation report and used in the assessment of uncertainty in the risk assessment.

C3.8.1 Model Domain

The model domain is presented in **Figure C3-1**. The domain is approximately 36 km by 28 km and extends from Northwest Brampton in the east to the Eramosa River in the west, Erin in the north to Milton and Mississauga in the south. The total area is approximately 745 km^2. The model domain was developed based on current and potential future municipal well locations identified by Halton is consistent with the model domain established for the Cedarvale Model (EarthFX, 2009).

The model domain follows the interpreted groundwater divides or is located at sufficient distance to minimize the bias of boundary conditions on the area of interest (Acton and Georgetown municipal supply wells). The model domain is consistent with the hydrological model domain and represents the key subwatersheds influencing Acton and Georgetown including: Blue Springs Creek, Black Creek, Silver Creek, and parts of the main Credit River subwatersheds.

C3.8.2 Model Grid

Figure C3-1 shows the finite element mesh designed for the Tier Three assessment. The finite element mesh consists of 2,867,355 elements (1,539,488 nodes) within the 15 model layers (16 slices). This equates to 191,157 elements per layer, and 96,218 nodes per slice.

The largest elements are approximately 100 m in areas outside municipal wellfields or distant from streams. Along the streams in Silver Creek and Black Creek subwatersheds, the mesh was refined to 25 m element size with nodes following the stream channel. Around the municipal pumping wells, additional refinement was used to achieve a typical element size of 3 to 5 m to improve the representation of drawdown at these locations.

Vertically, the model domain is divided into 15 model layers and 16 slices. The layers are of variable thickness representing the interpreted thickness of the hydrostratigraphic units represented in the layer as determined through cross-section analysis.

C3.8.3 Hydrostratigraphic Layer Structure

The numerical model was subdivided into 15 hydrostratigraphic layers. Each of the units represented by the FEFLOW model has a variable model layer thickness. The elevations associated with the top of each model layer (i.e., the model slices) were assigned based on the two-dimensional cross-sections generated and interpreted across the model domain (see Conceptual Model Report for more detail). Layers are continuous through the model domain, and where a hydrostratigraphic unit “pinches-out”, the layer thickness is reduced to a minimum (i.e., 0.1 m) and the hydraulic conductivity of the layer at this location is changed to the value of the overlying or underlying unit.

The foundation report describes the refined hydrostratigraphic framework used as the layer structure in the initial groundwater flow model. Following initial runs of the model, it became apparent that further subdivision of the Maple/Oak Ridges Equivalent was appropriate to better represent vertical head gradients. As a result, this overburden unit composed of sand, gravel, and silt, was subdivided into multiple units based on the lithologic descriptions and the observed hydrogeologic conditions near the wellfields.

Model layers one to eight represent overburden sediments while layer nine represents weathered bedrock (also referred to as the ‘contact zone’). Layers 10 to 15 represent bedrock units. **Table C3-11** lists the 15 model layers and the hydrostratigraphic units represented in each layer.

Table C3-11: Model Representation of Hydrostratigraphic Units

Model Layer	Hydrostratigraphic Unit	Predominant/General Lithology
1	Aquitard	Weathered Halton Till & Wentworth Till /Surficial Units/Streambed and Pond Conductance layers
2	Aquitard	Halton Till & Wentworth Till/Surficial Units
3	Aquifer	Maple/Oak Ridges Equivalent – mix of silt, sand
4	Aquifer	Maple/Oak Ridges Equivalent – mix of silt, sand
5	Aquifer	Maple/Oak Ridges Equivalent – mix of silt, sand, gravel and cobbles
6	Aquitard/Aquifer	Wentworth Till/Upper Newmarket Till/Lower Sediments*
7	Aquifer	Inter-Newmarket/Lower Sediments*
8	Aquitard/Aquifer	Lower Newmarket Till, Lower Sediments *
9	Bedrock Aquifer	Contact zone aquifer/ Weathered bedrock surface, all formations
10	Bedrock Aquifer	Guelph Formation
11	Bedrock Aquitard	Eramosa-Vinemount Member
12	Bedrock Aquifer	Gasport-Goat Island Formation
13	Bedrock Aquitard	Cabot Head Formation
14	Local Bedrock Aquifer	Manitoulin and Whirlpool Formations
15	Bedrock Aquitard	Queenston Formation

C3.8.4 Model Properties

Hydrogeologic properties assigned within the FEFLOW model included hydraulic conductivity values and estimates of specific storage and specific yield. The hydraulic conductivity of a hydrostratigraphic unit plays a significant role in the simulated hydraulic head distribution within that unit (constrained by the boundary condition values). Hydraulic conductivity values also represent the streambed conductance along streams and rivers and influence the flux of water flowing into or out of surface water features. Storage parameters are not used in steady-state simulations; however, under time varying (transient) conditions, specific yield and specific storage control the timing and response of the groundwater system to external stresses.

Hydraulic Conductivities

In the initial modelling simulations, hydraulic conductivity values were assigned to each hydrostratigraphic unit represented in each model layer. Zones, or groups of elements with common hydraulic conductivity values, are used to represent the understanding of the distribution of hydrostratigraphic units within each layer. Hydraulic conductivity zones were assigned based on surficial and bedrock geology maps, interpreted isopachs of hydrostratigraphic units and bulk hydraulic conductivity estimates. Hydraulic conductivity values were assigned based on literature values, available field measurements (e.g., slug tests), and the bulk conductivity estimates based on lithologic descriptions from well logs.

During model calibration, the hydraulic conductivity values were adjusted to achieve an acceptable fit between the model predicted and observed hydraulic head data. Initially in steady-state and then refined in transient calibration. Adjustments to the hydraulic conductivity values were constrained by literature values, field data, and understanding of relative hydraulic conductivity values based on lithologic descriptions of the units. **Table C3-12** shows the range of calibrated hydraulic conductivity values for each hydrostratigraphic unit.

The hydraulic conductivity distribution in model layers one and two was guided largely by the surficial geology mapping. These layers include Port Stanley, Wentworth and (weathered) Halton Tills, bog and glaciolacustrine deposits, alluvium, glaciofluvial outwash, sands and ice-contact stratified drift. Some of the underlying bedrock units outcrop at surface, including weathered Guelph, Eramosa-Vinemount, Gasport / Goat Island, Cabot Head, Manitoulin-Whirlpool, and Queenston Formations. In layer one, elements touching any stream or lake boundary condition nodes were grouped into separate stream bed conductivity zones to represent the exchange of water between surface water features and the underlying groundwater flow system.

The hydraulic conductivity values applied to represent the stream bed and lake bottom sediments differed across the model domain depending on the location of the surface water features. Within some headwaters areas, low streambed hydraulic conductivity values ($1x10^{-9}$ m/s) were applied to limit the leakage from the Type 1 boundary where the simulated water table is lower than the head assigned to the boundary condition while allowing the representation of gaining or losing stream conditions. Outside wellfield areas, where stream orders were greater than 1, a value of $1x10^{-6}$ m/s was applied. Within wellfield areas, the hydraulic conductivity value applied to represent the streambed conductance varied from $1x10^{-7}$ m/s up to $5x10^{-4}$ m/s where streamflow measurements provided insight on local conditions. The high conductance riverbeds (i.e., conductance of $5x10^{-4}$ m/s) assume a stream length of 25 m (element length), stream width of 2 m, and riverbed thickness of 0.1 m. These values are consistent with the surficial geology units in the area.

Model layers three to five represent the Maple/ Oak Ridges Equivalent. These are overburden aquifer units that consist of silt, sand, gravel, and cobbles. To represent the vertical and horizontal variability of units in the Maple Formation derived from cross-sections and bulk hydraulic conductivity values, the original single layer of Maple Formation was subdivided to create three model layers. There are a total of 88 hydraulic conductivity zones applied in these three model layers and they were developed through model calibration. For details on the methodology applied see Appendix A – Bulk Hydraulic Conductivity. Some of the conductivity zones have the same or similar values but were separated to enable individual adjustment during calibration where necessary.

Model layers six through eight represent the Upper Newmarket Till, Inter-Newmarket Sediments and Lower Newmarket Till or Lower Sediments. In the study area these units are typically mapped as discontinuous units in the bedrock valleys with the tills representing local aquitards and the coarse-grained Inter-Newmarket or Lower sediments representing a local aquifer unit.

The hydraulic conductivity zones in model layer nine represent weathered bedrock units and represent the subcrop of bedrock units in the study area. The distribution of these units and the assigned hydraulic conductivity values are derived from updated bedrock mapping.

Table C3-12: Calibrated Conductivity Estimates

	Calibrated Hydraulic Conductivity		
	Hydrostratigraphic Unit	**Kh Range [m/s]**	**Kh/Kv Range [-]**
Surficial Features	Stream Lake Bed	5.0E-7 - 1.0E-6	1
	Headwaters Stream Bed	1.0E-9	1
	Permeable Stream Bed	2.0E-4	1
	Pond	1.0E-9	1
	Quarry Excavation	3.0E-3	1
	Quarry Base	3.0E-3	1
Overburden	Weathered Halton Till	6.5E-6	1 – 2
	Clay	1.0E-8 - 1.0E-7	10
	Clay Till	5.0E-8 - 4.0E-7	10 – 12
	Silty Clay Till	5.0E-8	5
	Silt Clay	5.0E-8 - 7.0E-7	10
	Silt Sand Clay	3.5E-6	10
	Silty Till	2.5E-6 - 1.0E-5	10 – 12
	Silt Sand	5.0E-7 - 1.0E-3	1 – 11
	Sand	1.0E-6 - 3.0E-4	1 – 20
	Sand Gravel Cobbles	5.0E-4 - 1.0E-2	1 – 20
Bedrock Units	Fractured Dolostone	4.0E-3 - 1.0E-1	1
	Guelph Formation Dolostone	1.0E-5 - 4.5E-4	10
	Weathered Guelph Formation Dolostone	7.0E-5	10
	Vinemount Formation Shale	5.0E-6	10
	Weathered Vinemount Formation Shale	1.0E-5	10
	Gasport Goat Island Formation Dolostone	2.5E-5	10
	Weathered Gasport/Goat Island Formation Dolostone	7.0E-5	10
	Cabot Head Formation Shale	6.5E-7	10
	Weathered Cabot Head Formation Shale	5.0E-6	10
	Manitoulin Formation Dolostone/ Whirlpool Formation Shale	1.0E-5	10
	Weathered Manitoulin Formation Dolostone/ Whirlpool Formation Shale	4.5E-5	10
	Queenston Formation Shale	1.0E-8	10
	Weathered Queenston Formation Shale	1.0E-6 - 7.0E-6	10

Model layers 10 to 15 represent the deeper bedrock units as described in **Table C3-11**. These layers dip to the south-southwest and outcrop/ subcrop in model layer 9 at the Niagara Escarpment. Subcropping model layers must be carried through the entire model domain; where they reach top of bedrock elevation, the model layer thickness is set to equal 0.1 m and the conductivity value of the underlying unit is applied. The calibrated hydraulic conductivity values for the unweathered and weathered bedrock formations, as well as their anisotropy ratios are listed in **Table C3-12**.

Unsaturated Zone properties

The FEFLOW model simulations were completed using a variably-saturated simulation mode to approximate the water table position without representing the details of the unsaturated zone. This was achieved by globally assigning a smoothed pressure-saturation function and a linearized hydraulic conductivity-saturation function to the unsaturated zone. This approach provided numerical stability while representing the specific yield component of aquifer storage. Parameters for the unsaturated flow model are assigned to all model layers, however, hydraulic conductivity is adjusted based on the degree of saturation only where model layers become (partially) unsaturated during the simulation. Where a unit is partially saturated specific yield contributes to the storage response.

C3.8.5 Model Boundary Conditions

Boundary conditions are used in numerical groundwater models to represent various features that add or remove groundwater from the numerical model domain. In Acton and Georgetown, the primary input of water to the groundwater flow system is groundwater recharge derived from precipitation. A secondary input is recharge or leakage from surface water features. Groundwater outputs include discharge to surface water features and extraction via pumping wells. There are three types of model boundary conditions used in the groundwater flow model:

- Constant-head boundary conditions are boundaries where the value of the hydraulic head is assigned to specific nodes within the model, and the amount of flow into or out of the model node fluctuates to satisfy the head condition. Physically, these boundary conditions (constant heads) are used to simulate areas where aquifer potentials are expected to remain at a constant level. Since the water levels (heads) along streams and lakes are usually assumed to be constant in groundwater models, specified head boundary conditions are used to implement those surface water features;
- Specified-flux boundary conditions are boundary conditions for which a flux value is assigned to specific model node. The hydraulic head at the node is allowed to fluctuate to meet that flux condition. This type of boundary condition is commonly used to specify recharge to the groundwater system from precipitation on top of the model. No-flow boundaries are one type of specified-flux boundary where the rate of lateral flow across the boundary is assumed to be negligible or equal to zero. No-flow boundaries are applied to simulate groundwater divides or impermeable geologic units; and
- Well boundary conditions are a type of specified flux boundary. For wells a total discharge is specified for a set of vertical nodes at the same spatial location in the model domain. While nodes with specified fluxes are treated individually, the flux from individual layers will be proportional to the transmissivity of the layer. As the name suggests, this type of boundary condition is used to represent pumping wells in the model.

Boundary conditions applied in this model include groundwater recharge estimated from the calibrated hydrological model, flow into and out of surface water features such as streams, rivers, lakes, discharge to Acton Quarry and municipal and non-municipal pumping wells.

Recharge

The output from the calibrated hydrological model (MIKE SHE) was used as input recharge rates (specified flux) into the top layer of the FEFLOW model. To provide consistency between the two models, data was transferred by intersecting MIKE SHE’s regular grid with FEFLOW’s finite element mesh and assigning area weighted averages to each element to maintain rate and volumes between models. Average annual recharge rates were estimated from the 1991 through 2005 hydrologic model calibration, representing a long-term average. Note that at the time of steady-state calibration, the climate data for 2006 through 2009 were not available. Subsequently the climate data for 2006 to 2009 became available and confirmed consistency with longer-term average of 1991-2005. Precipitation in 1991-2005 was 846 mm/year compared to 2005-2009 at 868 mm/year at the Georgetown climate station.

Rivers, Streams, Ponds, Lakes and Wetlands

The virtual drainage mapping provided by CVC was used as the basis for representing streams in the groundwater model. The virtual drainage was compared against satellite imagery to check that mapped streams were continuous and in the correct location. Rivers and streams with stream order greater than 1, and first-order streams with observed perennial flow were included in the finite element mesh generation to enable placement of nodes along the stream course to closely represent the stream location in the model. Type 1 (fixed-head) boundary conditions were assigned at nodes along the streams with stream stage estimated from the 5 m DEM, and hydraulically corrected to ensure stream elevations are monotonically decreasing in the downstream direction.

Acri Pond, located near the Davidson Well(s), and Fairy Lake located next to the Prospect Park Well in Acton were represented using Type 1 (fixed-head) boundary conditions. The fixed head values were based on pond/ lake elevations. The sediments at the base of lakes and ponds were represented with a separate hydraulic conductivity zone to be able to represent interaction between surface-water and groundwater. A conductivity value representing a silt-clay unit of $1x10^{-6}$ m/s (assumed thickness of 0.1 m) was used for these ponds (see discussion of stream bed hydraulic conductivity **Section C3.8.4**). All other ponds, lakes, and wetlands were represented in the recharge distribution derived from the calibrated hydrological model and were not limited by specified heads or flux boundary conditions. Where the hydrologic model represented a wetland, lake or pond as recharging the groundwater flow system, this value was applied in the groundwater model as recharge. Where groundwater was simulated to discharge to the surface water feature in the hydrologic model, a value of zero recharge was applied in the groundwater flow model. This approach may underestimate total discharge to ponds or wetlands disconnected from the stream network.

Perimeter Boundaries

The model boundary was assumed to follow natural groundwater divides with no regional groundwater flow entering or leaving through the model perimeter. As such, no boundary conditions were specified along the model perimeter, which in FEFLOW are no-flow boundaries by default.

Pumping Wells

All municipal pumping wellfields for the communities of Georgetown and Acton were represented in the groundwater flow model. These included the Prospect Park, Davidson, and Fourth Line wellfields servicing Acton, and the Lindsay Court, Princess Anne and Cedarvale wellfields servicing Georgetown. Well boundary conditions (Type 4) were specified according to their reported well screen depths. The pumping rates simulated in the model varied depending on the model simulation (e.g., long-term steady-state, transient pumping test simulation, recharge pulse). **Table C3-13** lists all pumping wells represented in the model, including their locations, screen depths, and 2005-2009 average pumping rates.

Table C3-13: Municipal Pumping Well Simulation Detail

Municipal Well		Aquifer	NAD 83, Zone 17		Elevation (mASL)		Model Layer Screen	2005-2009 Average Pumping Rates [m³/day]	Static Water Level** [mASL]
			Easting	Northing	Screen/ Open Hole Top	Screen/ Open Hole Bottom			
Georgetown	Cedarvale 1a	Overburden	587068	4833224	213.7	210.7	5 – 6	937	228.0
Georgetown	Cedarvale 3a	Overburden	587453	4832861	211.4	206.8	5 – 8	931	227.5
Georgetown	Cedarvale 4	Overburden	587444	4832968	216.2	208.7	5 – 9	0	228.5
Georgetown	Cedarvale 4a	Overburden	587459	4832965	216.0	209.8	5 – 8	459	230.0
Georgetown	Lindsay Court 9	Overburden	584832	4833369	250.8	244.7	8 – 9	5,083	263.5
Georgetown	Princess Anne 5	Overburden	586154	4833170	235.6	224.3	5 – 7	2,671	251.0
Georgetown	Princess Anne 6	Overburden	586147	4833162	238.6	229.2	7 – 10	2,794	251.0
Acton	Fourth Line A	Bedrock	577022	4835284	367.9	356.1	9 – 11	808	369.5
Acton	Davidson Well 1*	Bedrock	576873	4833288	365.9	355.6	9 – 11	1,024	367.0
Acton	Prospect Park Well 2*	Overburden	576814	4830878	329.1	323.0	8	1,331	343.5
							Total	**16,038**	

Transient Model Setup

The calibrated steady-state model was used as the starting point to develop two transient simulations as follows:

- Transient Calibration - 2008 through 2010: This simulation consisted of daily time-steps to evaluate the ability of the model to represent observed changes in groundwater levels during the Prospect Park long-term pumping tests, the Cedarvale shutdown, and normal operations of the other municipal wells; and
- Long-Term Verification Simulation - 1995 through 2005: This simulation consisted of monthly time-steps to evaluate the ability of the model to represent the observed recharge-pulse, whereby seasonal water level responses in the Princess Anne wellfield lag behind the responses in the Lindsay Court wellfield located up-gradient.

The steady-state model simulation results, representing average groundwater conditions in 2005 through 2009, were used as the initial conditions for both transient simulations. The pumping well and recharge boundary conditions were updated to reflect the daily values for the transient calibration and the monthly values for the long-term verification simulation. The water levels are considered representative of conditions after the first year of each simulation as the influence of the differences between the initial condition and the historical pumping are minimized. Through the transient calibration process, minor modifications in hydraulic conductivity were made to better represent transient well responses as documented.

C3.9 Groundwater Flow Model Calibration

Numerical groundwater flow models are calibrated by systematically adjusting the model input parameters and boundary conditions to determine the optimum match (within an acceptable margin of error) between the simulated results and field observations. The model's ability to represent observed conditions is assessed qualitatively to assess trends in water levels and distribution of groundwater discharge and quantitatively to achieve acceptable statistical measures of calibration.

The groundwater model calibration process included calibration to steady-state conditions to represent existing conditions (2005 to 2009), as well as a transient calibration to Prospect Park Pumping Long-Term Tests in Acton, and the Cedarvale wellfield shutdown (2009 to 2010) in Georgetown. Model verification was established through a long-term, transient groundwater simulation that used monthly recharge and pumping rates to evaluate the model's ability to represent seasonal conditions between 1995 and 2005.

Calibrated recharge estimates developed for the steady-state simulations were iteratively calibrated in both the hydrologic and groundwater flow models. Additional iterations were completed using transient data for 1995-2005 and 1995-2010 to adjust storage coefficients. The hydrologic model used the groundwater model layers, hydraulic conductivity and storage parameters (transient only) as direct input to represent the saturated zone. The iterative process was as follows:

- An initial hydrologic model (MIKE SHE) was set-up and calibrated to streamflow data. The results were used as preliminary recharge input to FEFLOW;
- The FEFLOW model was calibrated to heads and baseflows. Adjusted conductivity (and storage) values were populated back into the MIKE SHE model; and
- MIKE SHE model was recalibrated using the updated hydraulic conductivity (and storage) values. New recharge estimates were passed on to FEFLOW.

This process was repeated several times until the simulated groundwater heads and baseflow estimates in both models were reasonably close. The consistency of the model parameterization allowed the recharge output from the MIKE SHE model to be used directly in the FEFLOW model, requiring only spatial averaging to apply the MIKE SHE gridded distribution to the FEFLOW mesh.

The calibrated model results are non-unique; in that there are several combinations of boundary conditions and model input parameters that may result in a similar fit between observed and simulated values. Other combinations may simulate different conditions in areas between observations, where the model is being used to understand existing or future conditions (e.g., groundwater recharge or discharge conditions along a stream).

To reduce the number of potential parameter combinations and distributions, the model was calibrated to both steady-state and transient conditions for multiple time periods. The simulation of different stresses at different locations in the groundwater system provided further information on plausible parameters. In addition, the simulation and evaluation of the groundwater flow system in the groundwater and hydrologic model further reduced the range of potential parameter values.

Overall, the iterative-multiple simulation calibration approach, incorporating the field data collected as part of the Tier 3 assessment, decreased the uncertainty in the model input parameters compared to previous studies, which had few observation data and only considered steady-state conditions (EarthFX, 2009 and AquaResource Inc., 2009).

Details on both the steady-state and transient groundwater flow calibration can be found in the foundation report.

C3.9.1 Calibration Targets – Long Term Model Verification

In general, a model is considered to be well calibrated qualitatively if there is a good fit between the observed head contours and the model predicted contours, and it may also be considered well calibrated from a quantitative perspective if the model predicted heads and groundwater discharge estimates fall within the range of reported values. The general philosophy followed during this calibration exercise was to achieve calibration results 'as good as possible' using reasonable parameter estimates. Local adjustments to hydraulic conductivity zones were not made without having reliable geology data to support modifications to the conceptual model.

The groundwater flow model is considered to be well calibrated for the following reasons:

- Aquifer and aquitard property estimates are within expected ranges of values and similar to those used in the CVC watershed-scale groundwater flow model (AquaResource, 2006) and the Cedarvale Model (EarthFX, 2009) and the Guelph Tier 3 Draft Model Results (unpublished);
- The hydraulic conductivity and recharge estimates produced simulated hydraulic heads in the higher quality monitors that are largely within the range of measured head values;
- Hydraulic heads collected in a few monitoring well nests or clusters that are screened across multiple aquifer units allowed for examination of vertical head differences between aquifers. The model calculated hydraulic head difference between the aquifer units was comparable to the observed hydraulic head differences except where the shallow unit represented a perched sand lens within a less permeable extensive unit;
- Statistical measures of water level calibration accuracy, including mean error, root mean squared error, and normalized root mean squared error are within the acceptable range of typical state of practice 'rules of thumb' (Anderson and Woessner, 1992); and
- Simulated groundwater discharge is consistently within the range and distribution of observed values, indicating that the overall groundwater recharge rate is appropriate.

C3.10 Tier 3 Water Budget Modelling Results

The combined results of the two water budget models produce an improved understanding of the hydrologic and hydrogeologic flow systems. The following sections quantify and outline the water budget components within subwatersheds 10 and 11 of the Credit River watershed. Each of the components presented were calculated assuming no net change in stored water occurs over the time period 2005 to 2009 and were based on the limitations and assumptions of the long-term climate dataset.

C3.10.1 Groundwater Recharge

Figure C3-16 shows groundwater recharge simulated in the calibrated groundwater flow model. The recharge values range from zero in areas of net groundwater discharge (e.g., some wetlands) to greater than 500 mm/yr. in areas of depression storage where sand and gravel lie at ground surface and receive enhanced recharge due to runoff from adjacent areas.

C3.10.2 Water Table Contours

Figure C3-17 illustrates the simulated water level contours produced in the steady-state groundwater flow for the Maple Formation, the primary overburden aquifer. Water level contours generally mimic surface topography (and bedrock topography) and flow converges towards the buried bedrock valleys and higher order streams and wetlands. The groundwater elevation contours generally compare well with the observed water level contours. Water level contours show inflections around most surface water features suggesting an influence of the streams on groundwater flow in the Maple Formation. The contours on Black Creek at 20th Sideroad (Beeney Creek) suggest losing stream conditions, which is consistent with observed conditions. Minor inflection of the water level contours is observed along Silver Creek in Georgetown, which is consistent with the observed neutral gaining/losing conditions. Upstream of the Cedarvale wellfield, water level contours show little to no influence from surface water. This suggests at depth a portion of groundwater flow in the bedrock bypasses Silver Creek at this location and flows towards the Main Credit River.

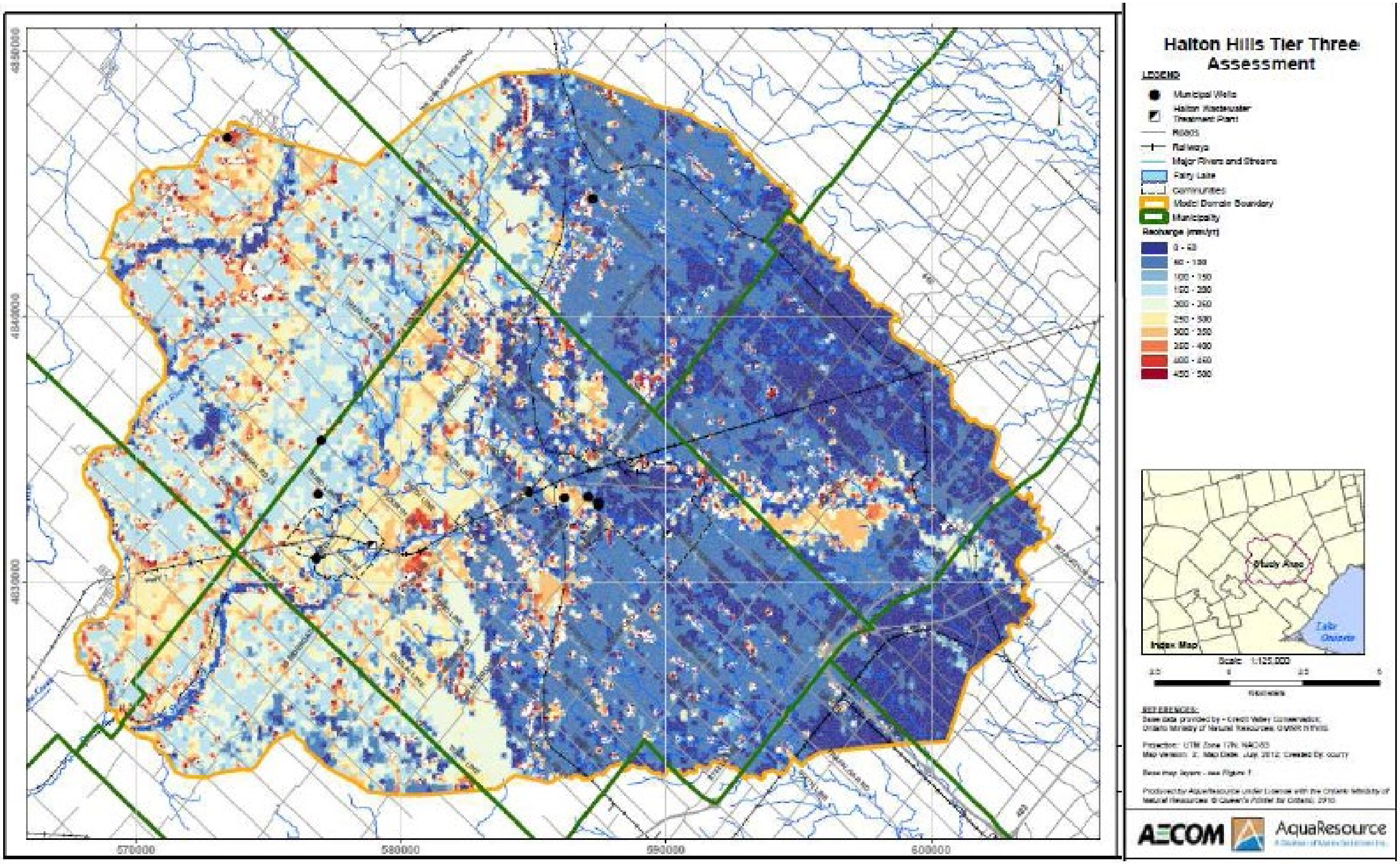

Figure C3-16: Groundwater Recharge

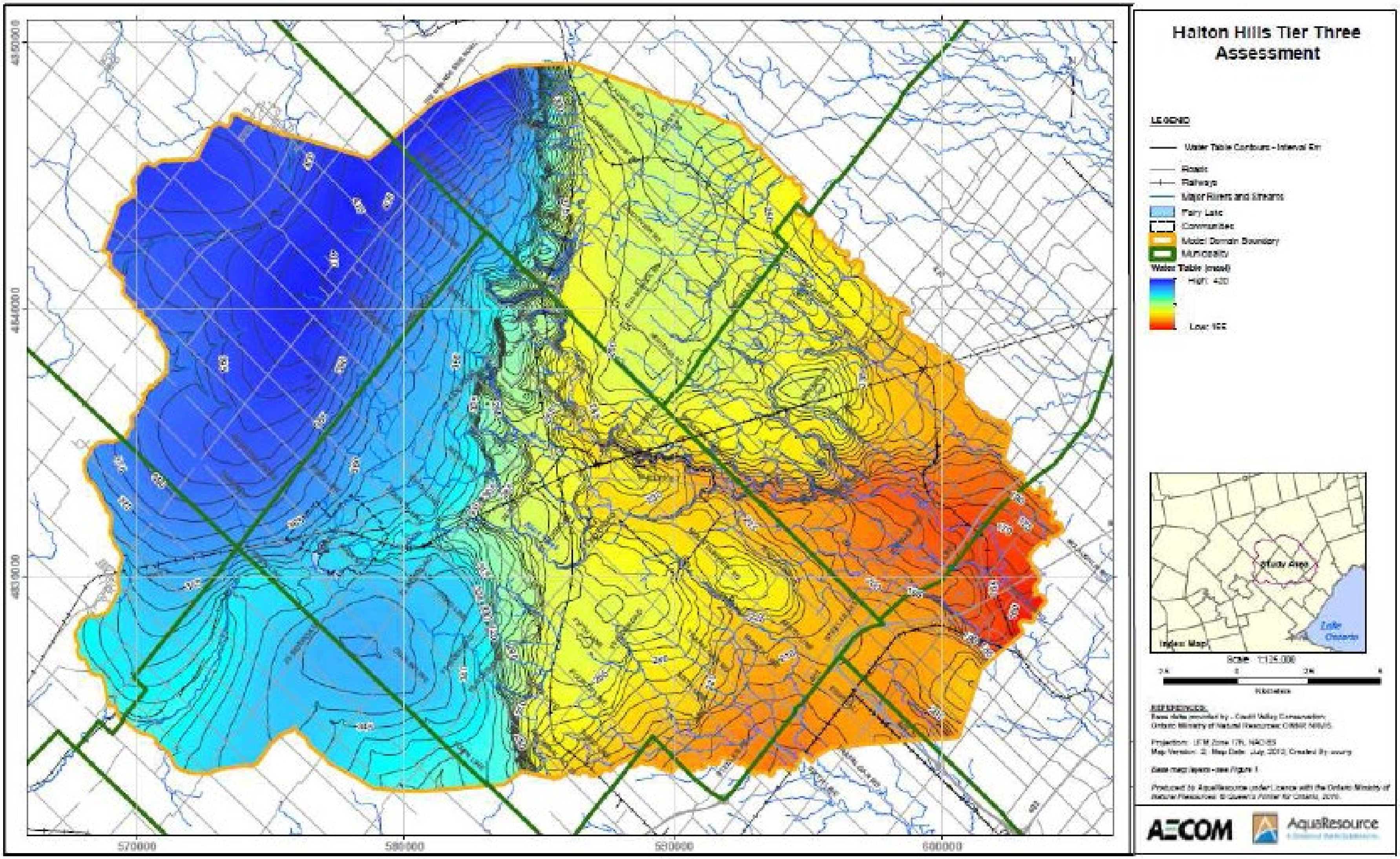

Figure C3-17: Simulated Overburden Aquifer Water Levels

The largest gradients (closely spaced contours) are observed along the escarpment reflecting the topographic influence. In addition, flow from the escarpment is directed toward the buried valley area between Limehouse and Cedarvale, indicating potential inflow to the valley at these locations.

C3.10.3 Bedrock Water Level Contours

Figure C3-18 illustrates the simulated bedrock potentiometric surface contours, which consists of predominately Gasport dolostone west of the escarpment and Queenston shale east of the escarpment. The potentiometric surface contours are similar to the overburden water levels; however, the bedrock potentiometric surface exhibits a more subdued expression and is more heavily influenced by the bedrock topography. The bedrock potentiometric surface contours converge on buried bedrock valleys in Acton, Georgetown and along Blue Springs Creek and the Main Credit River. One dominating flow feature is the southwest flow from the topographic high of the escarpment toward Blue Springs Creek. The closely spaced contours at the escarpment highlight the convergence of flow on the steep slopes and the potential for seepage. The influence of surface water features is evident in the bedrock potentiometric surface, particularly in areas of thin overburden along the escarpment. The Acton Quarry, which is located southeast of Acton on 22nd Sideroad, extracts dolostone. Potentiometric surface contours are shown to converge on this feature and lower local groundwater levels in part due to the simulation of dewatering at the Quarry during the calibration period.

C3.10.4 Vertical Hydraulic Head Difference

Figure C3-19 illustrates the simulated vertical hydraulic head difference across the model domain, calculated as the head difference between the water levels in model layer five (Maple Formation) and the water levels in model layer 12 (Gasport/Queenston Formation). The map is shaded to show where groundwater heads are directed upwards (green) or downwards (blue). Areas where the overburden water level is lower than the bedrock potentiometric surface are predicted along the Credit River and its tributaries and some wetland complexes; a reflection of groundwater discharge to those areas. Upward gradients are also observed at Limehouse near Fifth Line between Acton and Georgetown where the buried valley narrows and shallows. Many of the areas in the map show neutral gradients. In these areas, layers 5 and 12 are at similar elevations and therefore do not show any vertical head difference.

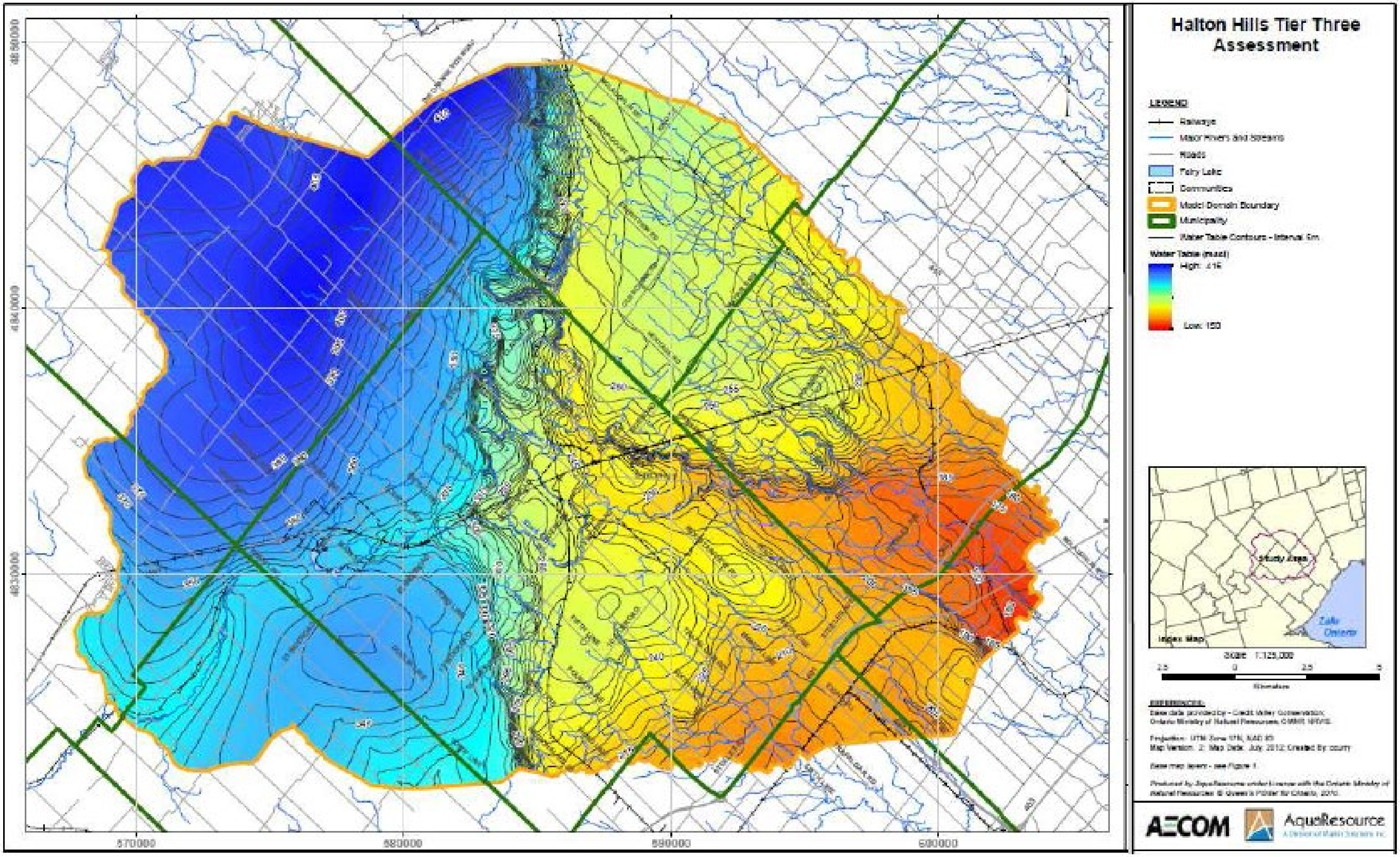

Figure C3-18: Simulated Bedrock Potentiometric Surface Contour

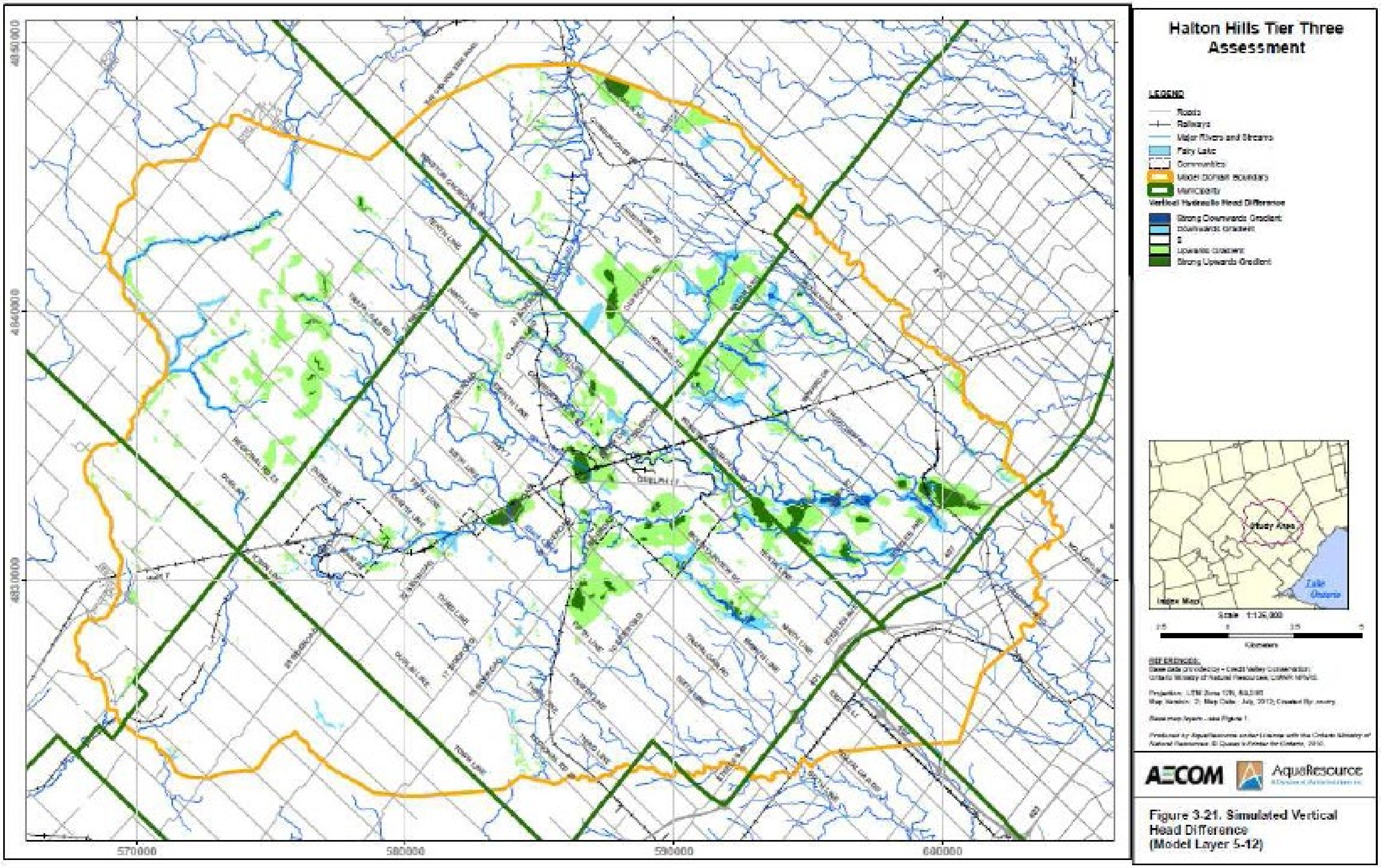

Figure C3-19: Vertical Hydraulic Head Difference

C3.11 Subwatersheds 10 and 11 Water Budget

As part of the water budget process, estimates of the water budget component fluxes were used to better understand the processes contributing to the water budget in the area.

Table C3-14 summarizes the estimated cross boundary flow between Black Creek and Silver Creek Subwatersheds (subwatersheds 10 and 11). Cross-boundary groundwater flow into Subwatershed 10 is significant along the southeast and west boundaries. These flows are interpreted to be the natural flow directions in the west hydraulic gradients are enhanced by Acton municipal pumping. Cross-boundary groundwater flow out of Subwatershed 10 is interpreted to be enhanced due to pumping within Georgetown. Cross-boundary flows into Subwatershed 11 are significant along the Subwatershed 10 and southeast boundaries. Groundwater flows out of Subwatershed 11 in the northeast to the main Credit River subwatershed as flow converges on the Niagara Escarpment.

Table C3-14: Summary of Cross Boundary Water Flow

Subwatershed 10 Boundary	Cross Boundary Flow (m^3/d)
From West boundary into Subwatershed 10	+3,700
Subwatershed 10 to Southwest boundary	-3,900
From Southwest boundary into Subwatershed 10	+5,100
Subwatershed 10 into Subwatershed 11	-15,400
Net Cross Boundary Groundwater Flow	**-10,500**
Subwatershed 11 Boundary	**Cross Boundary Flow (m^3/d)**
Subwatershed 11 to Northwest boundary	-1,000
Subwatershed 10 into Subwatershed 11	+15,400
From Southeast boundary into Subwatershed 11	+3,500
Subwatershed 11 into Northeast Boundary	-3,500
Net Cross Boundary Groundwater Flow	**+14,300**

Table C3-15 summarizes the estimated overall water budget fluxes for subwatersheds 10 and 11. The table summarizes watershed inflows including precipitation and groundwater interbasin flow. Outflows include evapotranspiration, streamflow, groundwater pumping, and groundwater interbasin flow. The water budget parameters are calculated based on information derived from both the surface water and groundwater flow models and are presented in units of m^3/d and mm/year. In addition to the bedrock potentiometric surface, **Figure C3-18** illustrates the estimated cross-boundary groundwater flow between the subwatersheds 10 and 11 and adjacent subwatersheds. The water budget components in **Table C3-15** are described in the following discussion.

The average annual precipitation in Subwatershed 10 is approximately 881 mm/year and 855 mm/year in Subwatershed 11. Groundwater modelling results indicate that 7% of the total inflow into Subwatershed 10 is from groundwater flow from adjacent subwatersheds. The groundwater inflow from adjacent subwatersheds to Subwatershed 11 is 13% of the total inflow and 11% (106 mm/year) of the cross-boundary inflow comes from Subwatershed 10. Cross-boundary flow is interpreted to occur under the non-pumping conditions but is enhanced by municipal pumping.

Table C3-15: Overall Water Balance for Black Creek and Silver Creek Subwatersheds

Subwatershed 10			
Inflows	**Flow (m^3/d)**	**Flow (mm/yr)**	**Percent of Total Inflow**
Precipitation	129,100	881	94%
Net Groundwater Flow in			
From west boundary into Subwatershed 10	3,700	25	3%
From Southeast boundary into Subwatershed 10	5,100	35	4%
Total Inflow	**137,900**	**941**	**100%**
Outflows	**Flow (m^3/d)**	**Flow (mm/yr)**	**Percent of Total Inflow**
Evapotranspiration	-77,100	-526	-56%
Streamflow	-33,200	-227	-24%
Pumping	-8,300	-57	-6%
Net Groundwater Flow out			
Subwatershed 10 to southwest boundary	-3900	-27	-3%
Subwatershed 10 into Subwatershed 11	-15400	-105	-11%
Total Outflow	**-137,900**	**-941**	**-100%**

Subwatershed 11			
Inflows	**Flow (m^3/d)**	**Flow (mm/yr)**	**Percent of Total Inflow**
Precipitation	125,300	855	87%
Net Groundwater Flow in			
From west boundary into Subwatershed 10	15,400	105	11%
From Southeast boundary into Subwatershed 10	3,500	24	2%
Total Inflow	**144,200**	**984**	**100%**
Outflows	**Flow (m^3/d)**	**Flow (mm/yr)**	**Percent of Total Inflow**
Evapotranspiration	-84,200	-580	-58%
Streamflow	-45,000	-311	-31%
Pumping	-10,400	-57	-7%
Net Groundwater Flow out			
Subwatershed 10 to southwest boundary	-1,000	-8	-1%
Subwatershed 10 into Subwatershed 11	-3,600	-25	-2%
Total Outflow	**-144,200**	**-984**	**-100%**

Outflows include evapotranspiration, streamflow (e.g., overland flow, interflow, and groundwater discharge), groundwater pumping, and groundwater flow out of the subwatersheds. Average annual evapotranspiration is approximately 526 mm/year in Subwatershed 10 and 580 mm/year in Subwatershed 11. Average annual streamflow is 227 mm/year from all streams across the Subwatershed 10 and 311 mm/year across Subwatershed 11.

Table C3-16 summarizes the water balance for groundwater within the subwatersheds. The water budget models predict an average annual groundwater recharge rate of 376 mm/year, or 55,100 m^3/d into the subwatershed.

Table C3-16: Groundwater Balance for Black Creek and Silver Creek Subwatersheds

Subwatershed 10			
Inflows	**Flow (m^3/d)**	**Flow (mm/yr)**	**Percent of Total Inflow**
Groundwater Recharge	55,100	375	80%
Cross Boundary Flows	13,700	93	20%
Total Groundwater Inflow	**68,900**	**469**	**100%**
Outflows	**Flow (m^3/d)**	**Flow (mm/yr)**	**Percent of Total Inflow**
Groundwater discharge	-33,800	-231	-49%
Permitted Wells	-8,300	-57	-12%
Acton Quarry Pumping	-2,500	-17	-4%
Cross Boundary Flows	-24,200	-165	-35%
Total Outflow	**-68,800**	**-469**	**-100%**

Subwatershed 11			
Inflows	**Flow (m3/d)**	**Flow (mm/yr)**	**Percent of Total Inflow**
Groundwater Recharge	27,900	190	80%
Cross Boundary Flows	22,400	153	20%
Total Groundwater Inflow	**50,300**	**343**	**100%**
Outflows	**Flow (m3/d)**	**Flow (mm/yr)**	**Percent of Total Inflow**
Groundwater discharge	-31,800	-217	-63%
Permitted Wells	-10,400	-71	-21%
Cross Boundary Flows	-8,100	-55	-16%
Total Outflow	**-50,300**	**-343**	**-100%**

Groundwater outflows include discharge to surface water (streams and wetlands), groundwater wells, and groundwater flow out of the subwatersheds. Total groundwater discharge to surface water in Subwatershed 10 is approximately 33,800 m^3/d or 231 mm/year and is 31,800 m^3/d or 217 mm/year in Subwatershed 11. Subwatershed 10 groundwater pumping is 8,300 m^3/d, or approximately 12% of the total groundwater inflow (recharge plus cross-boundary flows). Subwatershed 11 groundwater pumping is 10,400 m^3/d, or approximately 21% of the total groundwater inflow into the subwatershed. These values are within 10% of those estimated using the Tier 2 (watershed-scale) FEFLOW model; however, discharge to streams is better represented within the Tier 3 model based on the calibration to continuous gauges and additional spot streamflow measurements. The differences in water budget parameters between the two models are attributed to the conceptual and numerical model updates made in the MIKE SHE and groundwater model and the refined local-scale calibration. The Tier 3 assessment water balance estimates should be considered more reliable than those from the Tier 2 assessment due to the refined conceptual and numerical models.

C3.12 Water Demand

C3.12.1 Municipal Water Demand

The municipalities of Acton and Georgetown have municipal water supplies within the study area. Each relies entirely on groundwater for their municipal drinking water needs.

Efforts to collect and confirm water demand estimates include:

- Review of the MOECC's Water Taking Reporting System (WTRS) to incorporate actual pumping rates for permit holders;
- Review of monitoring reports and discussions with permit holders to ensure that site conditions and operating practices are incorporated into the consumptive demand estimate, if possible; and
- Site visits if warranted to better estimate consumptive water use.

Acton

Drinking water for Acton is sourced from five active municipal wells, which are distributed among three wellfields. As listed in **Table C3-17**, there is one bedrock well at the Fourth Line wellfield (Fourth Line Well A), two bedrock wells at the Davidson wellfield (Davidson 1 and Davidson 2), and two overburden wells at the Prospect Park wellfield (Prospect Park 1 and Prospect Park 2). Extraction at each well is limited by the permitted maximum daily taking at the wells and wellfield as outlined in the table. Taking varies seasonally at Prospect Park to minimize impacts on Fairy Lake water levels and outflow to Black Creek. Permitted maximums are stipulated in the Permit to Take Water (PTTW) issued by the MOECC for operation of the wells. The Fourth Line and Davidson wells extract groundwater from Silurian dolostone units within the 20 m of ground surface (i.e., the Gasport and / or Guelph Formations). At the Prospect Park wells, groundwater is sourced from coarse grained deposits of the buried bedrock valley aquifer (BBVA) in the Acton-Georgetown bedrock valley system west of Limehouse. Municipal supply well locations are shown on **Figure C3-20**.

Table C3-17: Acton Water Supply Wells and Permitted Capacity

Permitted Taking	Pumping Well				
	Prospect Park 1	**Prospect Park 2**	**Davidson 1**	**Davidson 2**	**Fourth Line A**
Aquifer Type	Overburden	Overburden	Bedrock	Bedrock	Bedrock
Pumping Well Maximum Daily Taking (m^3/d)	2,273	2,273	1,250	1,250	1,309
Wellfield Max Daily taking (m3/d)	2,273 (June1-September 30) 1,137 (October 1-May 31)		-		-
Additional Restrictions	Wells 1 and 2 cannot be operated simultaneously		Sufficient flow to be maintained in stream on adjacent property to provide flow for the rearing of trout. Not required to maintain stream flow in excess of 304.5 L/min from May 1 – October 31 and 227 L/min from November 1 to April 30.		-
Permitted Emergency taking	Emergency excess taking is permitted as follows: • 3,456 m^3/day for up to 20 non-consecutive days (per year). • 4,546 m^3/day for up to 5 consecutive days (per year). • The above values represent combined taking from Wells 1 & 2. Wells 1 & 2 can be operated simultaneously during an emergency.		-		-

Georgetown

Drinking water for Georgetown is sourced from seven active municipal wells, which are distributed among three wellfields. All of the Georgetown wells are screened in overburden sediments of the BBVA in either the Acton-Georgetown or Georgetown-Mississauga bedrock valleys. As listed in **Table C3-18** and illustrated on **Figure C3-20**, there is one well at the Lindsay Court wellfield (Lindsay Court Well 9), two wells at the Princess Anne wellfield (Princess Anne 5 and Princess Anne 6), and four wells at the Cedarvale wellfield (Cedarvale 1A, Cedarvale 3A, Cedarvale 4 and Cedarvale 4A). Extraction at each well is limited by permitted daily and annual maximums, as outlined in the table.

Table C3-18: Georgetown Water Supply Wells and Permitted Capacity

<table>
<tr><th rowspan="2">Permitted Taking</th><th colspan="7">Pumping Well</th></tr>
<tr><th>Lindsay Court 9</th><th>Princess Anne 5</th><th>Princess Anne 6</th><th>Cedarvale 1A</th><th>Cedarvale 3A</th><th>Cedarvale 4</th><th>Cedarvale 4A</th></tr>
<tr><td>Aquifer Type</td><td>Overburden</td><td>Overburden</td><td>Overburden</td><td>Overburden</td><td>Overburden</td><td>Overburden</td><td>Overburden</td></tr>
<tr><td>Pumping Well Maximum Daily Taking (m^3/d)</td><td>6,545</td><td>4,582</td><td>13,091</td><td>2,618</td><td>3,931</td><td>7,855</td><td>5,891</td></tr>
<tr><td>Wellfield Max Annual Avg Daily taking (m^3/d)</td><td>-</td><td colspan="2">6,800</td><td colspan="4">6,800</td></tr>
<tr><td>Wellfield Max Daily taking (m^3/d)</td><td>-</td><td colspan="2">-</td><td colspan="4">14,404</td></tr>
<tr><td>Additional Restrictions</td><td>-</td><td colspan="2">Wells 5 and 6 cannot be operated simultaneously</td><td colspan="4">Wells 4 and 4A combined cannot be pumped in excess of 7,855 m^3/d</td></tr>
</table>

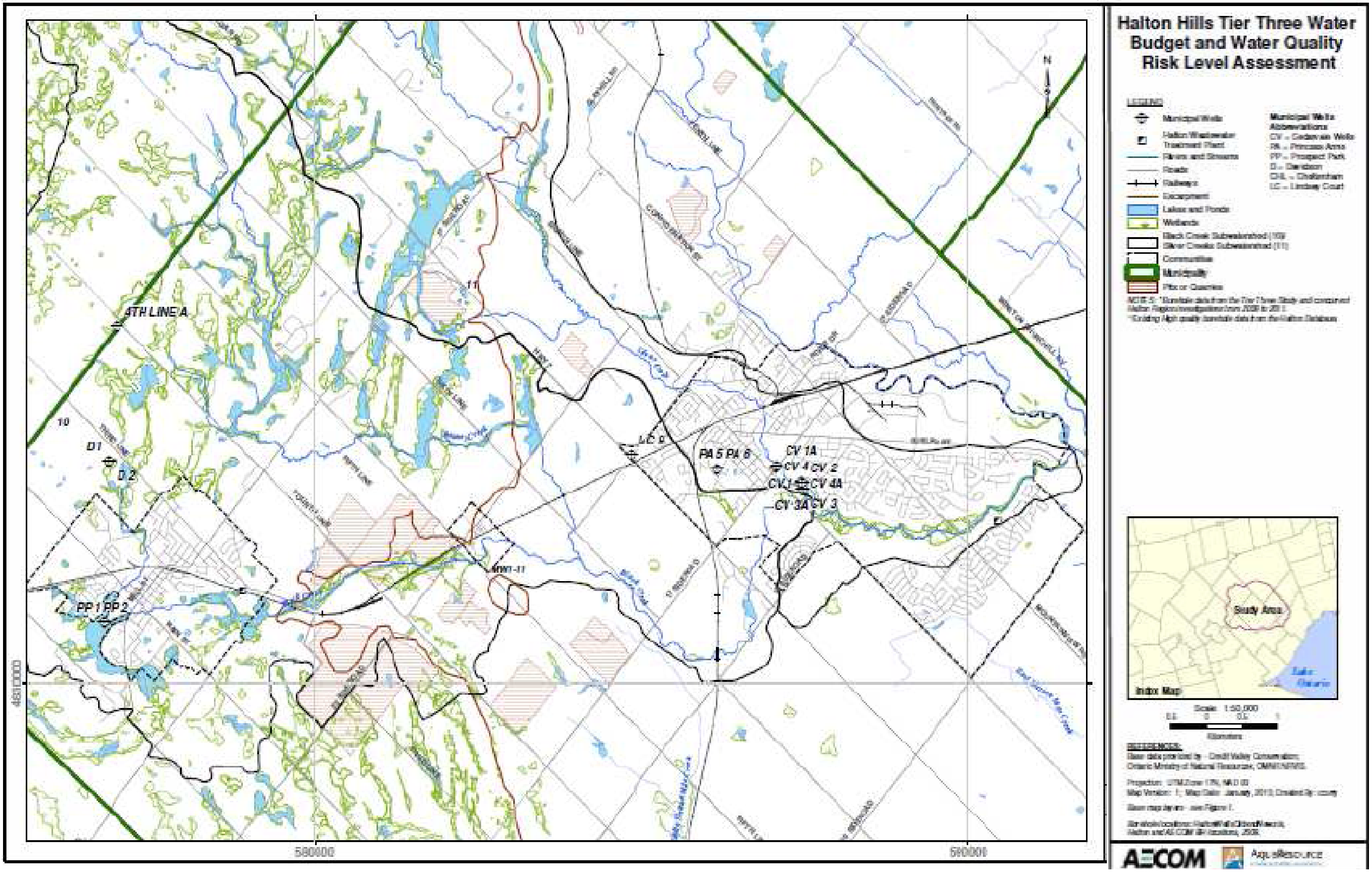

Figure C3-20: Municipal Systems serving Acton and Georgetown

C3.12.2 Safe Additional Drawdown

Safe additional drawdown is defined 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 existing conditions pumping rate. To establish the safe additional drawdown for each municipal well, the following components need to be evaluated or calculated for each well:

1. Safe Water Level Elevations - The lowermost elevation that an Operator can pump the water levels in a municipal pumping well. This elevation may be related to the well screen elevation, pump intake elevation or similar operational limitations.

2. Existing Water Level Elevations in the Pumping Wells - The elevation of the observed average annual pumped water level within each municipal well for the 2005 to 2011 time period during periods of normal operation.

3. Estimated Non-linear Well Losses at Each Well - Drawdown within the well in response to well inefficiencies (e.g., entrance losses, turbulent flow around pump fittings) created during groundwater extraction.

The safe water level elevation at each municipal water supply well was supplied by Halton Region staff. The safe water levels or set points for operation are based on historical and recent testing and operation of the wells. Halton Region has demonstrated that a feasible safe water level for all overburden wells is the top of the well screen plus 1 m, and for bedrock wells the safe water level is at or above the base of the deepest producing fracture zone. This information provides the basis for assigning the individual safe water levels regardless of the present pump intake location in the well. Halton Region has indicated that it is feasible that they could lower the pumps as needed to accommodate the specified safe water levels.

The average pumped water level represents the average water level at the well when it is pumped at rates consistent with normal operational patterns water level data measured during uncharacteristically high or low production, as would occur during aquifer testing or well maintenance, was not used to calculate the average pumped water level.

The safe additional in-well drawdown is a measure of the additional drawdown within a well, regardless of the non-linear head losses at each well that are due to turbulent flow of water through the well screen and casing to the pump intake. The safe additional in-well drawdown is calculated as the difference between the average pumped water level and the safe water level. **Table C3-19** lists the safe water level elevations for the municipal wells within the study area, while **Figure C3-21** shows as an example, the derivation of safe additional drawdown at the Fourth Line well in Acton.

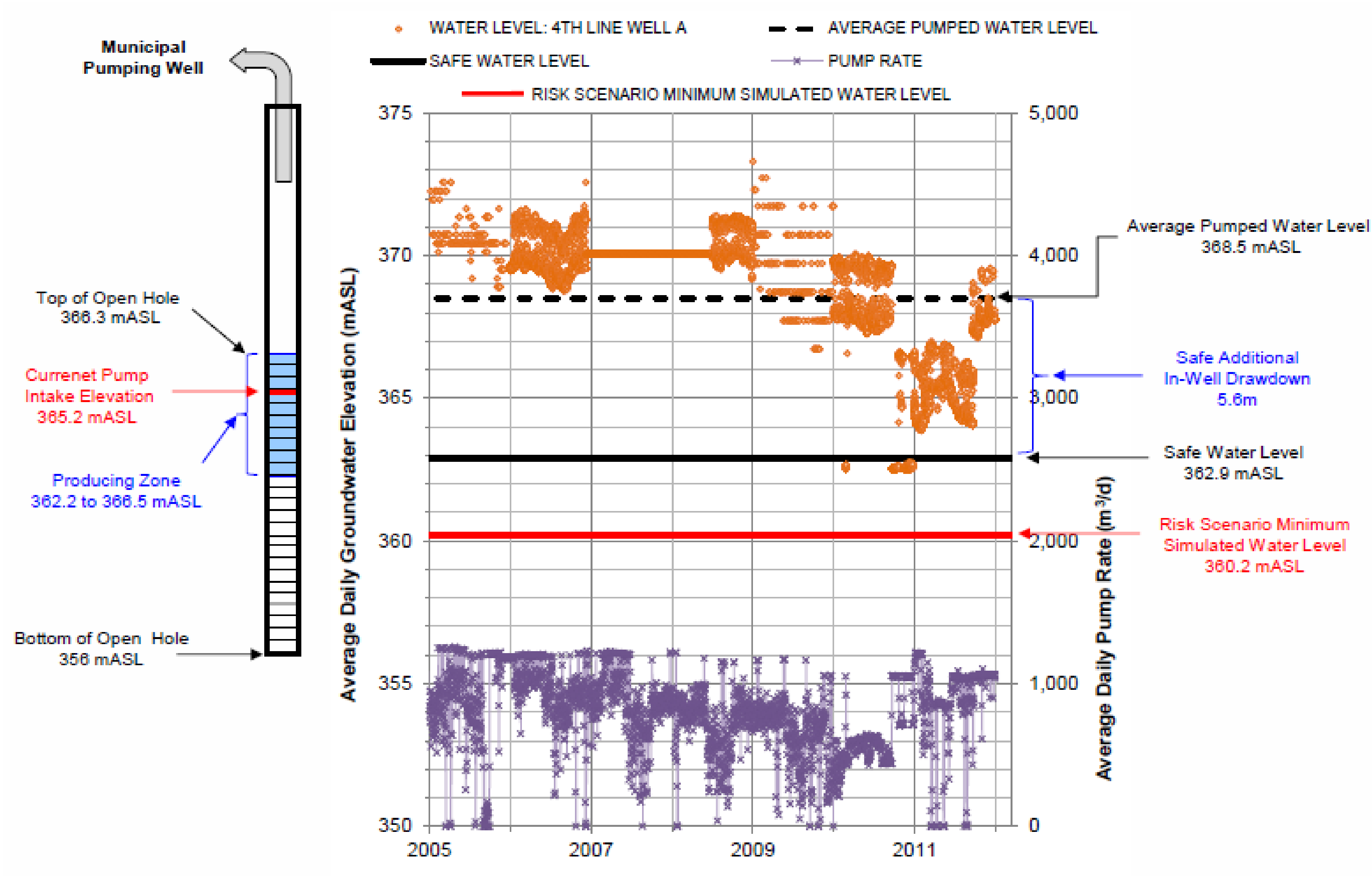

Figure C3-21: Safe Additional Drawdown Calculation – Fourth Line Well (Acton)

Table C3-19: Safe Additional In-well Drawdown

Well	Safe Water Level [1] (mASL)	Average Pumped Water Level for 2005-2011	Safe Additional In-Well Drawdown[1] (m)
Acton			
Fourth Line Well A	362.9	368.5	5.6
Davidson Well 1	358.2	366.4	8.2
Davidson Well 2	358.2	366.3	8.1
Prospect Park Well 1	328.8	344.4	15.6
Prospect Park Well 2	330.1	342.5	12.4
Georgetown			
Lindsay Court Well 9	251.8	261.9	10.1
Princess Anne Well 5	236.6	251.3	14.7
Princess Anne Well 6	239.6	252.6	13.0
Cedarvale Well 1A	214.7	226.5	11.8
Cedarvale Well 3A	212.4	222.5	10.1
Cedarvale Well 4	217.2	226.0	8.8
Cedarvale Well 4A	217.0	221.5	4.5

Notes: 1 - defined by Halton Region
2 - as observed from water level measured during periods of normal well operation and production
3 - relative to average observed water level for the 2005 -2011 period

Convergent Head Losses

Convergent head losses derived from differences in simulating an average water level at a finite element node and the pumping well are negligible due to the small node spacing around the wells in the model and therefore of less importance than the in-well losses. They are therefore not considered in the analysis.

Non-Linear In-Well Losses

Non-linear head losses refer to the difference between the theoretical drawdown in a well and the observed drawdown and are due to factors such as turbulence in the well itself as water flows into the pump.

These well losses need to be considered in the Tier 3 assessment, as the additional available drawdown refers specifically to the water level in the well and not the average water level in the aquifer in the vicinity of the well. The in-well losses are calculated as the additional drawdown that is expected within the pumping well due to the incremental increase from the existing to the allocated rates.

An assessment of the non-linear head losses is essential in estimating the additional drawdown in the aquifer due to an increase in pumping rate at the municipal wells. These head losses increase with pumping rate and cause greater drawdown than expected at the wellhead and surrounding aquifer given aquifer hydraulic characteristics.

The two components of observed drawdown in a given pumping well are described in the following equation (Jacob, 1947; Hantush, 1964; Bierschenk, 1963):

$$\Delta s_{in\text{-}well} = BQ + CQ^2$$

Where: S is drawdown
Q is the pumping rate
B is the aquifer loss coefficient, which increases with time (Thesis, 1935)
C is the well loss coefficient, which is constant for a given pumping rate

The first term of the equation (BQ) describes the linear component of the drawdown (i.e., doubling the pumping rate leads to a doubling of the drawdown). This term accounts for the head losses in the formation in the vicinity of the well. The second term of the equation (CQ^2) describes the non-linear well-loss component of drawdown (Jacob, 1947) in the well itself. This is the additional component that was quantified in this assessment.

Non-linear in-well losses are estimated using step test results. Step tests are hydraulic tests where a pumping well is pumped at a series of pumping rates and the drawdown throughout the test is recorded. Non-linear well loss coefficients were estimated using the step test results presented in Burnside (2005), and Burnside and Gartner Lee (2004).

Well loss coefficients are calculated by plotting graphs of specific drawdown (drawdown divided by pumping rate) against time for the individual step tests. Plotting the specific drawdown against time (time since the start of the test) will yield observation points that lie along a straight line as drawdown increases with increasing time. The slope of the line fit to the data points is equal to the well loss coefficient (*C*). The intercept of the line fit to the data points on the y-axis is equal to the aquifer loss coefficient (*B*).

The results were plotted using this process to estimate the well loss coefficient, which was then used in the following equation (Jacob, 1947) to calculate the drawdown due to in-well losses for the increased pumping from existing to the allocated rates:

$$\Delta s_{in-well} = C\left[\left(Q_{2008} + \Delta Q\right)^2 - Q_{2008}^2\right]$$

where Q_{2008} is the existing (2008) pumping rate, and ΔQ represents the increase in pumping from existing to the allocated rates. Based on this analysis, the in-well losses were calculated for each well in the study area, and are listed in **Table C3-20**.

Table C3-20: Estimated Drawdown due to Non-Linear Head Losses

Well Name	Well Loss Coefficient (C)	Existing Conditions (2005-2009) versus (2005-2011)		Existing Conditions (2005-2009) versus Existing plus Committed plus planned	
		Pumping Rate Increase	Drawdown due to Non-Linear Head Losses	Pumping Rate Increase	Drawdown due to Non-Linear Head Losses
	m/(m³/day)	m³/day	m	m³/day	m
Acton					
Fourth Line Well A	4.00E-07	-3	0.0	501	368.5
Davidson Well 1	2.55E-07	37	0.0	650	366.4
Davidson Well 2	2.55E-07	19	0.0	826	366.3
Prospect Park Well 1	3.00E-08	73	0.0	93	344.4
Prospect Park Well 2	2.55E-07	73	0.0	93	342.5
Georgetown					
Lindsay Court Well 9	3.00E-09	-104	0.0	1492	0.1
Princess Anne Well 5	2.00E-09	-92	0.0	729	0.1
Princess Anne Well 6	5.00E-10	-205	0.0	606	0.0
Cedarvale Well 1A	7.00E-08	73	0.0	510.5	0.1
Cedarvale Well 3A	6.00E-08	620	0.1	516.5	0.1
Cedarvale Well 4	8.00E-08	1087	0.1	1447.5	0.2
Cedarvale Well 4A	1.00E-07	843	0.1	988.5	0.2

C3.12.3 Existing Demand

Existing municipal demand for the municipalities of Acton and Georgetown is based on the 2005 to 2011 average annual pumping rates at each municipal well. Existing municipal demand rates are shown in **Table C3-21**.

C3.12.4 Population Growth and Committed / Planned Demand

As part of the Sustainable Halton planning process, a water demand assessment was completed by AECOM (2011) to quantify future water supply needs in Halton Hills and identify the potential servicing options required to meet those needs. The assessment was based on population growth targets to 2031, which assume a 75% increase over 2011 population levels and a 27% increase over 2011 employment levels, with most of the growth occurring in Georgetown due to expansion of the urban envelope. Modest growth is projected for Acton where it is associated with infill within the existing urban envelope.

Table C3-21: Municipal Water Demand

	Permitted (m^3/d)				Allocated Municipal Demand(m^3/d)		
	Maximum Daily Taking at Well [1]	**Maximum Annual Average Daily Taking at Wellfield**[1]	**Maximum Daily Taking at Wellfield**[1]	**Municipal Drinking Water Licence WTP Capacity**[2]	**Existing**[3]	**Existing plus Committed plus Planned**[4]	**Comments**
Fourth Line A	1,309	1,309	1,309	n/a	805	1,309	
Davidson Well 1	1,250	2,500	2,500	n/a	1,080	2,500	Two wells represented by one boundary node in model
Davidson Well 2	1,250			n/a			
Prospect Pk Well 1	2,273	1,517*	1,517*	2,270	1,477	1,517*	Two wells represented by one boundary node in model
Prospect Pk Well 2	**2,273**						
Total Acton	**8,355**	**5,326**	**5,326**	**n/a**	**3,362**	**5,326**	
Lindsay Court 9	6,545	6,545		n/a	4,979	6,545	
Princess Anne 5	4,582	6,800	13,021	n/a	2,579	3,400	Max ann avg daily taking divided equally based on historical & planned extraction
Princess Anne 6	13,091			n/a	2,589	3,400	
Cedarvale 1a	2,618				1,064	1,447.5	Max ann avg daily taking divided equally based on historical & planned extraction
Cedarvale 3a	3,931	5,790	14,404	12,960	1,551	1,447.5	
Cedarvale 4	7,855				1,087	1,447.5	
Cedarvale 4a	5,891				1,302	1,447.5	
Total Georgetown	**44,513**	**19,135**	**34,040**	**n/a**	**15,449**	**19,135**	

Notes: Abbreviations: n/a- not applicable, WTP – Water Treatment Plant.

1- Values from PTTW No. 7801-825PBJ for the Georgetown Municipal Water Supply, and PTTW No. 6281-7WFQB3 for the Acton Municipal Water Supply

2 - Refers to limits under Municipal Drinking Water License for Prospect Park and Georgetown Water Treatment Plants. Existing plus committed plus planned allocated rates must not exceed this limit. Applies only to allocated rates at the Prospect Park and Cedarvale wellfields.

3 - Average Annual daily taking for 2005 to 2011.

4 - Representative of maximum annual average daily taking at wellfield per PTTWs.

** - Blended rate given maximum daily taking of 2273 m^3/d for June 1 to September 30; and 1137 m^3/d for October 1 to May 30 of each calendar year*

Population and employment projections were based on Halton Region's 2011 Best Planning Estimates for their preferred growth option. Population projections are shown in **Table C3-22** and were used to develop estimates of future residential water demand, and employment projections were used to develop estimates for future industrial, commercial, and institutional uses. Water demand projections are summarized in **Table C3-23**.

The results of the water demand assessment showed that there was insufficient capacity in the Acton and Georgetown municipal supply systems to meet the average day demands for the 2031 planning horizon. Therefore, the full permitted capacity of all existing wells is required to meet the projected demand; however, a deficit will still exist. Additional strategies to meet demand with groundwater supply include the installation of backup wells at Lindsay Court and Princess Anne 6, and the installation of a new production well in north of Acton (North Acton), and the twinning of Fourth Line Well A. In addition, Halton Region is planning on integrating water takings from Lake Ontario to meet much of the demand associated with planned growth in Georgetown.

New groundwater and surface water intakes that were proposed to help meet future demand as part of the Sustainable Halton process are still being evaluated within the Environmental Assessment (EA) process. Planned demand, as defined above, includes only those demands that have been approved through the EA process. As none of the proposed wells or intakes has gone through the EA process, they are not considered for the Tier 3 assessment. However, projections from the Sustainable Halton process can be used to define planned demand at all existing and active municipal supply wells.

Projected pumping rates for existing wells from the Sustainable Halton process are the same as the maximum annual average daily taking at each wellfield currently approved in the Acton and Georgetown Permits to Take Water (**Table C3-21**), with the exception that the planned pumping rate for the Cedarvale Wellfield has contingent approval pending the results of an ongoing monitoring program. Per the Interim Guidance, the allocated quantity is considered as the combined amount of the existing plus any committed demand up to the current lawful PTTW.

Table C3-22: Population Projections for Acton and Georgetown

Population Projections*	2016	2021	2026	2031
Georgetown	41,042	44,410	57,452	71,332
Acton	9,798	10,379	12,874	13,981
Total	**50,838**	**54,789**	**70,326**	**85,313**

*Note: * Source: 2011 Best Planning Estimates as listed by AECOM (2011)*

Table C3-23: Water Demand Projection

Community	Average Daily Demand Production (m^3/day)			
	2016	2021	2026	2031
Georgetown	17,040	17,620	22,000	26,610
Acton	3,540	3,910	4,790	5,210
Total	**20,580**	**21,530**	**26,790**	**31,820**

*Note * Source: AECOM (2011)*

C3.12.5 Non Municipal Water Demand

Permitted Water Uses

Non-municipal groundwater users in the study area with water takings in excess of 50,000 L/day (500 m^3/day) are listed in **Table C3-24** and shown on **Figure C3-22**. These water users are referred to as large, non-municipal, water takers, and represent agricultural, commercial, and industrial uses. Non-municipal PTTW holders were identified from the 2006 MOECC PTTW database and cross-referenced with results of the Tier 2 water budget assessment.

The takings associated with the Acton Quarry (PTTW 02-P-3087) were represented in the model using actual estimates of the groundwater portion of the dewatering based on site monitoring data (AquaResource Inc., 2013).

PTTW 7530-8FP6GZ is the only large, non-municipal, water taker within the Local Area. The water taking occurs at an institutional complex where groundwater is extracted for the purpose of cooling building and equipment. The water taking is considered to be a net non-consumptive user of groundwater, since the extracted groundwater is returned to the supply aquifer through injection wells after it is cycled through the cooling system. Since PTTW 7530-8FP6GZ is not a consumptive user of groundwater, it was not represented in the developed numerical modelling tools as a groundwater taking.

Table C3-24: Non-Municipal Groundwater Permits within the Study Area

Permit Number[1]	Easting	Northing	Maximum Permitted Rate (m^3/d)	General Use	Specific Purpose
7530-8FP6GZ[2]	584997	4834153	1,635	Commercial	Cooling Water
92-P-3116	587642	4828367	202	Agricultural	Market Gardens/Flowers
95-P-3014	585952	4828367	687	Commercial	Aquaculture
96-P-3031	595876	4839029	3,273	Industrial	Aggregate Washing
96-P-3031	595876	4839029	3,273	Industrial	Aggregate Washing
97-P-3009	588202	4824224	163	Agricultural	Fruit Orchards
98-P-3006	591994	4836832	2,864	Industrial	Aggregate Washing
02-P-3087	580507	4830473	4,582	Dewatering	Pits and Quarries
02-P-3087	580609	4829783	3,928	Dewatering	Pits and Quarries
02-P-3087	580482	4828837	1,047	Dewatering	Pits and Quarries
98-P-2106	577179	4846206	393	Industrial	Aggregate Washing
01-P-2927	576640	4844411	426	Commercial	Golf Course Irrigation

Notes: [1] Every permitted water taking in the MOECC's 2006 PTTW database located in the study area was examined, and the reported volume of extracted groundwater summarized, where applicable. Permitted water sources in the study area, taken from the 2006 PTTW database. Of the 28 sources of water takings in the study area, eight sources did not have reported water takings in the Water Takings Reporting System (WTRS) database for years 2006 or 2008. To ensure permit data completeness in the Tier 3 assessment, each permit in the study area was examined to ensure each considered water taking is a true groundwater taking, as well as the nature of the relationship between multiple sources under one permit and data completeness by referencing original permits (accessed using the Environmental Registry at www.ebr.gov.on.ca). Compiled permits were also cross-referenced to permits examined as part of the Credit Valley Conservation Tier 2 Integrated Water Budget Report (AquaResource, 2009) to ensure all permits considered in the Tier 2 assessment are captured in the Tier 3 assessment. Comparison revealed three permits included by AquaResource (2009) not captured in the 2006 PTTW database. Each of these watering takings was referenced to the original permit, where possible.

[2] Updated per recent EBR Registry Number 011-2429, formerly regulated under PTTW No. 92-P-3051. (References water takings associated with the local high quality monitoring locations: WATCHTOWER 12, WATCHTOWER 2, WATCHTOWER 3, and WATCHTOWER 4 (Table B.1, Appendix C – Summary of Monitoring Data).

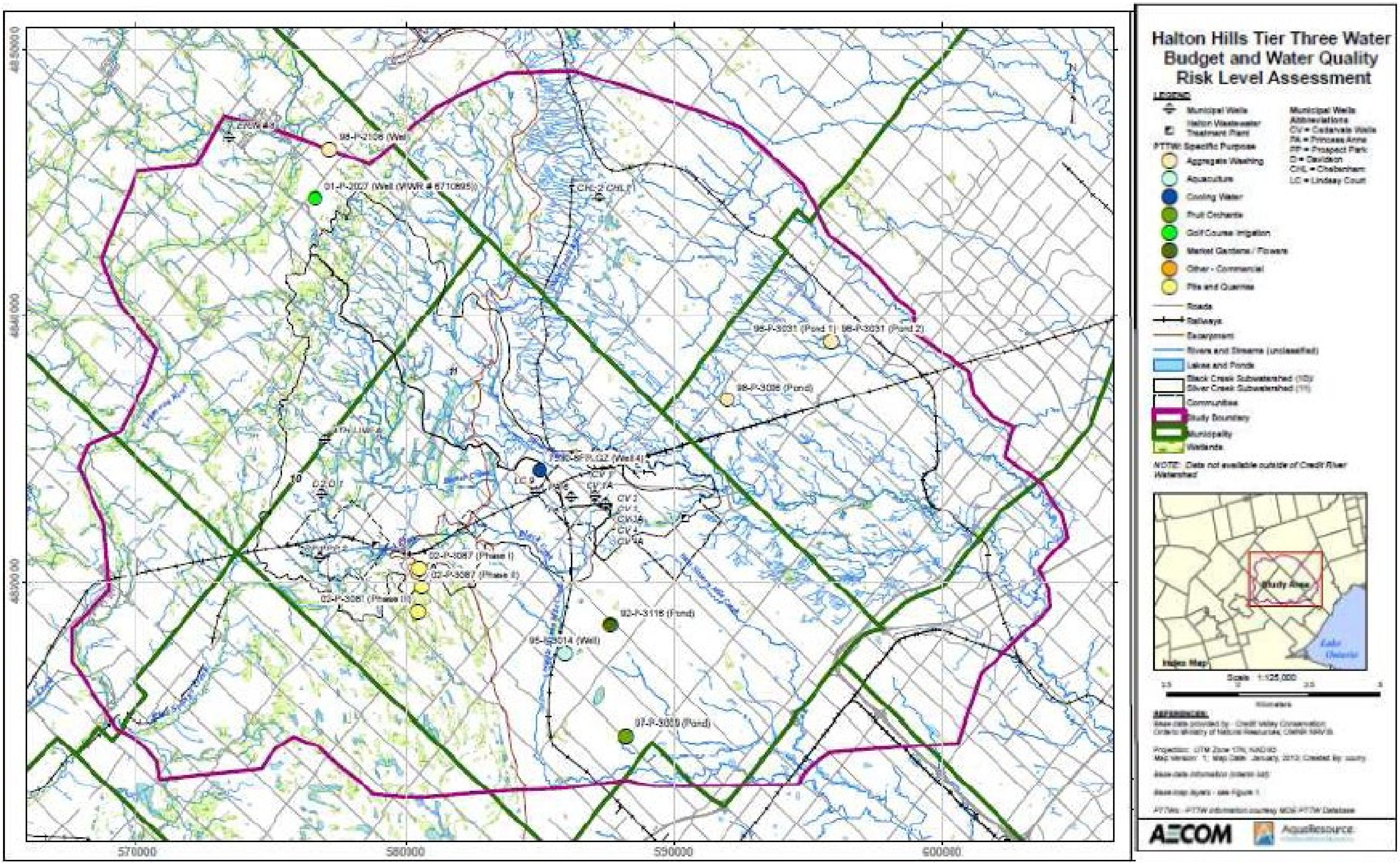

Figure C3-22: Non-municipal Permits to Take Water

Non-Permitted Water Uses

Several wells that are located in serviced areas pre-date the supply of serviced water to the area. Although these wells may no longer be used for potable supply, they may still be used for lawn watering or similar uses. Domestic water takers were not simulated in the groundwater flow model as their individual takings are relatively insignificant compared to municipal pumping. Consumptive water use from the unserviced domestic wells in subwatersheds 10 and 11 was estimated at 981 m^3/d (AquaResource Inc., 2009). This represents approximately 5% of the average annual water taking at municipal supply wells within Acton and Georgetown between 2005 and 2011, or less than 3% of the maximum permitted municipal water taking volume. As such, these water uses were not simulated in the groundwater flow model or considered in the water budget calculations.

C3.13 Land Use and Land Use Change

Tier 3 risk assessment scenarios must consider the impact of existing and future land development on groundwater recharge and sustainability of municipal water supplies. To achieve this, potential changes in recharge must be determined based on land imperviousness factors applied to areas where there is a change in land use between existing and future conditions. Land use changes were only considered within the study area where a reduction in recharge might have an impact on the availability of municipal water supply. As such, the project team reviewed land use patterns within Halton Region only.

C3.13.1 Existing Conditions Land Use

Existing land use within the study area is shown on **Figure C3-23** and is representative of the 2005-to-2009-time frame. The data set shown is the Southern Ontario Land Resource Information System (SOLRIS), Version 1.2, as distributed by the MNRF Science and Information Branch (2008). The existing urban land use is divided into Employment Areas (industrial/commercial land uses), Urban Areas (industrial /commercial/mixed use and residential land uses) and the Natural Heritage System (Green Belt Policy Area and other Natural Heritage Features, e.g., wetlands, woodlands, watercourses).

C3.13.2 Official Plan Land Use

Halton Region adopted the Regional Official Plan (ROP) in 2006, "*to give clear direction as to how physical development should take place in Halton to meet the current and future needs of its people*." (Regional Municipality of Halton, 2006). To accommodate the Ontario provincial planning regulations under the *Places to Grow Plan*, *Greenbelt Protection Plan*, and *Provincial Policy Statement*, Halton Region initiated the Sustainable Halton process. The Sustainable Halton process involved the creation of a growth management plan, as well as a basic and comprehensive review of the ROP. AECOM (2011) completed a water demand assessment as part of the Sustainable Halton process, which was summarized earlier. In 2009, the region adopted Amendment No. 38 to the Regional Official Plan, or ROPA 38, based on the results of the Sustainable Halton process and review of the ROP (Regional Municipality of Halton, 2009). ROPA 38 outlines the region's growth strategy to 2031. Planned future land uses under ROPA 38 are illustrated on **Figure C3-24**.

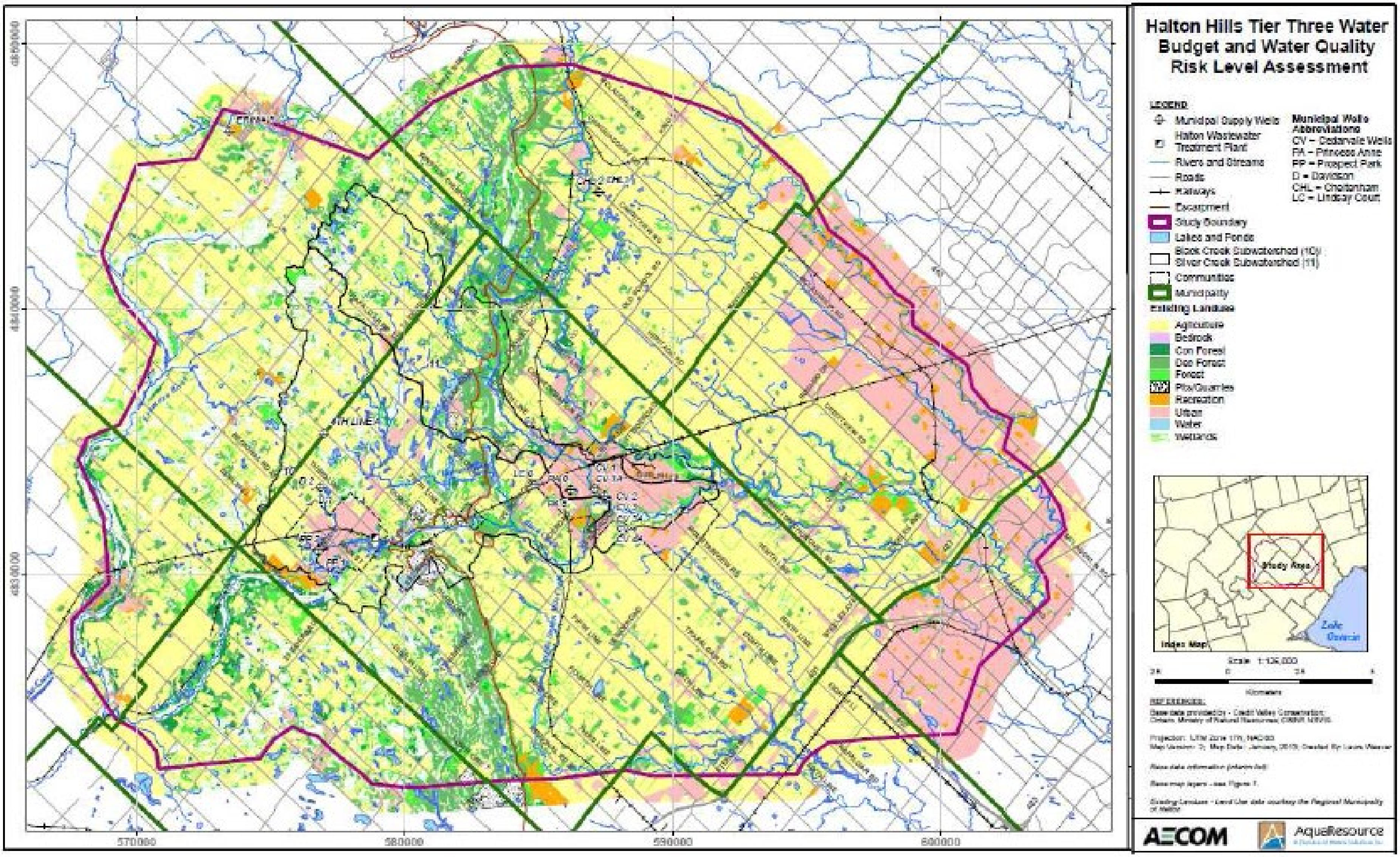

Figure C3-23: Existing Conditions Land Use

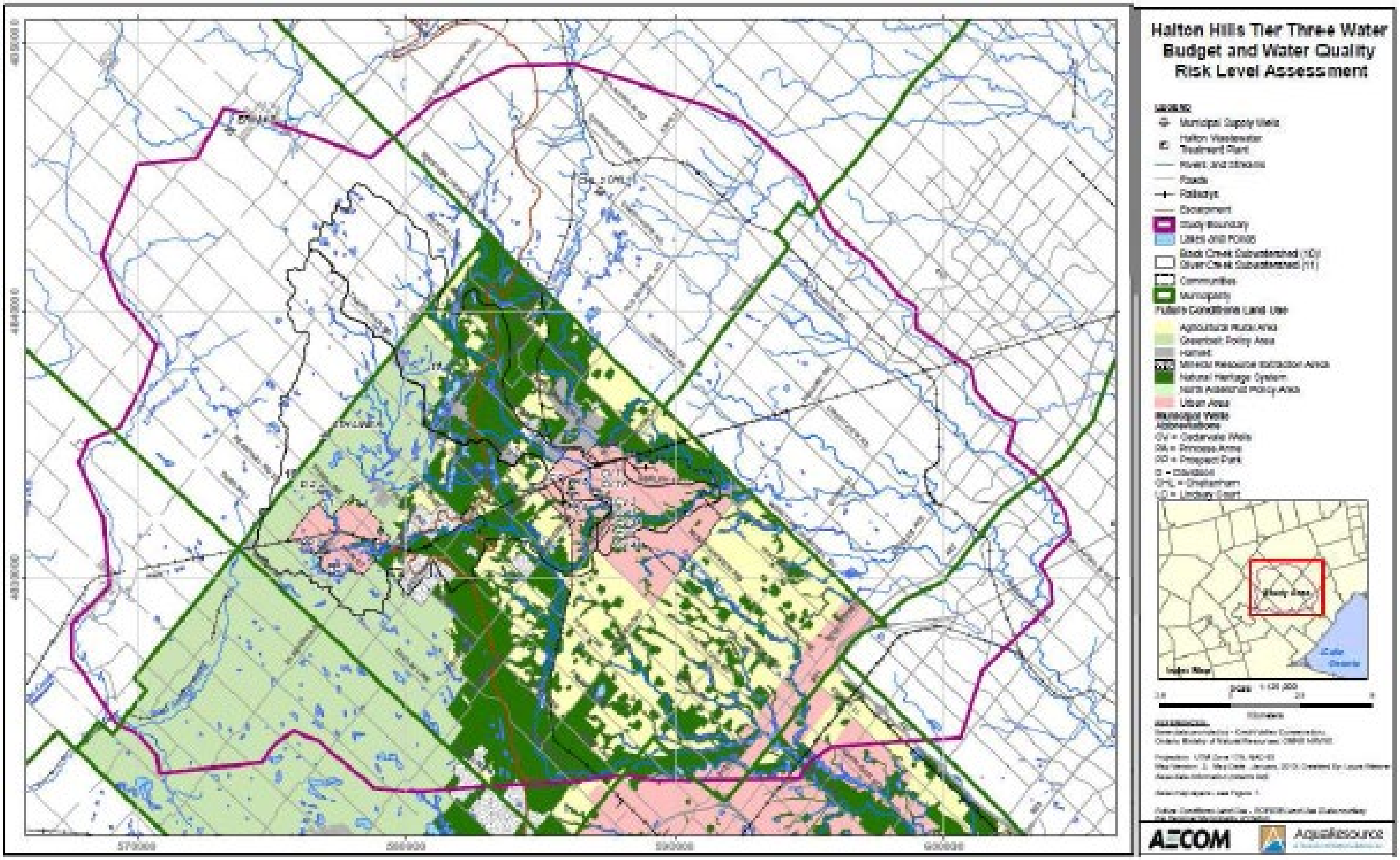

Figure C3-24: ROPA 38 (Future Conditions) Land Use

C3.13.3 Land Use Change

Areas of planned land use change for risk assessment scenarios are depicted in **Figure C3-25**. These areas were identified by comparing existing conditions and future (ROPA 38) land use patterns using a Geographic Information System (GIS). The most significant areas of change are south of Georgetown just to the northeast of Milton: along Steeles Road between 6th Line and Winston Churchill Boulevard; and along Highway 401 between 6th Line and 8th Line south to Derry Road. Specific future urban land uses are not identified in ROPA 38. To represent land use changes (imperviousness changes) assumptions about likely land uses were made based on the surrounding current land uses or developments underway.

C3.14 Local Area Risk Assessment

C3.14.1 Vulnerable area Delineation

WHPA-Q1

The WHPA-Q1 areas were delineated by examining the change in model predicted heads within the production aquifers between two model scenarios:

1. Steady-state model simulating existing land use, and no municipal pumping. This scenario establishes water levels that would exist without pumping.
2. Steady-state model simulating existing land use, and existing plus committed plus planned municipal pumping rates.

The model predicted heads in the production aquifer for each of the above scenarios, which were subtracted from one another. The average seasonal water level fluctuation within wells monitoring heads in the production aquifer is 1.0 m, and therefore, the 1.0 m drawdown contour interval was selected for use in delineating the WHPA-Q1 area for Acton and Georgetown. With respect to Georgetown's municipal wells, additional consideration was given to the neighbouring surface catchment area, which has been shown to contribute recharge to the Georgetown municipal aquifer. This is discussed below.

Recharge to Georgetown Municipal Aquifer from Beeney Creek

Groundwater modelling studies show that the lower reach of Beeney Creek, west of Georgetown, loses on average, approximately 4,130 m^3/day, through leakage from the base of the stream bed to the underlying aquifer (**Appendix C3**). This aquifer is part of the Acton-Georgetown buried bedrock valley aquifer east of Limehouse and is intersected by Lindsay Court and Princess Anne wellfields. The observed average leakage represents 27% of the existing pumping demand from the Georgetown wellfields. A measurable reduction in streamflow in Beeney Creek could arguably reduce leakage to the municipal aquifer and impact well production. The Beeney Creek catchment area is illustrated in **Figure 3.34** of the Assessment Report.

Model scenario analysis and calibration indicates that leakage from Beeney Creek provides an important recharge function for the buried bedrock valley aquifer. Recharge reduction activities in the catchment area of Beeney Creek could impact the ability of the wells to meet demand. In respect of this, the Province, Ministry of Natural Resources and Forestry, and CTC Source Protection Region have supported the inclusion of the Beeney Creek catchment area as part of the WHPA-Q1 for Georgetown.

Three WHPA-Q1 areas were delineated within Acton and Georgetown are shown in **Figure 3.35** of the Assessment Report. WHPA-Q1-A lies northwest of the Acton and is associated with Fourth Line Well A and Davidson Well. Drawdown associated with Prospect Park well was approximately 2 m, and the 1.0 m drawdown cone was restricted to a very small area immediately surrounding the well (< 100 m). Given the limited drawdown extent associated with Prospect Park well, a 100 m buffer area was drawn around the well to delineate WHPA-Q1-B. WHPA-Q1-C is the largest area delineated and is associated with Georgetown and the area west and north of the urban areas. No other users beyond the municipal wells were identified within the cones of influence.

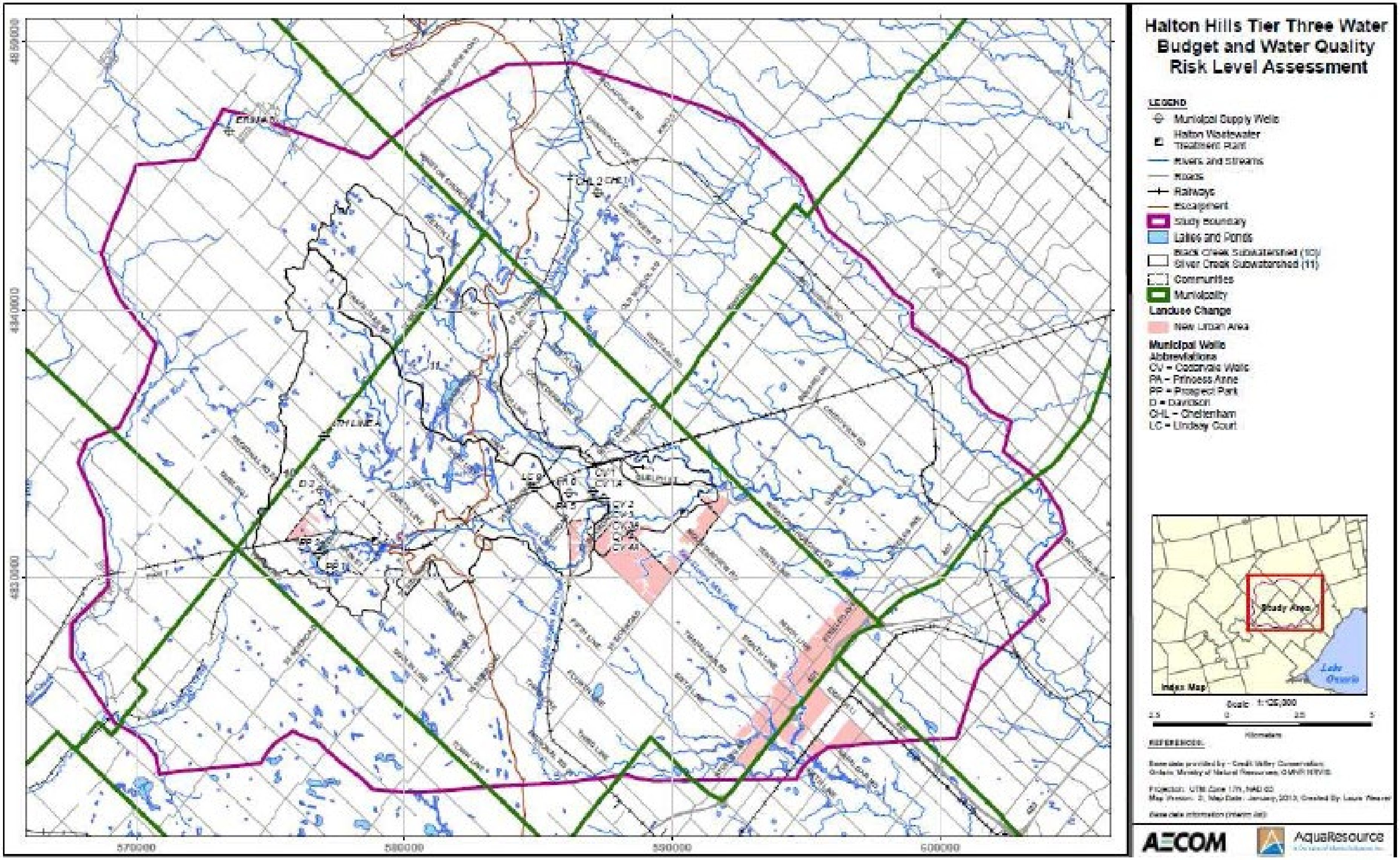

Figure C3-25: Land Use Change between Existing and Future Conditions

WHPA-Q2

The WHPA-Q2 is defined in the MOECC *Technical Rules (MOE, 2009)* as the WHPA-Q1 area, plus any area where a future reduction in recharge may have a measurable impact on that area. Proposed land development areas that had the potential to reduce the available drawdown in a municipal well were simulated in the groundwater model. These are primarily south and west of Georgetown and a small area in the western portion of the Acton boundary (**Figure 3.36** of the Assessment Report).

According to the Official Plan, proposed land use development areas that lie within the WHPA-Q1 area include infilling of high and low intensity urbanized land within the urban core. Some of these potential land use developments straddle the boundary of the WHPA-Q1 and extend beyond it. To assess the impact of land use changes on the water quantity for the municipal wells, the MIKE SHE surface model was updated to simulate the reduction in recharge for the areas designated for high or low intensity land use changes, by changing the vegetation, surface roughness, imperviousness and depression storage parameters in the areas to be developed using values assigned in similar existing developments modelled in the same area. This generally resulted in an increase in runoff and evapotranspiration and a reduction in recharge in areas of proposed development based on the general land use classification.

The simulated average annual groundwater recharge distribution from the MIKE SHE model was applied to the FEFLOW groundwater model, and the model was re-run. The reduction in hydraulic head due to the development of residential lands was predicted to be between 2 and 9 cm for the Georgetown municipal wells, and between 0 and 2 cm for the Acton municipal wells. The seasonal variation in water levels of approximately 1 m would mask this change. Further, the reduction in hydraulic head is much smaller than the available drawdown at all wells (> 4.5 m). Therefore, the reduction in recharge outside of the WHPA-Q1 is not considered to have a measurable impact on the wells. As such, the land use changes that lie outside the WHPA-Q1 area were not included in the WHPA-Q2 area. The WHPA-Q2 area is therefore coincident with the WHPA-Q1 area.

The development of residential lands was predicted to be between 2 and 9 cm for the Georgetown municipal wells, and between 0 and 2 cm for the Acton municipal wells. The seasonal variation in water levels of approximately 1 m would mask this change. Further, the reduction in hydraulic head is much smaller than the available drawdown at all wells (> 4.5 m). Therefore, the reduction in recharge outside of the WHPA-Q1 is not considered to have a measurable impact on the wells. As such, the land use changes that lie outside the WHPA-Q1 area were not included in the WHPA-Q2 area. The WHPA-Q2 area is therefore coincident with the WHPA-Q1 area. **Figure 5-2** illustrates the WHPA-Q2 area within the study area as well as the proposed land use development areas (as specified in the Official Plan).

Local Area

The Local Area for this study area is illustrated on **Figure 3.37** of the Assessment Report. The Local Areas are delineated by combining the cone of influence of the municipal supply wells (WHPA-Q1) and the areas where a reduction in recharge would have a measurable impact on the cone of influence of the wells (WHPA-Q2). WHPA-Q1 and WHPA-Q2 areas are coincident reflecting low potential for measureable impact on water levels at the municipal wells under proposed changes in land use outside the WHPA-Q1. Local Area A includes Fourth Line Well A and Davidson Well. Local Area B includes Prospect Park Well. The cone of influence for the Georgetown municipal wells overlap and define a single local area, Local Area C.

C3.14.2 Risk Scenario Development

Information required to prepare the models for each risk assessment scenario was compiled as follows.

Scenario C – Existing Conditions, Average Climate

Scenario C evaluates the ability for existing municipal water supply wells to maintain existing average annual pumping rates under average climate conditions. This scenario was simulated in steady state in the FEFLOW model using 2005-2011 average (existing) pumping rates (**Table C3-21**) and the average annual groundwater recharge distribution from the calibrated MIKE SHE model (1960 to 2005 simulation). The groundwater flow model was constructed and calibrated to predict groundwater levels in the aquifer at the municipal pumping wells, and to predict groundwater levels and/or groundwater discharge rates under existing water demand and average climate conditions.

Scenario D – Existing Conditions, Drought

Scenario D aims to evaluate whether each municipal well is able to pump at their allocated rates (existing rates) during a drought period. This scenario was simulated using the calibrated Tier 3 groundwater flow model in continuous transient mode for a period of 50 years. Average monthly recharge rates from the MIKE SHE model were applied in the groundwater flow model throughout the duration of the simulation (1960 to 2005), which included several drought periods (i.e., late 1960s and late 1990s droughts). The recharge was simulated as spatially variable, and the magnitude of fluctuation was different for each soil and land use type. Monthly pumping rates were applied in the groundwater flow model based on the monthly average use from the 2005-2011 period to be representative of existing pumping and seasonal variations.

The *Technical Rules* refer to a 10-year period to define drought conditions for the scenarios. However, this assessment went beyond the requirements of the *Technical Rules* (MOE 2009) and examined two drought periods that occurred within the 45-year climate period examined (i.e., 1960s and 1990s). The 45-year period examined with the transient model included the two drought periods, and also periods where precipitation (and in turn recharge) were above normal.

As outlined in the *Technical Rules* (MOE, 2009), the impacts of municipal pumping on other uses were not considered in this drought scenario. As a result, the main output parameters for this scenario are water levels at each of the municipal wells.

Scenario G – Existing Plus Committed Plus Planned Demand, Future Land Development, Average Climate

Scenario G evaluates the ability for existing and planned wells to maintain existing plus committed plus planned pumping rates under average climate conditions and reductions in recharge. This scenario was simulated using the calibrated Tier 3 groundwater flow model in steady state conditions using groundwater recharge rates that reflect long-term average climate conditions. Scenario G is subdivided into three scenarios (G(1), G(2), and G(3)). The purpose of subdividing into these scenarios is to isolate the impacts of municipal pumping from land development. Only the scenario representing increased municipal pumping is considered when evaluating the impact of the scenarios on wetlands and coldwater streams (Scenario G(2)).

Scenario G(1) - this scenario evaluated the cumulative impact of increased municipal pumping rates (existing plus committed plus planned rates) and reductions in recharge (assuming increased imperviousness) due to future land use changes defined in the Official Plans, on the municipal wells, and other uses. **Table C3-21** lists the existing plus committed plus planned water demands applied to evaluate this scenario. **Table C3-25** summarizes the imperviousness values applied to the land use areas that, according to the Official Plans, will be modified in the future. These values were obtained by comparing each soil class used within the recharge estimation for both urban and non-urban settings within the Halton Region portion of the study area. The recharge rates assigned for these areas were calculated by multiplying the impervious value by the recharge rate estimated for undeveloped conditions.

Table C3-25: Imperviousness Estimates Applied for Future Land Use Areas

Soil Type	Sample Average Recharge		Imperviousness (Recharge Reduction)
	Urban	Non-Urban	
Halton Till	36	74	49% (51%)
Wentworth Till	133	218	61% (39%)
Clay	9	16	55% (45%)
Sand	288	375	77% (23%)
Bedrock	245	376	65% (35%)
Gravel	284	323	88% (12%)

Scenario G(2) - this scenario evaluated only the impact of increased municipal pumping rates (existing plus committed plus planned rates) on the municipal wells and other water uses. The existing conditions land use was simulated in this scenario to isolate the influence of municipal pumping from land development. Only this scenario is considered when evaluating the impact of the scenarios on wetlands and coldwater streams. Baseflow reductions arising from land use development are independent from increased groundwater pumping, and only those impacts associated with groundwater pumping (e.g., Scenario G(2)) should be used to evaluate the Water Quantity Risk Level relating to the impact to other uses.

Scenario G(3) - this scenario evaluated only the impact of reductions in recharge (due to increases in imperviousness) due to future land use changes defined in the Official Plans, on the municipal wells and other water uses. Existing municipal pumping rates were used in this scenario to isolate the influence of land development from existing plus committed plus planned demand.

Scenario H – Existing Plus Committed Plus Planned Demand, Future Land Development, Drought Conditions

Scenario H evaluated the ability for existing wells to maintain allocated municipal pumping rates (existing plus committed plus planned) through a drought period (same temporal period as Scenario D). The groundwater flow model was run transiently to examine the combined impact of drought conditions, land use development, and additional municipal pumping on water levels at the municipal wells. Impacts to other water uses are not considered in Scenario H. Monthly pumping rates were applied in the groundwater flow model based on the monthly average use from the 2005-2011 period and scaled to the average annual allocated rate to be representative of allocated pumping and seasonal variations. Similar to Scenario G, this scenario was subdivided into Scenario H(1), H(2) and H(3) to evaluate the relative contribution of municipal water takings and land use development at each municipal well under drought conditions.

Scenario H(1) - this scenario evaluated the cumulative impact of increased municipal pumping rates (existing plus committed plus planned rates), reductions in recharge (due to increases in imperviousness) due to future land use developments defined in the Official Plans, and drought conditions on the municipal wells. As noted above, the impact was only evaluated at the municipal wells and not on other water uses.

Scenario H(2) - this scenario evaluated only the impact of increased municipal pumping rates (existing plus committed plus planned rates) on the municipal wells during a drought period. The existing conditions land use was simulated in this scenario.

Scenario H(3) - this scenario evaluated the impact of reductions in recharge (due to increases in imperviousness) due to future land use developments defined in the Official Plans and drought conditions on the municipal wells. As noted above, the impact was only evaluated at the municipal wells and not on other water uses.

C3.15 Sensitivity Analysis of Scenarios

The representation of the groundwater flow system was manually calibrated to available hydraulic head data and baseflow measurements using a set of parameters (e.g., recharge and hydraulic conductivity) that are consistent with the conceptual model. However, this set of parameters is non-unique, and other parameter sets may produce an equally well-calibrated model. An uncertainty analysis was therefore conducted on the scenarios to:

- Determine which parameters could be adjusted and still be considered to be acceptably calibrated;
- Create multiple sets of model input files, each containing different combinations of suitable model parameters that are considered to be acceptably calibrated; and
- Evaluate two scenarios for each of the model input files and predict the result in terms of water level drawdown (G(1)) and baseflow reduction (G(2)). From these results, it is possible to estimate conservatively, if the water level or groundwater discharge reduction criteria will be violated (or satisfied) by the model if other supported parameter values or conceptualizations are considered.

C3.15.1 Parameter Sensitivity

PEST (Dougherty, 2004) facilitates a series of model runs where each model parameter (e.g., hydraulic conductivity zone) is adjusted individually to determine the sensitivity of the model calibration to an incremental change in parameter value. The calibration sensitivity gives insight on the parameterization of the model and identifies:

- The parameter values that are well-supported by field observations;
- The parameters that can be estimated using automated parameter estimation routines (e.g., PEST) to optimize model calibration;
- The relative influence of each parameter in model calibration; and
- The potential for new observations to improve the estimation of a parameter.

A single-parameter sensitivity analysis approach was used and it involves multiple model simulations whereby each model input parameter (e.g., hydraulic conductivity or recharge zone) is modified one at a time from the base case model. The updated model is run and the simulation output (e.g., head or discharge) is compared to the output from the base case model. The goal of this analysis is to identify those parameters that have the largest influence on the simulations and to evaluate the observation data that are available to constrain/estimate that parameter.

C3.15.2 Uncertainty Analysis Results

The base case model is one realization of a set of parameters that produced a calibrated model. The model and the input parameters are a generalized representation of a complex hydrogeological system, and the assumptions used to generalize the model have associated uncertainty. Throughout the calibration process, it was noted that changed input parameters, or combinations of changed input parameters, had little impact on the model calibration at the municipal wells. The model calibration changes very little with changes in these conceptualized parameters; a much larger range of plausible values will produce a calibrated model. These parameters have a higher degree of uncertainty (when calibrating the model to higher quality well data) and their impact on the model prediction was tested and examined.

A set of sensitivity analysis was completed to quantify how uncertainty associated with various model input parameters (e.g., recharge, buried valley infill continuity, hydraulic conductivity, etc.) influences additional drawdown and baseflow reduction. Four scenarios were conducted to examine the impact of uncertainty in hydraulic conductivity, recharge, and alternative conceptual models within the area. In all sensitivity scenarios, variations in parameters are imposed to seek additional impacts to the hydraulic head at each well while maintaining calibration. These parameters used in the sensitivity scenarios are plausible but are conceptually less consistent with field data.

Model Predictions- Hydraulic Heads

The model output was compiled to provide insight into how the uncertainty associated with the model input parameters may affect the model predictions. The initial objective was to identify if these conditions would cause the hydraulic head in the aquifer at the municipal well to violate the safe additional drawdown at the well. **Table C3-26** shows the results of these simulations for scenario G(1). The model predicted drawdown for the Halton Region municipal wells, for the most part, exceeds the model predicted drawdown of the base case scenario, but does not exceed the safe additional drawdown, except in one scenario at two Cedarvale wells. The safe additional drawdown is exceeded in scenario 2 (decreased K and increased recharge) at the Cedarvale Wellfield (within Local Area C). However, the simulated values are interpreted to be less representative of the system as they are on the low end of the range of field-tested hydraulic conductivity (transmissivity) and result in a slightly poorer calibration. Therefore, the base case results that do not exceed the safe additional drawdown are considered more representative.

Table C3-26: Uncertainty Analysis – Alternative Calibration Model Drawdown Results

Well Name	Safe Additional Drawdown (2005-2011)	Simulated Additional Drawdown (m) by Scenario				
		Base Case	Scenario 1 Increased K Decreased Recharge	Scenario 2 Decreased K Increased Recharge	Scenario 4 Decreased Beeney Creek Leakage	Scenario 5 Continuity of Buried valley Sediments near to Lindsay Court
Fourth Line Well A	5.6	5.2	5.4	5.0	5.2	5.2
Davidson Well 1	8.2	1.9	1.3	4.4	1.9	1.9
Prospect Park Well	14.0	0.8	0.1	3.8	0.8	0.8
Lindsay Court Well 9	10.1	2.3	2.4	2.1	4.5	2.8
Princess Anne Well 5	14.7	5.5	5.0	6.0	7.1	4.1
Princess Anne Well 6	13.0	6.3	5.7	7.4	8.0	4.9
Cedarvale Well 1A	11.8	7.0	2.7	18.5	7.3	6.8
Cedarvale Well 3A	10.1	1.9	-1.4	10.1	2.1	1.7
Cedarvale Well 4	8.8	4.8	1.9	11.7	5.0	4.6
Cedarvale Well 4A	4.5	-1.2	-4.1	5.8	-1.0	-1.4

Table C3-27 shows the impacts to groundwater discharge. As illustrated in **Table C3-27**, most of simulated groundwater discharge reductions for the G(2) scenario are less than 10%. The potential baseflow reduction associated with recharge reductions for Lower Black Creek is 13% under sensitivity scenario 4. The potential baseflow reductions associated with recharge reductions for Upper Black Creek is 16% and 12% for sensitivity scenarios 1 and 4, respectively. Consistent with the risk scenarios, Lower Beeney Creek and Hospital Tributary have baseflow reductions larger than 20%.

Table C3-27: Impacts to Groundwater Discharge and Stream Leakage reducing Baseflow Scenario G

Stream/Reach	Net Stream/Reach Condition	Scenario C GW Discharge/ Stream Leakage (m^3/d)	Reduction in Baseflow (m^3/d) Percentage Change for Increased Demand Scenario G (2)			
			Scenario 1 Increased K Decreased Recharge	Scenario 2 Decreased K Increased Recharge	Scenario 4 Decreased Beeney Creek Leakage	Scenario 5 Continuity of Buried valley Sediments near to Lindsay Court
Silver Creek	Gaining	-31875	4%	4%	8%	3%
Lower Black Creek	Gaining	-5020	6%	6%	13%	7%
Upper Black Creek	Gaining	-25620	16%	3%	12%	7%
Lower Beeney Creek	Losing	6280	22%	22%	3%	26%
Upper Beeney Creek	Gaining	-10715	3%	-4%	5%	4%
Hospital Tributary	Gaining	-595	39%	49%	89%	40%

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