Municipal Groundwater Flow Modelling Study, Town of Torbay, NL

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Sign-off Sheet

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Stantec

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Executive Summary

The Town of Torbay commissioned a steady-state groundwater model that can be used as a tool for evaluating the cumulative impacts of residential subdivision and commercial development on groundwater resources.

In order to meet the request of the Town of Torbay, a three-dimensional groundwater flow model was constructed using MODFLOW to evaluate existing conditions and the effects of future development on existing well users and stream baseflow. The model was prepared using a simple conceptual hydrostratigraphic model, and assumed homogeneous properties within each defined unit. The model was calibrated to observed water levels and estimated stream baseflow targets. The parameter values for hydraulic conductivity and groundwater recharge are reasonable and align with other analyses and field observations.

The base case established current groundwater flow conditions within the defined Study Area. Predictive simulations assessed the effects of completing existing subdivisions, new subdivisions and commercial development, and the feasibility of a municipal well field. Predicted drawdown and changes in baseflow are local to development areas. Residential and commercial development is not expected to induce adverse effects (drawdown) on existing well users. Reductions in stream baseflow are noted. A municipal well field is not likely feasible in this setting due to low yield and available drawdown constraints.

The groundwater model is designed to be a tool for adaptive groundwater resource management and land use planning. Future information obtained from water well records, pumping tests, and the direct observation of changes due to development within the Study Area should be used to update the conceptual and numerical models in an effort to refine the tool for this purpose.

1.0 INTRODUCTION

Stantec Consulting Ltd. (herein referred to as “Stantec”) was retained by the Town of Torbay to carry out a Municipal Groundwater Flow Modelling study. It is understood that the purpose of this groundwater modelling study was to create a representative numerical groundwater flow model that simulates local hydrogeological conditions and can be used to evaluate and understand the cumulative, town-wide effects of unserviced development on groundwater supply and the overall sustainability of the community’s groundwater resources, and can be used as a planning tool to assess groundwater supply potential in unserviced areas of the town with respect to various future residential and commercial development schemes.

This report presents the description and results of the development and application of a steady-state numerical groundwater flow model developed for the Town of Torbay, NL (referred to hereafter as the “Town”).

1.1 Scope

The scope of work for this study included the development of a steady-state numerical groundwater flow that is calibrated to available data. The main tasks for this study included:

- Developing a conceptual groundwater flow model using existing data;
- Constructing a three-dimensional (3-D) hydrogeological model;
- Developing a calibrated 3-D steady-state groundwater flow model of existing conditions; and,
- Predicative modelling utilizing the base steady-state groundwater flow model to simulate three future development scenarios for the municipality.

The groundwater flow model developed as part of this study relied solely on existing reports and other available sources of information, including various federal and provincial government databases, and did not include any hydrogeologic field investigations to collect new data in support of the project.

2.0 BACKGROUND

Based on information provided in the project’s Terms of Reference (TOR), dated July 23, 2014, the Town of Torbay (the “Town”) currently provides municipal water supply sourced from North Pond to approximately 30 percent of the community, leaving the remaining 70 percent of the community reliant on private groundwater wells for its potable supply. Furthermore, the Town is growing and recent development has continued to expand out into unserviced regions of the municipality. The Town is concerned that the form and pace of growth in unserviced development areas may deplete or otherwise adversely affect local groundwater supply, as well as impact the municipality’s overall groundwater resources. As such, the Town has requested that a steady-state groundwater model be completed to characterize groundwater conditions and assist in evaluating and understanding of the cumulative, town-wide effects of unserviced development on groundwater supply and the overall sustainability of the community's groundwater resources. It is hoped that this groundwater flow model can be used as a planning tool to assess groundwater supply potential in unserviced areas of the town with respect to various future residential and commercial development schemes.

2.1 Physical Setting

Torbay is located 12 km to the north of St. John's on the eastern side of the Avalon Peninsula in Newfoundland and Labrador. The Town Limit (Municipal Planning Area) covers approximately 36 km^2 (Figure 1).

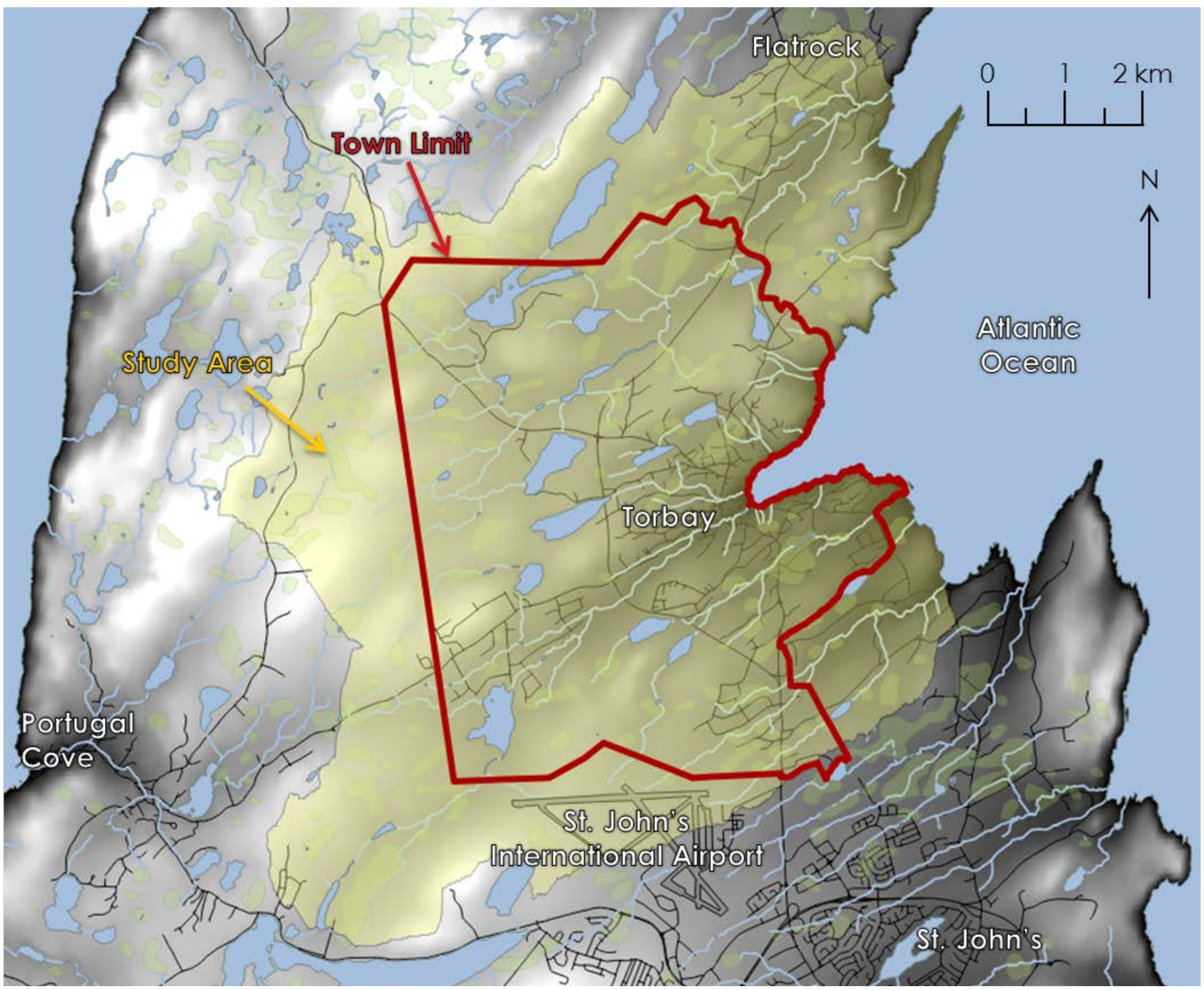

Figure 1 Study Area location plan showing the town limit (red outline) and the model Study Area (yellow shading), Located north of St. John's on the Avalon Peninsula, NL.

2.2 Topography and Drainage

The Town of Torbay is characterized by northeast-southwest trending bedrock-controlled undulating ridges/hills and valleys and a steep rugged coastline. Elevations in the general area range from 0 m to 210 m above sea level (masl) (Figure 1; with range in elevations from 0 masl shown in black to 210 masl shown in white). In natural, undeveloped portions of the study area, ground cover is predominantly boreal softwood forest and wetland.

The Town Limit spans numerous watersheds and sub-watersheds; each containing a network of wetlands, streams and ponds that ultimately flow northeast and discharge into the Atlantic Ocean (Figure 1). For purposes of defining the model Study Area, watersheds defined through topographic analysis were combined to include the Town Limit. This larger 76.2 km^2 combined watershed forms the Study Area for this project (Figure 1).

2.3 Climate

Torbay is located within the Maritime Barrens Ecoregion, which is characterized by cold summers with frequent fog and strong winds, and relatively mild winters with intermittent snow cover (Department of Natural Resources 2015).

Climate normals between 1981-2010 for station "St. John's A" are available from Environment Canada (2015). Average daily temperatures range between -4.9 °C (February) and 16.1 °C (August). Average annual total precipitation is 1534.2 mm, 1206.4 mm of which is rain. June through September are the only months consistently without snowfall.

2.4 Regional Geologic Setting

Surficial geologic materials in the Study Area are predominantly glacial till that occurs as a veneer (<1.5 m thick) and/or as linear ridges, as well as organic deposits (Batterson 2000). Bare rock or bedrock concealed by vegetation is mainly found along the coast.

Bedrock underlying the till or exposed at surface is comprised of the Late Precambrian Signal Hill Group (red, grey and green sandstone, conglomerate and shale), St. John's Group (black shale and slate), and Connecting Point and Conception Groups (grey and green sandstone, siltstone, shale and conglomerate) (King 1990a).

The bedrock has been deformed by the Precambrian Avalonian and mid-Paleozoic Acadian Orogenies with regional metamorphism during the latter. Geologic structure is quite complex with the presence of numerous large-scale dominantly northeast-trending structural features (faults) and large-scale anticlines and synclines (some doubly plunging to form domes and basins, respectively). Mapped bedding planes range in orientation from near-horizontal (10 degrees below horizontal in portions of the Torbay Dome) to vertical (King 1990b).

3.0 CONCEPTUAL MODEL

3.1 Model Approach

The development of a conceptual model is the fundamental first step in the preparation of a numerical groundwater model that represents the groundwater flow system underlying Torbay. The purpose of the conceptual model is to consolidate site hydrogeologic and hydrologic data into a set of assumptions and concepts that can be evaluated quantitatively and represented mathematically in the numerical groundwater flow model. A conceptualized hydrogeologic model of the Torbay Study Area was developed by taking into consideration available well drilling data and aquifer test results for the study area, as well as other relevant hydrogeologic and geological interpretations and surface water hydrologic data (e.g., rivers, streams and lakes). A detailed description of the various geologic, hydrogeologic, and hydrologic data sources utilized as part of this study are provided in the proceeding sections. These data sets were used to develop the conceptual hydrogeologic model, and construct the geologic and hydrogeologic framework of the numeric groundwater flow model for the Study Area. The general approach used to develop the conceptual and numerical models for the study area was to add complexity only as warranted by the available data and to achieve the goals of the numerical modeling (see Section 1.0).

3.2 Data Sources

3.2.1 Baseflow and Estimates of Groundwater Recharge

Stream flow is comprised of two components: direct runoff (overland flow) and baseflow (groundwater discharging into the surface watercourse). Baseflow is essentially equal to groundwater recharge in shallow groundwater systems. No hydrometric stations are present within the Study Area from which to obtain stream flow data for baseflow analysis to derive estimates of groundwater recharge. However, daily stream flow records are available for many monitored streams on the Avalon Peninsula outside of the Study Area (Environment Canada, 2015). A number of these hydrometric stations were used in the present study to obtain stream flow data for baseflow analysis and groundwater recharge estimation, based on their reported catchment area. The objective was to consider a range of catchment areas that are similar in scale to those for the larger streams in the Study Area. Based on this selection criterion, a total of 11 hydrometric stations on the Avalon Peninsula were selected for base flow analysis. These are listed in Table 1 along with a summary of stream flow data for each hydrometric station.

Numerous base flow separation methods have been developed to “filter” the baseflow “signal” out of daily flow data for a stream. For this study, the recursive digital filter developed by Eckhardt (2005) for perennial streams with hard rock aquifers was used to derive estimates of baseflow at each of the hydrometric stations. The results of the baseflow analysis for each of the hydrometric stations are provided in Table 1, and indicate that the proportion of stream flow that is baseflow ranges between 20% and 24% for the 11 hydrometric station stream flow data sets. Figure 2 provides a rating curve for the expected baseflow for a given catchment area based on the results presented in Table 1. Given the distribution of the data, the rating curve is likely best suited for catchment areas ranging between 10 km^2 and 100 km^2.

The Study Area can be subdivided into six surface water catchment areas that cover approximately 71.9 km^2, and with a total estimated baseflow is 60,222 m^3/d (Figure 3, Table 2). The volume of daily precipitation for this 71.9 km^2 combined catchment area is 302,000 m^3/d (1534.2 mm/yr × 1 m/1000 mm × 1 yr/365.25 d × 71.9 km^2 × 1,000,000 m^2/km^2). Based on these estimates, baseflow is determined to be approximately 20% of total precipitation. Since baseflow is generally considered to equal groundwater recharge in shallow groundwater systems, this implies that steady-state groundwater recharge is equivalent to about 20% of total annual precipitation in the Study Area.

Table 1 Baseflow Estimates from Hydrometric Station Daily Flow Records

Station Code	Location	Catchment Area (km^2)	Daily Flow Record	Mean Stream Flow Rate (m^3/d)	Proportion of Stream Flow that is Base Flow[1]	Estimated Baseflow (m^3/d)
02ZM006	Northeast Pond River at Northeast Pond	3.63	1953 - 2011	11,837	0.205	2,247
02ZM021	South Brook at Pearl Town Road	9.21	1986-1998	35,338	0.221	7,810
02ZM018	Virginia River at Pleasantville	10.7	1981-1996	46,656	0.244	11.384
02ZM010	Waterford River at Mount Pearl	16.6	1981-1996	63,763	0.239	15,239
02ZL004	Shearstown Brook at Shearstown	28.9	1983 - 2009	77,328	0.239	18,481
02ZK003	Little Barachois River Near Placentia	37.2	1983 - 2010	137,030	0.225	30,832
02ZN001	Northwest Brook at Northwest Pond	53.3	1966 - 1996	269,654	0.240	64,717
02ZK002	Northeast River Near Placentia	89.6	1979 - 2011	350,525	0.239	83,775
02ZK004	Little Salmonier River near North Harbour	104	1983 - 2011	453,600	0.215	97,524
02ZM001	Petty Harbour River at Second Pond	134	1962 - 2010	479,002	0.152	72,808
02ZK001	Rocky River near Colinet	301	1948 - 2011	970,790	0.238	231,048

Note:
[1] Estimated using WHAT Analysis for a perennial stream with hard rock aquifers (Eckhardt 2005)

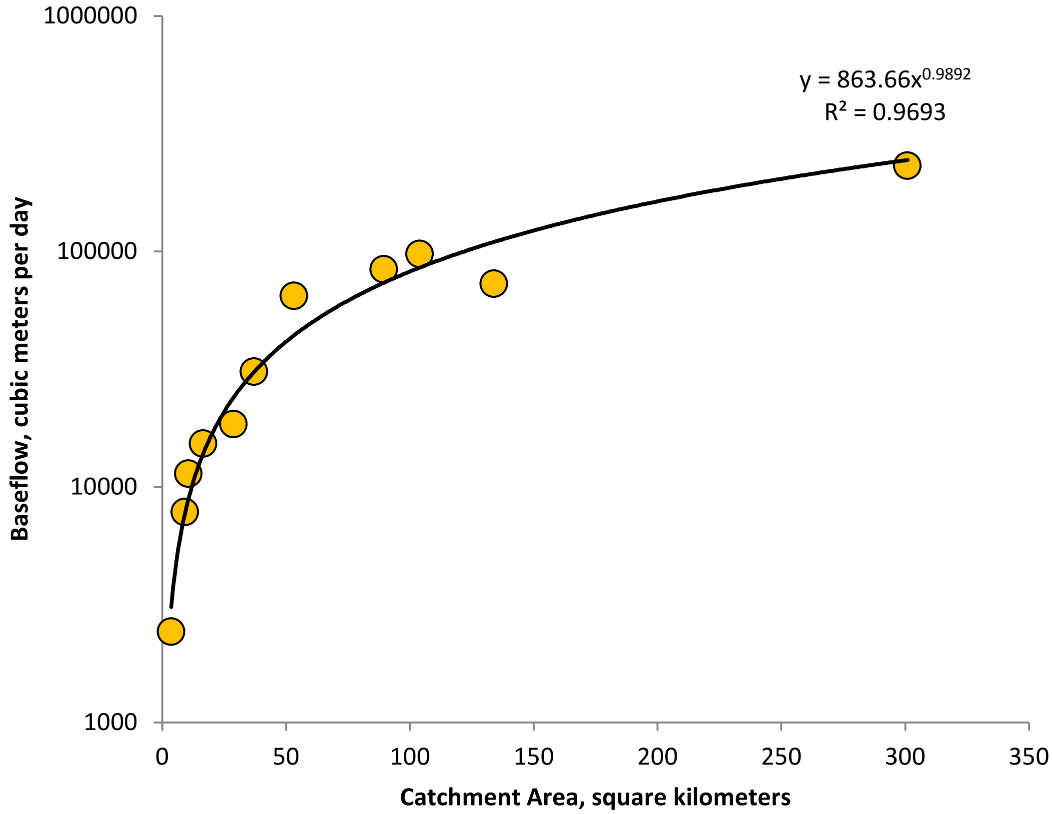

Figure 2 **Rating Curve for Estimating baseflow in a given surface water catchment area based on data from 11 hydrometric stations on the Avalon Peninsula.**

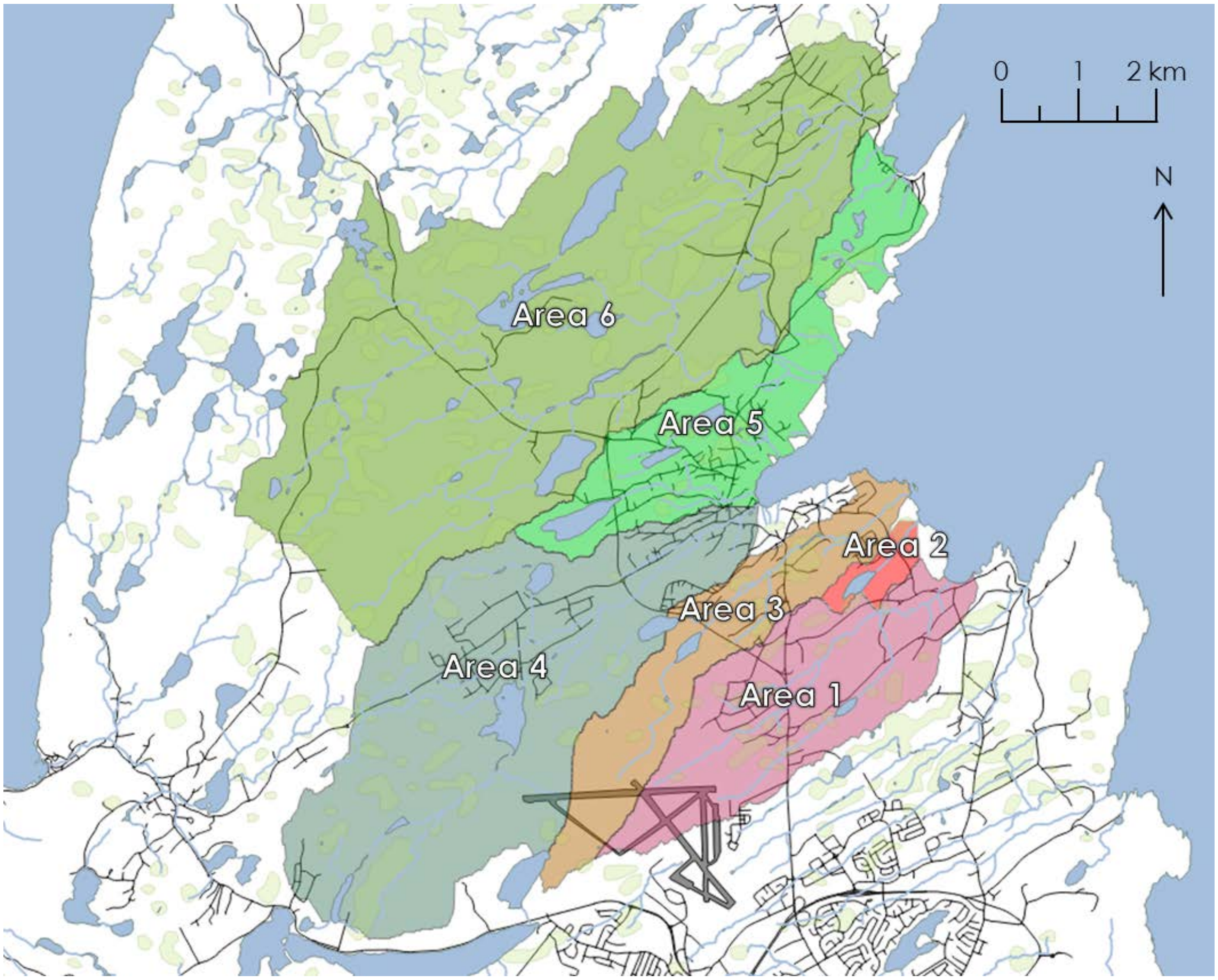

Figure 3 **Six surface water catchment areas defined within the Study Area for the purpose of baseflow estimation.**

Table 2 Baseflow Estimates for Surface Water Catchment Areas Defined within the Study Area

Catchment Area	Main Watercourse	Area (km^2)	Estimated Baseflow (m^3/d)[1]
1	Kennedys Brook	7.8	6,589
2	Jones Pond Brook	0.7	607
3	North Pond Brook	5.9	4,999
4	Island Pond Brook	17.7	14,820
5	various	7.5	6,338
6	Big River	32.3	26,869
	TOTAL	71.9	60,222

Note:
[1] Estimated using the power curve fit function in Figure 2.

3.2.2 Drilled Water Well Records

Drilled water well records were provided in database format by the Department of Environment and Conservation. These records include wells drilled between 1978 and 2014. More recent well records have a Global Positioning System (GPS) coordinate for the location of the wellhead; while older well logs often rely on civic addresses to locate the wells. Plotting the well locations on a map, identified a number of errors in the reported well log coordinates.

A total of 988 records were reviewed for this study, of which Stantec's review yielded a total of 531 records with useable spatial coordinates. Unfortunately, many data fields were blank in the 531 water well data set, and data for the parameters discussed below were not always available for each well.

The ground surface elevation for each well location was extracted from the digital elevation map (DEM) for the 1N10 1:50,000-scale topographic map sheet sourced from Natural Resources Canada. Depth measurements reported in the records were converted to elevations relative to mean sea level.

It should be noted that the provincial drilled water well record database does not include dug overburden wells, which are used in some locations for drinking water supply within the Study Area.

3.2.2.1 Depth to Bedrock

Depth to bedrock (or overburden thickness) was obtained from 451 records. The mean depth from surface to rock was 4.3 m (ranging from 0 m to 14 m). Figure 4 shows that a linear relationship is present between ground surface elevation and bedrock surface elevation. Based on the relationship, the overburden is thinner at higher elevations and thicker in topographic lows, as would be expected.

3.2.2.2 Casing Length

Casing length was obtained from 472 well records. Casing length ranges from 4.87 m to 21 m with an average of 9 m. Figure 5 shows that a linear relationship is present between ground surface elevation and the bottom of casing elevation.

3.2.2.3 Well Depth

Well depth was obtained from 526 records. Well depths range between 15 m and 170 m with an average of 87 m. This is similar to the Department of Environment and Conservation's (2014) reported average depth of 73 m for wells drilled in Torbay based on 622 records. There is no relationship between ground surface elevation and well depth (Figure 6). Wells are typically drilled until enough water-bearing features have been intersected to supply domestic water use demands.

The large range noted in well depth is consistent with what is expected in a sparsely fractured bedrock aquifer where fracture orientations are inclined.

3.2.2.4 Well Yield

The Department of Environment and Conservation (2014) reports the average well yield is 14.42 L/min. This estimate is approximately half of the average yield (27 L/min, range 1 – 454 L/min) reported for a much larger scale on the Avalon Peninsula (Newfoundland Department of Environment and Lands 1988).

3.2.2.5 Static Water Level

Static water levels were obtained from 110 records. Values ranged from 1 m to 106.7 m below ground surface. It is expected that four values (14 m, 18 m, 46 m and 106.7 m below ground surface) are erroneous. An improper measurement of the static water level occurs when taken before the water level in the well stabilizes.

Static water levels were also obtained from various Level II Groundwater Supply Assessments completed for unserviced residential developments in the Torbay area. Static water levels ranged from 14.79 m to "flowing" (i.e., the water level is above the top of the casing) based on 28 records.

Figure 7 shows a linear relationship between ground surface elevation and static water level (converted to an elevation above mean sea level). The trend line shows that the static water level elevation is generally 97% of the ground surface elevation.

3.2.2.6 Water-Bearing Zones

Of the 531 water well records analyzed, only 73% had water-bearing zones reported. Of the records with water-bearing zones reported, 63% only had one zone, 31% had two zones, 5% had three zones and 1% had four zones.

Figure 8 shows poor correlation between ground surface elevation and the elevation of the uppermost water-bearing feature.

3.2.2.7 Available Drawdown

Figure 9 is a schematic illustrating the concept of available drawdown. As discussed in the previous section, there are relatively few water-bearing features intersecting the typical well in the Study Area. Available drawdown is the distance between the static water level and the water-bearing feature within the well. The concept is that this is the distance the water level can drop during pumping (drawdown) before the water level is below the feature. This is an adverse condition because the feature can de-water and no longer contribute to flow into the well.

Many of the water well records do not report both the static water level and the position of water-bearing features. In these cases, the static water level was estimated for the purpose of the calculation using the relationship with ground surface elevation shown in Figure 7. Figure 10 provides a histogram of available drawdown based on the elevation of the static water level and the elevation of the uppermost feature identified in a record.

The range of available drawdown is essentially the same as the large range noted in the elevation of water-bearing features and well depth. No patterns were established when available drawdown was sorted by development within the Study Area. Two wells in close proximity might have a large difference in available drawdown even if the same water-bearing fracture intersects both if its orientation is inclined.

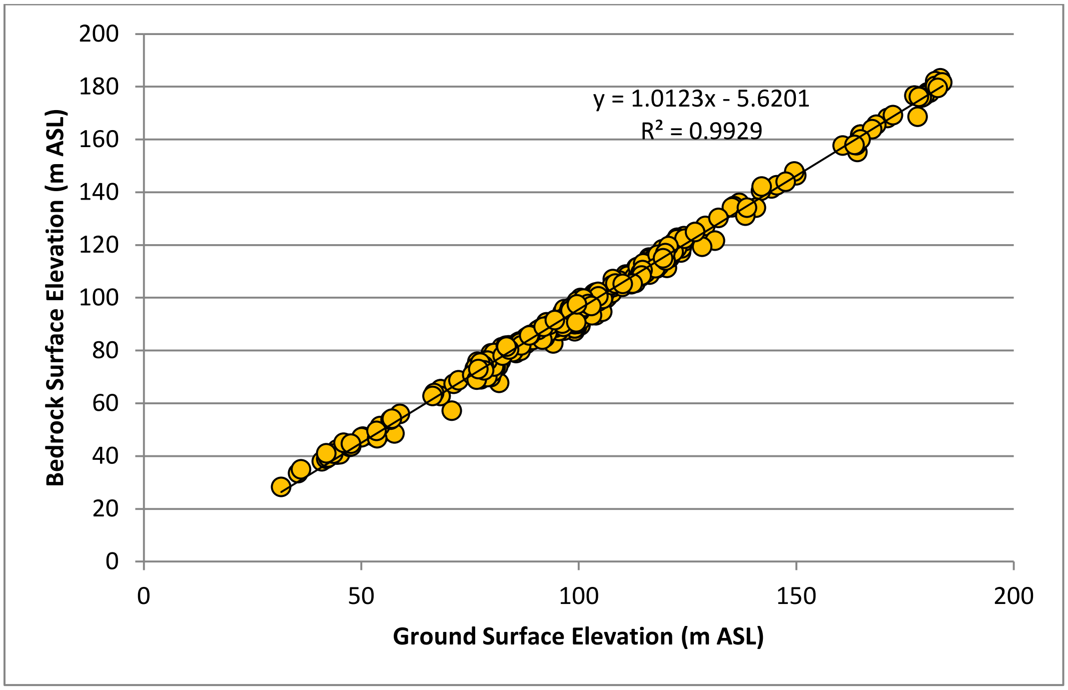

Figure 4 **Bedrock surface elevation from well records in the Study Area.**

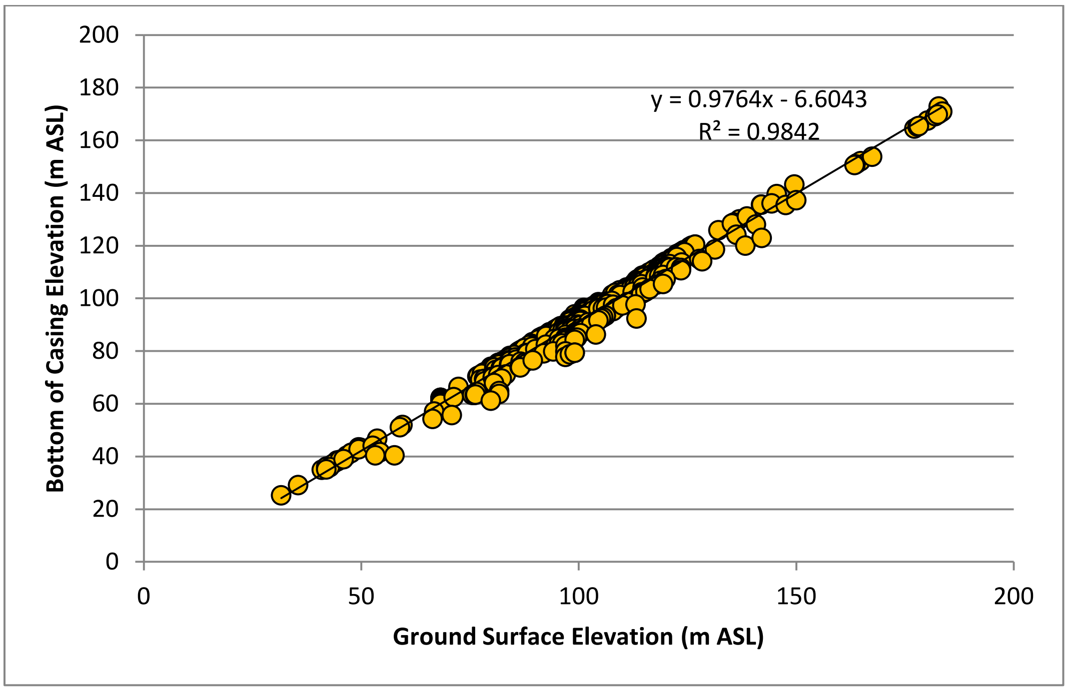

Figure 5 **Bottom of casing elevation from well records with the Study Area.**

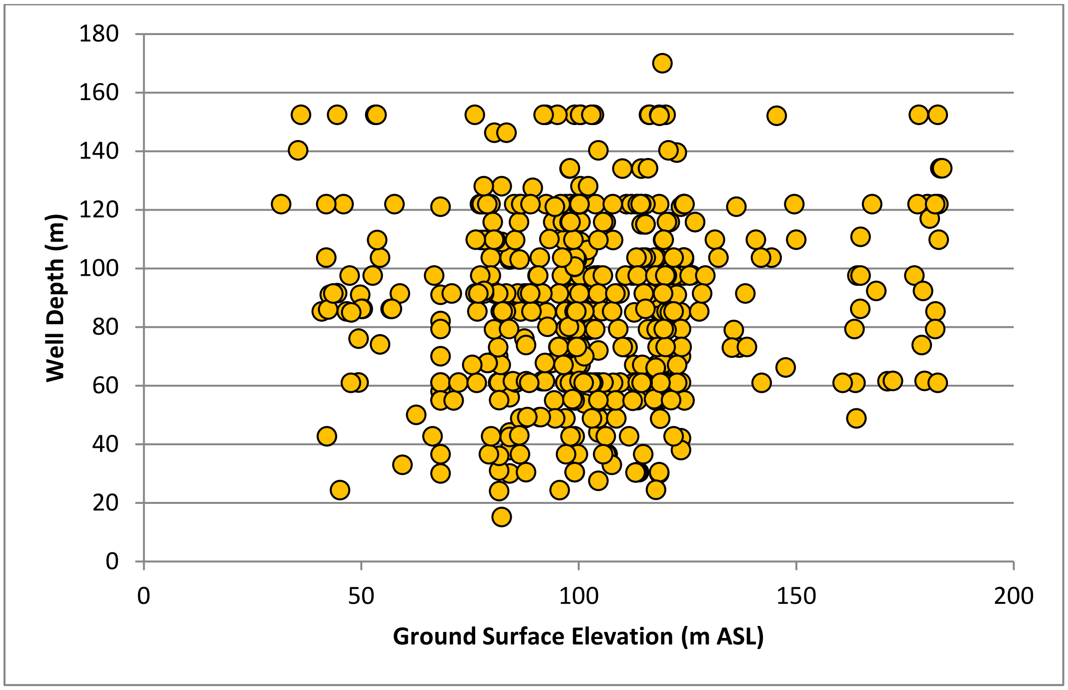

Figure 6 Comparison of well depth to ground surface from well records in the Study Area.

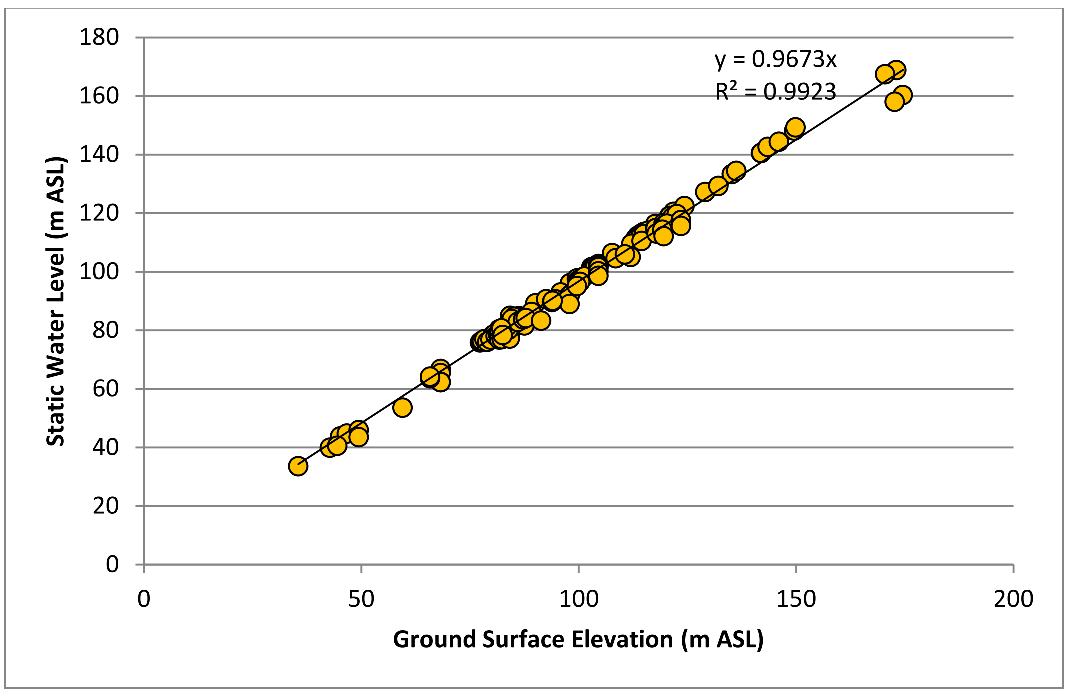

Figure 7 Comparison of static water level to ground surface from well records in the Study Area.

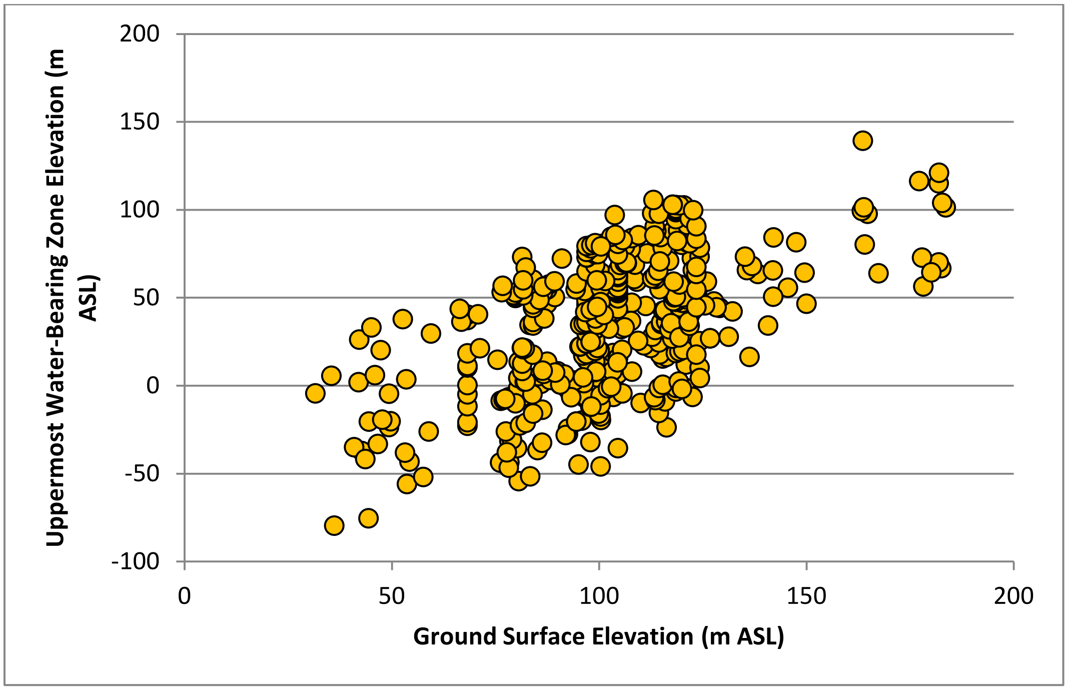

Figure 8 Comparison of elevation of uppermost water-bearing zone to ground surface from well records in the Study Area.

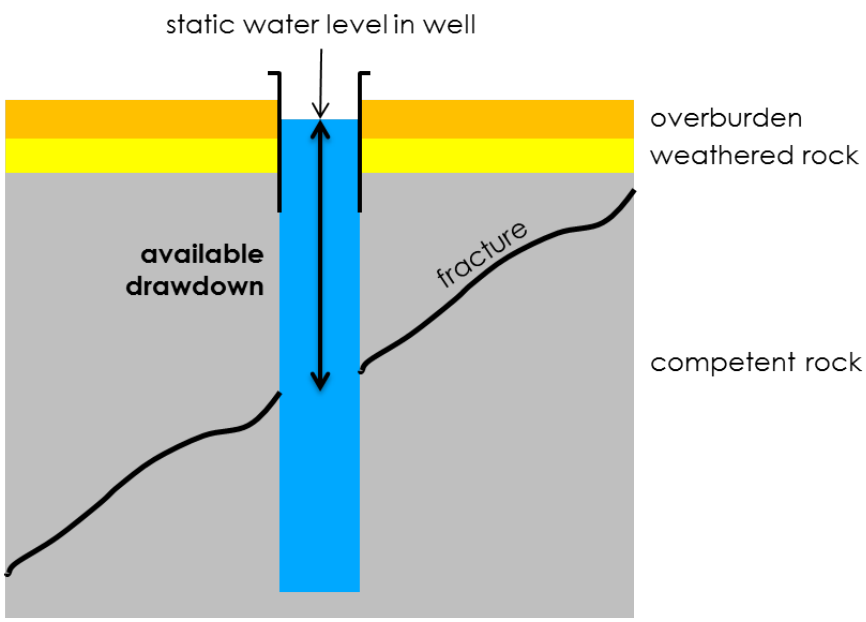

Figure 9 Available drawdown in a drilled bedrock well. The inclined fracture is the only water-bearing feature in the well. Available drawdown is the length between the static water level and the fracture.

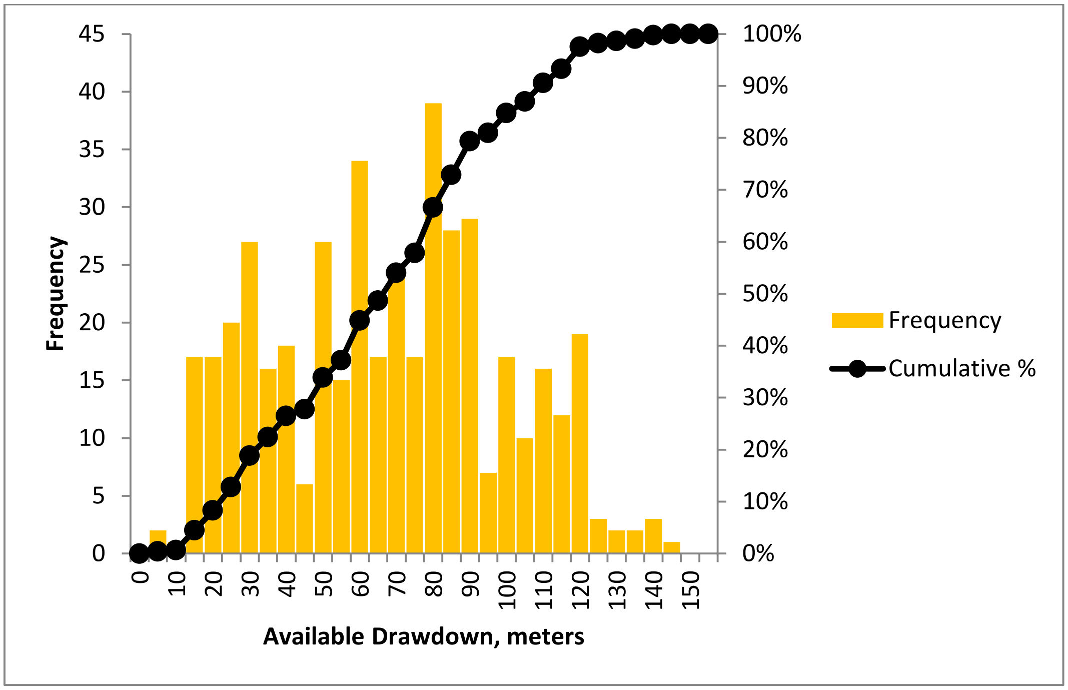

Figure 10 **Histogram of calculated available drawdown from records in the Study Area. For the purpose of this study, available drawdown is measured as the distance between the static water level and the uppermost water-bearing feature reported.**

3.2.3 Aquifer Testing

Several Level I and Level II Groundwater Supply Assessments have been conducted as part of sub-division development within the Study Area. Level II assessments include constant rate pumping tests and step drawdown tests to evaluate the hydraulic properties of the aquifer and test well, respectively. Hydraulic properties are estimated by interpreting the drawdown and recovery data collected during each test in the pumping well and observation well(s). The interpretation is made with simplified analytical solutions which yield interpreted values of horizontal transmissivity and, in the case of pumping test with a pumping well and an observation well, aquifer storativity.

The constant rate, multi-well pumping tests (pumping well and observation well(s)) are most useful for the purpose of this study because the interpreted transmissivity is more representative of the aquifer. Table 3 provides a summary of results from seven (7) Level II assessments conducted within the Study Area. The reported aquifer transmissivity from each interpreted test was converted to an aquifer hydraulic conductivity by dividing the transmissivity by the aquifer thickness (the vertical distance between the bottom of the casing and the bottom of the well). If the drawdown/recovery for a well was interpreted with more than one analytical solution (e.g., Cooper-Jacob, Theis, residual recovery methods), the geometric mean of the hydraulic conductivity was calculated. A representative hydraulic conductivity for each sub-division was calculated by taking the geometric mean of the hydraulic conductivity for each well (pumping or observation) and is the value reported in Table 3.

The estimated hydraulic conductivity for the bedrock zone in which the typical open portion of a residential well is located ranges from 2.9×10^{-3} m/d (Eagle Nest Ridge) to 5.3×10^{-1} m/d (Quarry Road) with a geometric mean of 1.8×10^{-2} m/d. This range fits well within the hydraulic conductivity range expected for fractured sedimentary and metasedimentary rocks (approximately 1×10^{-5} m/d to 10 m/d reported in Freeze and Cherry (1979)). Based on Stantec's experience, the geometric mean from these assessments is also comparable to the geometric mean of 2.2×10^{-2} m/d obtained from hundreds of single well pumping tests conducted in similar geologic/hydrogeologic terrain in Nova Scotia.

exp Services Inc. (2012) drilled six test wells in their assessment of the Venice Holdings/Gibraltar Development Subdivision, located in the community of Logy Bay – Middle Cove, along the southeast boundary of the Study Area. The objective of the well configuration and constant rate testing was to quantify the horizontal anisotropy in hydraulic conductivity due to north-south oriented lithologic and structural constraints. The outcome failed to quantify anisotropy because no drawdown was observed in any of the observation wells.

None of the Level II Groundwater Supply Assessments completed within the Study Area have included testing to evaluate the hydraulic properties of surficial material or the upper zone of the bedrock, which is weathered from surface processes and glacial events.

Table 3 Compilation of Aquifer Horizontal Hydraulic Conductivity Estimated from Constant Rate Pumping Tests

Location	Geometric Mean of Horizontal Hydraulic Conductivity (m/d)	Source
Eagle Nest Ridge	2.9×10^{-3}	Stantec Consulting Ltd. (2013a)
Logy Bay	5.3×10^{-3}	exp Services Inc. (2012)
Martin's Meadows	2.7×10^{-2}	Stantec Consulting Ltd. (2013b)
Outer Cove	4.8×10^{-3}	exp Services Inc. (2014)
Pine Ridge	4.6×10^{-2}	Stantec Consulting Ltd. (2013c)
Quarry Road	5.3×10^{-1}	Stantec Consulting Ltd. (2011)
Scenic View Ridge	1.2×10^{-2}	Stantec Consulting Ltd. (2013d)
GEOMETRIC MEAN	1.8×10^{-2}	

3.3 Hydrostratigraphic Framework

The hydrostratigraphy of the Study Area is generally conceptualized as a three-layer (unit) system consisting of a layer of unconsolidated deposits including glacial till and organics (referred to collectively as "overburden"), underlain by weathered and competent bedrock.

Given the limited hydrogeological information available for the area, a conceptual model using homogeneous properties for each layer is considered appropriate. Vertical anisotropy is inferred by the nature of the mapped geologic structure, weathering, and depositional environment.

3.3.1 Overburden

Based on Drilled Well records and working knowledge of the area, the glacial till is poorly sorted with particle sizes ranging from clay to gravel. It ranges in thickness from 0 m to 14 m with an average of 4.3 m (Section 3.2.2.1), and is generally thinner at higher elevations and thicker at lower elevations.

Given the nature of deposition, the properties of the till are expected to be homogenous and isotropic. The hydraulic conductivity of the overburden has not been directly measured. A range on the order of 0.0432 m/d to 4.32 m/d is expected based on the type of geologic material and literature values (e.g., Freeze and Cherry 1979).

3.3.2 Bedrock

As outlined in Section 2.4, the Study Area is predominantly underlain by sequences of metamorphosed coarse- to fine-grained clastic sedimentary rocks that are complexly faulted and folded. The orientation of geologic structure does vary but is often inclined, and is expected to influence groundwater flow.

Field aquifer tests have only been conducted in competent rock. As discussed in Section 3.2.2.7 and summarized in Table 3, the geometric mean of horizontal hydraulic conductivity estimated from these tests is 1.8×10^{-2} m/d. This is comparable to the geometric mean of 2.2×10^{-2} m/d compiled by Stantec from hundreds of single well pumping tests conducted in similar terrain in Atlantic Canada. These values are likely lower than what would be expected at the regional scale.

Vertical hydraulic conductivity has not been measured in the field setting. It is expected that the vertical hydraulic conductivity could be up to 100 times greater than horizontal hydraulic conductivity given the inclined orientation of the structural fabric.

The upper portion of bedrock is known to be weathered in this setting. The thickness of the weathered zone has not been quantified. Glacial loading and unloading and other surface processes are expected to have induced additional horizontal fracturing in the weathered rock compared to the underlying competent rock resulting in more isotropic hydraulic properties. The hydraulic conductivity of the weathered rock has not been measured but is expected to be on the order of an order of magnitude higher than competent bedrock and close to isotropic.

3.4 Groundwater Flow System

Local groundwater flow directions and gradients are expected to vary around Torbay due to topography and the presence of numerous watercourses. In general, groundwater flow is expected to closely follow topography and flow northeast towards the Atlantic Ocean. Groundwater divides are expected to mimic surface water divides.

Local groundwater flow systems are expected within the overburden and weathered bedrock with recharge occurring at topographic highs and discharge occurring at adjacent topographic lows into streams as baseflow. Vertical hydraulic gradients between the overburden and deeper competent bedrock have not been quantified due to a lack of information. It is expected that the bedrock system is semi-confined by the overburden and that flow is locally controlled by the orientation and connectivity of extension and shear fractures associated numerous geologic processes such as with regional deformation, regional stress fields, erosional unloading, glacial loading/unloading. These discrete structural controls are expected to be adequately connected on the larger scale to allow an “equivalent porous medium” approach to be used in the simulation of regional groundwater flow.

3.5 Groundwater Sources/Sinks

In three-dimensional groundwater flow models, the source/sink terms are used to describe water flowing in (source) or out (sink) of the system, and are represented as positive or negative volumes of water per volume of the porous medium, respectively.

3.5.1 Groundwater Recharge

Groundwater recharge estimates from baseflow separation (Section 3.2.1) are on the order of 305 mm/yr (20% of total annual recharge).

3.5.2 Pond and Stream Levels

There are no hydrometric stations to measure stream flow or level within the Study Area. Assumptions pertaining to stream geometry are outlined in Section 4.3.4.1.

Pond levels were obtained from the DEM. No water depth or bathymetry information was available.

3.5.3 Residential Wells

The number of existing houses using private wells was estimated from Google Earth imagery (as of September 9, 2014) and subdivision lot layout plans in areas not serviced by municipal water according to the Servicing Plan provided by the Town. A total of 2,073 homes were identified within the Study Area, and were each assigned an assumed daily household demand of 1,360 L (i.e., estimated daily demand for a 4-person home (NLDEC-WRMD, November 2009). The Jack Byrne Arena is also included with an estimated average daily usage of 34,200 L (Fracflow Consultants Inc. 2008).

4.0 GROUNDWATER FLOW MODEL CONSTRUCTION

A numerical groundwater flow model is a simplified representation of a groundwater system that divides space and/or time into discrete pieces and is a set of mathematical equations that describe and approximation the physical processes and boundaries of a groundwater system (after Barnett et al. 2012).

The primary tasks involved in developing the groundwater flow model for the Town of Torbay included:

1. Identifying a suitable computer code
2. Selecting the vertical and horizontal extent of the model domain
3. Constructing a finite-difference grid for the model domain
4. Overlaying the hydrostratigraphy onto the finite difference grid
5. Assigning boundary conditions within the model domain
6. Specifying hydraulic property values for each stratigraphic unit or layer

The following sections describe these tasks in more detail.

4.1 Model and Graphical User Interface Selection

MODFLOW was chosen as the numerical groundwater-software application for this evaluation because it is considered an international standard for simulating and predicting groundwater flow. The code and a variety of utilities are available for free through the U.S. Geological Survey (USGS) at: http://water.usgs.gov/ogw/modflow/.

The version of MODFLOW used in this study was MODFLOW-NWT (Niswonger, et al, 2011), which is the Newton-Raphson formulation for MODFLOW-2005. This particular version was chosen because it is numerically stable and able to quickly converge on a steady-state flow solution.

Groundwater Vistas (Environmental Simulations International 2014) was chosen as the graphical user interface with MODFLOW-NWT. Groundwater Vistas is a pre- and post-processor for MODFLOW models and other technologies for sensitivity analysis and model calibration. Groundwater Vistas writes the input files in native MODFLOW format, which can be readily imported into other graphical user interfaces (such as the USGS's ModelMuse which is available for free) and can be run directly using USGS versions of the MODFLOW executable files.

4.2 Model Domain

4.2.1 Delineating the Study Area

As mentioned previously in Section 2.2, the Study Area was defined by combining watersheds that encompass the Town Limit and include the catchment areas for Kennedys Brook, Island Pond Brook and Big River (Figure 3). This was conducted using the Watershed Layer function in Surfer 12 (Golden Software 2015). Digital elevation map (DEM) raster data for the 1N10 1:50,000 topographic map sheet was obtained from the Natural Resources Canada and imported into Surfer to create a grid.

It is assumed that the surface water watershed that defines the limits of the Study Area coincides with the underlying flow boundaries of the groundwater system.

4.2.2 Model Grid

A model grid was constructed to fully encapsulate the Study Area. The grid is composed of 99 rows (uniform row spacing of 100 m) and 132 columns (uniform column spacing of 100 m). Grid cells located outside of the Study Area are designated "inactive." The total active area of the model is 76.19 km^2.

The grid is rotated by 30° to align the northeast-trending physical features of the natural environment with the x-direction of the model grid.

The model was discretized into four (4) model layers using the hydrostratigraphic units presented in Figure 11. Competent bedrock is divided into two layers (layers 3 and 4) based on the elevation of the bottom of residential well casings as reported in the drilled well records. Layer 4 represents the open borehole zone in competent rock. The equations defining the Bedrock Surface Elevation and Bottom of Casing Elevation come from Figure 4 and Figure 5, respectively.

The model grid forms a total of 52,272 cells, of which 30,476 are active.

4.3 Flow Model Boundary Conditions

Following the construction of the three-dimensional model grid, flow boundary conditions were applied. Specified head, no-flow, general head and source/sink boundary conditions were applied to represent the groundwater flow divide around the land perimeter of the Study Area, the ocean boundary, ponds, streams, and residential pumping wells.

4.3.1 Specified Head Boundary

A specified head boundary allows the head to be fixed in a cell. A specified head of 0 masl is assigned to active coastal cells in all layers to represent the Atlantic Ocean (Figure 12).

4.3.2 No-Flow Boundary

A watershed boundary is by definition a surface water flow divide. It is assumed that the groundwater system mimics the surface water system on this scale. Therefore, the land perimeter of the Study Area is inferred to be the flow divide for the groundwater watershed and is represented by a no-flow boundary in all four model layers (boundary between active and inactive cells in Figure 12). The bottom of the model domain is also a no-flow boundary condition.

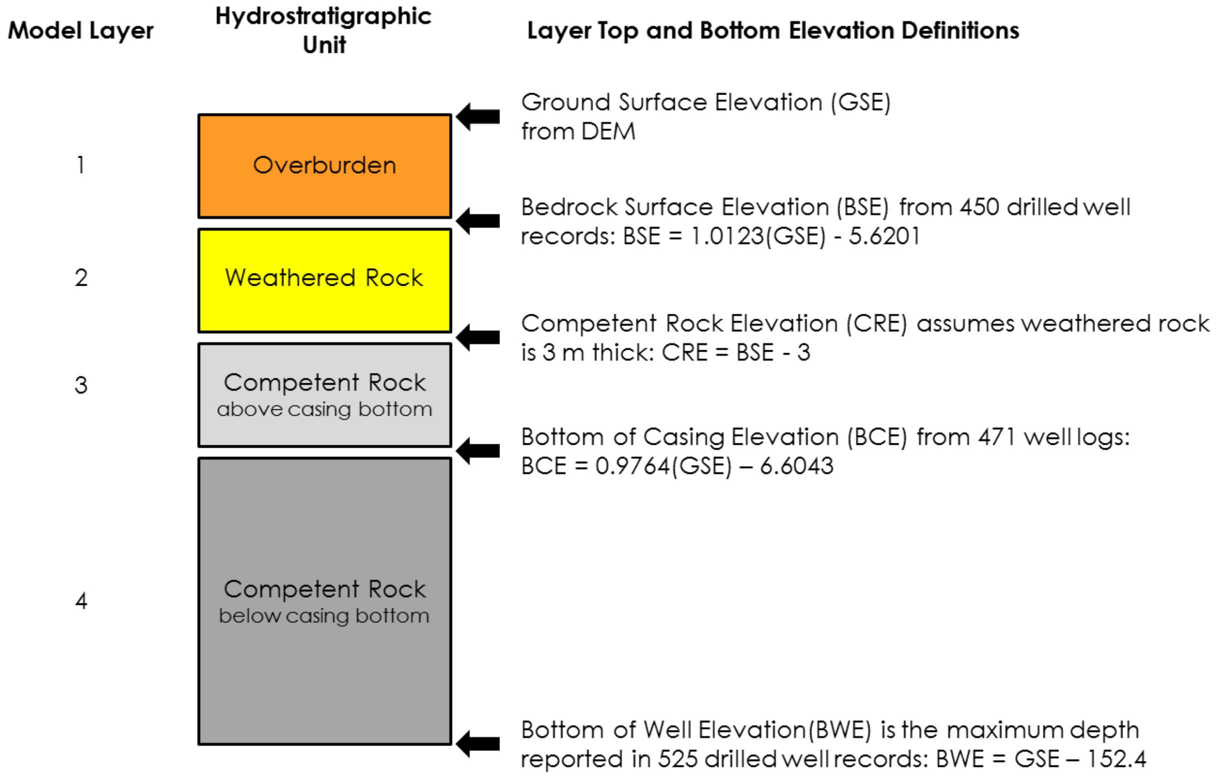

Figure 11 Model layer top and bottom elevation definitions.

4.3.3 General Head Boundary

A general head boundary is a form of head-dependent flux boundary where a reference head and a conductance are specified. If the modelled head in the cell is equal to the reference head the flux into the groundwater system is zero. If the modelled head in the cell is greater than the reference head, water leaves the groundwater domain through the general head boundary. The relationship between flux and head is linear.

General head boundaries are assigned in layer 1 to represent 32 ponds within the Study Area (Figure 13). The reference head for each pond was obtained from the DEM, which captures the surface elevation of each pond to the nearest metre. The conductance term is arbitrarily set to a higher value of 10,000 m^2/d, which results in flow not being restricted in and out of the groundwater system.

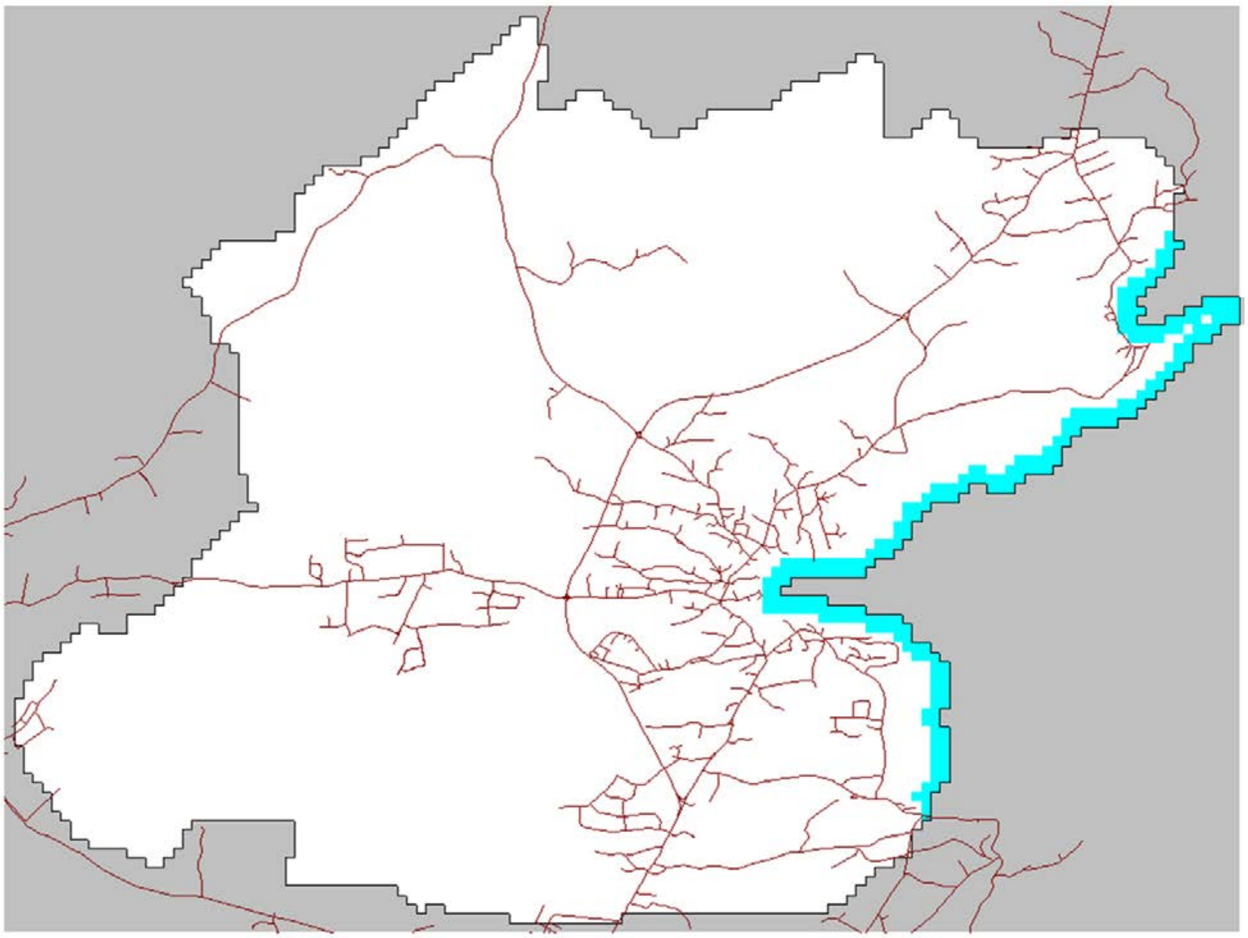

Figure 12 Plan view of constant head boundaries (blue) defined in cells along the coast (layers 1 to 4) in the active model domain (white). Grey cells are inactive. The road network is shown (brown lines).

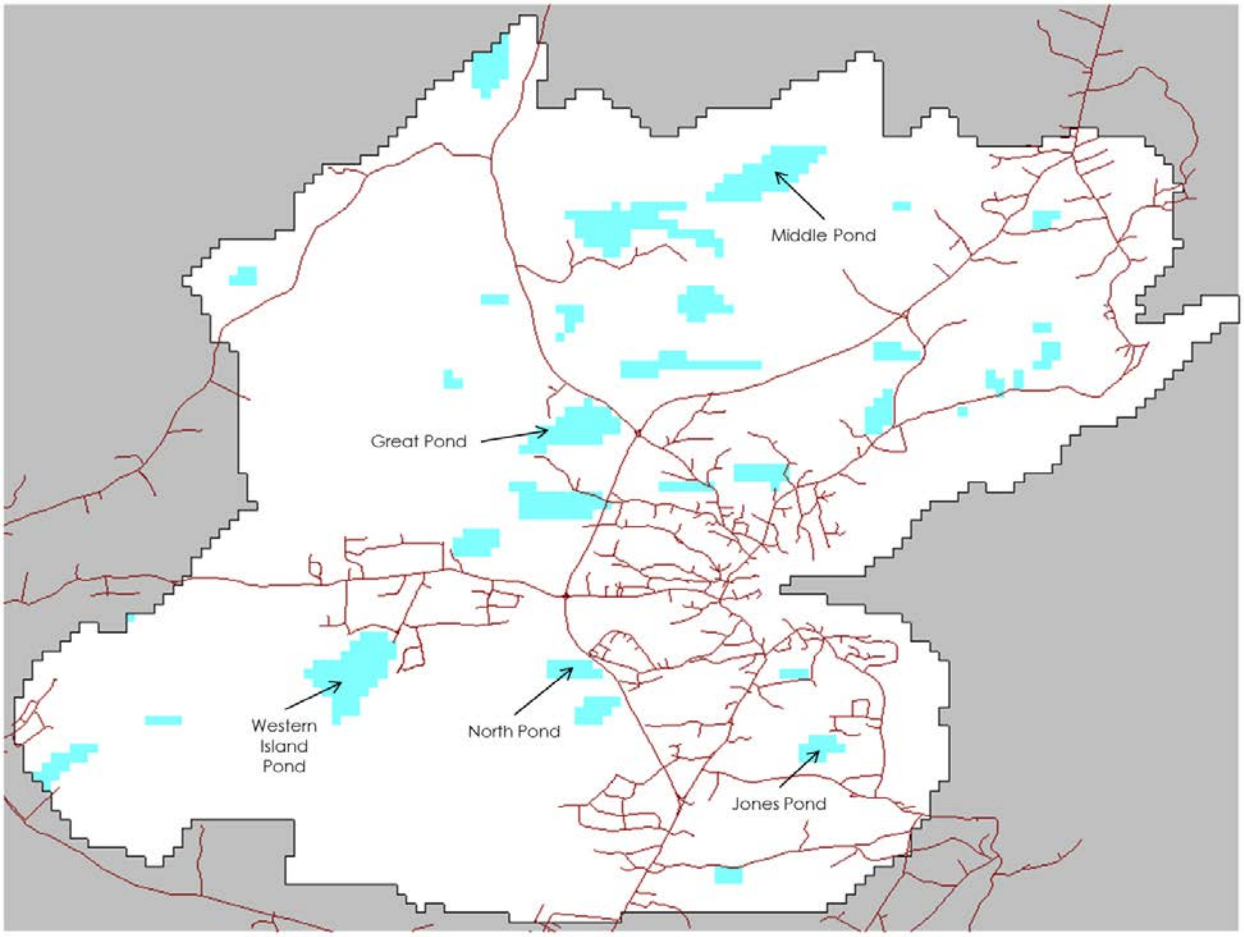

Figure 13 Plan view of general head boundaries (blue) assigned in layer 1 to represent ponds within the Study Area.

4.3.4 Sources and Sinks

4.3.4.1 Streams

River boundary conditions were assigned in layer 1 to represent numerous stream segments within the Study Area (Figure 14). This is a head-dependent flux boundary condition. Flow into or out of the groundwater system is dependent on the assigned head (stage) and the conductance of the riverbed. If the simulated head is higher than specified stage, water is removed from the groundwater system. If the simulated head is lower than the stage but higher than the bottom elevation of the river, water enters the groundwater system. No gain or loss occurs if the simulated head is below the bottom elevation of the river bed.

Stream segments were assigned an order based on how they connect moving downstream. Streams starting at the watershed boundary are designated first-order streams. When two first-order streams meet, the large downstream segment past where they converge becomes second-order. If a first-order stream flows into a second-order stream, the segment downstream of the join remains second-order. Thus, the order of a stream remains the same until it joins with a higher-order stream. The order of the stream increases by one past the point where two streams of equal order meet.

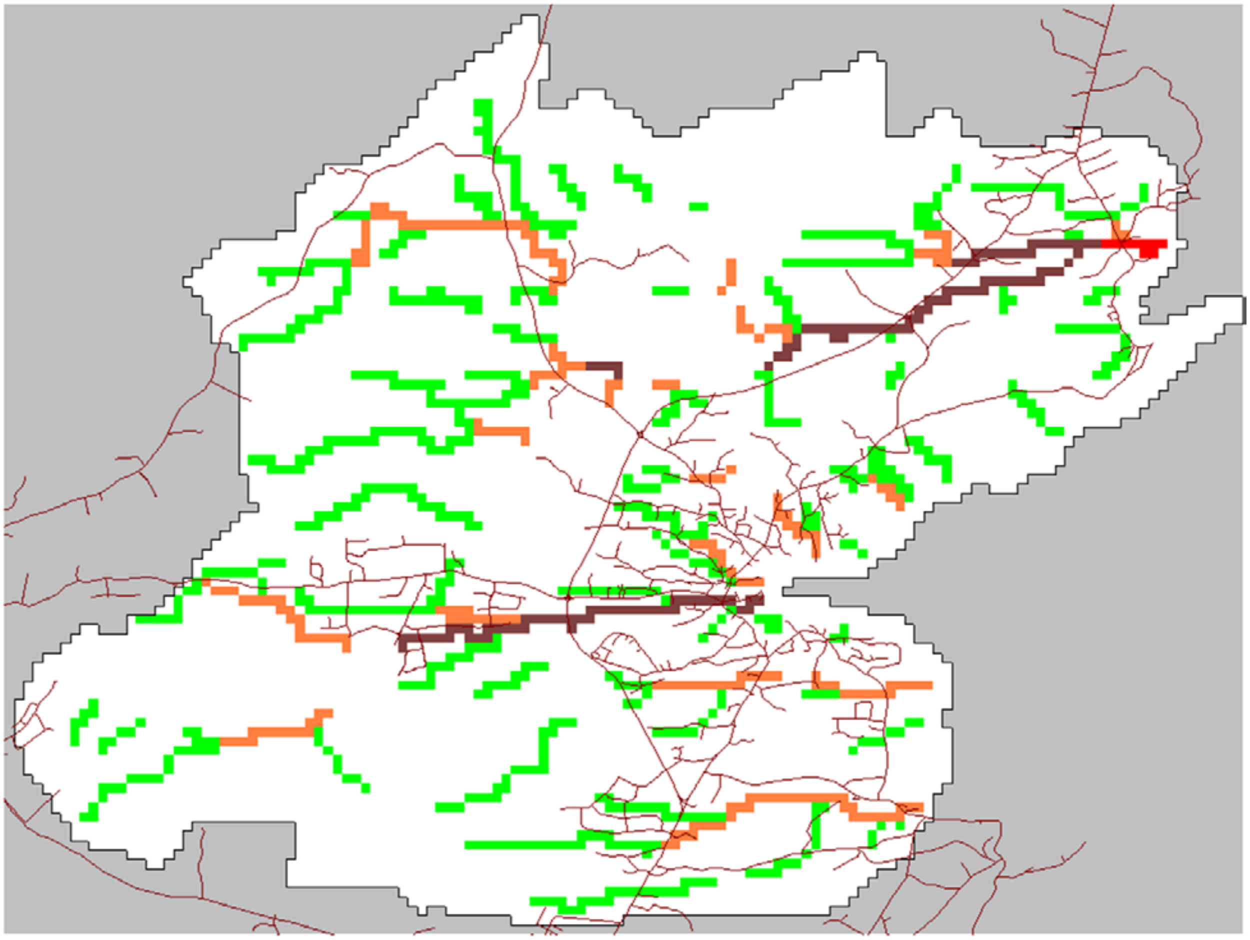

Figure 14 Plan view of river boundary conditions assigned in layer 1 to represent streams within the Study Area. Green – 1st order, orange – 2nd order, brown = 3rd order, red = 4th order.

Table 4 summarizes the characteristics of each order specified in the model. These values are assumed in the absence of field data. The hydraulic conductivity and thickness of the river bed are arbitrarily set as to not restrict flow in or out of the groundwater domain through a river boundary. This means that the properties of the aquifer control the flow rate.

Table 4 Prescribed Characteristics for River Boundaries in Layer 1

Order	Number of Segments	Stage	Bottom Elevation	Width
1	73	Top Elevation of Cell in Layer 1 from DEM	Top Elevation minus 0.2 m	1 m
2	19		Top Elevation minus 0.5 m	5 m
3	7		Top Elevation minus 1 m	10 m
4	1		Top Elevation minus 2 m	15 m

4.3.4.2 Residential Wells

Well boundary conditions are applied to select cells in layer 4 (the layer that represents the open portion of the borehole in competent bedrock).

Residential wells are divided into 17 development areas within the Study Area (Figure 15). The road network and distribution of houses are used to determine which cells were assigned a well boundary condition. The total number of houses within a development is used to calculate the total daily water usage. The total daily pumping rate is divided by the number of cells selected for the well boundary condition to determine the daily pumping rate per cell for a particular development. This pumping rate typically represents the demands of two to four houses.

4.4 Hydraulic Parameters

Property zones for hydraulic conductivity were constructed using the values outlined in the conceptual model (Section 1.1) and the defined model layers (Figure 11). Figure 16 shows where the weathered rock property zone (layer 2) is extended into layer 1 for the case where bare or concealed bedrock is mapped at surface based on Batterson (2000).

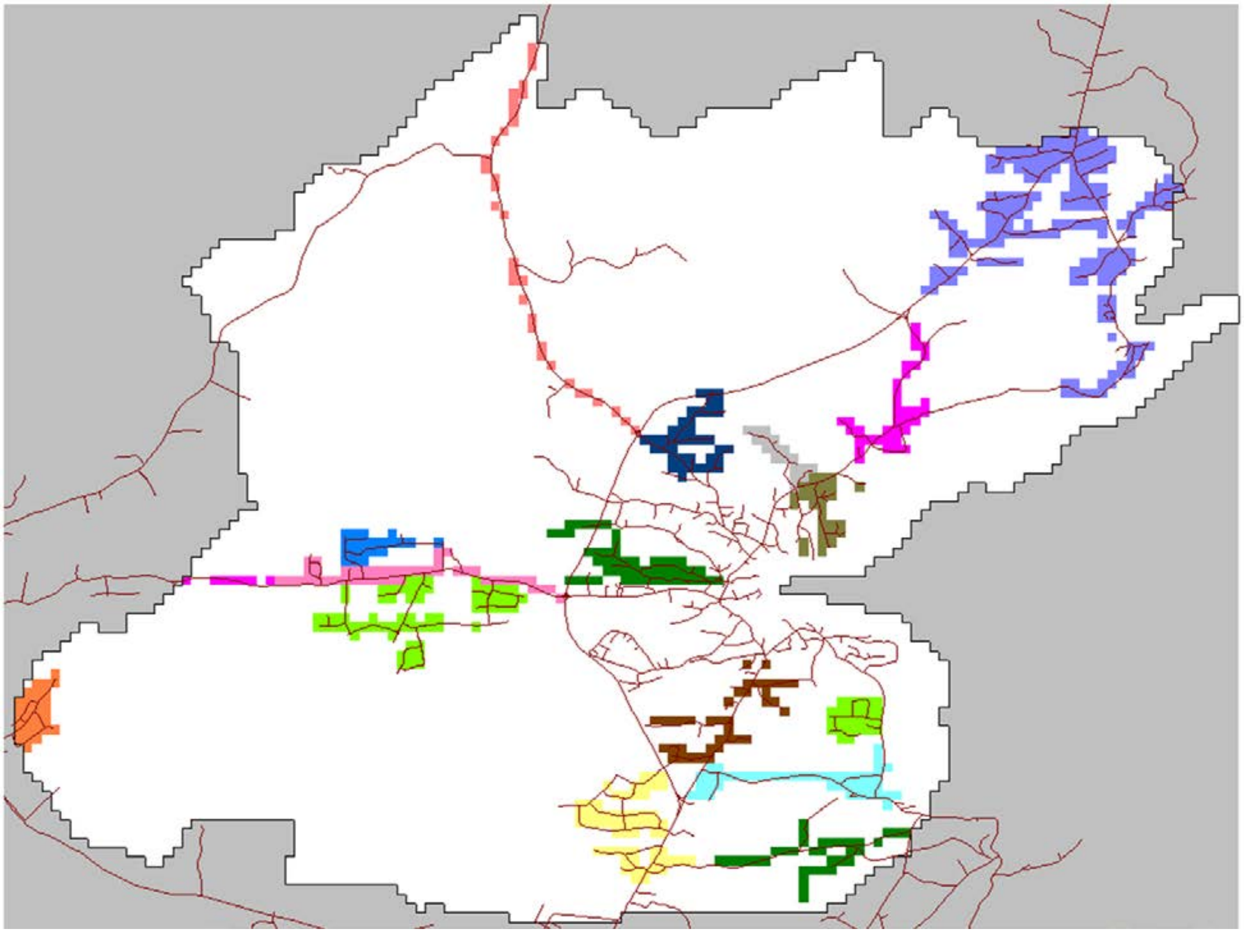

Figure 15 Plan view of pumping well boundary conditions assigned to cells in layer 4. Each coloured cluster represents a particular unserviced sub-division or area of development.

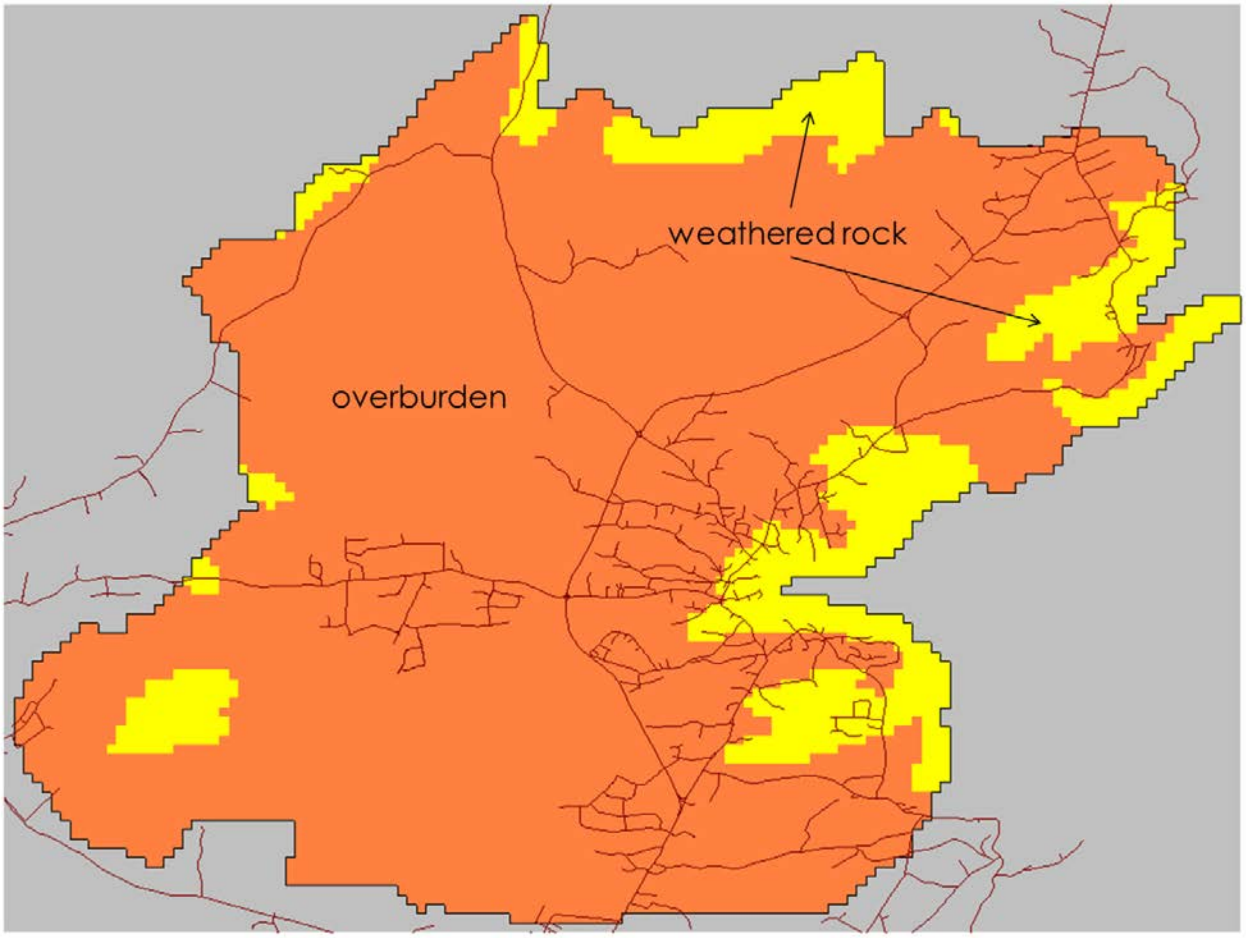

Figure 16 Property zones in layer 1. Weathered rock (yellow) extends from layer 2 where bare or concealed bedrock has been mapped at surface.

5.0 MODEL CALIBRATION

The goal of the current study is to create a numerical model that captures the behaviour of the groundwater flow system within which Torbay is situated. The conceptual model establishes the general framework of how the system is thought to work and what processes are relevant to larger-scale, steady-state groundwater flow in this setting. This includes consideration for boundary conditions, and parameter values and their potential range of uncertainty.

The next step is to see how well the numerical model performs in the task of simulating hydraulic head and groundwater flow compared to real world observations. This is carried out through a process of model calibration, whereby model input parameters values are adjusted within their defined potential range of uncertainty to minimize the difference between calculated and observed data.

Model calibration can be done manually using a trial-and-error approach. However, this approach can be quite tedious and time consuming. Alternatively, Model-Independent Parameter Estimation and Uncertainty Analysis (PEST) (Watermark Numerical Computing 2005) was used in the current study to calibrate the constructed groundwater flow model. PEST is a parameter estimation model calibration tool that can interface with MODFLOW through Groundwater Vistas. Like the trial-and-error approach, PEST runs the model and compares simulated results with calibration targets (head and flow observed in the real world) but does so in an automated fashion. Again, PEST can only vary parameter values within the range allowed by the user.

PEST finishes operating once it establishes what combination of parameter values provides the best match to calibration targets.

5.1 Specification of Calibration Targets

Both head and flow targets were used to calibrate the steady-state groundwater flow model.

A total of 132 head targets in layer 4 were identified within the Study Area. Of these 105 are static water levels reported in drilled well records between 1985 and 2012. The remaining 27 are static water levels in monitoring wells reported as part of Level II Groundwater Assessments conducted between 2012 and 2014. The number of head targets was reduced to 83 using the target thinning option in Groundwater Vistas (Figure 17). This option allows only one target per cell and it was set to retain the value closet to the mean.

Six flow targets were defined based on estimated stream baseflow (the component of stream flow that is from the groundwater system) for a given catchment area (Figure 3). Baseflow targets for the six defined catchment areas are summarized in Table 2.

A computer code was written to read the cell-by-cell flow file generated by MODFLOW, extract the flow information at each cell with a river boundary condition (for streams) or general head boundary condition (for ponds), and sum the flows to/from the river and general head boundaries for cells in each of the defined catchment areas. This sum is equal to the baseflow with negative values indicating that water is leaving the groundwater system. PEST was configured to run this computer code after each MODFLOW model run during the calibration process.

Head and flow calibration targets are weighted so that the residuals (difference between the simulated and target values) are of similar magnitude.

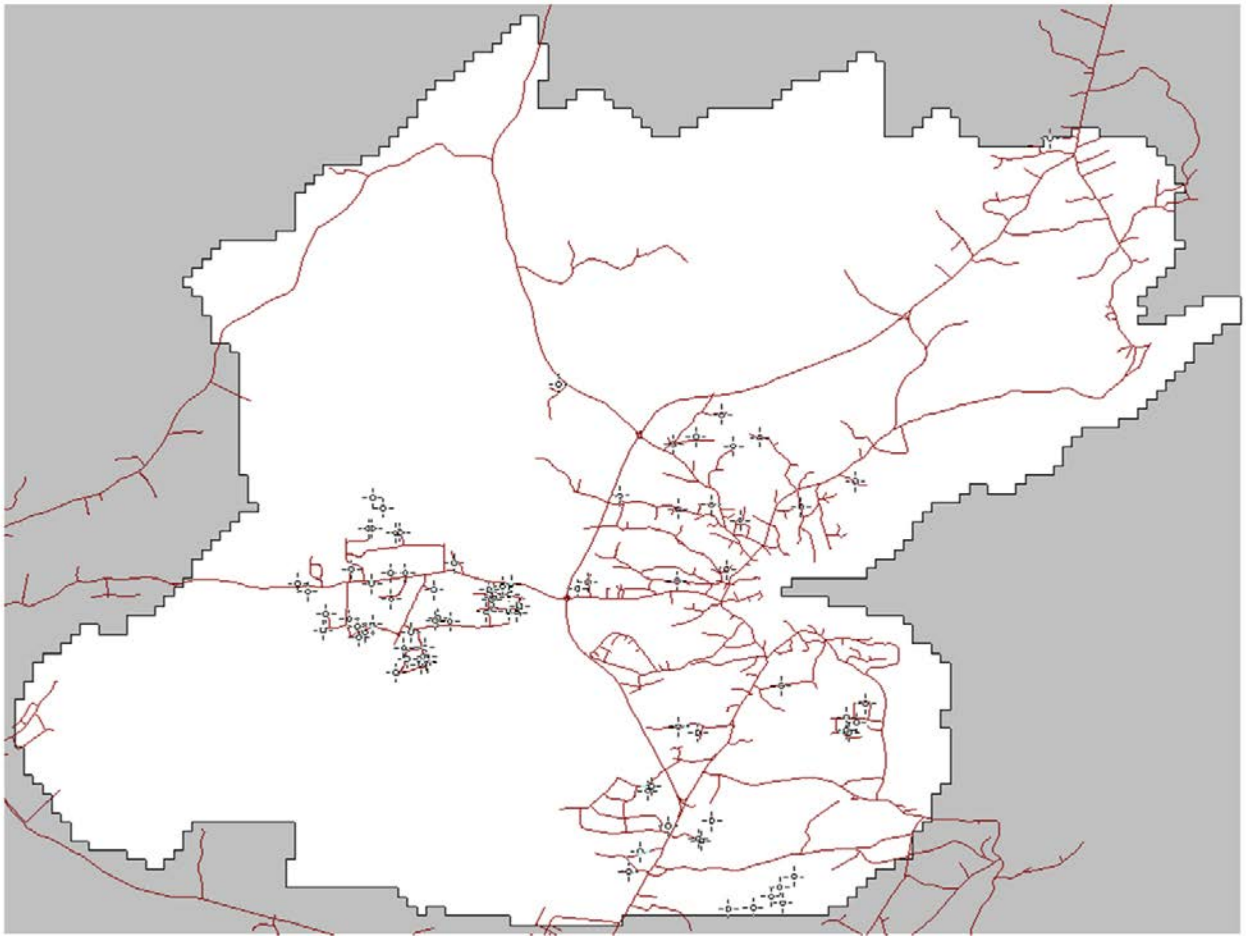

Figure 17 Head target locations in layer 4. The road network is shown (brown lines).

5.2 Flow and Mass Balance Errors

Flow and mass balance errors from the simulations were monitored with the goal of maintaining errors less than 1%. For the steady-state model calibration presented below, the mass balance errors were routinely less than 1% (reported as 0% in the model output due to the number of decimal places reported).

5.3 Residual Analysis

Model residuals, or the difference between the target (observed) value and the simulated value, are analyzed to evaluate how well the model is able to match observed conditions in the Study Area.

Figure 18 provides a visual comparison between observed and simulated water levels following model calibration. The objective of the calibration process is to reduce the residuals. The dashed line in Figure 18 represents a perfect match. There is a symbol for each of the 83 head targets. The further a symbol is away from the dashed line, the greater the residual. The overall fit is good and there are cases where the model over predicts (symbol is above the dashed line) and under predicts (dot is below the dashed line) head.

Four statistical parameters were used to evaluate the degree of fit, including the mean residual, mean absolute residual, the normalized root mean squared residual (NRMS), and the correlation coefficient. In general, groundwater models are considered to be adequately calibrated if:

- The mean error is close to zero;
- The absolute mean error is as small as possible;
- The NRMS is less than 10% (Spitz and Moreno 1996); and,
- The correlation coefficient is close to a perfect correlation of one.

Based on the head targets alone, the mean error is -0.89 m ASL, the absolute mean error is 2.57 m ASL, the NRMS is 2.2%, and the correlation coefficient is 0.99.

Flow target residuals are shown in Table 5. A flow target residual less than 20% is considered a good match. This condition is met for catchment areas 1, 4 and 6, which collectively cover approximately 80% of the Study Area. This is also a good result considering the majority of development (existing and future) is within these areas.

Higher flow residuals for catchment areas 2 and 3 are likely the result of the quality of the power function used to estimate baseflow (Figure 2) for small areas. As previously stated, the power function is likely best suited for catchment areas ranging from 10 km^2 to 100 km^2.

The high flow residual for catchment area 5 is likely due to the catchment area being ill-defined. Unlike the other catchment areas, catchment area 5 feeds numerous first-order streams that are not connected and discharge into the ocean.

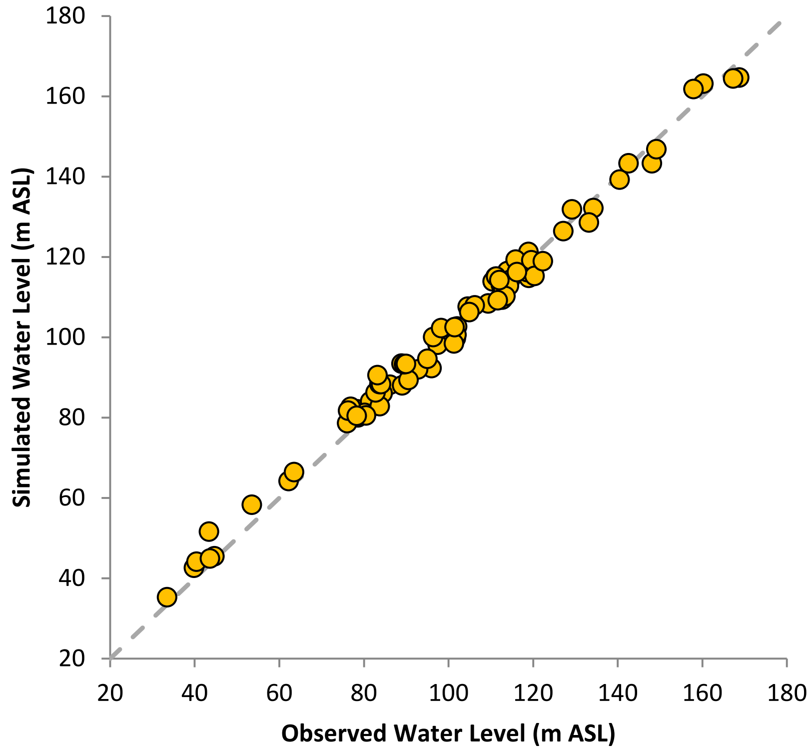

Figure 18 Comparison of observed and simulated water level.

Table 5 Baseflow Calibration Residuals

Catchment Area	Area (km^2)	Target "Observed" Baseflow (m^3/d)[1]	Simulated Baseflow (m^3/d)	Residual (m^3/d)	% Residual
1	7.8	6,589	5,678	-911	-14
2	0.7	607	374	-233	-38
3	5.9	4,999	3,767	-1,232	-25
4	17.7	14,820	14,713	-107	-1
5	7.5	6,338	2,687	-3,651	-58
6	32.3	26,869	25,712	-1,157	-4

Note:
[1] Estimated using the power curve fit function in Figure 2

5.4 Steady–Stage Model Calibration

The results of the model calibration indicate that a reasonably good match of hydraulic head and baseflow is achievable in such a complex setting based on a simplified distribution of hydraulic conductivities in three hydrostratigraphic units, and a uniform groundwater recharge rate. Table 6 presents the calibrated parameters.

Table 6 Parameters Values Assigned from Model Calibration

Parameter	Initial Value	Calibration Range	Calibrated Value	Anisotropy (Kv/Kh)
Groundwater Recharge	153 mm/yr	77 – 614 mm/yr	303 mm/yr	-
Overburden Horizontal Hydraulic Conductivity (K_h)	0.432 m/d	0.0432 – 4.32 m/d	4.32 m/d	1
Overburden Vertical Hydraulic Conductivity (K_v)	0.432 m/d	0.0432 – 4.32 m/d	4.32 m/d	
Weathered Bedrock Horizontal Hydraulic Conductivity (K_h)	0.216 m/d	0.0216 – 2.16 m/d	0.340 m/d	2
Weathered Bedrock Vertical Hydraulic Conductivity (K_v)	0.216 m/d	0.0216 – 2.16 m/d	0.688 m/d	
Competent Bedrock Horizontal Hydraulic Conductivity (K_h)	0.0216 m/d	0.00216 – 0.216 m/d	7.5×10^{-3} m/d	29
Competent Bedrock Vertical Hydraulic Conductivity (K_v)	0.0216 m/d	0.00216 – 0.216 m/d	0.216 m/d	

The calibrated hydraulic conductivity values compare well with what is expected for the given geology. The overburden hydraulic conductivity is greater than what is typically expected for glacial till (8.64×10^{-8} to 8.64×10^{-2} m/d from Freeze and Cherry (1979)). However, given that the overburden thickness might be closer to the regional value of 1.5 m (Batterson 2000) in areas where there aren't drilled wells and there are other more permeable unconsolidated materials present, this result is not surprising. The calibrated overburden hydraulic conductivity is compensating for these factors at a larger scale and is sufficient for the objective of the model.

As expected, the weathered bedrock has a higher hydraulic conductivity than the competent bedrock. More interestingly, the calibrated hydraulic conductivities of the weathered and competent bedrock yield anisotropies (K_v/K_h) that are greater than 1 (i.e., these units are more permeable in the vertical direction than the horizontal direction). This result is consistent with an aquifer that has inclined fracture features that control groundwater flow. In the case of the Study Area, bedrock structure is observed to be sub-vertical in many places.

It is expected that the proportion of baseflow in total streamflow be equal to the groundwater recharge rate in the case of a shallow groundwater flow system at steady state. Thus, baseflow as a percentage of total stream flow should be similar to groundwater recharge as a percentage of total annual precipitation. From Table 1, the average percentage of baseflow in total stream flow is 22% in this terrain (minimum = 15%, maximum = 24%). The calibrated value of 303 mm/yr for groundwater recharge is 20% of the 1534 mm/yr total annual precipitation recorded at the St. John's Airport (Environment Canada 2015)) and matches the groundwater recharge estimated in Section 3.2.1. This result is expected given the low residuals for flow targets in catchment areas 1, 4 and 6.

5.4.1 Parameter Sensitivity

PEST keeps track of parameter sensitivity during the calibration process. The final parameter sensitivity (Figure 19) provides an indication of the relative strength each parameter has on minimizing target residuals. The groundwater recharge rate is by far the most sensitive parameter followed by the horizontal hydraulic conductivities of the three hydrostratigraphic units. The flow solution is least sensitive to changes in the vertical hydraulic conductivity of the three hydrostratigraphic units.

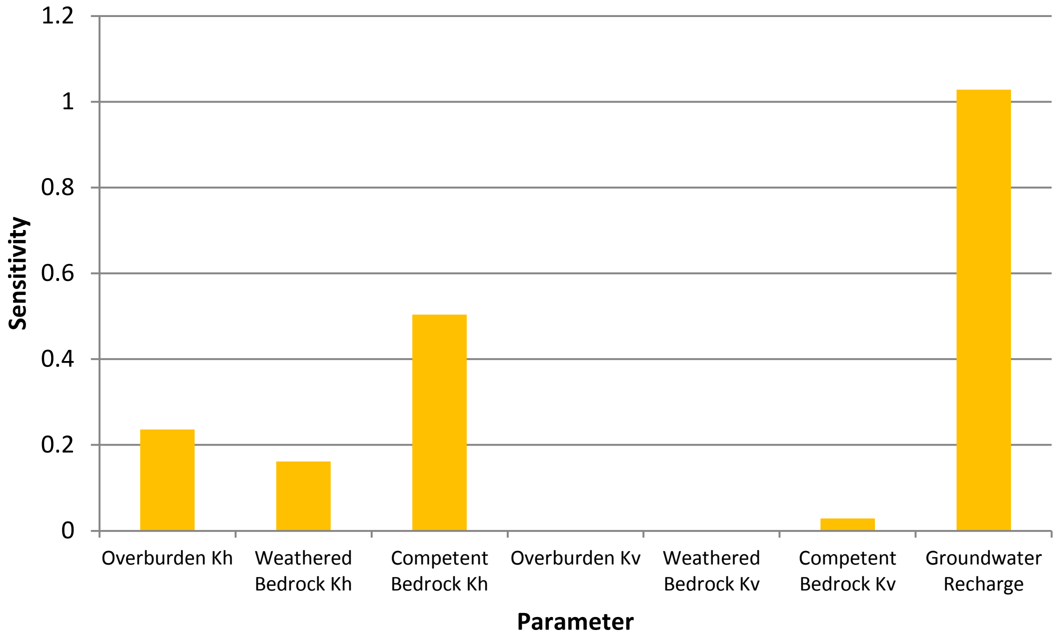

Figure 19 Final Calibrated Parameter Sensitivity.

6.0 MODEL APPLICATIONS

Increased groundwater extraction from new residential developments within the Study Area have the potential to alter the water balance within the watershed. This could result in the lowering of the water table at existing wells, or the reduction in baseflow to streams and ponds. Three sub-division development scenarios were constructed to evaluate if these effects have the potential to occur and if so, the magnitude of these effects.

6.1 Existing Conditions – Base Case

The base case for the sub-division development scenarios is the steady-state existing conditions developed as a result of the model calibration. This condition will be used to evaluate drawdown (a decline in water level) that will result from new unserviced development.

Figure 20 and Figure 21 show the calibrated, steady-state hydraulic head contours in the overburden (model layer 1) and pumped bedrock aquifer (model layer 4), respectively.

An important component of the base case simulation is the flow balance. In this case, the internal water balance within the domain has not been previously studied. Figure 22 provides a simplified schematic of the modeled groundwater flow balance expressed as a percentage of groundwater recharge.

The water balance shown in Figure 22 highlights a few points of interest. Firstly, the majority of groundwater recharge within the Study Area (approximately 85%) discharges to ponds and streams. Secondly, 10% of groundwater recharge enters the competent rock. This corresponds to only 2% of total annual precipitation. Finally, approximately 5% of the groundwater recharge within the Study Area is extracted by wells. This is a small volume of water within the water balance compared to the volume discharging to ponds and streams.

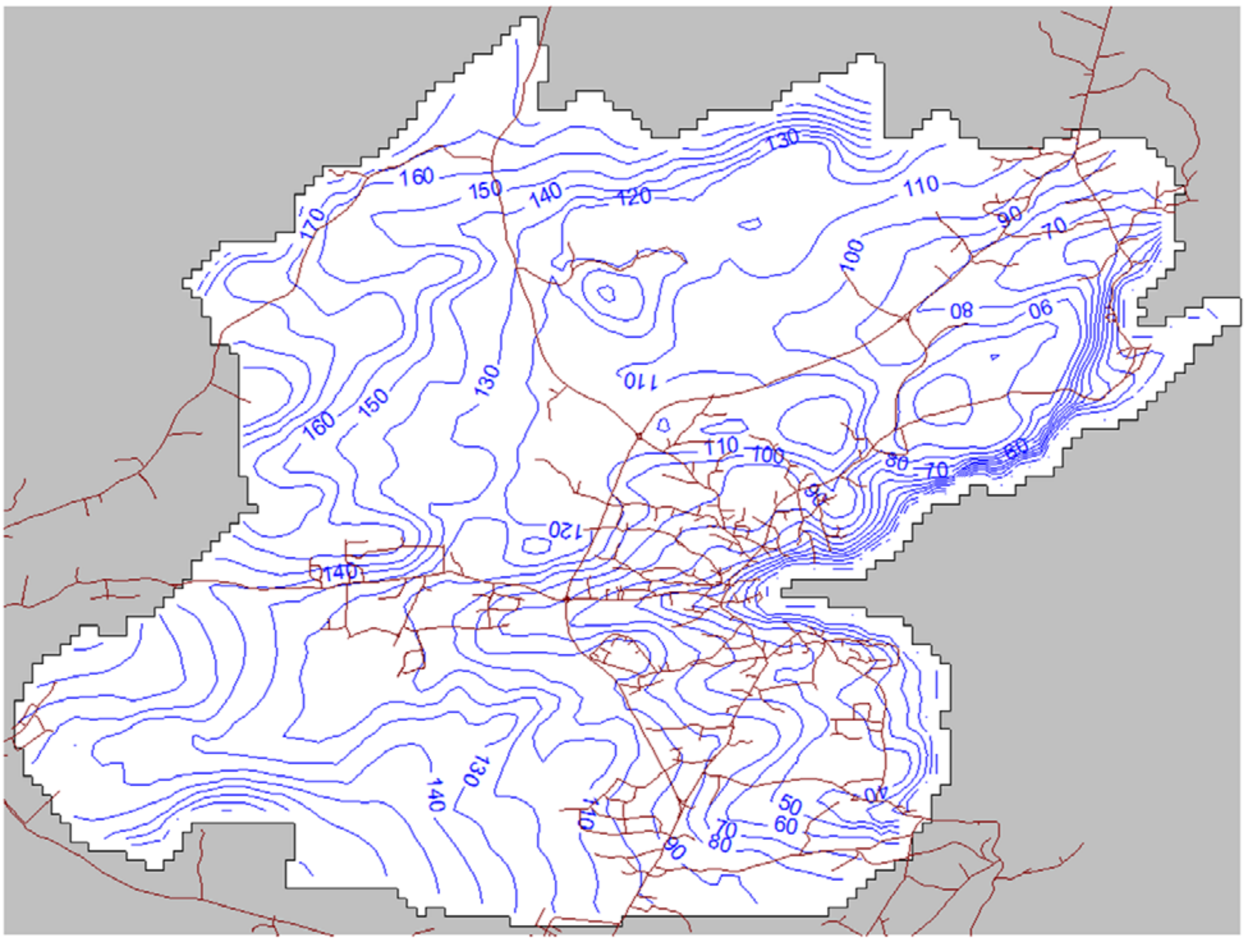

Figure 20 Base case simulated steady-state hydraulic head contours in overburden (model layer 1).

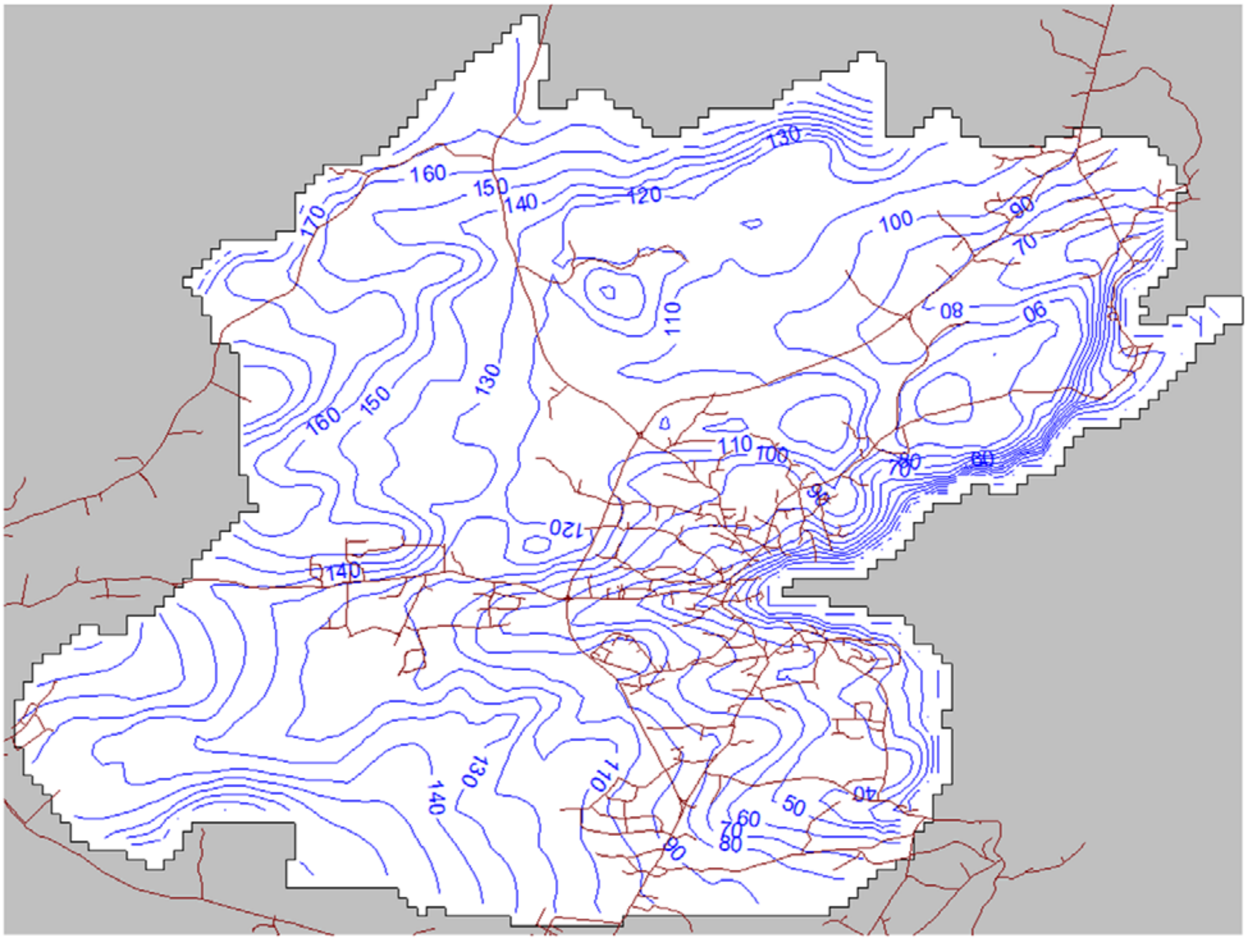

Figure 21 **Base case simulated steady-state hydraulic head contours in the pumped bedrock aquifer (model layer 4).**

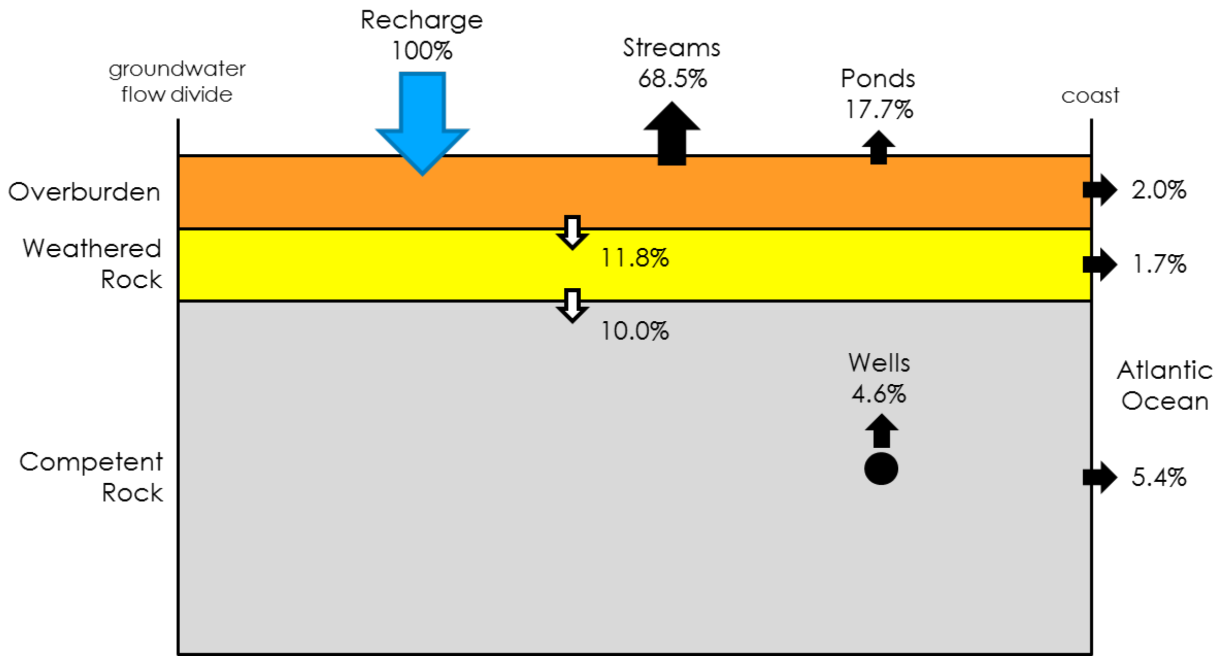

Figure 22 Base case simulated steady-state water balance. Black arrows indicate water is being removed from the groundwater system. White arrows indicate the direction of internal groundwater flow. Percentages are net values relative to groundwater recharge (the only source of water into the domain).

6.2 Predictive Scenarios

Upon the calibration of base case model parameters, several predictive scenarios were conducted to simulate the effects of new residential, commercial and municipal well development on the groundwater system, with particular focus on the potential interferences that changes in water levels may have on existing private wells.

However, before impacts on existing wells can be evaluated, it is necessary to define what an "adverse condition" might be, and discuss how to calculate the actual drawdown in a pumping well based on simulated results.

6.2.1 Defining an Adverse Condition

Evaluating an adverse impact requires the definition of an adverse condition that can be tested in the predictive scenarios. This study uses the available drawdown defined and quantified in Section 3.2.2.7. More specifically, the adverse condition is defined as when the calculated well drawdown exceeds the available drawdown in 5% of existing wells. From Figure 9, this threshold is met when the drawdown in a well exceeds 15 m.

6.2.2 Calculating the Actual Head in a Pumping Well

MODFLOW evaluates the average head in each cell. While a well boundary condition acts to remove water from a cell, MODFLOW does not output what the actual drawdown would be in a well of finite diameter pumping at a given flow rate (see Figure 23). The following correction is based on the Theim solution:

$$h_w = h_* - \frac{Q}{2\pi T} \ln\left(\frac{r_e}{r_w}\right) \qquad \text{[Eq. 1]}$$

where h_w is the head in the pumping well, h_* is head in the MODFLOW cell, Q is the pumping rate of the well, T is the transmissivity of the aquifer, and r_w is the radius of the well. The equivalent well-block radius r_e can be approximated by $0.198\Delta x$ (Peaceman 1983) where Δx is the length dimension of a cell (assuming the cells are square in plan view).

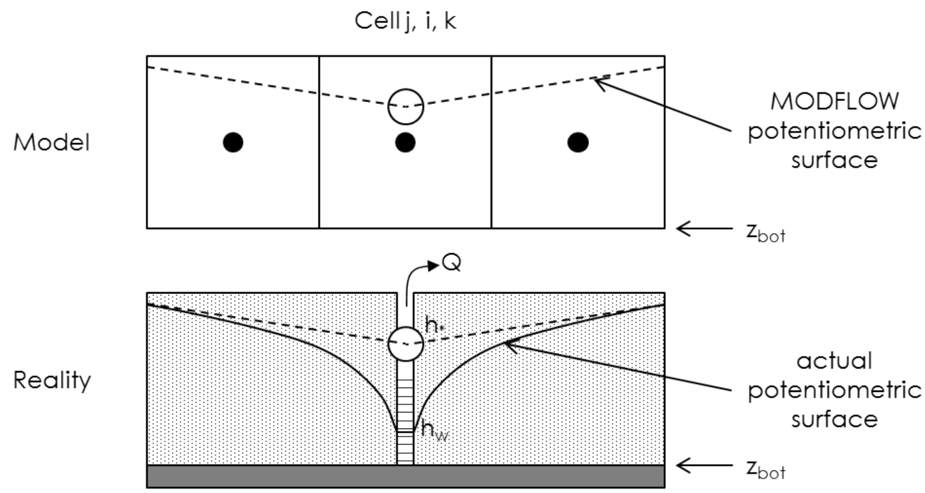

Figure 23 An example of the difference between MODFLOW results and the actual potentiometric surface in a pumping well.

The values used in this calculation are: T = 1.19 m^2/d and Δx = 100 m. For residential wells considered in scenarios 1 and 2, r_w = 0.0762 m (6"-diameter) and Q = 1.36 m3/d. The second term on the right-hand side of Equation 1 represents the head correction in the well and equals 1.1 m.

Scenario 3 considers high-capacity municipal wells with r_w= 0.127 m (10"-diameter) and Q = 136 m3/d. The head correction in the well is equal to 102 m, based on Equation 1.

6.2.3 Predictive Scenario 1 – Completion of Existing Subdivisions

This scenario involves the completion of all existing residential subdivisions. The location of each is shown in Figure 24 with the details summarized in Table 7. With the exception of Pine Ridge, new household demand is implemented using the well boundary condition in MODFLOW well boundary condition in new MODFLOW cells in layer 4. Thus, the “footprint” of the subdivision increases to the size of the planned or built road network. For Pine Ridge, the number of empty lots remaining is within the area of pumping cells defined in the base case. Thus, additional houses are accounted for by increasing the pumping rate per cell.

Figure 25 shows the simulated drawdown relative to the base case. In general, simulated drawdown is contained within the footprint of each development. The greatest impact to existing users is predicted along Flora Drive (Scenic View Ridge and Eagle Nest Ridge) with drawdown up to ~4 m. The actual drawdown in a residential pumping well within one of these model cells is therefore up to 5.1 m, based on the correction derived in Section 6.2.2.

The results of Prediction Scenario 1 do not show that the threshold allowable drawdown of 15 m in an existing well (see Section 6.2.1) is reached, as simulated.

Figure 26 provides the estimated change in baseflow relative to the base case. Analysis shows changes in baseflow between -4% and -57% in stream segments proximal to new development.

Table 7 Additional residential development considered in Predictive Scenario 1

Sub-division	Number of Additional Homes	Number of New MODFLOW Cells
Forest Landing	25	16
Forest Landing (Phase VIIA-C)	65	22
Scenic View Ridge and Eagle Nest Ridge	106	33
Logy Bay	67	24
Pine Ridge	12	0

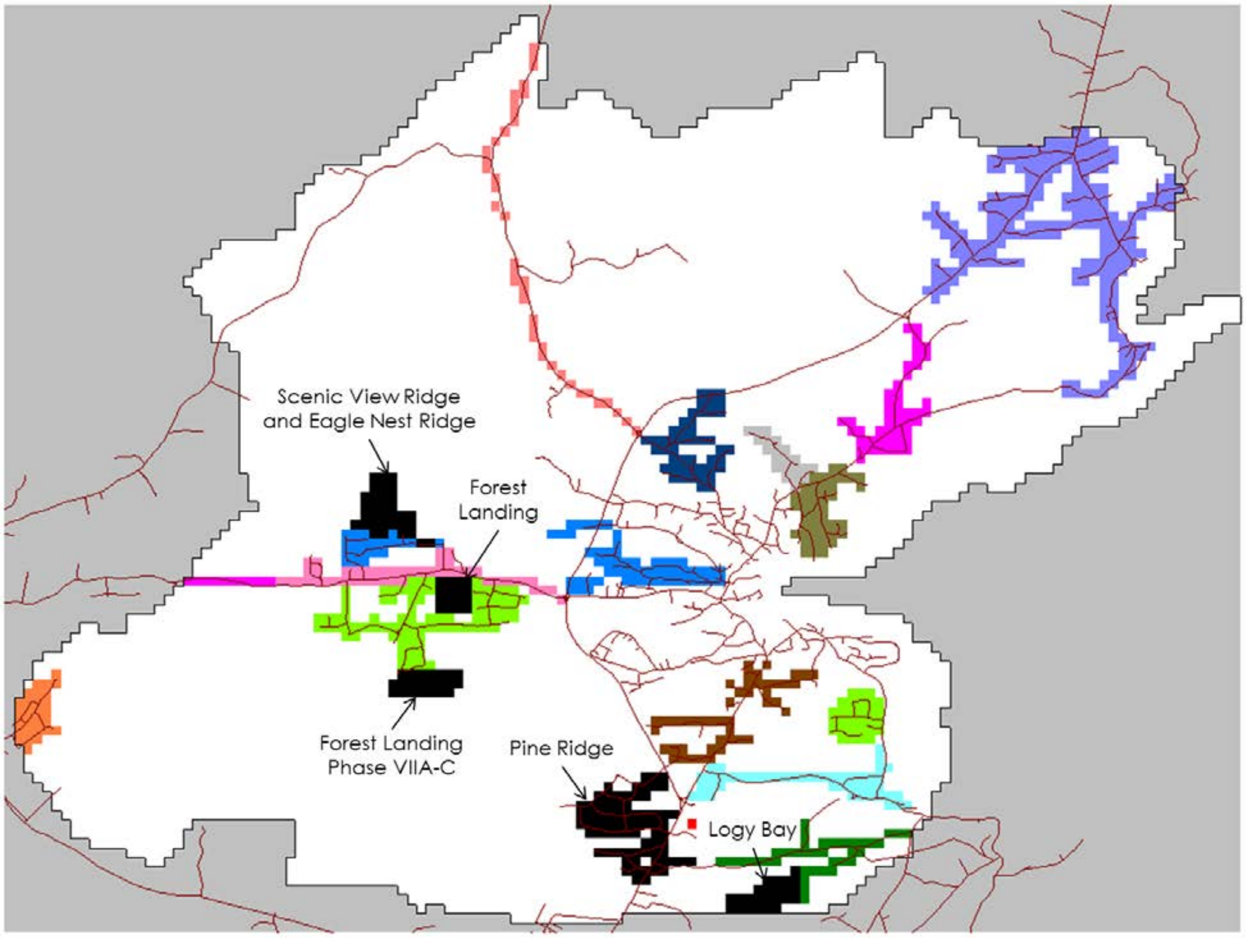

Figure 24 Predictive Scenario 1 plan view of pumping well boundary conditions assigned to cells in layer 4. Black cells represent added/altered pumping conditions corresponding to the completion of existing sub-divisions.

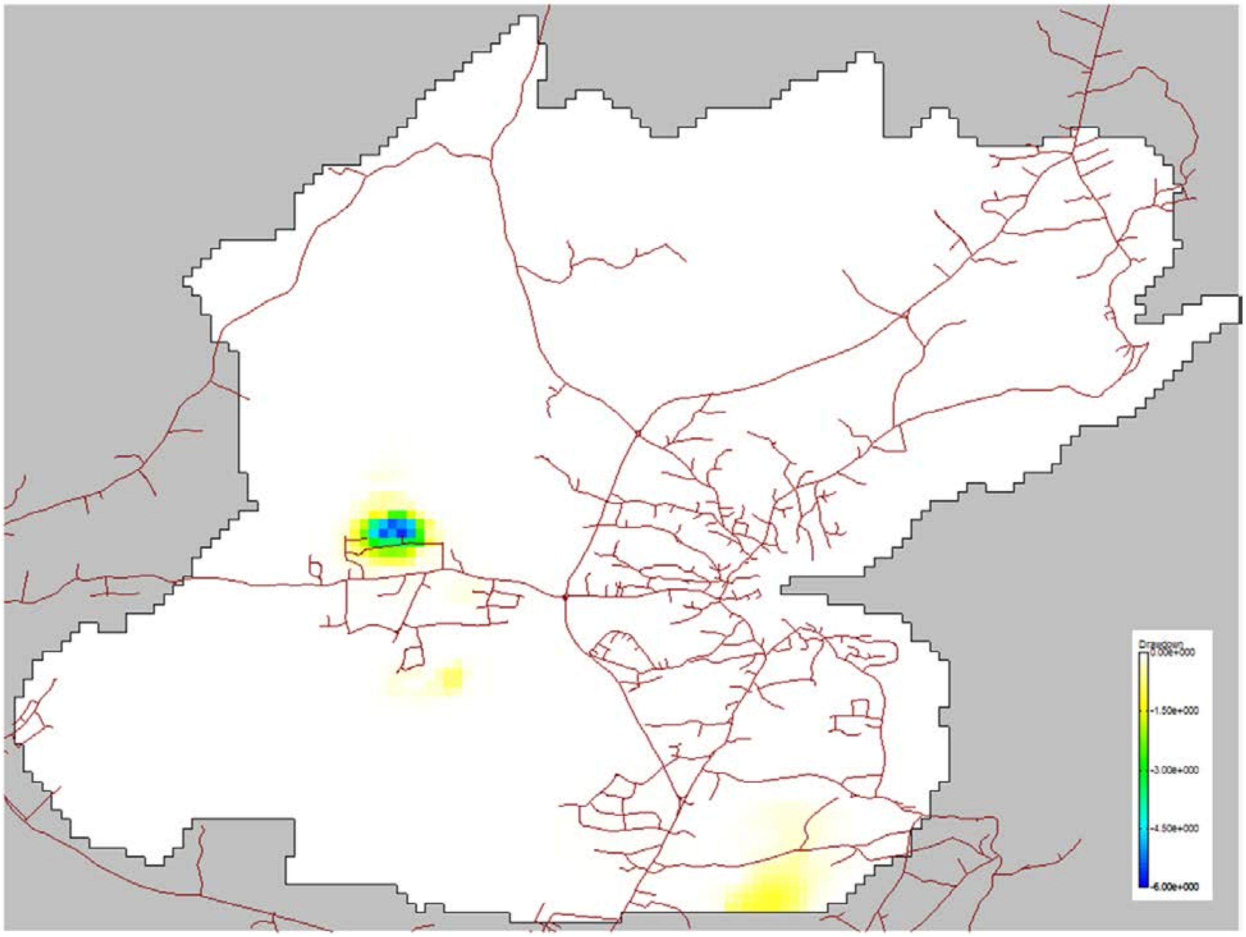

Figure 25 Predictive Scenario 1 development drawdown (in metres) in layer 4 relative to the base case.

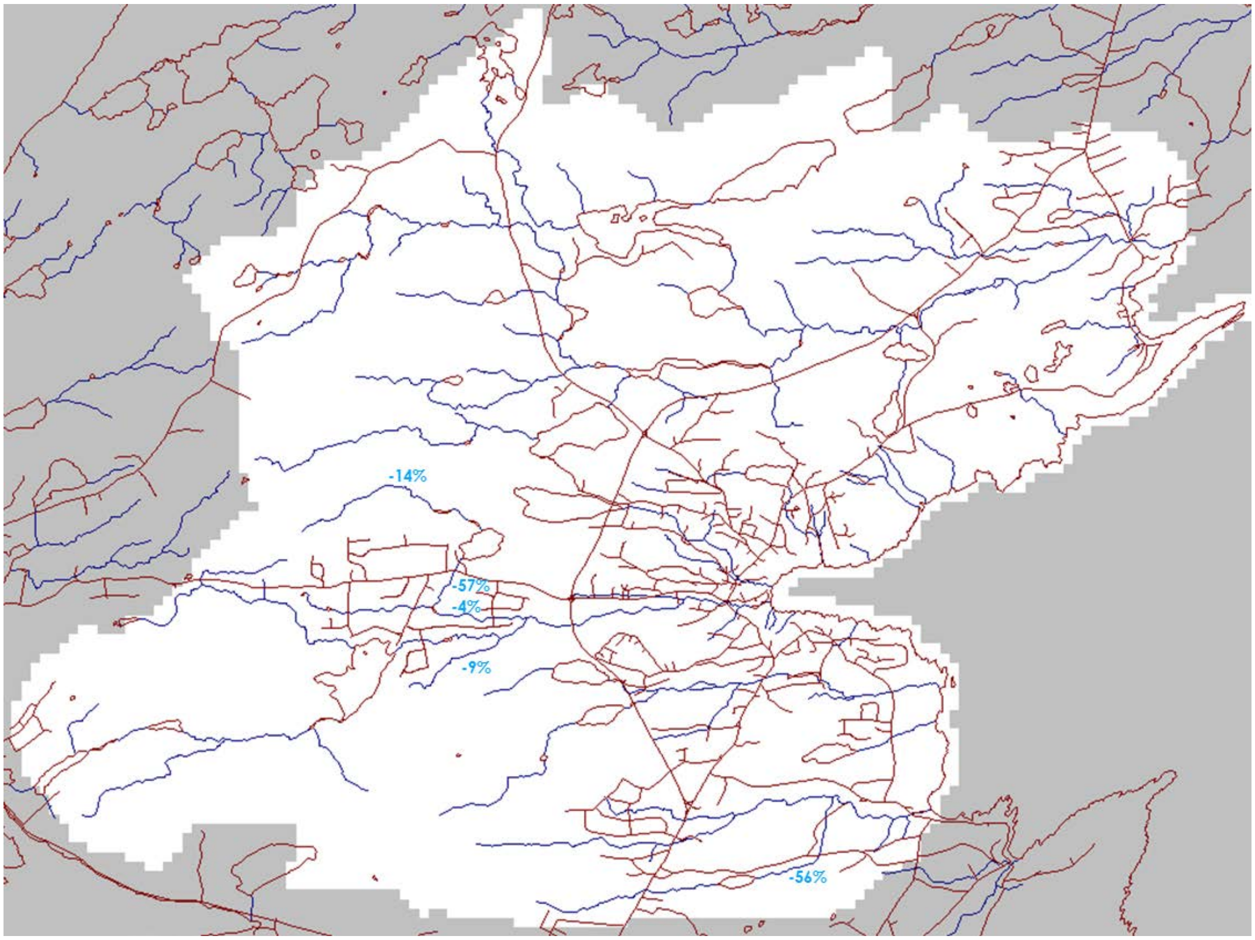

Figure 26 Predictive Scenario 1 estimated percentage change in baseflow to streams.

6.2.4 Predictive Scenario 2a – New Residential Development Areas

This scenario considers new residential development at potential growth areas identified along Bauline Line and near Jones Pond. The location of each is shown in Figure 27.

The development along Bauline Line is sized equivalent to a completed Forest Landing (350 homes represented by 123 MODFLOW cells). This is representative of a sub-division built with 0.75 acre lots.

The development at Jones Pond is equivalent in size and density to the newer neighbouring sub-division (76 homes along Torquay Place, Bixham Crescent, Sallesnik Lane and Paul's Place represented by 30 MODFLOW well cells, 1 acre lots).

Figure 28 shows the simulated drawdown relative to the base case. The greatest simulated drawdown is contained within the footprint of each development. Predicted drawdown in areas with existing users is up to approximately 2 m at Jones Pond and less than 1 m along Bauline Line. Therefore, the actual drawdown in an existing residential pumping well is up to 3.1 m, based on the correction derived in Section 6.2.2.

The results of Prediction Scenario 2a do not show that the threshold allowable drawdown of 15 m in an existing well (see Section 6.2.1) is reached, as simulated.

Figure 29 provides the estimated change in baseflow relative to the base case. Analysis shows changes in baseflow between -27% and +210% in stream segments proximal to new development.

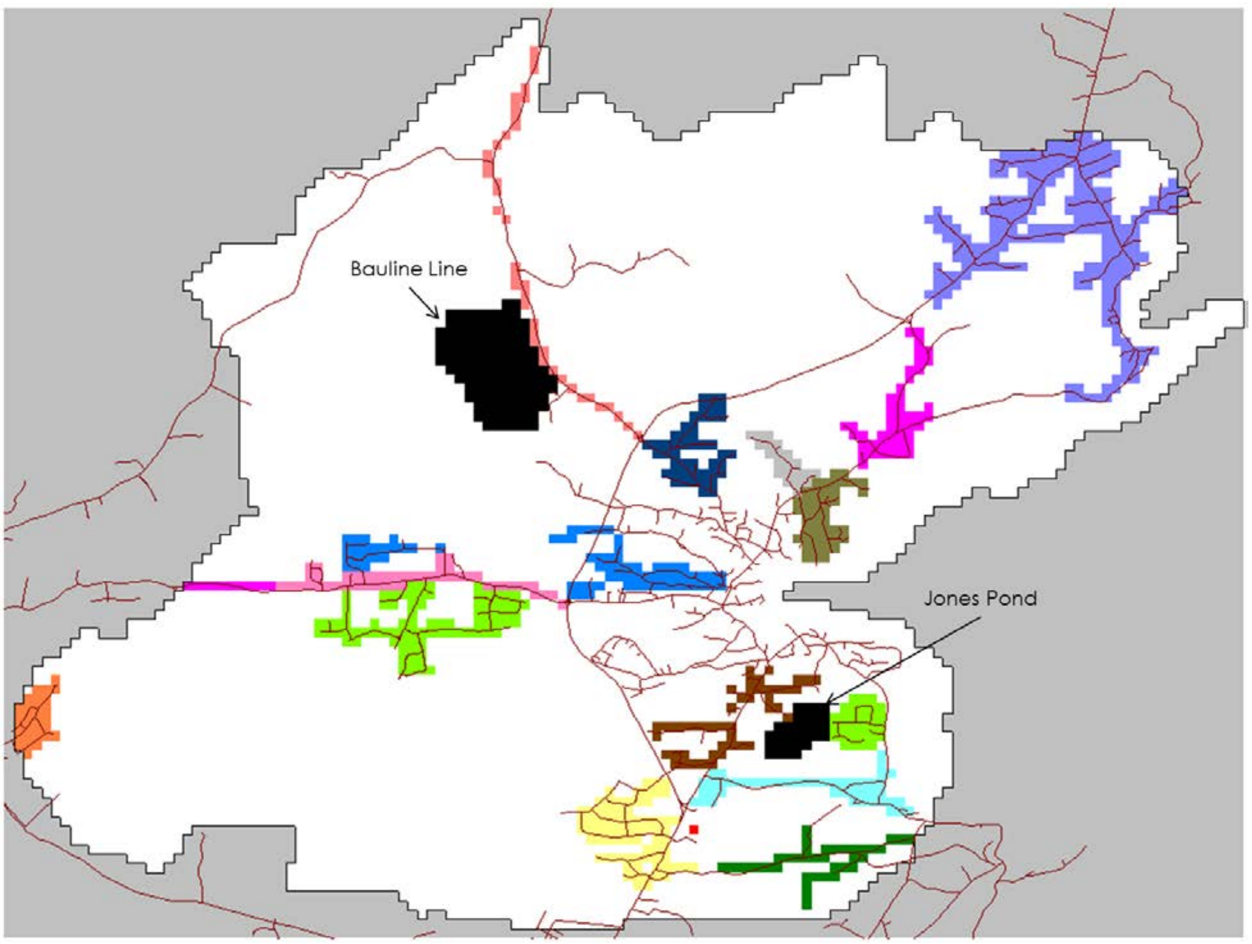

Figure 27 Predictive Scenario 2a plan view of pumping well boundary conditions assigned to cells in layer 4. Black cells represent added pumping conditions corresponding to new subdivisions.

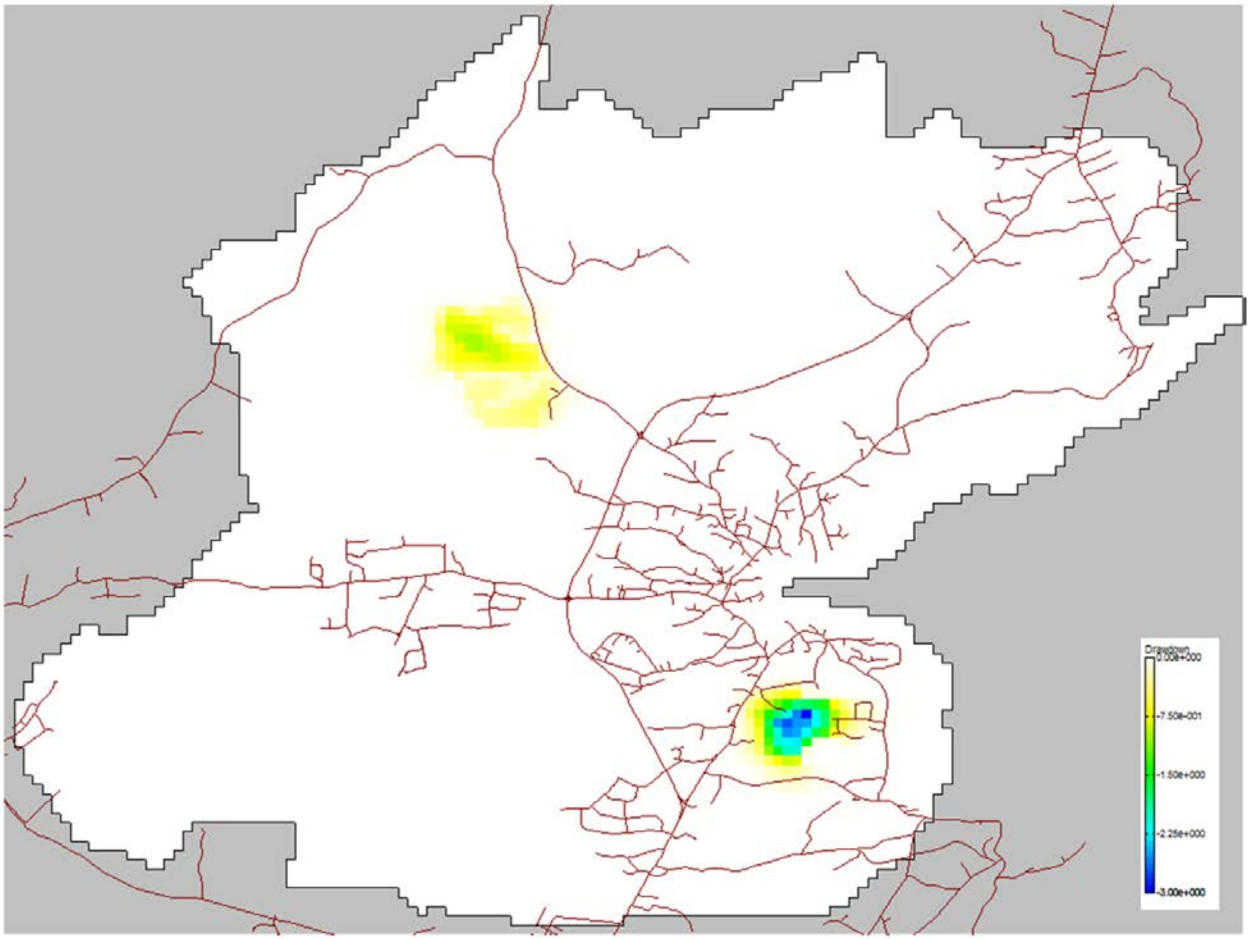

Figure 28 **Predictive Scenario 2a drawdown (in metres) in layer 4 due to new development relative to the base case.**

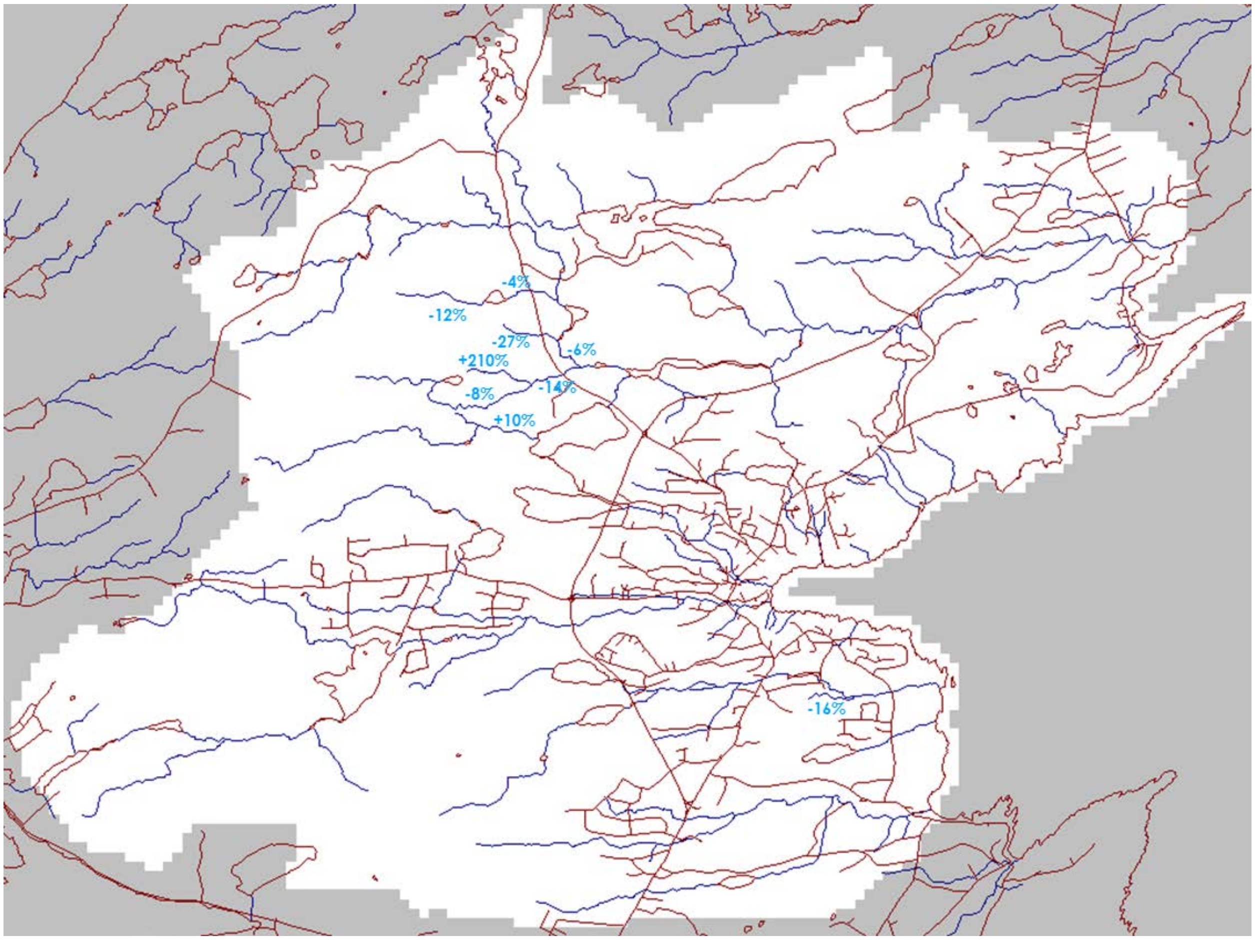

Figure 29 Predictive Scenario 2a estimated percentage change in baseflow to streams.

6.2.5 Predictive Scenario 2b – Pine Ridge Valley Residential

This scenario considers the proposed residential component of the Pine Ridge Valley development beside the Jack Byrne Arena (56 homes, 22 MODFLOW cells) shown in Figure 30.

The greatest simulated drawdown is contained within the footprint of the development. Predicted drawdown in areas with existing users is negligible (Figure 31).

The results of Prediction Scenario 2b do not show that the threshold allowable drawdown of 15 m in an existing well (see Section 6.2.1) is reached, as simulated.

Figure 32 provides the estimated change in baseflow relative to the base case. Analysis shows a change in baseflow of 2% in stream segment proximal to new development.

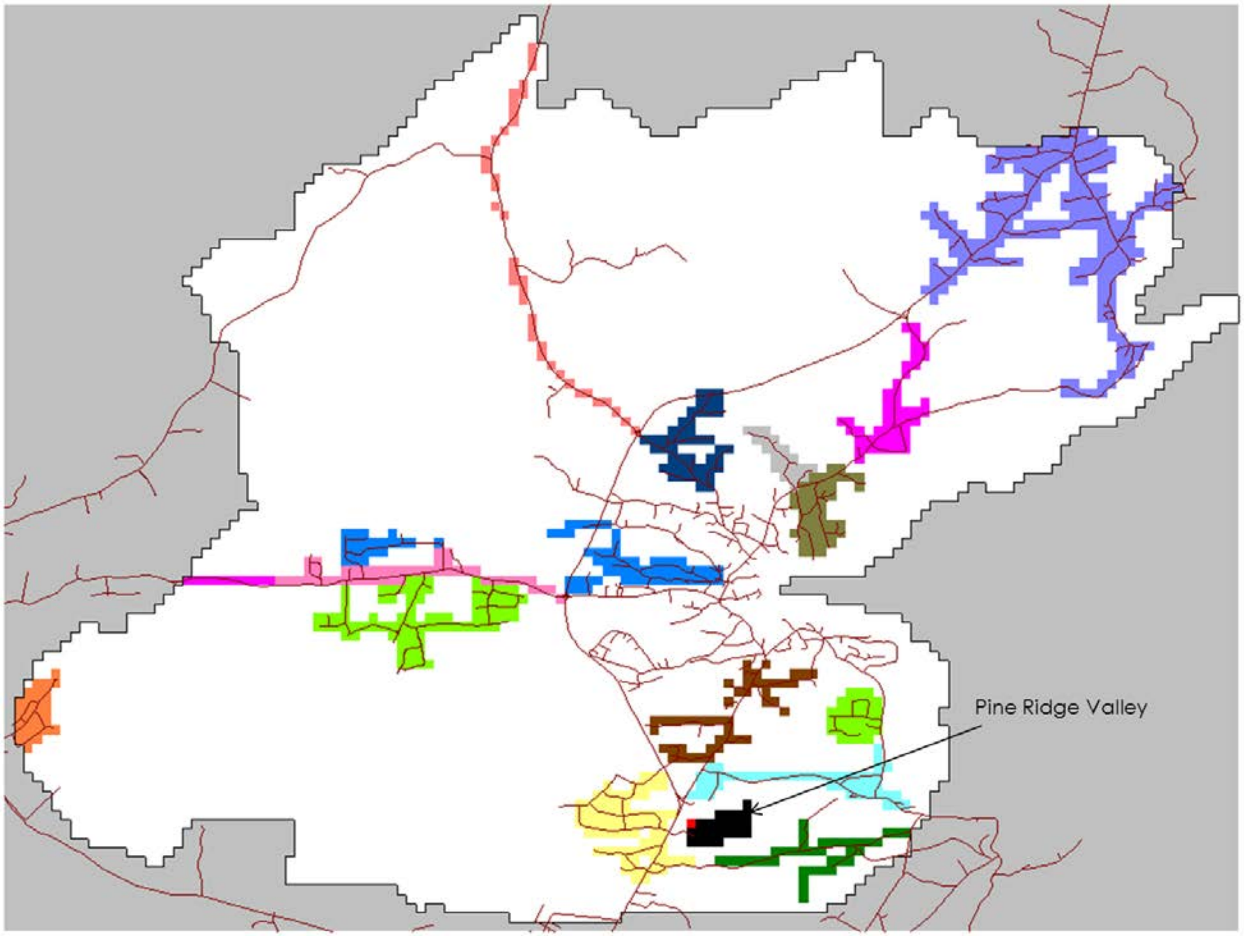

Figure 30 Predictive Scenario 2b plan view of pumping well boundary conditions assigned to cells in layer 4. Black cells represent added pumping conditions corresponding to residential development at Pine Ridge Valley.

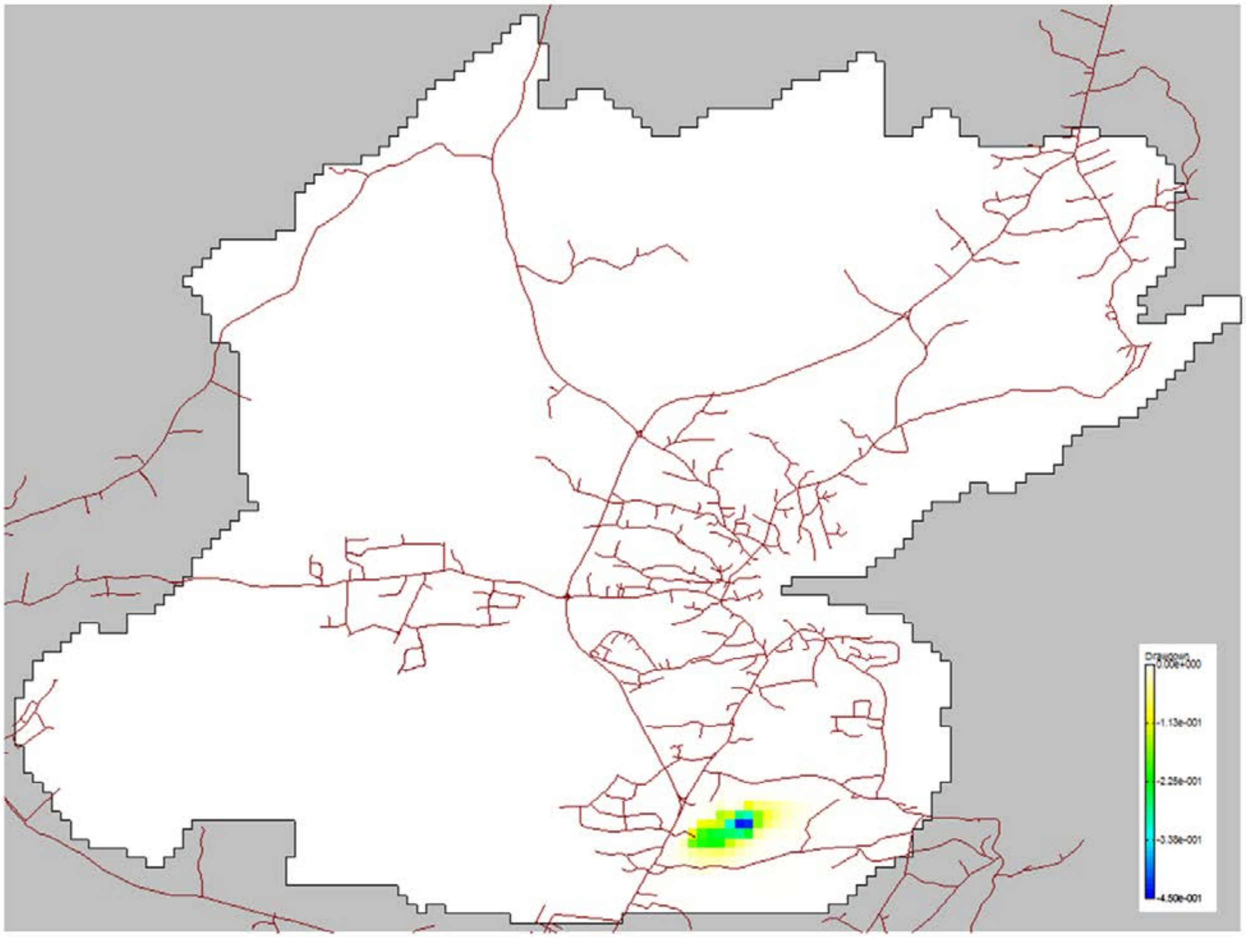

Figure 31 Predictive Scenario 2b drawdown (in metres) in layer 4 due to new development relative to the base case.

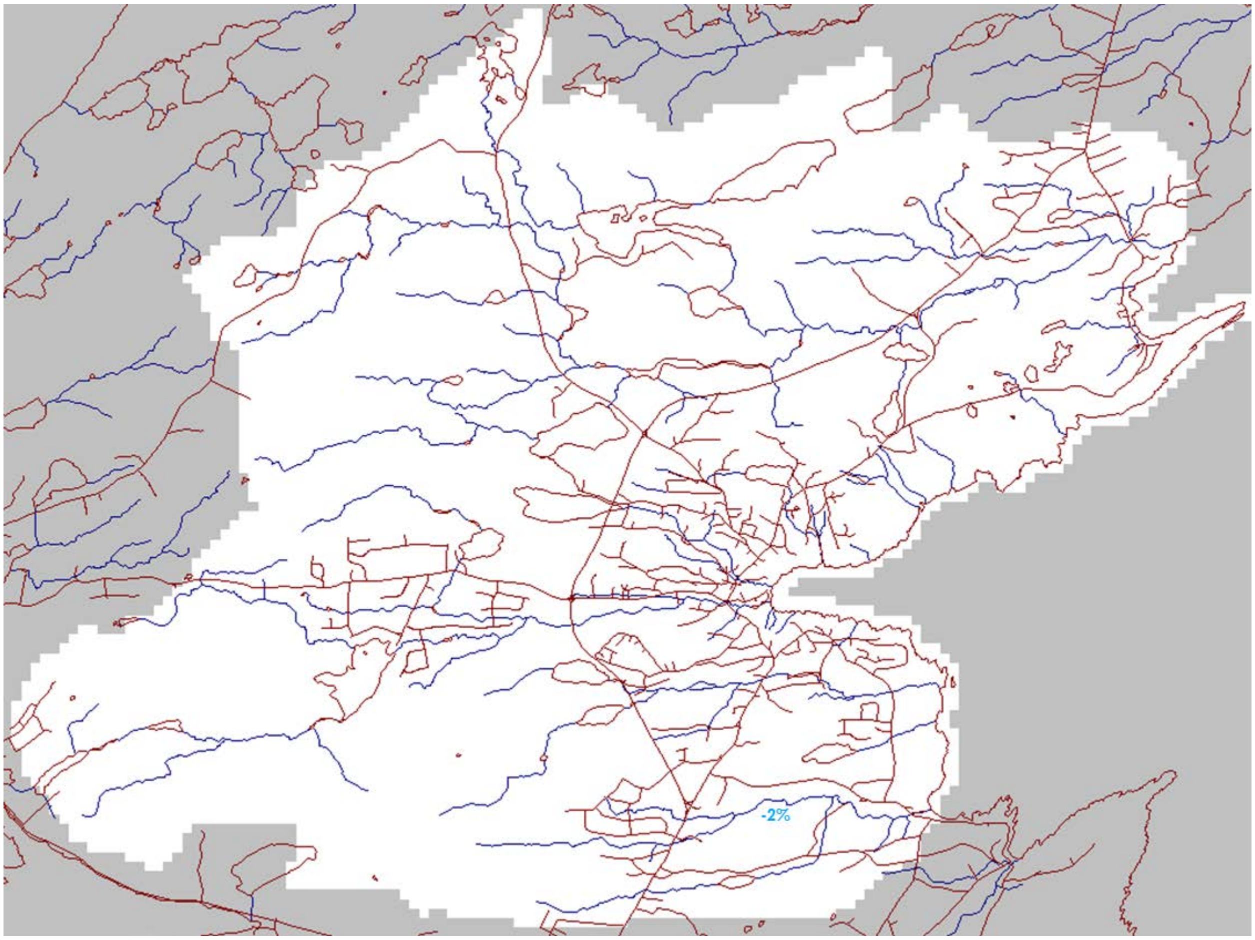

Figure 32 Predictive Scenario 2b estimated percentage change in baseflow to streams.

6.2.6 Predictive Scenario 2c – Pine Ridge Valley Residential and Commercial

This scenario considers the proposed residential and commercial components of the Pine Ridge Valley development beside the Jack Byrne Arena (Figure 33). Residential development represented the same way as in Prediction Scenario 2b (56 homes, 22 MODFLOW cells).

The proposed commercial development area is 35,000 m^2 (8.6 acres). This scenario considers developing the area with free-standing fast food restaurants as an example of a commercial development with higher water use compared to other commercial (e.g., office spaces where the primary use of water is washroom facilities). McDonald's reports that their typical free-standing restaurant inputs 4,100 m^3/y of water (2,255 m^3 goes to sewer). Each restaurant uses approximately 8.3 times the water of a four-person house for food preparation, beverages, cleaning, washrooms, etc.

A review of their restaurants in the St. John's area shows that the footprint of a typical restaurant is 1 – 1.5 acres (including parking). Thus, this scenario considers the case of six restaurants being built within the commercial development area. This is represented by pumping in three MODFLOW cells in Figure 33 (2 restaurants per cell).

Simulated drawdown (Figure 34) is greatest where commercial development is located, but is relatively minimal at values less than 1 m. Drawdown due to the residential development is similar to the results from Prediction Scenario 2b (Figure 31). Impacts on existing well users are negligible.

The results of Prediction Scenario 2c do not show that the threshold allowable drawdown of 15 m in an existing well (see Section 6.2.1) is reached, as simulated.

Figure 32 provides the estimated change in baseflow relative to the base case. Analysis shows a change in baseflow between <-1% to -3% in the stream segment proximal to new development.

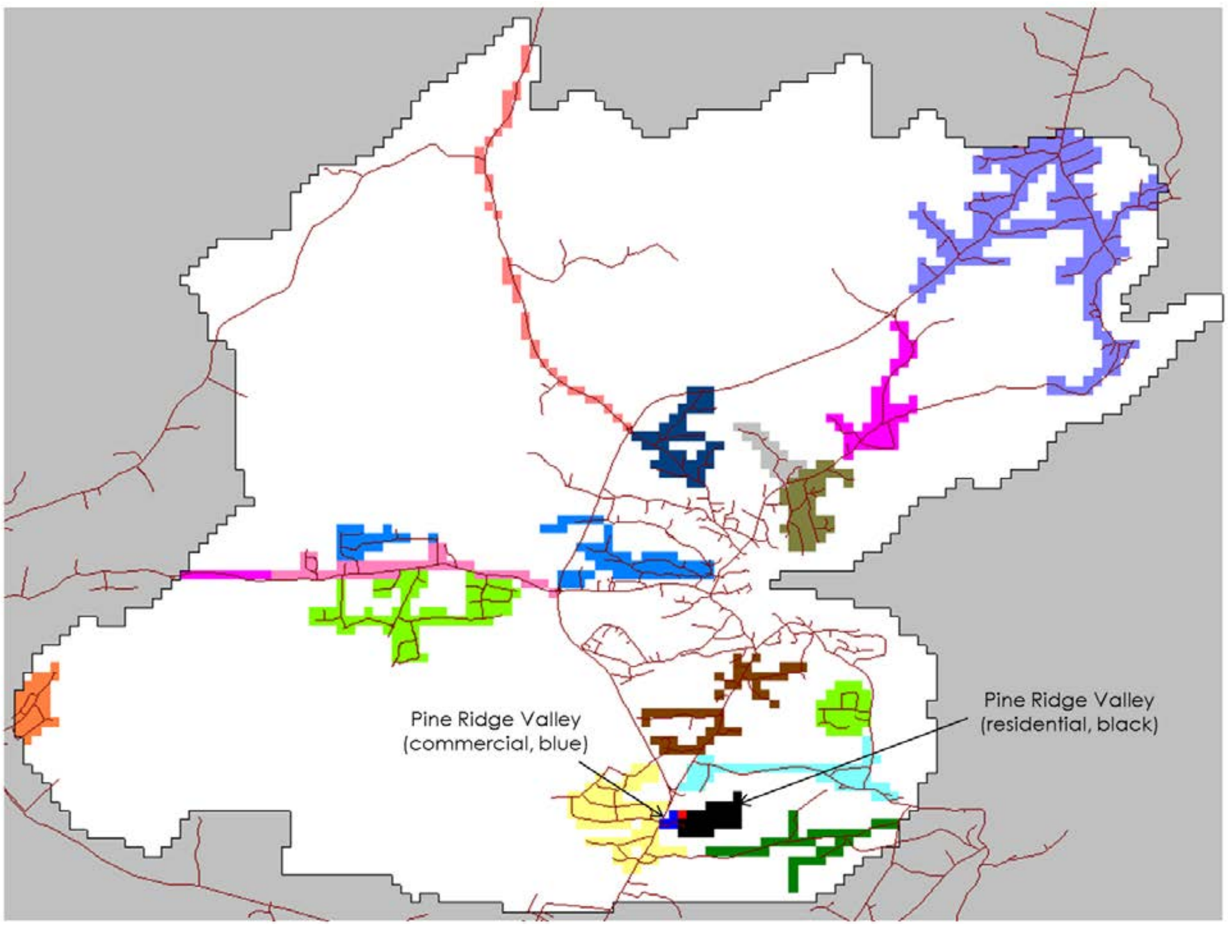

Figure 33 Predictive Scenario 2c plan view of pumping well boundary conditions assigned to cells in layer 4. Residential = black cells, commercial = blue cells.

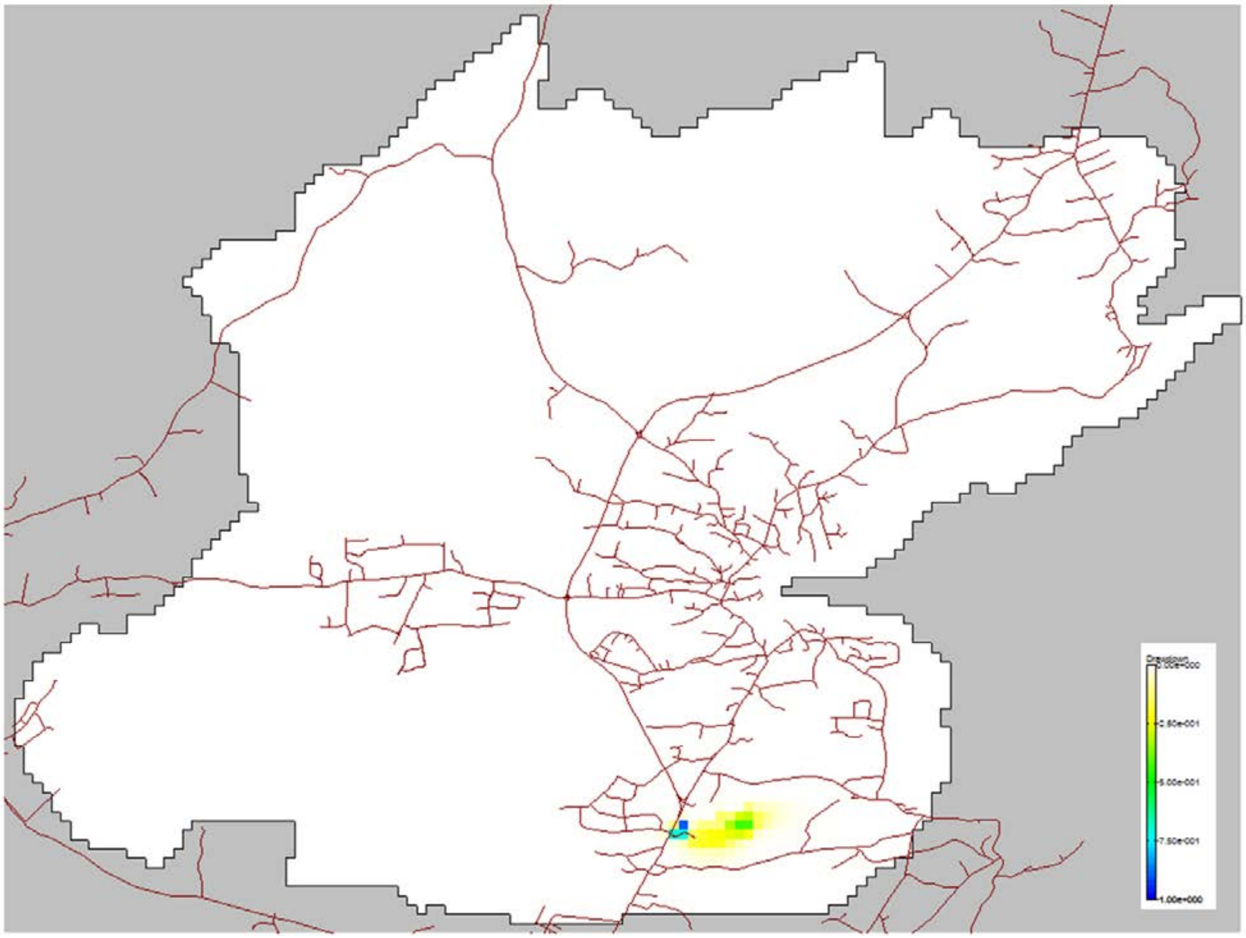

Figure 34 Predictive Scenario 2c drawdown (in metres) in layer 4 due to new development relative to the base case.

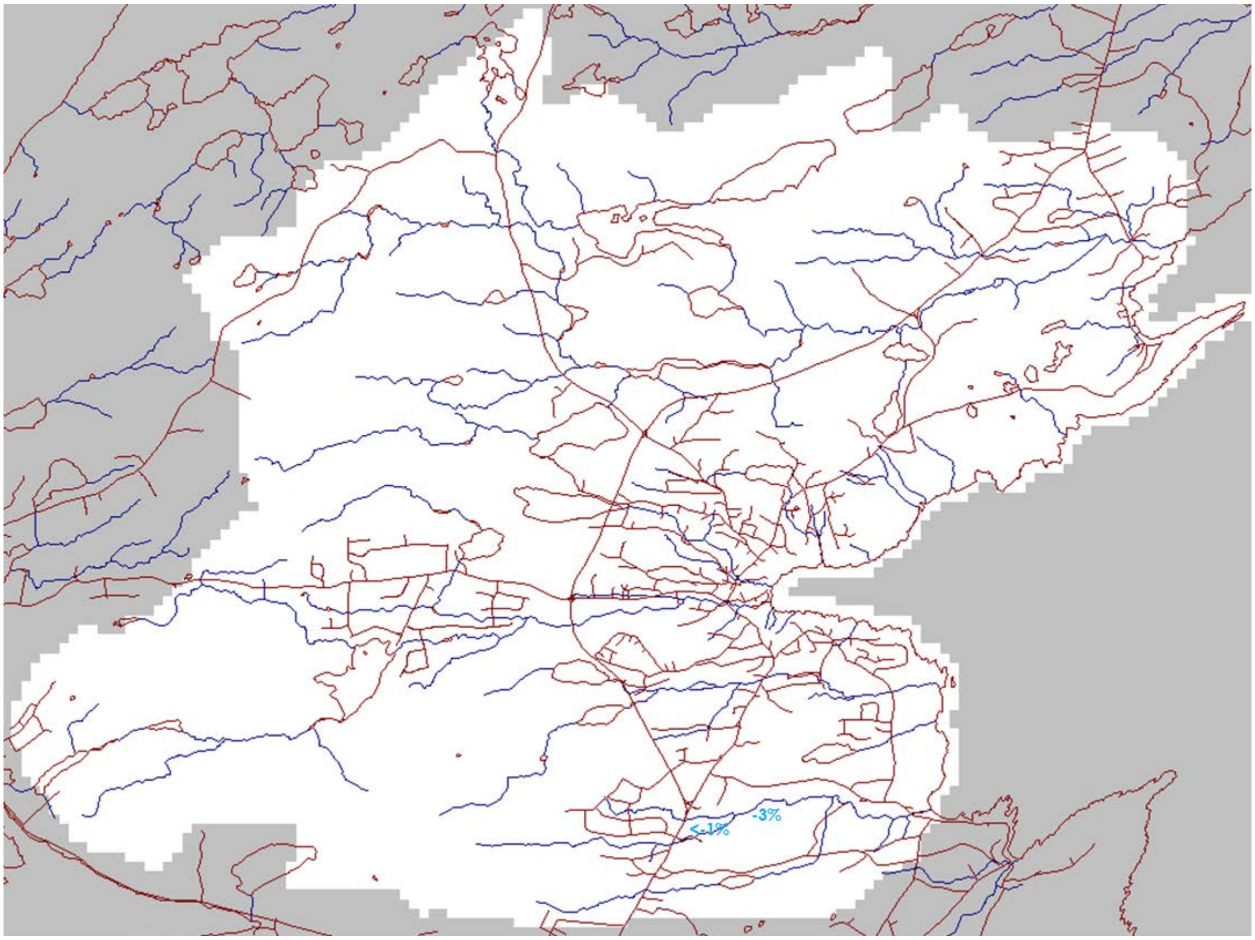

Figure 35 Predictive Scenario 2c estimated percentage change in baseflow to streams.

6.2.7 Predictive Scenario 3 – Municipal Well Field at North Pond

This scenario considers one configuration of four 0.254 m-diameter (10 inch) municipal groundwater wells designed to supply 400 homes (total pumping of 544 m^3/d, 136 m^3/d each well). The four wells were placed adjacent to North Pond (Figure 36). The location of the wells was based on proximity to the existing water treatment and distribution linear infrastructure. The design of the wells (diameter and spacing) is for demonstration purposes only.

Figure 37 shows that the simulated drawdown in the vicinity of the municipal wells is approximately 10 m with no apparent impact on surrounding existing well users.

Figure 38 provides the estimated change in baseflow relative to the base case. Analysis shows changes in baseflow between -2% and -51% in stream segments proximal to the municipal wells.

The well water level correction of 102 m determined in Section 6.2.2 increases the drawdown in municipal wells to approximately 112 m. Figure 10 provides a histogram of available drawdown based on records for existing residential wells within the Study Area. From Figure 10, approximately 90% of existing wells do not have this much available drawdown. The required pumping rate of 136 m^3/d (94 L/min) per well is far greater the average yield of 14.42 L/min for Torbay (Department of Environment and Conservation 2014). Thus, municipal well fields are not likely feasible in this setting.

Additional investigation could be conducted to identify potentially viable exploration targets based on known water-bearing geologic contacts or fracture zones. The numerical model built for this study does not incorporate such discrete and local features.

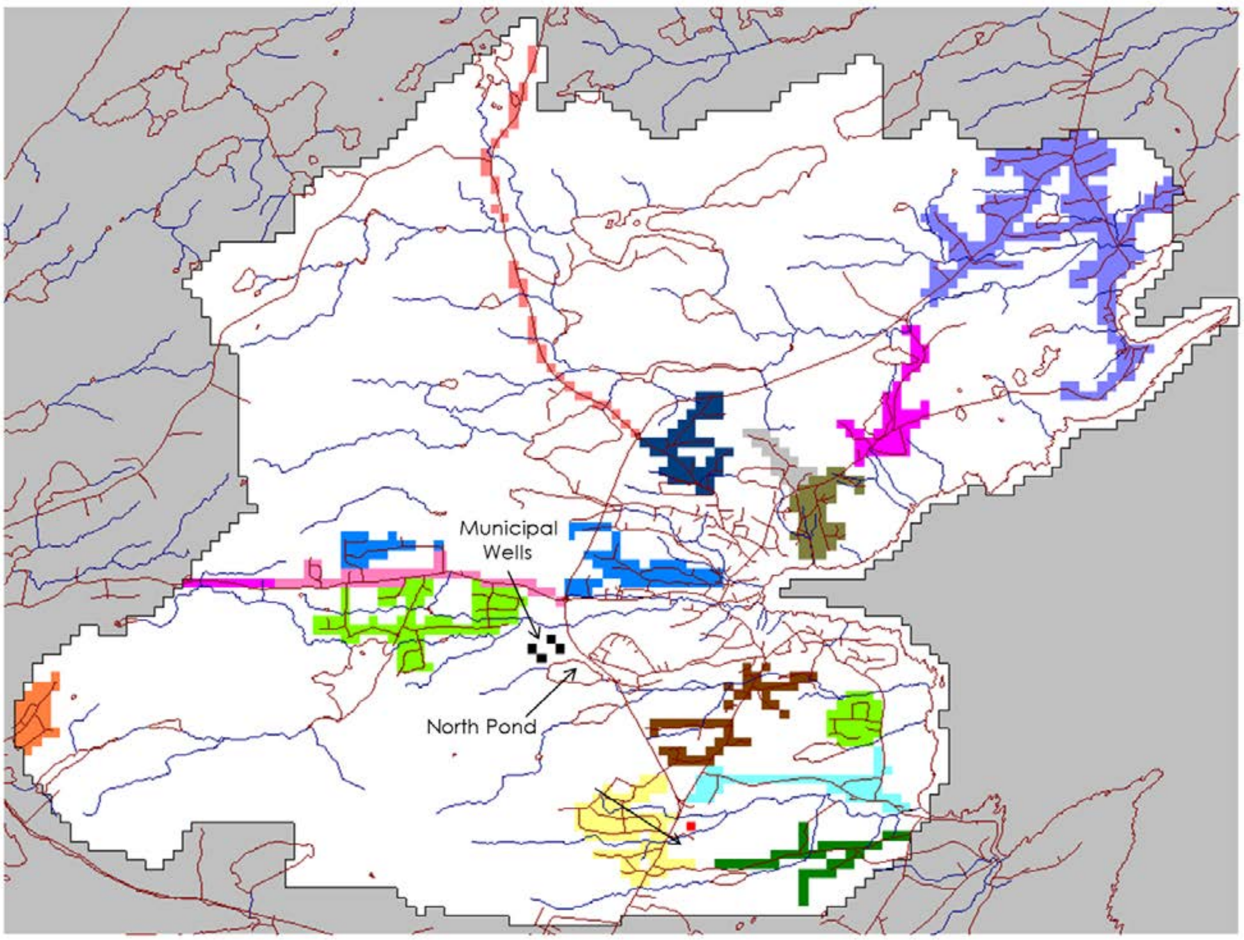

Figure 36 Predictive Scenario 3 plan view of pumping well boundary conditions assigned to cells in layer 4. Black cells represent four municipal wells adjacent to North Pond.

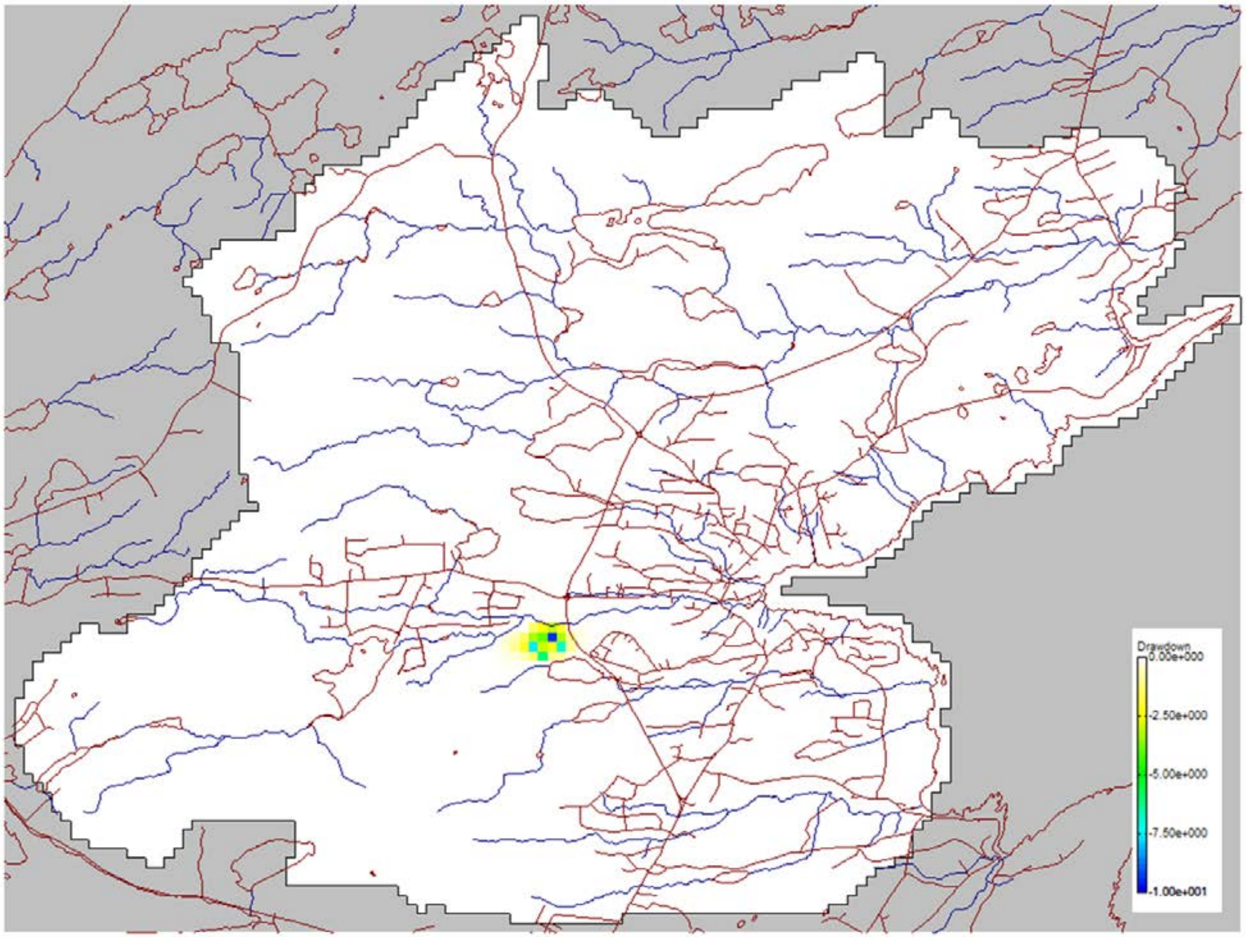

Figure 37 Predictive Scenario 3 drawdown (in metres) in layer 4 due to new municipal well development relative to the base case.

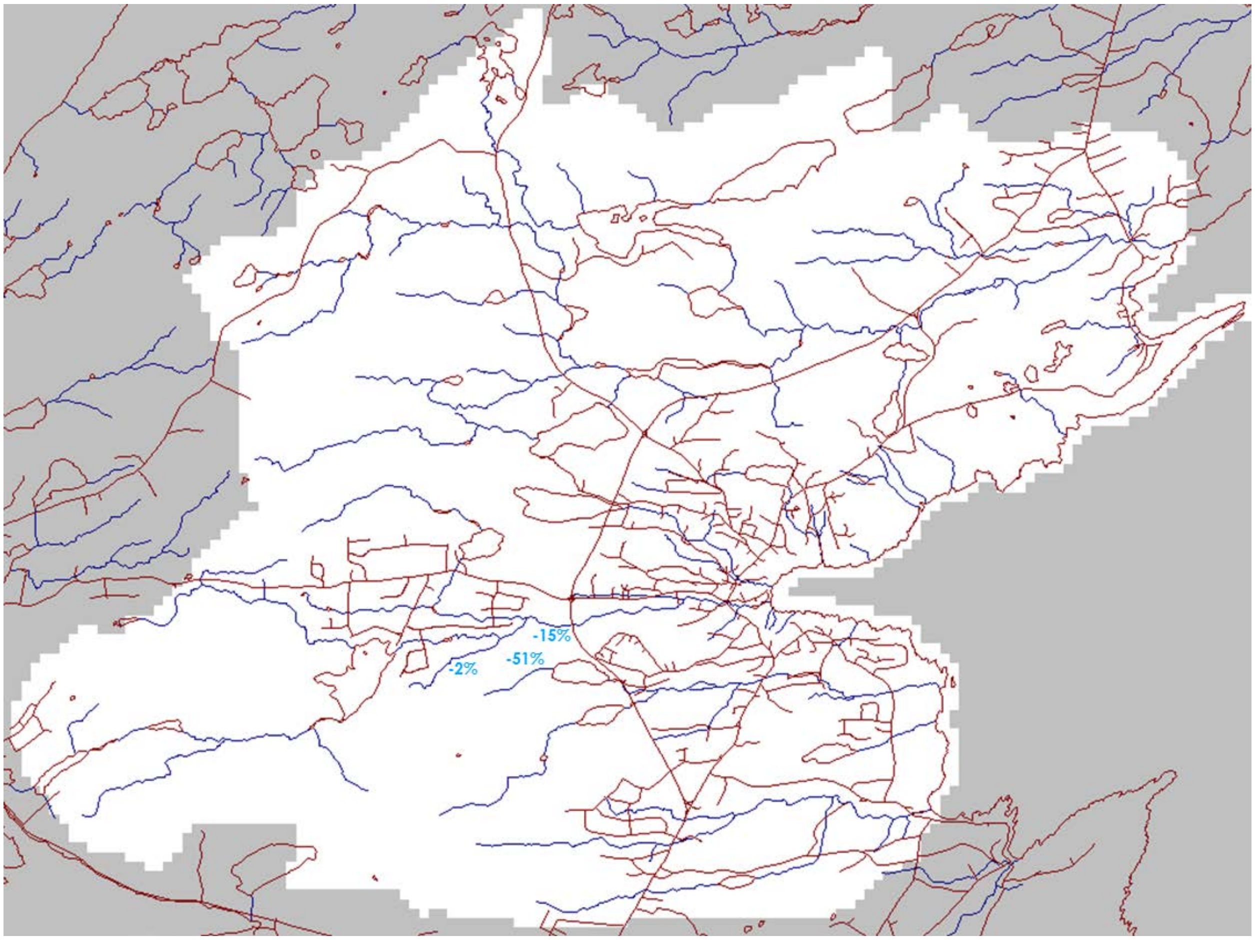

Figure 38 Predictive Scenario 3 estimated percentage change in baseflow to streams.

7.0 LIMITATIONS

The numerical model was prepared using a simple conceptual hydrostratigraphic model, and assumed homogenous properties. Features that may act as preferential pathways or barriers to flow in bedrock aquifers, such as fractures and faults, are not modeled discretely. Discrepancies between field observations and model predictions are likely to arise because of this and the fact that the model properties are calibrated to capture regional more than site-scale groundwater system behaviour.

The available drawdown metric used to define an adverse pumping condition does act to incorporate discrete fracture features in the assessment of drawdown due to new residential and commercial development. Available drawdown statistics were derived from records for wells within the Study Area. However, this could be refined to be more site-specific, if needed, and could benefit from the identification of water-bearing zones being a mandatory component of well records.

Estimates of changes in stream baseflow due to new unserviced development were computed at the scale of the stream segments identified during model construction. While this approach does help to show the “footprint” of new development on groundwater and surface water resources, it does not directly say what the implications are for baseflow impacts that are relevant to fisheries legislation. For example, while baseflow is estimated to exceed a 50% reduction in some cases in streams segments proximal to new development, the overall baseflow into the larger connected stream network is not as variable. Additional consideration for quantifying groundwater-surface water interaction in this setting and the implications for inland fishery regulatory compliance may be warranted if fisheries are even present, but is beyond the scope of this project.

The steady-state approach used here captures the long-term hydraulic response of the groundwater system to a stress (e.g. residential well pumping). It does not capture the transient behaviour induced by a residential well pump cycling on and off and typical patterns of daily use where demand is highest in the morning and early evening (i.e., peak demand). Times of peak demand are when well interference will be the greatest. Constructing a transient model requires the quantification of storage properties, recharge and time-dependent boundary conditions, and a robust time series dataset of hydraulic head in the domain from monitoring wells. This information is not currently available.

The numerical model employed in this report was built for the purpose of simulating regional groundwater flow and simulating future development scenarios. It is only as good as the data, assumptions, and conceptual model used to construct it and should be updated periodically with new information, if available. Developing a “daughter” model from this “parent” model is suggested for simulating smaller portions of the Study Area in more detail in the future.

8.0 SUMMARY AND CONCLUSIONS

A three-dimensional steady-state groundwater flow model was constructed using MODFLOW to simulate current groundwater conditions in the Study Area (base case), and evaluate the potential effects of new residential and commercial development on existing groundwater well users and stream baseflow, as well as the plausibility of supplementing municipal water supply with groundwater. The model was prepared using a simple conceptual model and hydrostratigraphic framework, and assumed homogenous properties within the units. A reasonable calibration of model parameters was obtained, as evaluated by comparing simulated and observed groundwater levels and estimated baseflow. The parameter values for hydraulic conductivity and groundwater recharge are similar to those obtained from other analyses of field observations.

Predictive simulations of the effects of new residential and commercial developments on existing unserviced developments show impact is low in terms of reduced available drawdown or dewatering of water-bearing features. Drawdown and changes to baseflow have been determined to be localized within the immediate vicinity of new development. However, future consideration of changes in stream baseflow and compliance with inland fisheries regulations may be warranted.

Supplementing municipal surface water supply with a groundwater source is not likely feasible because of low well yields and available drawdown constraints in the Study Area.

The groundwater model is designed to be a tool for adaptive groundwater resource management and land use planning. Future information obtained from water well records, pumping tests, and the direct observation of changes due to development should be used to update the conceptual and numerical models provided herein in an effort to refine the tool for this purpose.

9.0 CLOSURE

This report has been prepared for the sole benefit of the Town of Torbay. The report may not be used by any other person or entity without the express written consent of Stantec Consulting Ltd. and the Town of Torbay.

Any uses that a third party makes of this report, or any reliance on decisions made based on it, are the responsibility of such third parties. Stantec Consulting Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made, or actions taken, based on this report.

The information and conclusions contained in this report are based upon work undertaken by trained professional and technical staff in accordance with generally accepted engineering and scientific practices current at the time the work was performed. Conclusions and recommendations presented in this report should not be construed as legal advice.

CLOSURE
November 9, 2015

The conclusions presented in this report represent the best technical judgment of Stantec Consulting Ltd. based on the data obtained from the work. If any conditions become apparent that differ significantly from our understanding of conditions as presented in this report, we request that we be notified immediately to reassess the conclusions provided herein.

Regards,

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10.0 REFERENCES

Barnett, B., L.R. Townley, V. Post, R.E. Evans, R.J. Hunt, L. Peeters, S. Richardson, A.D. Werner, A. Knapton, and A. Boronkay. 2012. *Australian Groundwater Modelling Guidelines*.

Batterson, M.J. 2000. "Landforms and Surficial Geology of the St. John's Map Sheet (NTS 1N/10)." Newfoundland Department of Mines and Energy, Geological Survey, Map 99-19, Open File 001N/0661.

Department of Environment and Conservation. 2014. "Average Well Depth and Yield by Community." http://www.env.gov.nl.ca/env/waterres/cycle/groundwater/well/community.html.

Department of Natural Resources. 2015. "Ecoregions of Newfoundland." http://www.nr.gov.nl.ca/nr/forestry/maps/mbarrens_eco.html.

Eckhardt, K. 2005. "How to Construct Recursive Digital Filters for Baseflow Seperation." *Hydrological Processes* 19 (2): 507–15.

Environment Canada. 2015. "Canadian Climate Normals 1981-2010." http://climate.weather.gc.ca/climate_normals/results_1981_2010_e.html?stnID=6720&lang=e&province=NL&provSubmit=go&page=26&dCode=1.

Environmental Simulations International. 2014. "Groundwater Vistas Version 6.75 Build 27."

exp Services Inc. 2012. *Level II Groundwater Supply Assessment, Proposed Venice Holdings Ltdf. and Bibraltar Development Ltd. Residential Development, Logy-Bay-Middle Cove-Outer Cove, Newfoundland and Labrador*. Prepared for Venice Holdings Ltd. and Gibraltar Development Ltd.

———. 2014. *Level II Groundwater Supply Assessment, Nugent Property Development, Logy Bay-Middle Cove-Outer Cove, Newfoundland and Labrador*. Prepared for Rocky Hill Holding Inc.

Fracflow Consultants Inc. 2008. *Hydrogeological Assessment of the Proposed Torbay Road Commerical/Residential Development*. Prepared for Craig Williams, Realty Executives.

Freeze, R.A., and J.A. Cherry. 1979. *Groundwater*. Englewood Cliffs, NJ: Prentic Hall.

Golden Software. 2015. "Surfer 12.6.963." Golden, CO.

King, A.F. 1990a. "Geology of the St. John's Area." Government of Newfoundland and Labrador, Department of Mines and Energy, Geological Survey Branch, Report 90-2.

———. 1990b. "Geology of the St. John's Area." Newfoundland Department of Mines and Energy, Geological Survey Branch, Map 90-120.

Newfoundland Department of Environment and Lands. 1988. *Water Resources Reports, Groundwater Series II*.

Niswonder, R.G., S. Panday, and I. Motomu. 2011. "MODFLOW-NWT, A Newton Formulation for MODFLOW-2005." U.S. Geological Survey Techniques and Methods 6-A37.

Peaceman, D.W. 1983. "Interpretation of Well-Block Pressures in Numerical Reservoir Simulation." *Society of Petrolemum Engineers*, 531–43.

Spitz, K., and J. Moreno. 1996. *A Practical Guide to Groundwater and Solute Transport Modeling*. New York, NY: John Wiley & Sons.

Stantec Consulting Ltd. 2011. *Level II Groundwater Supply Assessment, Proposed Residential Subdivision, Quarry Road, Torbay, NL*. Prepared for P. Sullivan and Sons Co. Ltd.

———. 2013a. *Level II Groundwater Supply Assessment, Proposed Eagle Nest Ridge Development, Torbay, NL*. Prepared for MAE Design Ltd.

———. 2013b. *Level II Groundwater Supply Assessment, Proposed Martin's Meadows Subdivision, Flatrock, NL*. Prepared for Mr. Robert Martin.

———. 2013c. *Level II Groundwater Supply Assessment, Proposed Pine Ridge Valley Subdivision, Torbay, NL*. Prepared for MAE Design Ltd.

———. 2013d. *Level II Groundwater Supply Assessment, Proposed Phase II Scenic View Ridge Development, Torbay, NL*. Prepared for KMB Holdings Ltd.

Watermark Numerical Computing. 2005. *PEST, Model-Independent Parameter Estimation, User Manual: 5th Edition*. www.pesthomepage.org.