Municipal Groundwater Flow Modelling Study, Town of Logy Bay-Middle Cove-Outer Cove, NL

Stantec

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Executive Summary

The Town of Logy Bay–Middle Cove–Outer Cove (the Town) commissioned Stantec Consulting Ltd. to develop a groundwater model to evaluate potential impacts of residential development on groundwater availability in the Town. The model is meant to be used as a planning tool to assess groundwater supply potential in the town with respect to various future residential and commercial development scenarios.

A three-dimensional groundwater flow model was constructed using the MODFLOW software package; an international standard for simulating and predicting groundwater flow. The model was calibrated by comparing model-predicted groundwater levels to recorded groundwater levels in selected water wells in the vicinity of the Town. Once the model was considered calibrated to baseline conditions, it was used to predict groundwater level changes and changes to river and stream flow resulting from additional groundwater withdrawal from further residential expansion as per a development plan provided by the Town.

The model indicates that water levels in the underlying aquifer are not expected to drop below acceptable levels if the current policy of the development of residential lots with a minimum area of one acre is maintained by the Town. Although predicted groundwater level changes are considered acceptable, larger lot sizes (2 to 5 acres) may be more appropriate in more sensitive areas (*i.e.*, along Marine Drive near Stack's Point, north of the Marine Drive-Middle Cove Road intersection, the end of Doran's Lane, and the area southeast of Cobbler Crescent) to further minimize changes in groundwater levels. The overall impact on stream baseflow was predicted to be with acceptable limits in the scenarios that were modelled. Changes to stream baseflow are limited to the immediate vicinity of new development and are generally most pronounced in smaller tributaries to larger watercourses. The presence of wetland areas within the Town should be preserved to maintain baseflow conditions in local streams and recharge to the underlying aquifer.

Future development in unserviced areas of the Town are still subject to existing Newfoundland Department of Municipal Affairs and Environment (NLDMAE) guidance for groundwater extraction. This includes the restriction on the use of open loop geothermal systems. Further, re-evaluation of the model predictions may be required if the Town's development plan changes (e.g. commercial development, development outside the currently proposed areas, etc.).

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, aquifer tests, and the direct observation of changes due to development within the Town should be used to update the model to refine the tool for this purpose.

1.0   INTRODUCTION

Stantec Consulting Ltd. (herein referred to as "Stantec") was retained by the Town of Logy Bay-Middle Cove-Outer Cove (the "Town") 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 that 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. The model is meant to be used as a planning tool to assess groundwater supply potential in 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.

1.1   Scope

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

- Developing a conceptual groundwater flow model using existing data;
- Constructing a three-dimensional (3-D) numerical groundwater flow model;
- Calibrating the groundwater flow model; and,
- Preparing model predictions of potential long-term impacts of different future development scenarios on groundwater resources.

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.

1.2   Study Areas

Stantec (2015) prepared a similar model for the Town of Torbay. The Torbay model domain, as shown in Figure 1 was delineated using watershed boundaries as they represent more realistic physical limits than jurisdictional borders (e.g., town limits or municipal planning area boundaries) with respect to regional surface water and groundwater flow regimes. The Torbay model domain overlaps with the Logy Bay-Middle Cove-Outer Cove (LMO) municipal planning boundary in the area including part of Middle Cove Road and Nugent subdivision (Sandalwood Drive and Killick Drive, etc.) area off Pine Line.
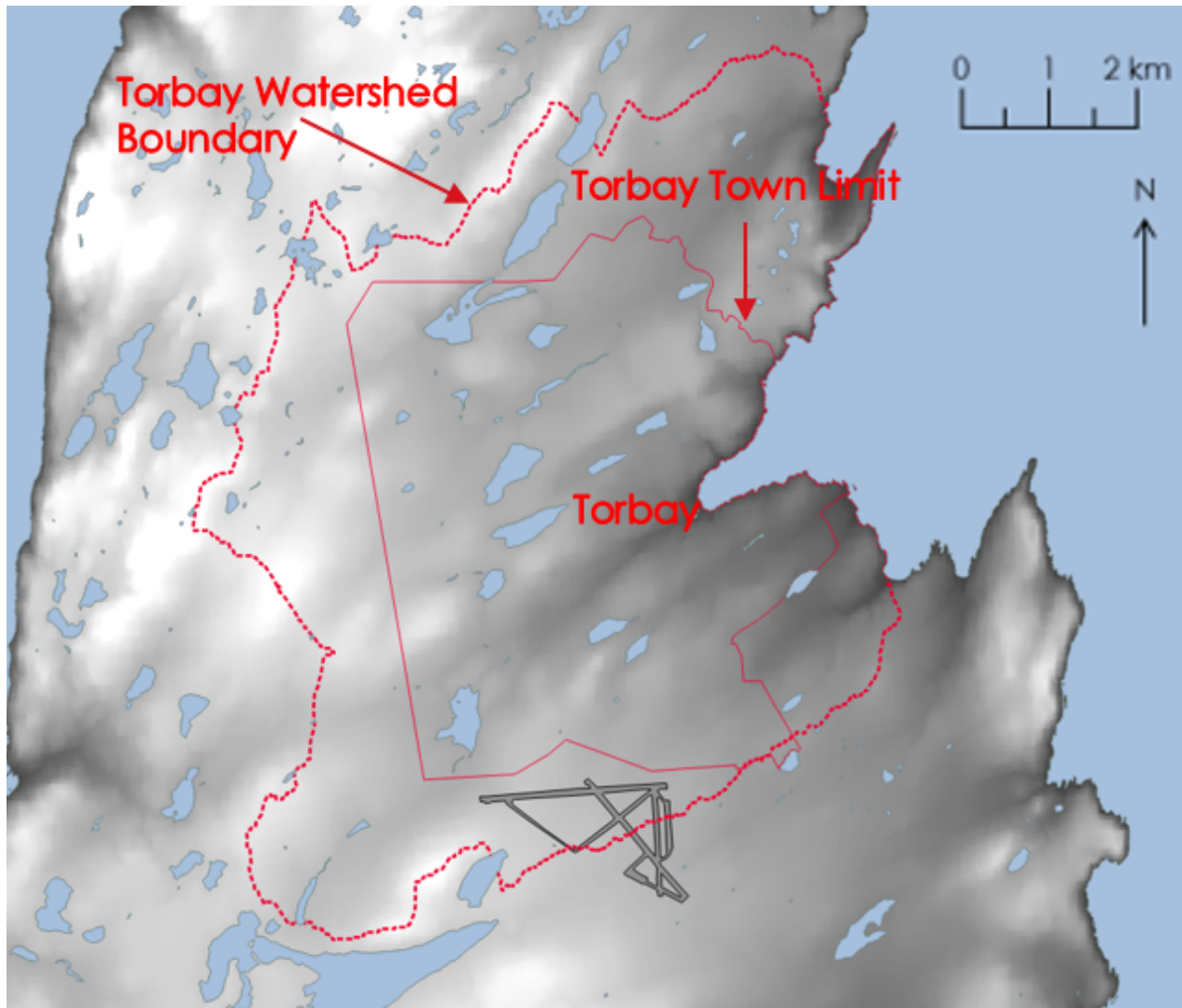
Figure 1   Torbay model domain showing the town limit and the model study area (corresponds to watershed boundaries).

The general approach used in the current modeling exercise was to begin with the Torbay model (with approval from the Town of Torbay) and to expand the model domain to include the watershed areas that encompass the LMO municipal planning boundary – thus forming a larger domain. The dataset used to construct and calibrate the Torbay model is still included in the current model. Using the larger dataset allows the creation of a higher-quality groundwater flow model that provides consistency between the two models (Figure 2), particularly for watersheds where the municipal areas are adjacent to each other.
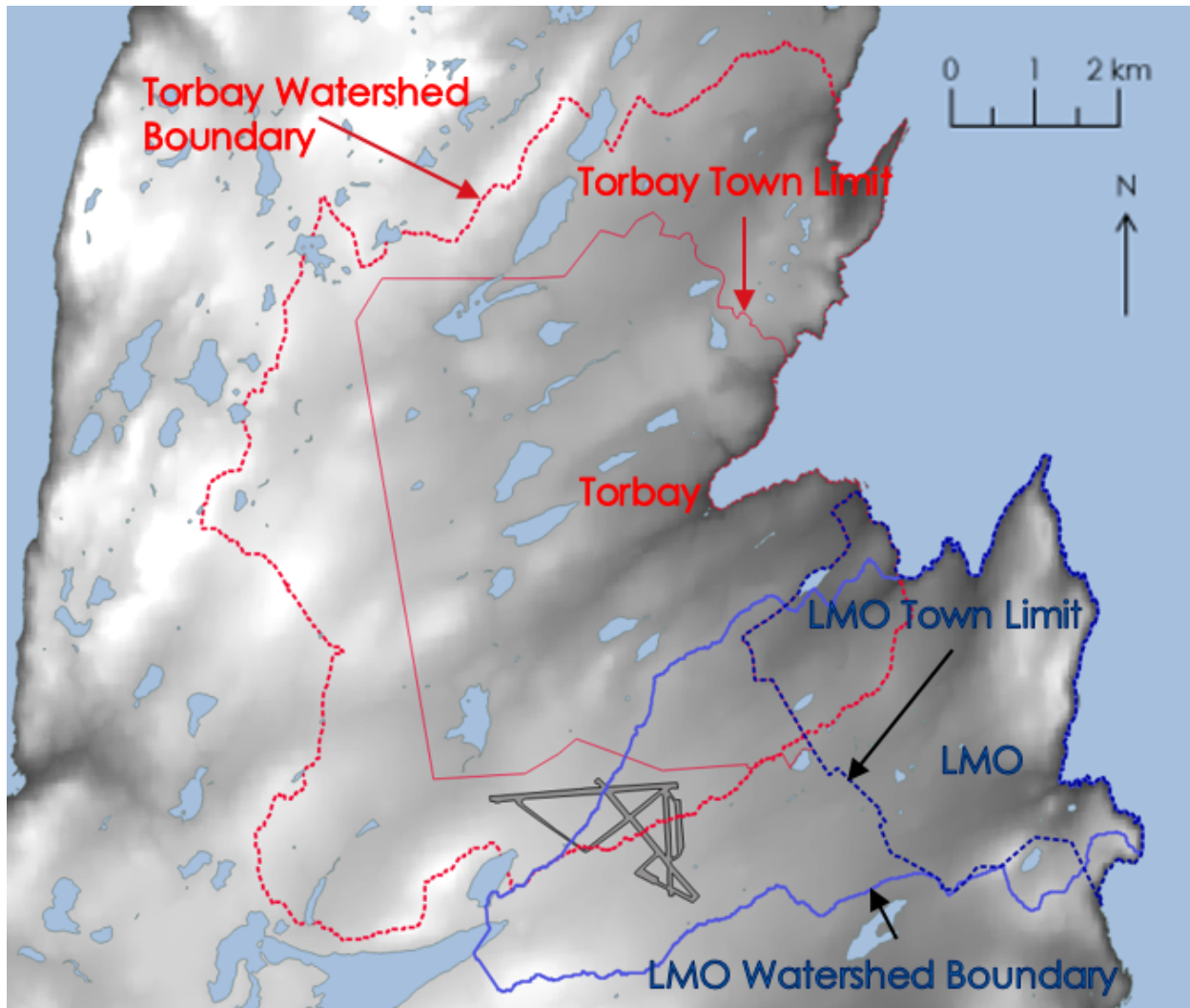
Figure 2   **Study Area for the current evaluation includes both the Torbay watershed boundaries and the LMO watershed boundaries.   Town limits shown for reference.**

Two study areas are considered in this report, a Local Study Area (LSA) which includes the watersheds that overlap with the LMO municipal planning boundary, and a Regional Study Area (RSA) that includes the LSA as well as the watersheds that overlap with the Town of Torbay municipal planning boundary. Although the larger model domain associated with the RSA was considered in the construction and calibration of the numerical model, the base case and predictive scenarios were prepared for only the portion of the model overlapping the LSA.

2.0   BACKGROUND

Based on information provided in the Terms of Reference (TOR) for the project, dated February 22, 2017, all development in the Town is unserviced and landowners obtain their domestic water supply from on-site groundwater wells. Furthermore, LMO is growing and there is concern about impacts new development may have on existing users, the overall cumulative effects of increased groundwater use, and the long-term sustainability of groundwater resources.

Prior to the current study, there has been no town-wide examination of the sustainability of the development of 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 the cumulative, town-wide effects of development on groundwater supply, and the overall sustainability of the community's groundwater resources. The purpose of this groundwater flow model is to be used as a planning tool to assess groundwater supply potential in unserviced areas of the Town with respect to various future residential development schemes.

2.1   Physical Setting

The Town is located north of and adjacent to St. John's on the eastern side of the Avalon Peninsula in Newfoundland and Labrador. The LMO municipal planning boundary covers approximately 17 square kilometers (km^2) (Figure 2).

2.2   Topography and Drainage

The topography in the area of the Town (i.e., the LSA) ranges from 0 to 180 metres above mean sea level (m AMSL) but reaches 210 m AMSL in parts of Torbay. The LSA is characterized by northeast-southwest trending bedrock-controlled undulating ridges/hills and valleys, and a steep rugged coastline. The LSA includes the north-south oriented Flagstaff Hill along the coast between Outer Cove and Logy Bay. In natural, undeveloped portions of the LSA, ground cover is predominantly boreal softwood forest and wetland.

The municipal planning boundaries for LMO and Torbay span numerous surface water watersheds; each containing a network of wetlands, streams, and ponds that ultimately flow northeast and discharge into the Atlantic Ocean. For the purposes of defining a model boundary and Regional Study Area (RSA), watersheds defined through topographic analysis were combined to include the LMO and Torbay municipal planning boundaries. The LSA is 30.8 km^2, and the RSA is 97.3 km^2, accounting for the overlap of the Torbay and LMO components (Figure 2).

2.3   Climate

The Town 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 RSA are predominantly glacial till that occurs as a veneer (<1.5 m thick) and/or as linear ridges, as well as some 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 Conception Group (grey and green sandstone, siltstone, shale and conglomerate), St. John's Group (black shale and slate), and Signal Hill Group (red, grey and green sandstone, conglomerate and shale) (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 north and northeast-trending faults and anticlines and synclines (some doubly plunging to form domes and basins, respectively). Mapped bedding planes range in orientation from near horizontal (10 degree below horizontal in portions of the Torbay Dome) to vertical (King 1990b). Note that beddings planes underlying most of the LSA are oriented north, dipping 60-85° to the east.

2.5   Land Use Zoning

Schedule C of the *Municipal Plan* (Town of Logy Bay-Middle Cove-Outer Cove 2005) defines the minimum lot areas for the following land use zones:

- Residential low density – 4,050 m^2 (except 8,100 m^2 on Doran's Lane);
- Residential medium density – 2,025 m^2;
- Residential estate – 20,250 m^2;
- Commercial-tourism – 12,100 m^2;
- Mixed development – shall conform to residential low-density standards; and
- Agricultural – 4050 m^2.

    Note: 4,050 m^2 = 1 acre

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 the RSA. 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 RSA was developed by taking into consideration available well drilling data and aquifer test results for the RSA, 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 RSA. The general approach used to develop the conceptual and numerical models for the RSA was to add complexity only as warranted by the available data and to achieve the goals of the numerical modeling.

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 RSA from which to obtain stream flow data for baseflow analysis. However, daily stream flow records are available for many monitored streams on the Avalon Peninsula outside of the RSA (Environment Canada, 2017). 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 unregulated and similar in scale to those for the larger streams in the RSA. 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 3.1 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 contributed by baseflow ranges between 20% and 24% for the 11 hydrometric station stream flow data sets. Figure 4 provides a rating curve for the expected baseflow

for a given catchment area based on the results presented in Table 3.1. 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 RSA can be subdivided into eight surface water catchment areas that cover approximately 92.1 km^2 (does not include approximately 5.2 km^2 of land along the coast that discharge directly to the Atlantic Ocean), and with a total estimated baseflow of 75,751 m^3/d (Figure 3; Table 3.2). The volume of daily precipitation for this 92.1 km^2 combined catchment area is approximately 386,860 m^3/d (1,534.2 mm/yr × 1 m/1000 mm × 1 yr/365.25 d × 92.1 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 RSA.
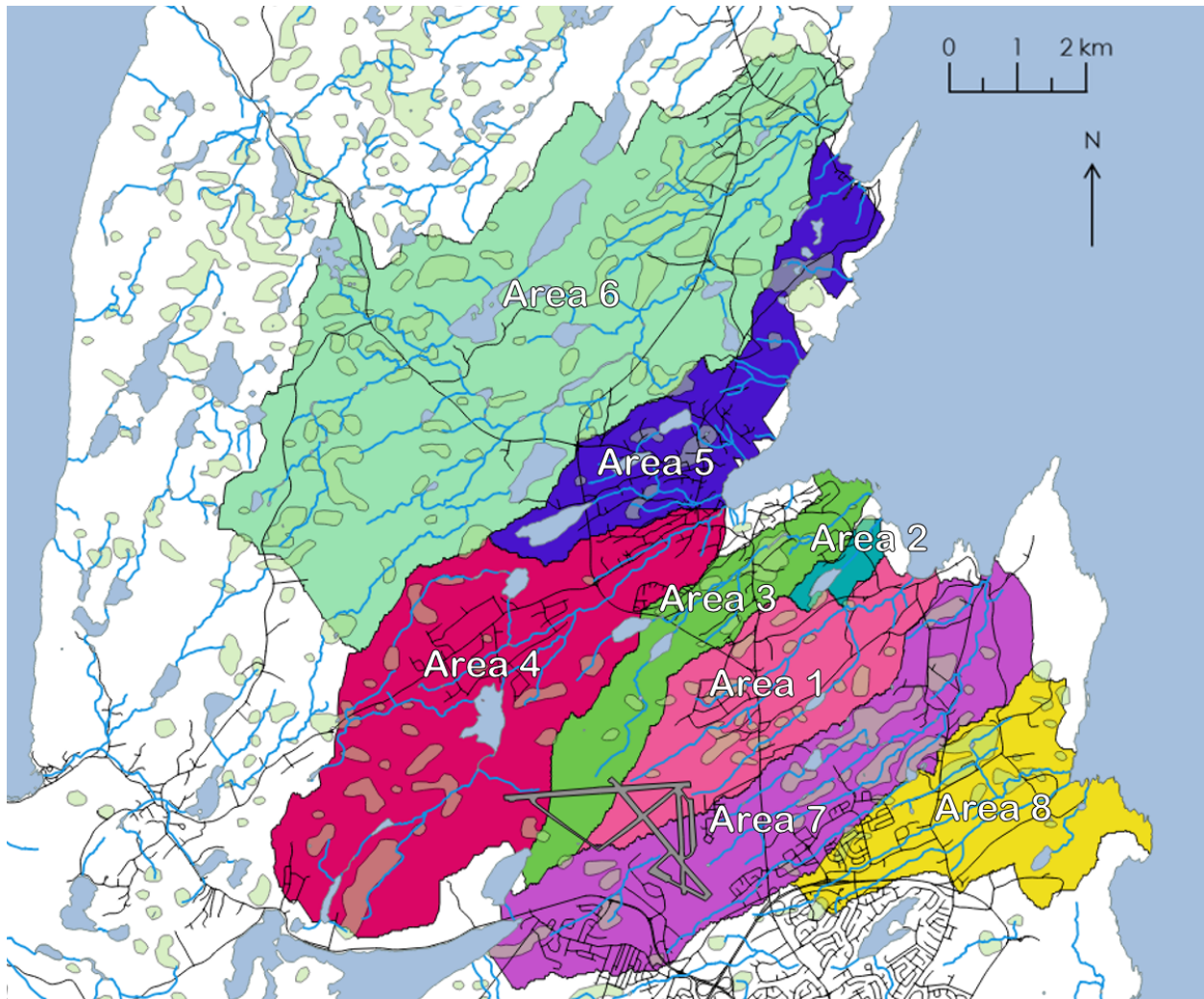
Figure 3   Distribution of surface water catchment areas within the Regional Study Area.

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Table 3.1   Baseflow Estimates from Hydrometric Station Daily Flow Records

Station Code	Location	Catchment Area (km²)	Daily Flow Record	Mean Stream Flow Rate (m³/d)	Proportion of Stream Flow that is Base Flow[1]	Estimated Baseflow (m³/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
				Average	0.223	-

Note:
[1] Estimated using WHAT Analysis for a perennial stream with hard rock aquifers (Eckhardt 2005)
Mean Stream Flow Rate data source: Environment Canada (2017)

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The chart shows the equation:

$$y = 863.66x^{0.9892}$$
$$R^2 = 0.9693$$

X-axis: Catchment Area, square kilometers
Y-axis: Baseflow, cubic meters per day
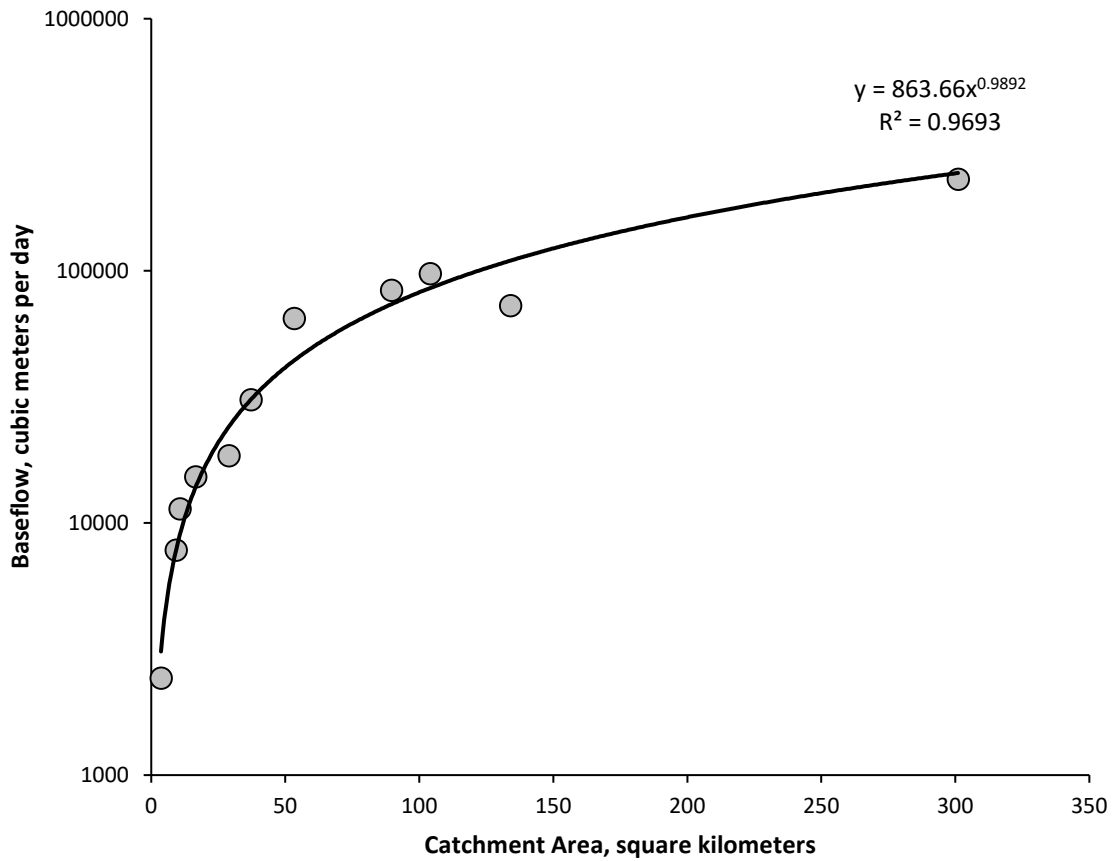
Figure 4   Rating Curve for Estimating baseflow in a given surface water catchment area based on data from 11 hydrometric stations on the Avalon Peninsula.

Table 3.2   Baseflow Estimates for Surface Water Catchment Areas Defined within the Regional Study Area

Catchment Area	Main Watercourse	Area (km²)	Estimated Baseflow (m³/d) [a]
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

Table 3.2   Baseflow Estimates for Surface Water Catchment Areas Defined within the Regional Study Area

Catchment Area	Main Watercourse	Area (km²)	Estimated Baseflow (m³/d) [a]
7*	Stick Pond and Outer Cove Brooks	12.8	10,755
8*	Coakers River and Drukens River	7.4	6,254
TOTAL		92.1	75,751

Note:
[a] Estimated using the power curve fit function in Figure 1.
* These catchment areas incorporate the Local Study Area

3.2.2   Drilled Water Well Records

Drilled water well records were provided by the Department of Municipal Affairs and Environment (NLDMAE, 2014). This information was added to the existing database constructed for the Torbay project (Stantec, 2015).

More than 1,000 records were reviewed for this study, of which Stantec's review yielded at total of 695 records with useable coordinates. Unfortunately, many of the data fields were blank, 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 (Natural Resources Canada, 2017). 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 RSA.

3.2.2.1   Depth to Bedrock

Depth to bedrock (or overburden thickness) was obtained from 668 records. The mean depth from surface to rock was 4.5 m (ranging from 0 m to 76 m) within the RSA. Figure 5 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. As shown on Figure 5, this relationship is consistent between the LSA and the RSA.

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y = 1.0168x - 6.1868
R^2 = 0.983

Ground Surface Elevation (m ASL)

○ Torbay   ○ Logy Bay - Middle Cove - Outer Cove
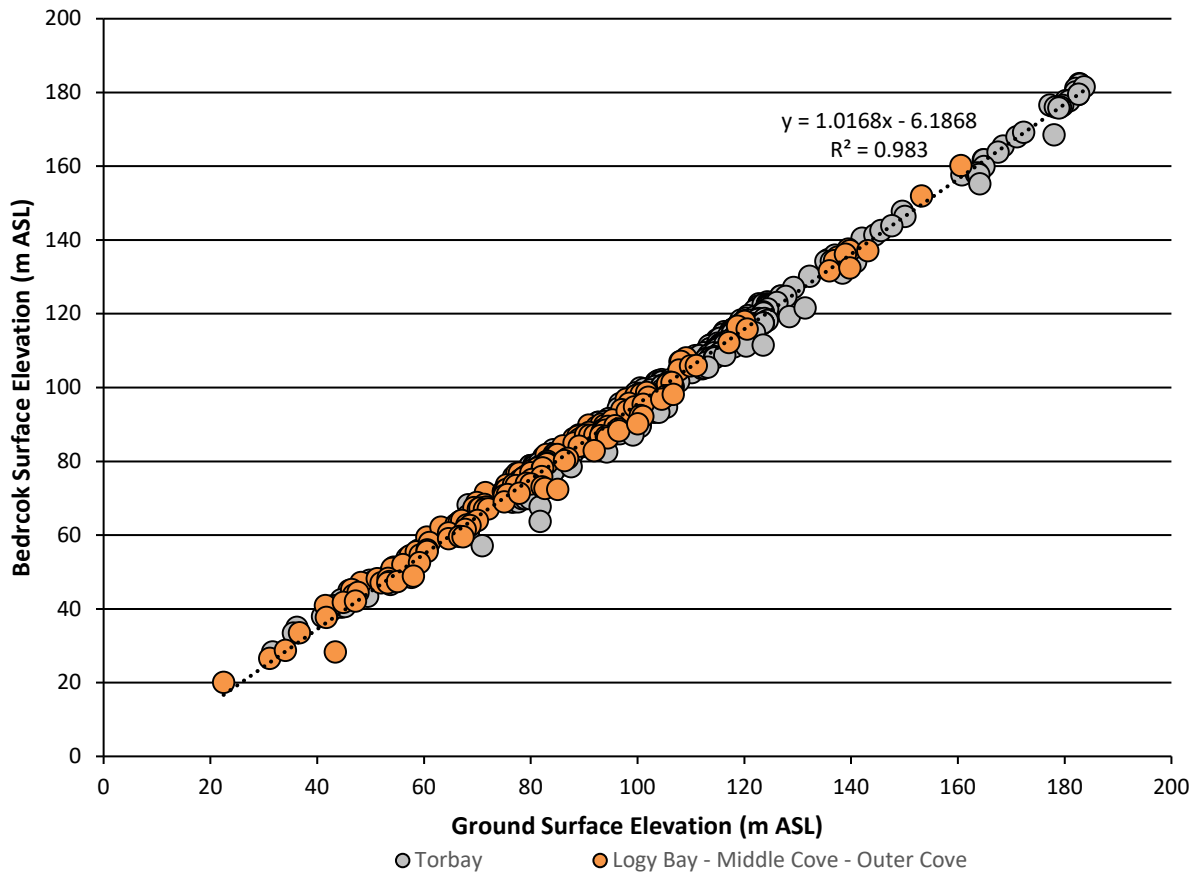
Figure 5   Bedrock surface elevation from well records in the Regional and Local Study Areas.

3.2.2.2 Casing Length

Casing length was obtained from 586 well records. Casing length ranges from 3.6 m to 21 m with an average of 9.1 m. Figure 6 shows that a linear relationship is present between ground surface elevation and the bottom of casing elevation. As shown on Figure 6, this relationship is consistent between the LSA and the RSA.
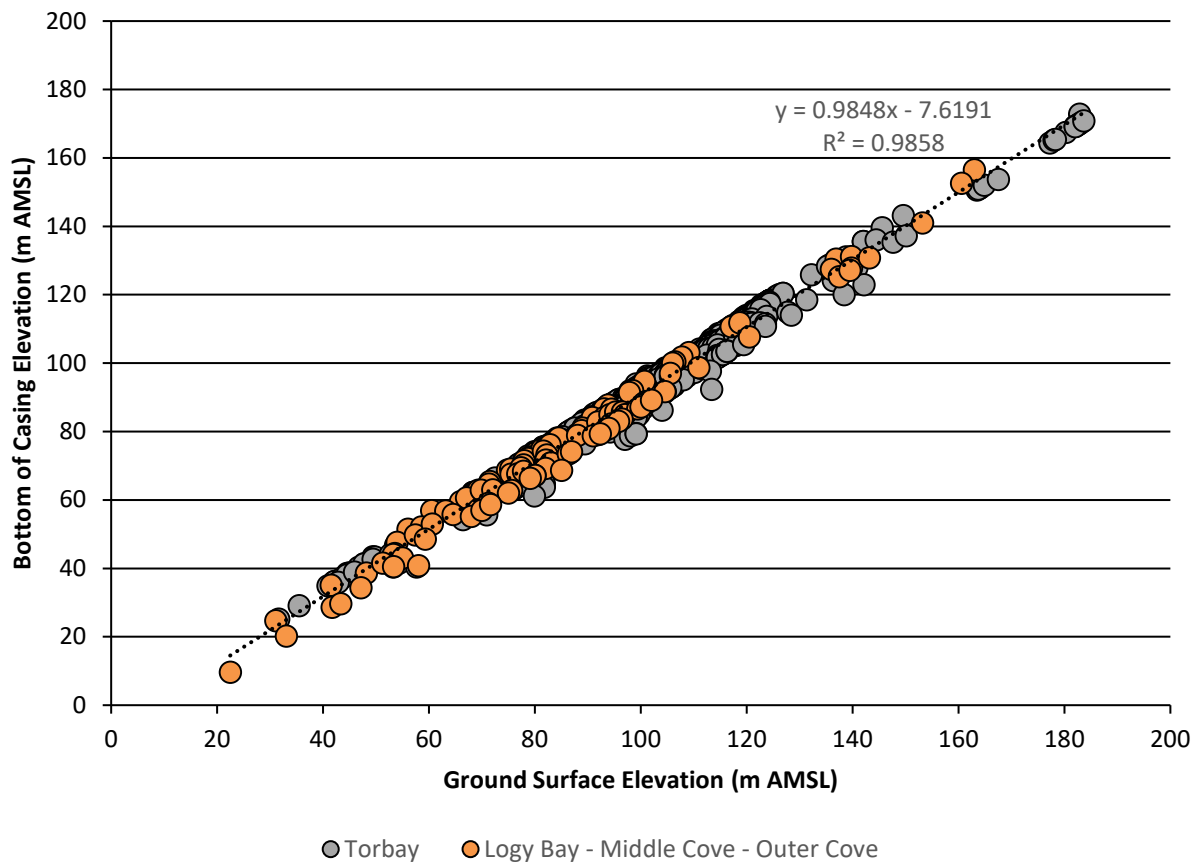
Figure 6   Bottom of casing elevation from well records in the Regional and Local Study Areas.

3.2.2.3   Well Depth

Well depth was obtained from 690 records. Drilled well depths range between 9.5 m and 212 m with an average of 85.7 m. This is similar to the NLDMAE (2014) reported average depth of 60.5 m in Logy Bay (based on 131 wells) and 72.7 m in Torbay (based on 622 records). There is no relationship between ground surface elevation and well depth (Figure 7). 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 (water-bearing features) orientations are inclined. As shown on Figure 7, wells located in the LSA have a similar range to that observed in the RSA.
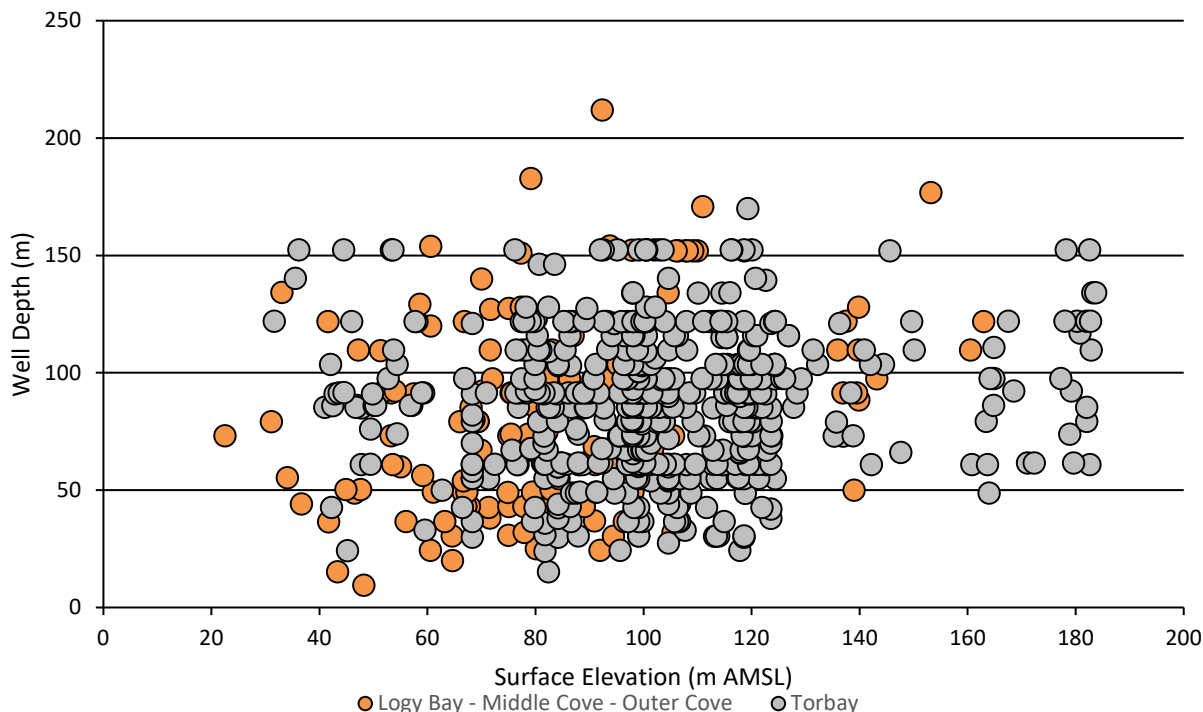
Figure 7   Comparison of well depth to ground surface from well records in the Regional and Local Study Areas.

3.2.2.4   Well Yield

NLDMAE (2014) reports the average well yield in Logy Bay is 12.21 L/min and 14.42 L/min in Torbay. 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 168 records, which was then reduced to 142 records based on values that appeared to be erroneous. Improper measurement of the static water level often occurs when it is measured before the water level in the well stabilizes. Static water level values ranged from 1 m to 18.2 m below ground surface, averaging 3.8 m.

Static water levels were also obtained from various Level II Groundwater Supply Assessments completed for unserviced residential developments in the model domain. Static water levels ranged from 14.79 m to "flowing" (i.e., the water level is above the top of the casing) based on 34 records.

Figure 8 shows a linear relationship between ground surface elevation and static water level (converted to an elevation above mean sea level). Analysis of the available data indicates that the static water level

elevation is generally 96% of the ground surface elevation. As shown on Figure 8, this relationship is consistent between the LSA and the RSA.
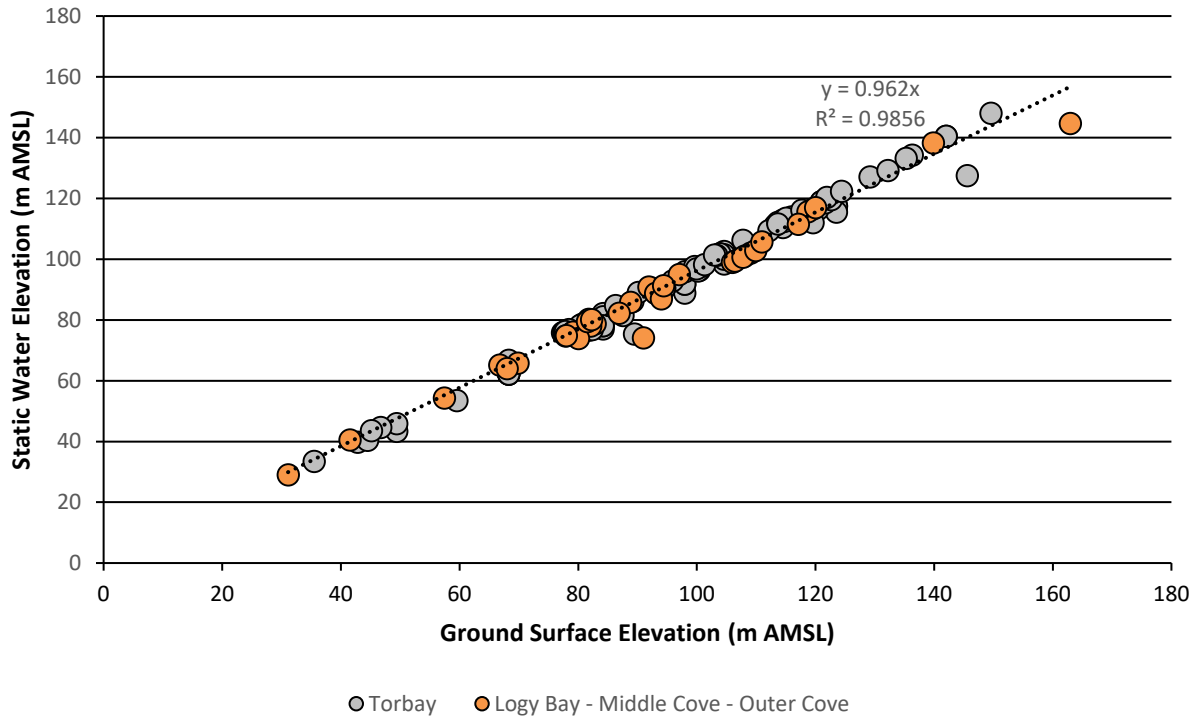
Figure 8   **Comparison of static water level to ground surface from well records in the Regional and Local Study Areas.**

3.2.2.6 Water-Bearing Zones

Of the 695 water well records analyzed, only 535 had water-bearing zones reported. Of the records with water-bearing zones reported, 68% had only one producing zone, 27% had two zones, 5% had three zones, and <1% had four zones.

Figure 9 shows poor correlation between ground surface elevation and the elevation of the uppermost water-bearing feature. As shown on Figure 9, this relationship is consistent between the LSA and the RSA.
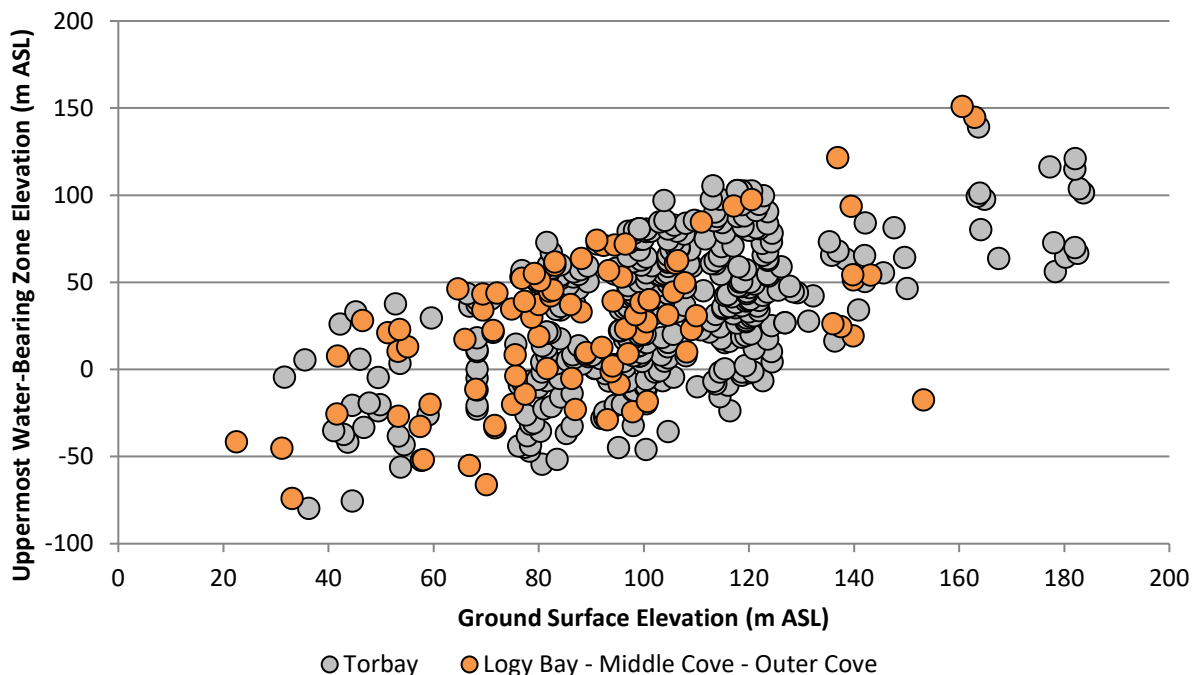
Figure 9   Comparison of elevation of uppermost water-bearing zone to ground surface from well records in the Regional and Local Study Areas.

3.2.2.7   Available Drawdown

Figure 10 is a schematic illustrating the concept of available drawdown. Available drawdown is the distance between the static water level and the water-bearing feature within the well. This is the distance the water level can drop during pumping (drawdown) before the water level is below the feature. Drawdown past water-bearing features can impact flow to a well as de-watered features may close and no longer contribute to flow into the well.

As noted in the previous section, there are relatively few water-bearing features intersecting a typical well in the RSA. 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 8. Figure 11 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 observed when available drawdown was sorted geographically within the RSA.
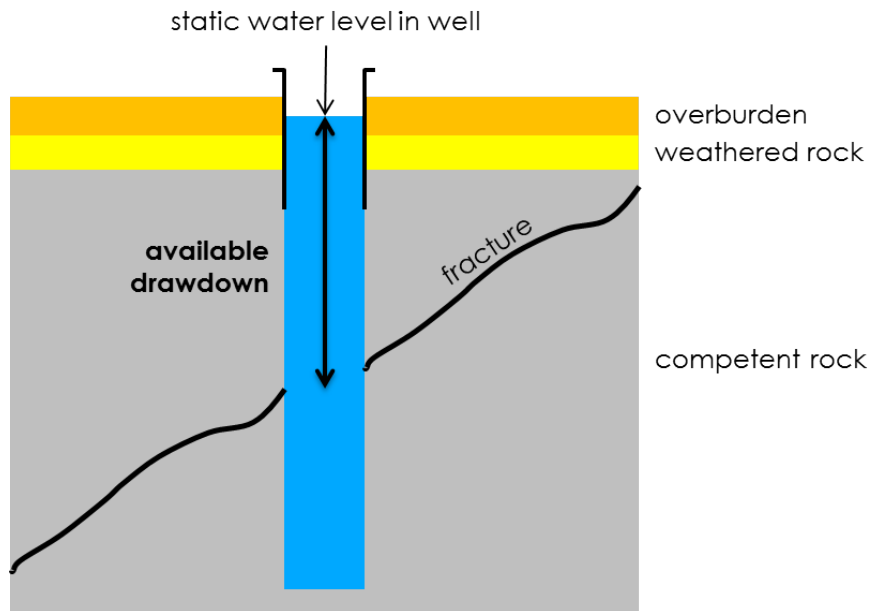
Figure 10 **Available drawdown in a drilled bedrock well. The inclined fracture is the only water-bearing feature in the well. Available drawdown is the length between the static water level and the fracture.**

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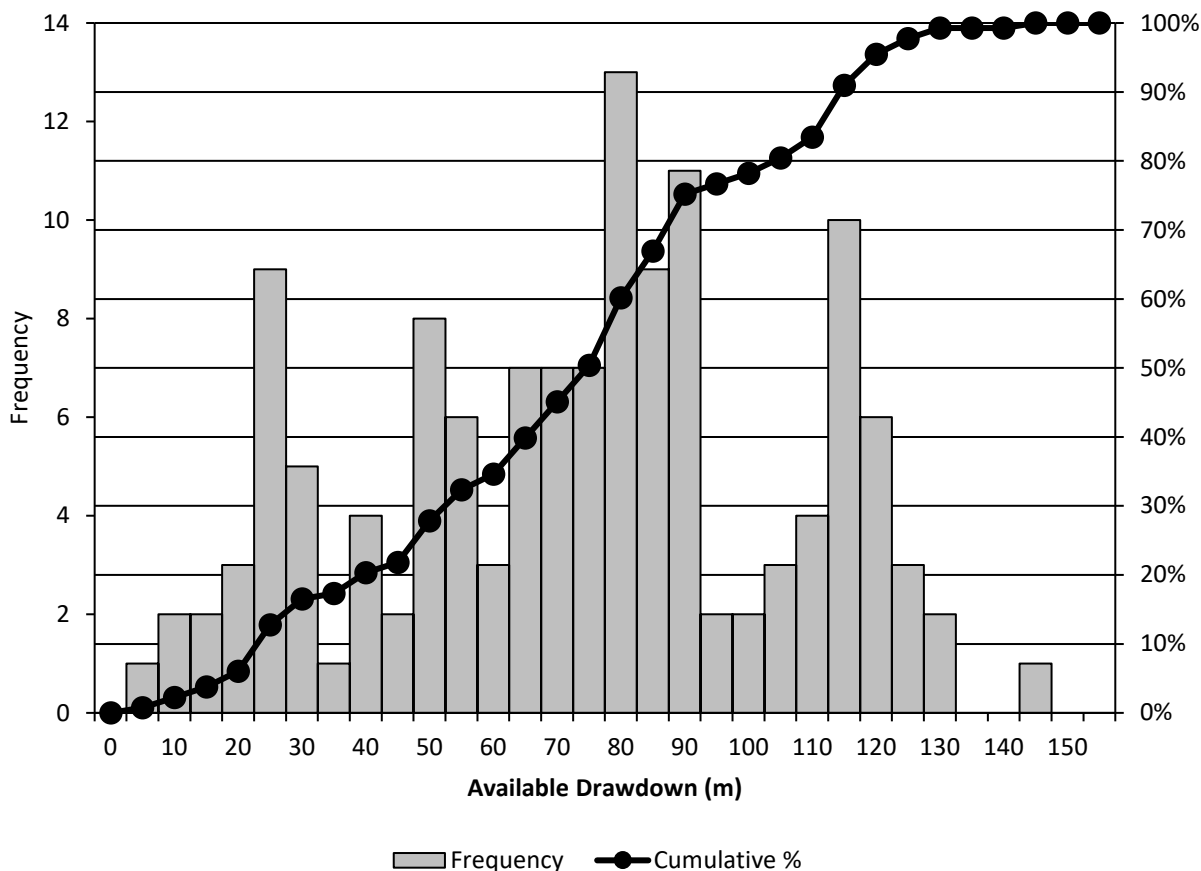
Figure 11 **Histogram of calculated available drawdown from records in the Regional Study Area. For the purpose of this study, available drawdown is measured as the distance between the static water level and the uppermost water-bearing feature reported.**

3.2.3 Aquifer Testing

Several Level I and Level II Groundwater Supply Assessments have been conducted as part of subdivision development within the RSA. Level II assessments include constant rate aquifer 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), if available. 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 3.3 provides a summary of results from seven Level II assessments conducted within the RSA. 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 3.3.

The estimated hydraulic conductivity for the bedrock zone in which the typical open portion of a residential well is located ranges from 1.3×10^{-3} m/d (Marine Drive) to 5.3×10^{-1} m/d (Quarry Road) with a geometric mean of 1.3×10^{-2} m/d. The values fit well within the range hydraulic conductivities expected for fractured sedimentary and metasedimentary rocks (approximately 1×10^{-5} m/d to 10 m/d reported in Freeze and Cherry (1979)).

exp Services Inc. (2012) drilled six test wells in their assessment of the Venice Holdings/Gibraltar Development Subdivision, located in the outer cove - along the southeast boundary of the RSA. 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 in the area. 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 RSA have included testing to evaluate the hydraulic properties of surficial material or the upper zone of the bedrock, which is typically weathered from surface processes and glacial events.

Table 3.3   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)
Marine Drive	1.3×10^{-3}	Fracflow Consultants Inc. (2016)
GEOMETRIC MEAN	1.3×10^{-2}	

3.3   Hydrostratigraphic Framework

The hydrostratigraphy of the RSA 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.

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 RSA 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 3.3, the geometric mean of horizontal hydraulic conductivity estimated from these tests is 1.3×10^{-2} m/d.

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 approximately 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 within the LSA and RSA 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 with numerous geologic processes such as regional deformation, regional stress fields, erosional unloading, and 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 RSA. 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 properties using private wells in the Town was estimated from satellite imagery (Google Earth, 2017) and subdivision lot layout plans according to the Servicing Plan provided by the Town. A total of 1,264 homes were identified within the Town and were each assigned an assumed daily household demand of 1,360 L (i.e., estimated daily demand for a 4-person home (NLDMAE-WRMD, 2009)).

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 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 selected as the numerical groundwater-software application for the 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/. Further, it was demonstrated during the evaluation of municipal groundwater in the Town of Torbay that calibration of MODFLOW parameters is reasonable for the Study Area.

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) 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 RSA was defined by combining watersheds defined through topographic analysis of both the LMO and Torbay Municipal Planning Areas. 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 Natural Resources Canada (2017) 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 136 rows (uniform row spacing of 100 m) and 132 columns (uniform column spacing of 100 m). Grid cells located outside of the RSA are designated "inactive." The total active area of the model is 92.1 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 model layers using the hydrostratigraphic units presented in Figure 12. 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 5 and Figure 6, respectively.

The model grid forms a total of 71,808 cells, of which 40,664 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 m ASL is assigned to active coastal cells in all layers to represent the Atlantic Ocean (Figure 13).

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 13). The bottom of the model domain is also a no-flow boundary condition.
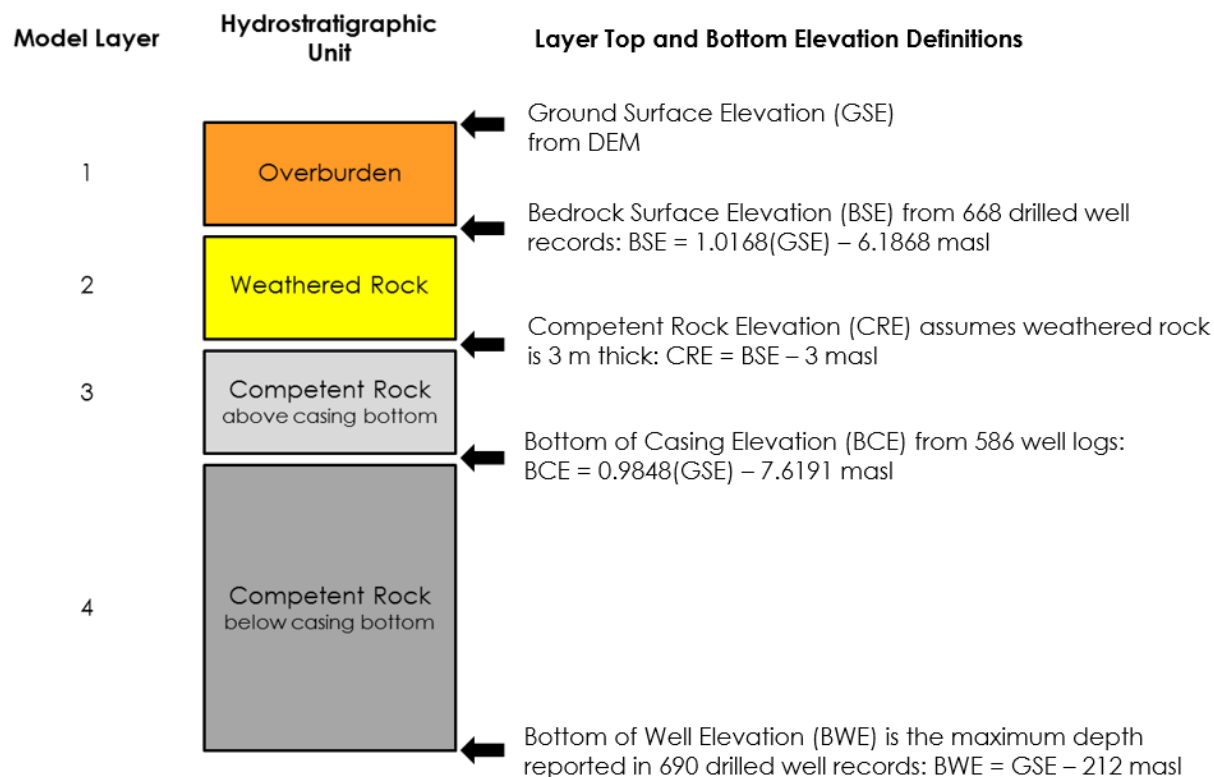
Figure 12 Model layer top and bottom elevation definitions.

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 38 ponds within the RSA (Figure 14). 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 13   Plan view of specified (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 for reference (brown lines).

Figure 14   Plan view of general head boundaries (blue) assigned in layer 1 to represent ponds within the Regional 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 RSA (Figure 15). 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,

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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.
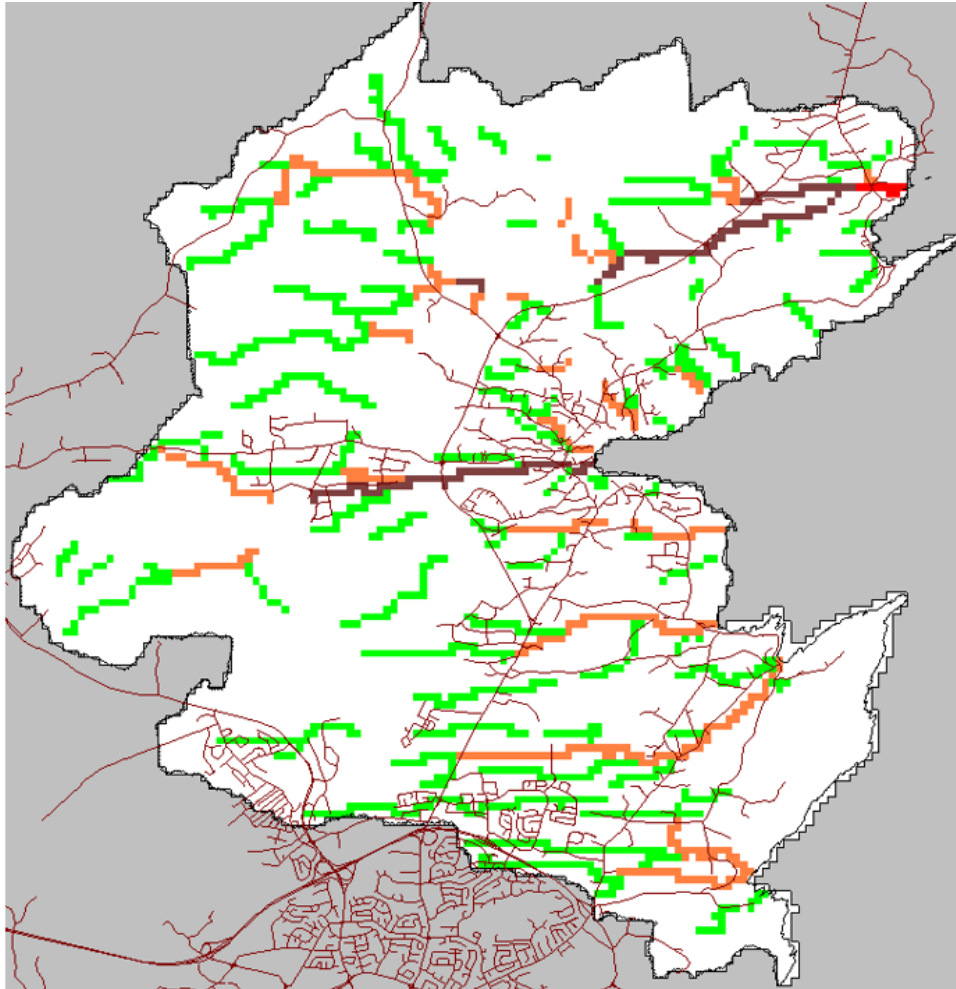
Figure 15   Plan view of river boundary conditions assigned in layer 1 to represent streams within the Regional Study Area. Green = 1st order, orange = 2nd order, brown = 3rd order, red = 4th order.

Table 4.1 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 4.1   Prescribed Characteristics for River Boundaries in Layer 1

Order	Number of Segments	Stage	Bottom Elevation	Width
1	102	Top Elevation of Cell in Layer 1 from DEM	Top Elevation minus 0.2 m	1 m
2	22		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 RSA (Figure 16). 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 3.3) and the defined model layers (Figure 12). Figure 17 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 16 **Plan view of pumping well boundary conditions assigned to cells in layer 4. Each coloured cluster represents a particular unserviced subdivision or area of development (yellow represents existing wells added to the model during the current evaluation).**

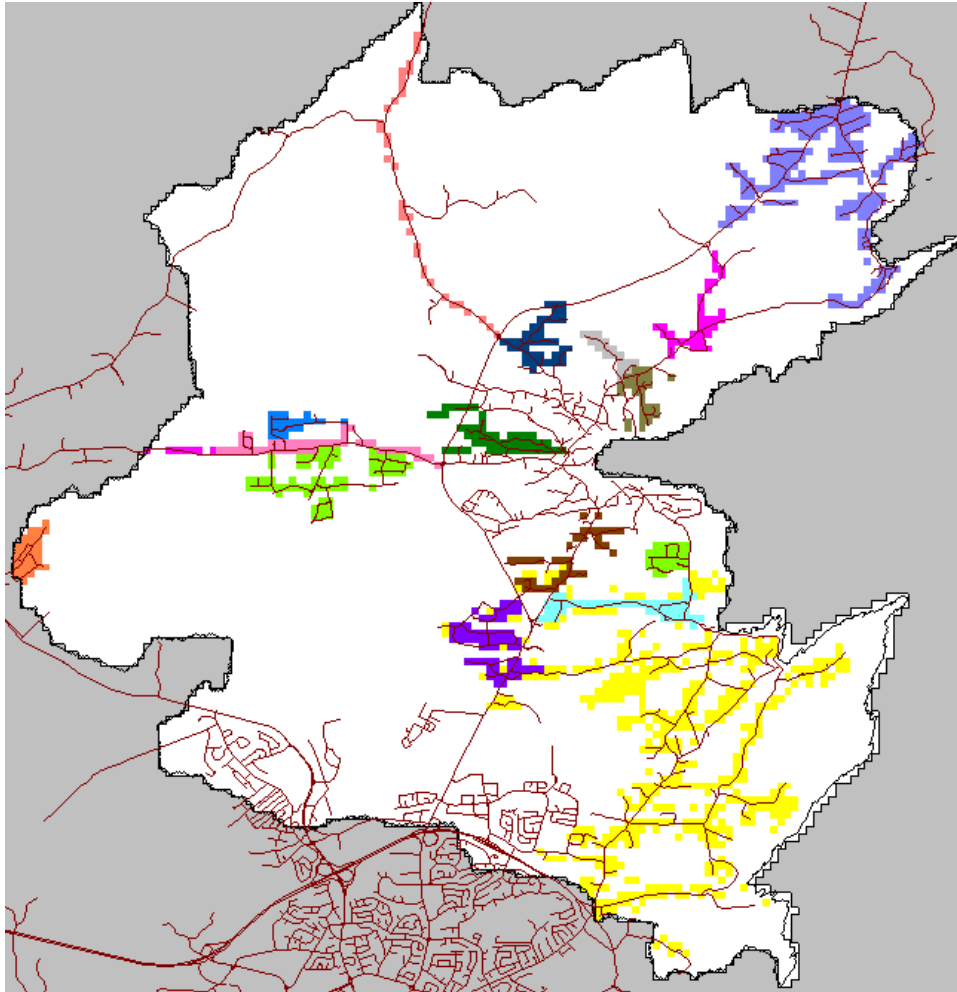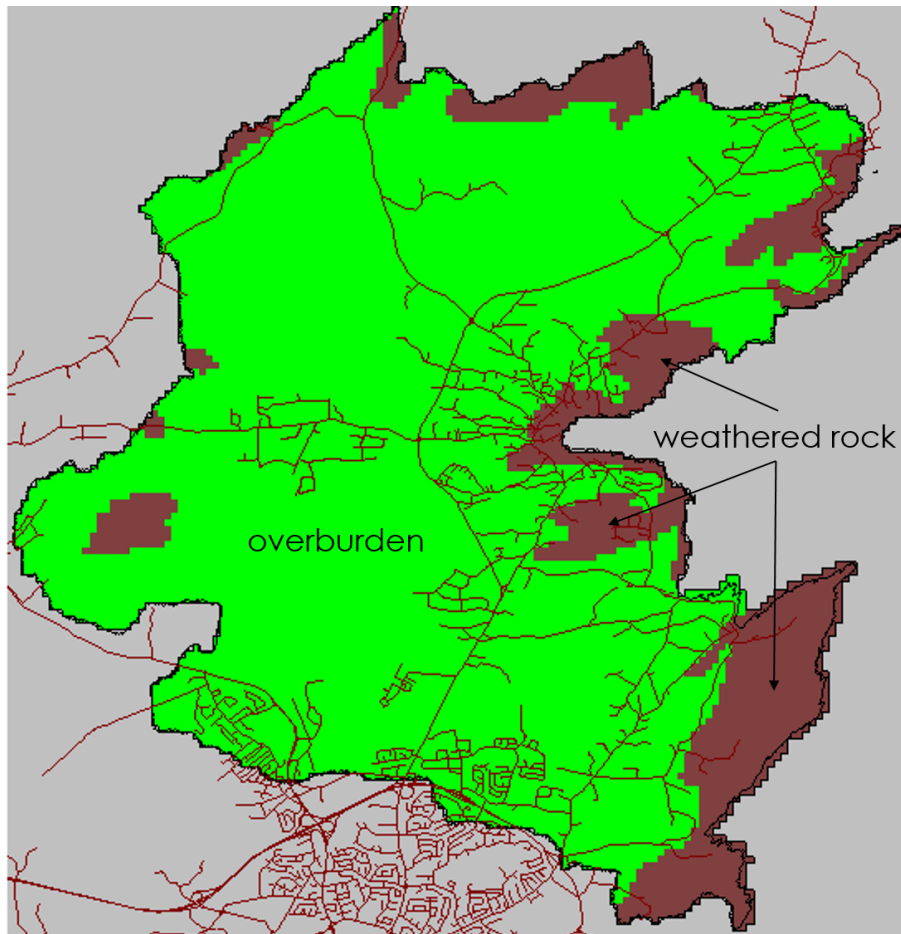**Figure 17   Property zones in layer 1. Weathered rock (brown) extends from layer 2 where bare or concealed bedrock has been mapped at surface.**

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 the RSA 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 facilitate the calibration of 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. PEST can only vary parameter values within the range allowed by the user.

PEST establishes a combination of parameter values that 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 169 head targets in layer 4 were identified within the RSA; 34 of which are within the LSA. Of the 169 head targets, 142 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 2016.   The total number of head targets was reduced to 111 using the target thinning option in Groundwater Vistas (Figure 18). This option allows only one target per cell and it was set to retain the value closet to the mean.

Eight 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. Baseflow targets for the eight defined catchment areas are summarized in Table 3.2.

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

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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.
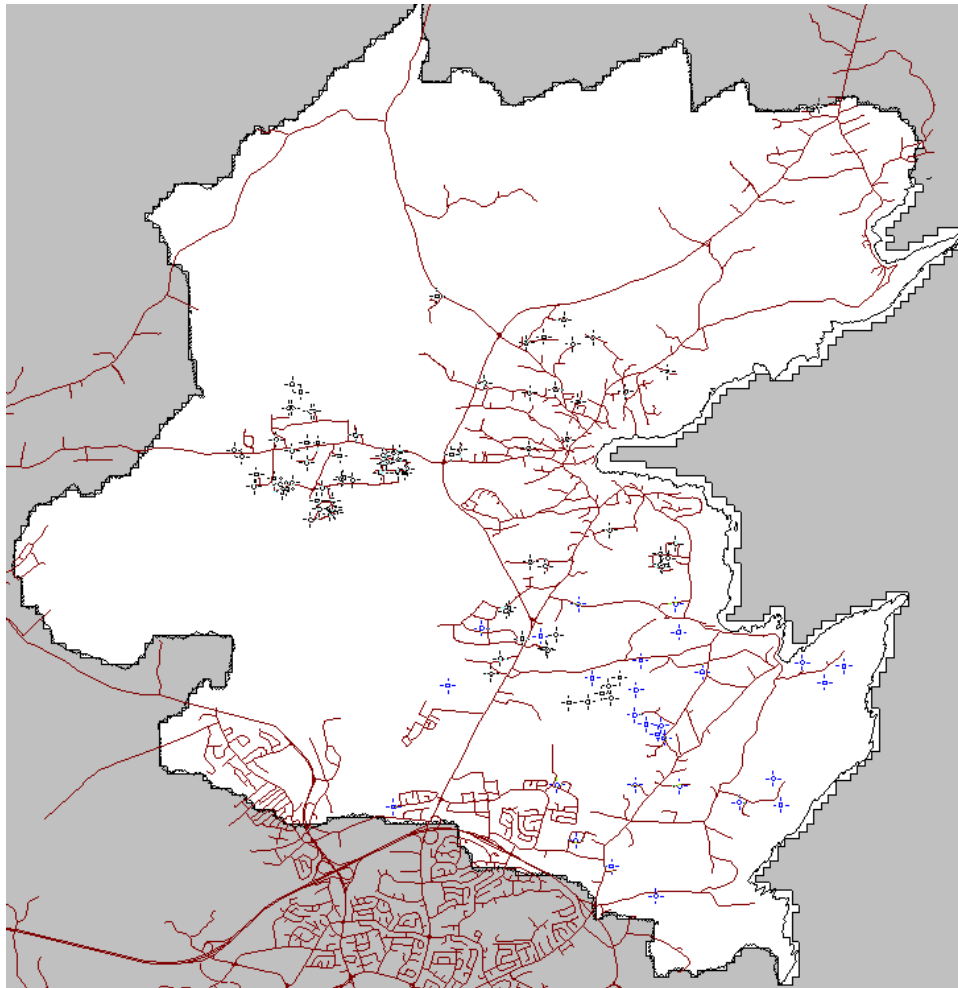
Figure 18   **Head target locations in layer 4. The road network is shown (brown lines). Blue head targets were added during the current evaluation.**

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 RSA.

Figure 19 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 19 represents a perfect match. There is a symbol for each of the 111 head targets in the RSA, 23 of which are within the LSA. 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 -0.77 m ASL, the absolute mean error is 2.66 m ASL, the NRMS is 2.3%, and the correlation coefficient is 0.99.

Flow target residuals are shown in Table 5.1. A flow target residual less than 20% is considered a good match. This condition is met for catchment areas 1, 3, 4, 6, 7, and 8 which collectively cover approximately 90% of the RSA. This is also a good result considering both sub-basins in the LSA (areas 7 and 8) are within this area.

As shown in Figure 19, the relationship between observed and simulated water levels is consistent between the LSA and RSA.

Higher flow residuals for catchment area 2 is likely the result of the quality of the power function used to estimate baseflow (Figure 4) for small areas. As previously stated, the power function is 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.
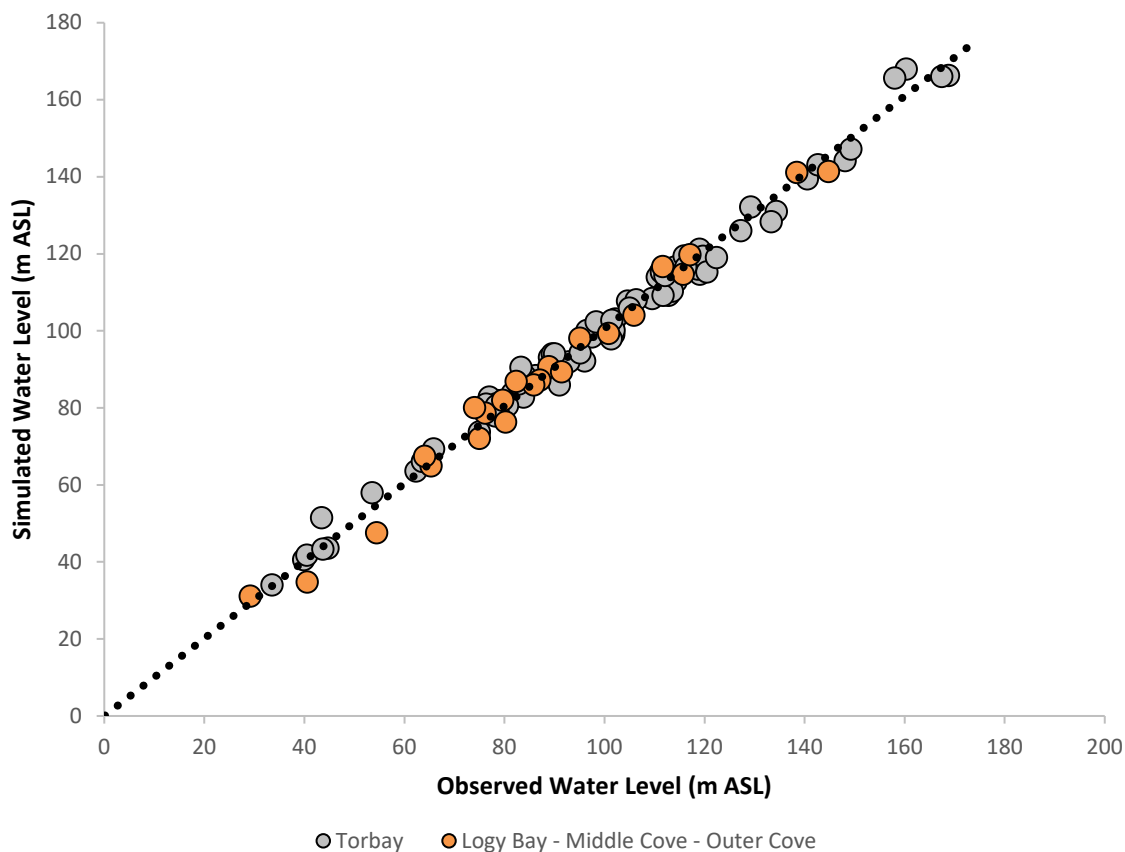
Figure 19   Comparison of observed and simulated water level.

Table 5.1   Baseflow Calibration Residuals

Catchment Area	Area (km²)	Target "Observed" Baseflow (m³/d)[1]	Simulated Baseflow (m³/d)	Residual (m³/d)	% Residual
1*	7.8	6,589	6267	-321	-5
2*	0.7	607	306	-301	-50
3*	5.9	4,999	4430	-569	-11
4	17.7	14,820	16,590	1,770	11
5	7.5	6,338	1,774	-4,564	-72
6	32.3	26,869	29,054	2,185	8
7*	12.7	10,671	11,973	1,302	11
8*	7.4	6,254	5,357	-897	-14
Notes: [1] Estimated using the power curve fit function in Figure 4 * Baseflow target is located in the LSA					

5.4   Steady–State 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 5.2 presents the calibrated parameters.

Table 5.2   Parameters Values Assigned from Model Calibration

Parameter	Initial Value	Calibration Range	Calibrated Value	Anisotropy (Kv/Kh)
Groundwater Recharge	153 mm/yr	77 – 613 mm/yr	346 mm/yr	-
Overburden Horizontal Hydraulic Conductivity (K_h)	0.432 m/d	$2.2×10^{-4}$ – 43.2 m/d	3.05 m/d	0.35
Overburden Vertical Hydraulic Conductivity (K_v)	0.432 m/d	$2.2×10^{-4}$ – 43.2 m/d	1.06 m/d	
Weathered Bedrock Horizontal Hydraulic Conductivity (K_h)	0.216 m/d	$2.2×10^{-4}$ – 43.2 m/d	1.26 m/d	0.84
Weathered Bedrock Vertical Hydraulic Conductivity (K_v)	0.216 m/d	$2.2×10^{-4}$ – 43.2 m/d	1.06 m/d	
Competent Bedrock Horizontal Hydraulic Conductivity (K_h)	0.0216 m/d	$2.2×10^{-4}$ – 43.2 m/d	$2.2×10^{-3}$ m/d	3916
Competent Bedrock Vertical Hydraulic Conductivity (K_v)	0.0216 m/d	$2.2×10^{-4}$ – 43.2 m/d	8.62 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 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 RSA, bedrock structure is observed to be sub-vertical in many places. It is noted that the anisotropic ratio for competent bedrock is extremely high. However, for the current calibration of the model, the parameter was not sensitive in layer 4 and therefore did not have a significant effect on the flow solution. Sensitivity is discussed in Section 5.4.1.

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 3.1, the average percentage of baseflow in total stream flow is 22.3% in this terrain (minimum = 15%, maximum = 24%). The calibrated value of 346 mm/yr for groundwater recharge is 22.6% 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, 3, 4, 6. 7, and 8.

5.4.1   Parameter Sensitivity

PEST reports the sensitivity of parameters adjusted during the calibration process to the residuals of the calibration targets. The final parameter sensitivity (Figure 20) provides an indication of the relative effect adjustments to each parameter has on minimizing target residuals. The groundwater recharge rate is 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.
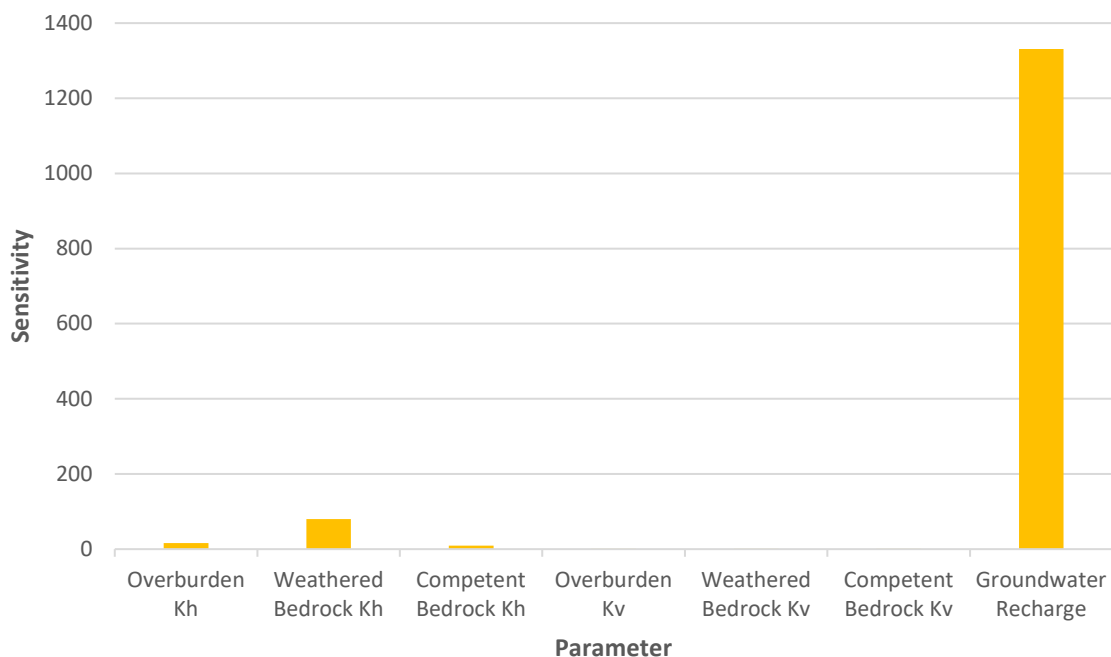

Figure 20   Final Calibrated Parameter Sensitivity.

6.0   MODEL APPLICATIONS

Increased groundwater extraction from new residential developments within the LSA have the potential to alter the water balance within the watershed. This could result in the lowering of water levels in existing wells, or reduction in baseflow to streams and ponds. Four development scenarios were constructed to evaluate if these effects have the potential to occur and if so, the magnitude of these effects. All of these scenarios were conducted using the calibrated RSA but focused on the LSA.

6.1   Existing Conditions – Base Case

The base case for the sub-division development scenarios is the existing steady-state conditions developed as a result of the model calibration. This condition will be used to evaluate drawdown (a decline in water level) resulting from new unserviced development. Figure 21 shows the calibrated, steady-state hydraulic head contours in the competent bedrock (model layer 4). Hydraulic head would be expected to be nearly identical in all layers.

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 LSA (approximately 83%) discharges to ponds and streams. Secondly, 7.3% of groundwater recharge enters the competent rock. This corresponds to only 1.6% of total annual precipitation. Finally, approximately 4% of the groundwater recharge within the LSA is extracted by wells. This is a small volume of water within the water balance compared to the volume discharging to ponds and streams.
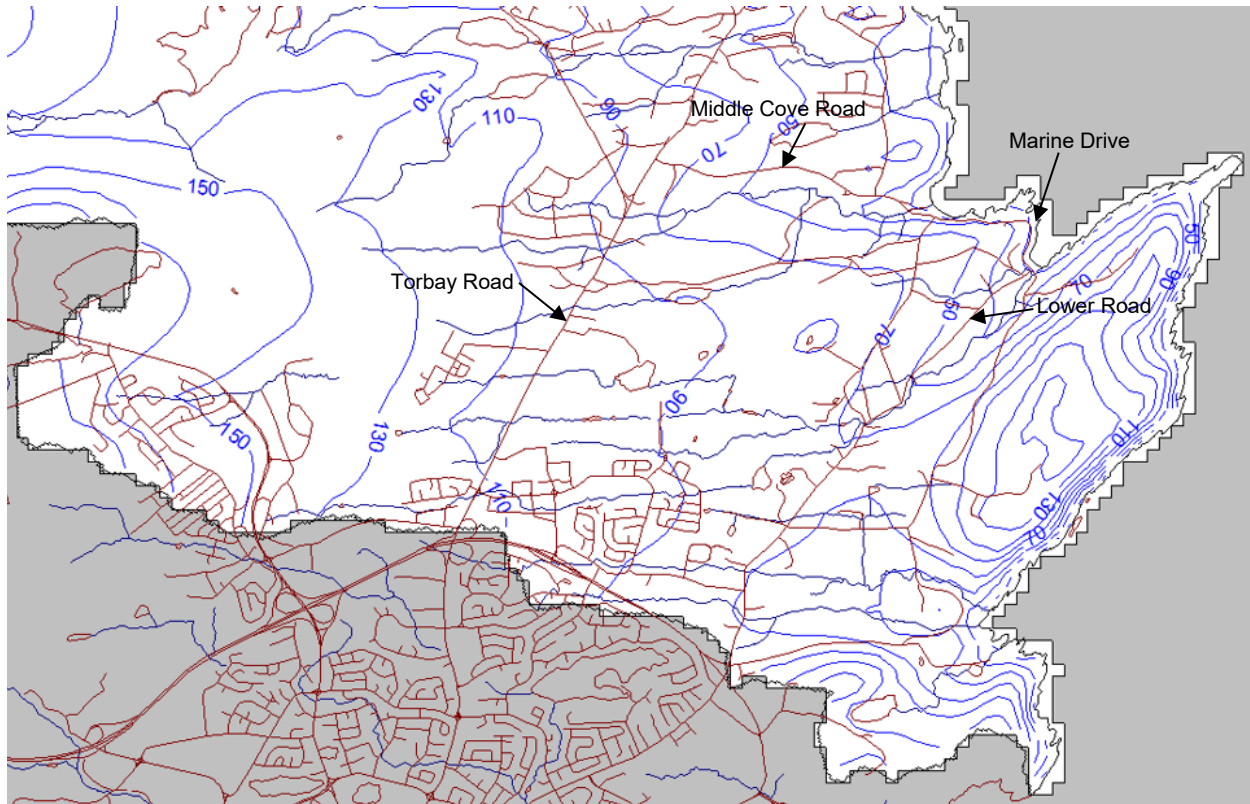
Figure 21 Steady-state hydraulic head contours of model layer 4. Base case simulation. Contours measured in metres.
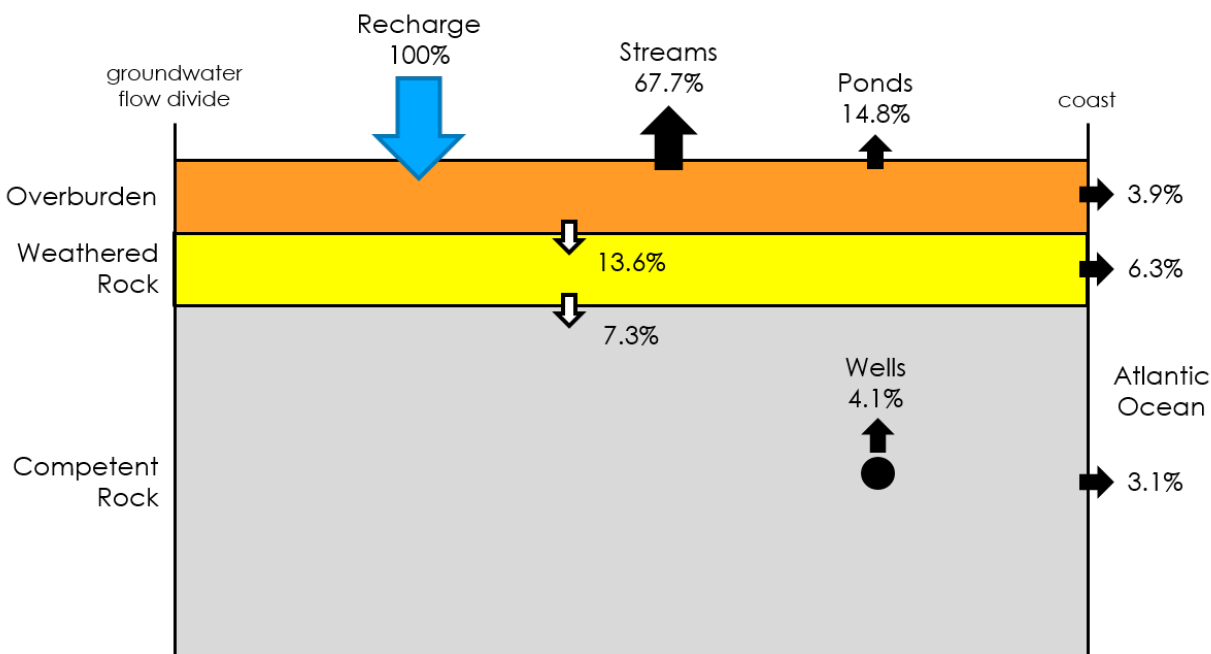
Figure 22   Base case simulated steady-state water balance. Black arrows indicate water is being removed from the groundwater system. White arrows indicate the direction of internal groundwater flow. Percentages are net values relative to groundwater recharge (the only source of water into the domain).

6.2   Predictive Scenarios

The base case model was modified to simulate several predictive scenarios to assess the effects of new residential development on the groundwater system, with particular focus on the potential interferences that changes in water levels may have on existing private wells.

Baseflow was also simulated for each sub-basin to quantify the change in baseflow for each predictive scenario. Predicted changes in baseflows for each scenario are presented in Table 6.1 along with a general discussion of impacts to baseflow at the end of this section.

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 11, 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). Therefore, a correction factor is applied to account for the difference between calculated drawdown in a model cell and actual drawdown in a well. 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).
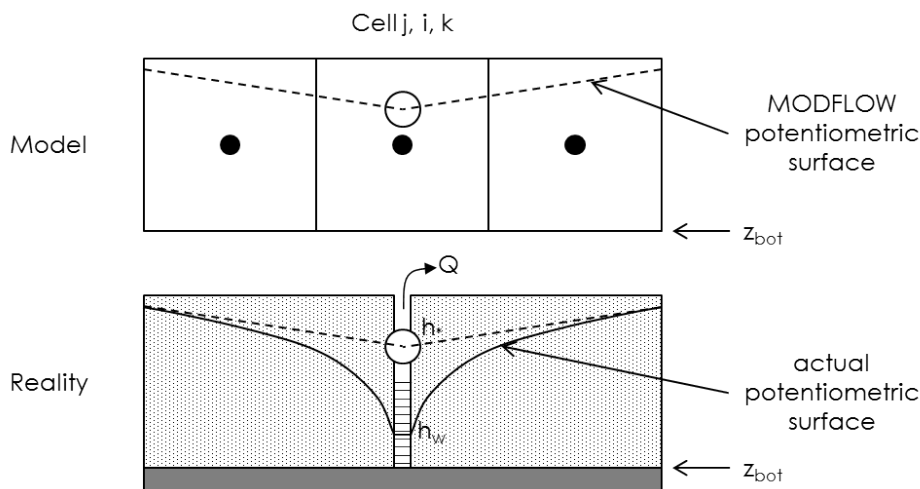

Figure 23   An example of the difference between MODFLOW results and the actual potentiometric surface in a pumping well.

The values used in this calculation are: T = 1.05 m²/d and Δx = 100 m. Transmissivity was estimated based on the geometric mean of transmissivities recorded during Level II assessments in the area. For residential

wells added during the current evaluation, 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.

6.2.3 Predictive Scenario 1 – Completion of Existing Subdivisions

This scenario involves the completion of all existing residential subdivisions and further development as outlined in correspondence with the Town in November 2017. New household demand is implemented by adding the well boundary condition to model cells in layer 4. Thus, the "footprint" of the development increases to the size of the planned or built road network. In areas where a new development cell overlaps with cells from the baseline case, the highest pumping rate between the two is applied to the cell. The locations of existing wells (shown in yellow) and further proposed development (shown in orange) are shown in Figure 24.
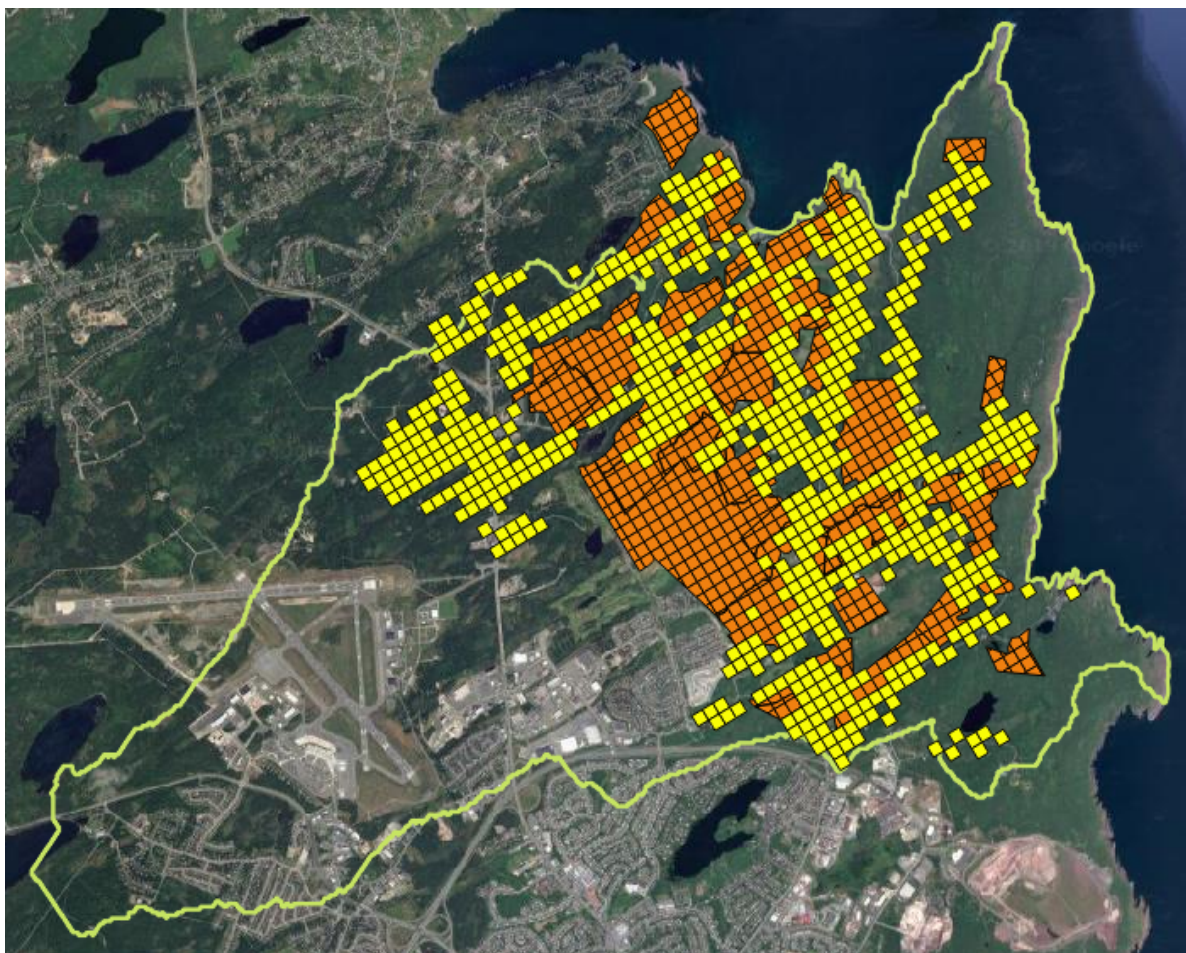

Figure 24   Scenario 1: Yellow cells represent existing wells used in the base case and orange cells represent proposed future development.

Figure 25 shows the simulated drawdown imposed by the additional development area relative to the base case. In general, simulated drawdown is contained within the footprint proposed development area. The largest drawdowns at existing wells are simulated along Marine Drive with drawdowns up to ~6 m in the aquifer. This corresponds to a drawdown in a residential pumping well of 7.1 m, based on the correction presented in Section 6.2.2.
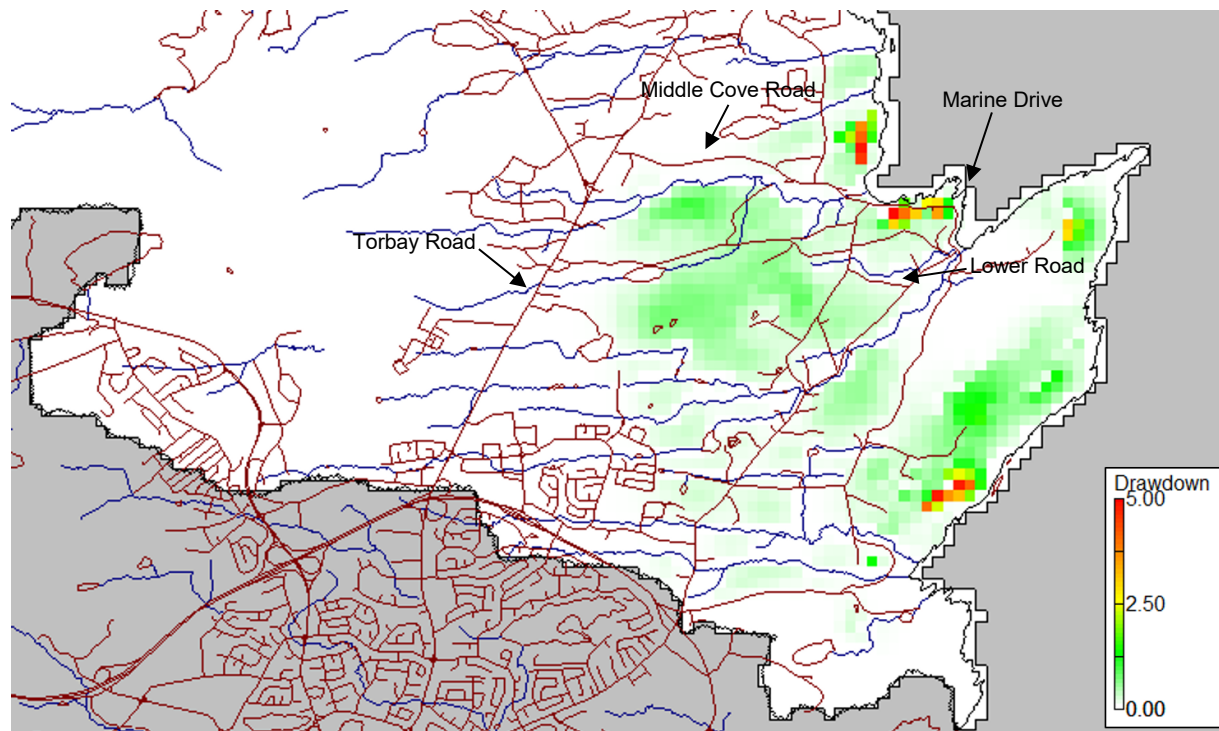
Figure 25 **Scenario 1: Development drawdown (in metres) in layer 4 relative to the base case. All future development as 1-acre lots.**

The results of Prediction Scenario 1 indicate that the threshold allowable drawdown of 15 m in an existing well (see Section 6.2.1) is not exceeded in this simulation. The areas with the greatest predicted drawdown generally correspond to local topographic highs that would be expected to receive less upland recharge than other developed areas.

6.2.4 Predictive Scenario 2 – Exclude the most sensitive areas from future development

Although there were no exceedances of the allowable threshold for available drawdown in Scenario 1, there were areas where relatively large drawdowns were predicted. Therefore, Scenario 2 was run to determine the effect of removing the most sensitive areas (drawdown >4 m) from the development plan. The locations of the excluded cells are shown in Figure 26. These areas generally correspond to local topographic highs that would be expected to receive less upland recharge than other developed areas.

Figure 27 shows the simulated drawdown for Scenario 2 relative to the base case. The results are similar to Scenario 1 except for a reduction in drawdown in the vicinities of the sensitive areas identified in Scenario 1. The maximum drawdown is still along Marine Drive, but is limited to ~3 m. Therefore, the actual drawdown in an existing residential pumping well is up to 4.1 m, based on the correction derived in Section 6.2.2.

Figure 26   Scenario 2: White cells represent future development areas that were removed to assess the sensitivity of wells in those areas.

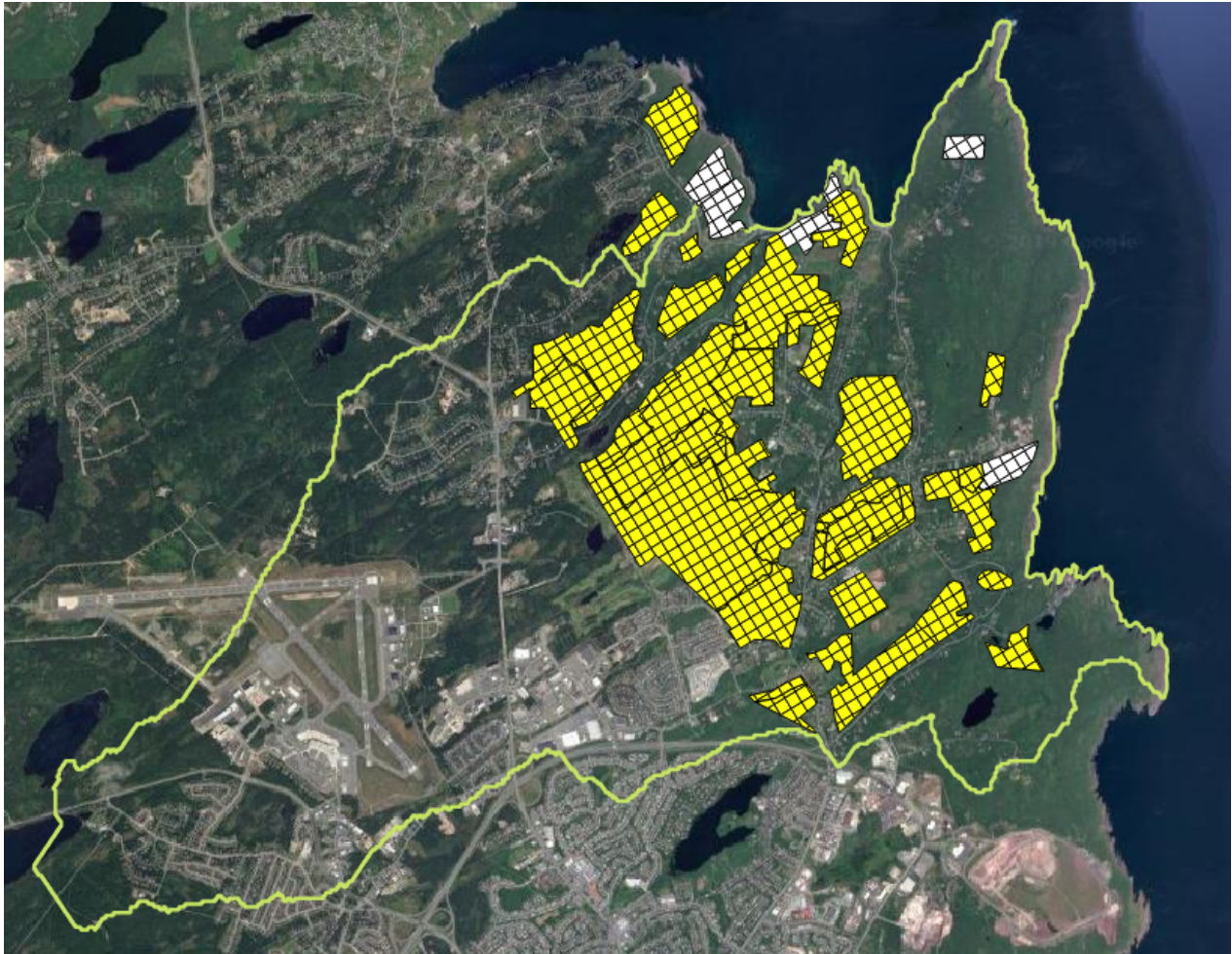The results of Prediction Scenario 2 indicate that the threshold allowable drawdown of 15 m in an existing well (see Section 6.2.1) has not been exceeded in this simulation.
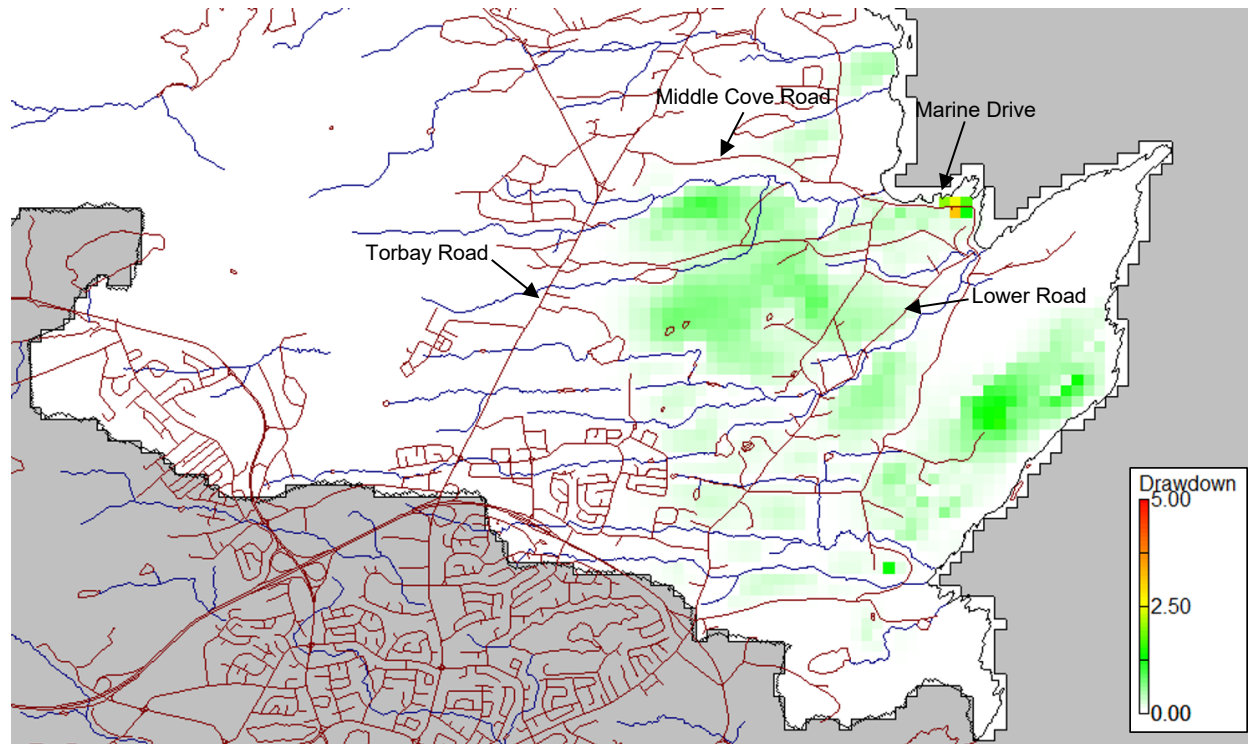
Figure 27   Scenario 2: Development drawdown (in metres) in layer 4 relative to the base case. Sensitive areas remain undeveloped.

6.2.5   Scenario 3: Lot size increase in sensitive areas

Scenario 3 was run to simulate the possibility of development in the sensitive areas using a larger lot size (i.e., 2 or 5 acre lots). Scenario 3a simulated 2 acre lots, and Scenario 3b simulated 5 acre lots. As expected, the magnitude of drawdown in the developments is between values observed in Scenario 1 and Scenario 2. The greatest drawdown continues to be along Marine Drive but is limited to ~3.5 m for 2-acre lots, and ~3 m for 5-acre lots. Therefore, the actual drawdown in an existing residential pumping well is up to 4.6 m for 2-acre lots and 4.1 m for 5-acre lots, based on the correction derived in Section 6.2.2. Simulated drawdowns are shown for Scenario 3a in Figure 28 and for Scenario 3b in Figure 29.

The results of Prediction Scenario 3 indicate that the threshold allowable drawdown of 15 m in an existing well (see Section 6.2.1) is not exceeded in this simulation.

Figure 28 **Scenario 3a: Development drawdown (in metres) in layer 4 relative to the base case. Sensitive developments as 2-acre lots.**
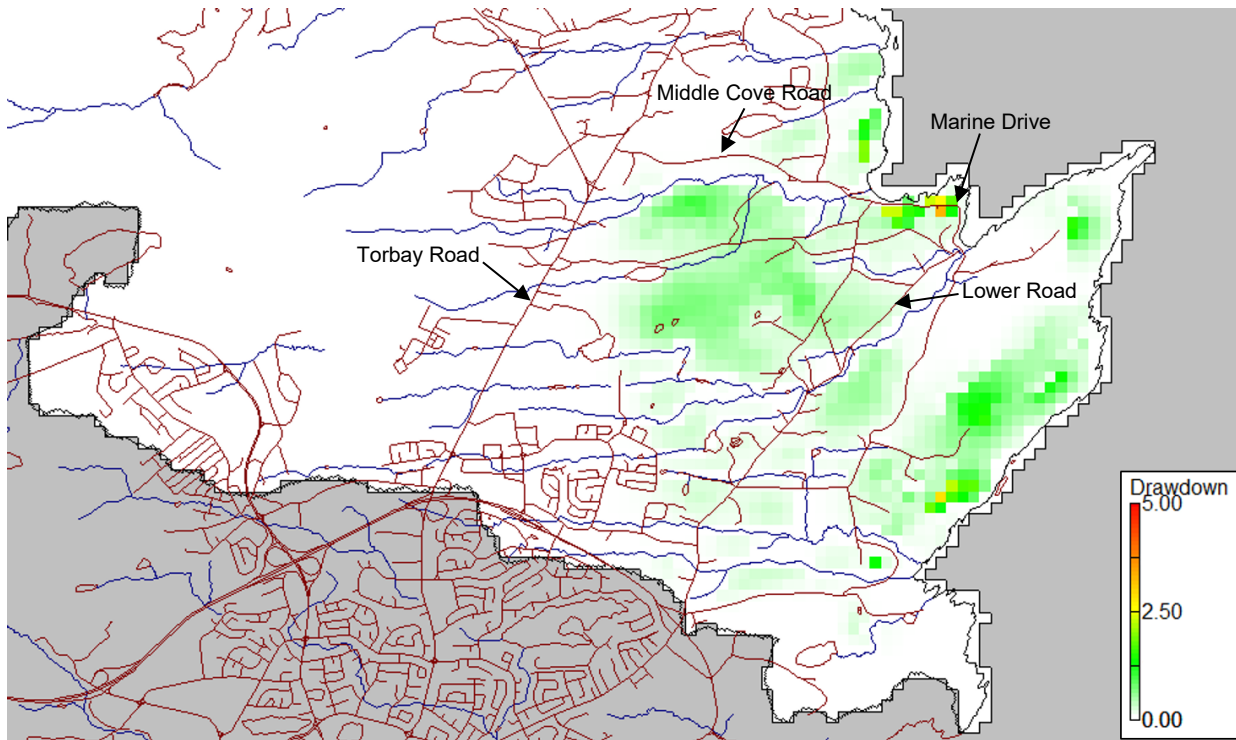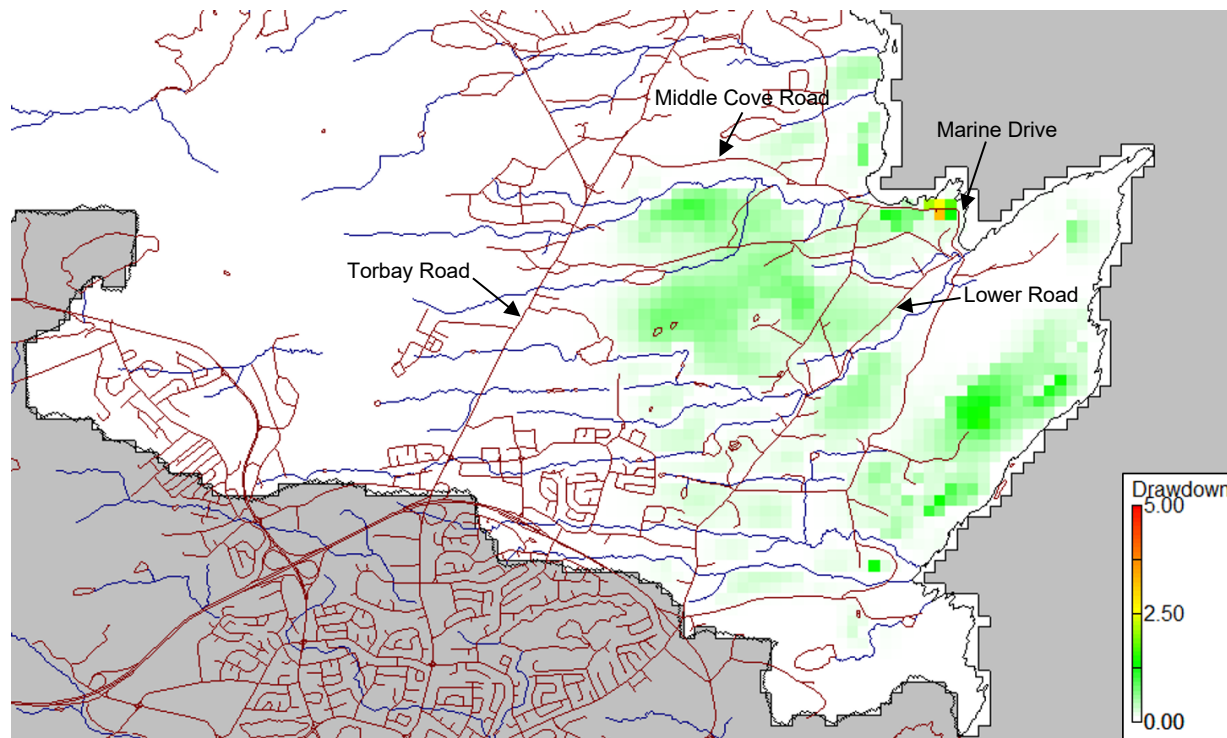
Figure 29   Scenario 3b: Development drawdown (in metres) in layer 4 relative to the base case. Sensitive developments as 5-acre lots.

6.2.6   Predictive Scenario 4 – Reduced recharge in development areas.

Scenario 4 considers the possibility that further development could have impacts on the amount of available recharge since impermeable surfaces on newly developed lots (e.g., roofs, driveways, pools, sheds, impermeable diversion to storm water and sewer) reduce available recharge, and could increase observed drawdowns, particularly in larger areas of development. It was assumed that a typical developed 1-acre lot would consist of a 150 m^2 home and other impermeable surfaces equal to the dimensions of the house for a total developed area of about 300 m^2. This would represent approximately 7.4% of the 1-acre lot being developed. It also assumes that all precipitation falling on these surfaces will be diverted to storm water or otherwise removed from recharge. This would be the conditions represented by the simulation completed in Scenario 1.

Increased development of impermeable surfaces on a lot were simulated by reducing the percentage of assigned recharge in Scenario 4. The recharge was reduced to 95% (Scenario 4a) and 90% (Scenario 4b) compared to the base case in the new development areas which represents approximately 10.9% to 15.6% of a 1-acre lot being developed. Note that the sensitive areas identified in Scenario 1 were not included in Scenario 4.

The area of reduced recharge, as entered in the model, is shown in Figure 30, and simulated drawdown for Scenarios 4a and 4b are shown in Figure 31 and Figure 32, respectively. While there is an increase in

the predicted drawdowns in the aquifer of up to 6.4 m (7.5 m in wells), the results of Prediction Scenario 4 indicate that the threshold allowable drawdown of 15 m in an existing well (see Section 0) is not exceeded in these simulations.
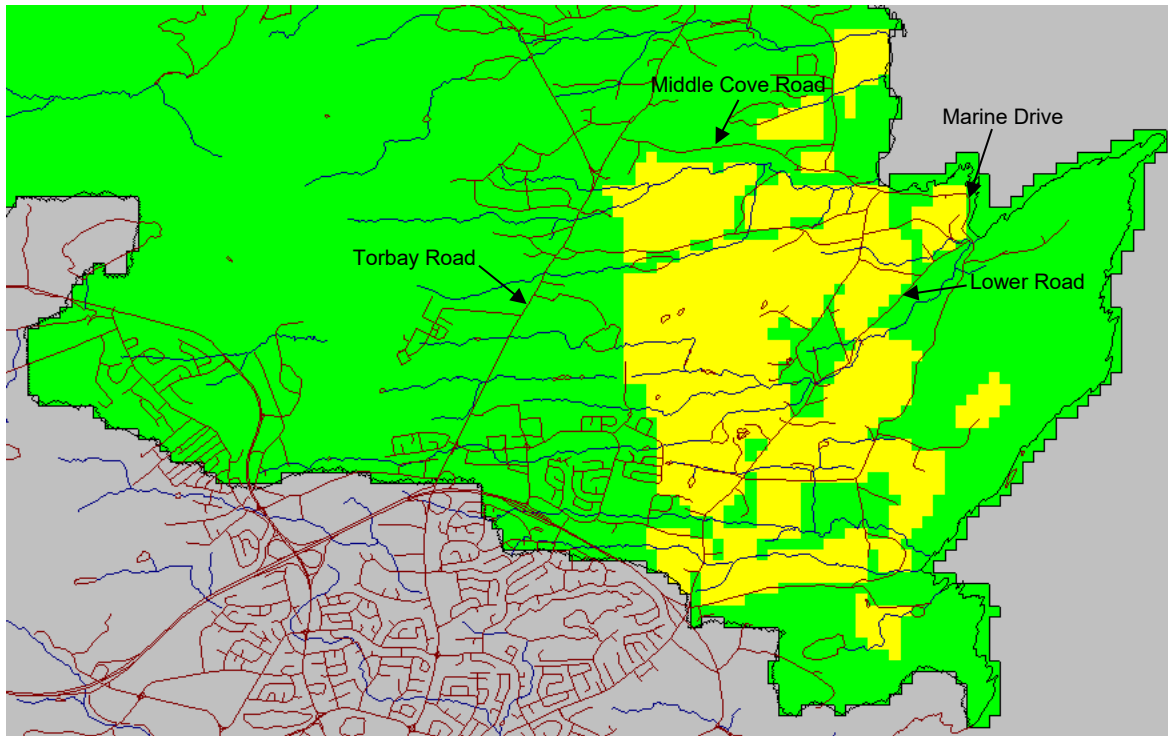
Figure 30   Scenario 4: Yellow cells represent areas where recharge was reduced to 95% and 90% of base values.
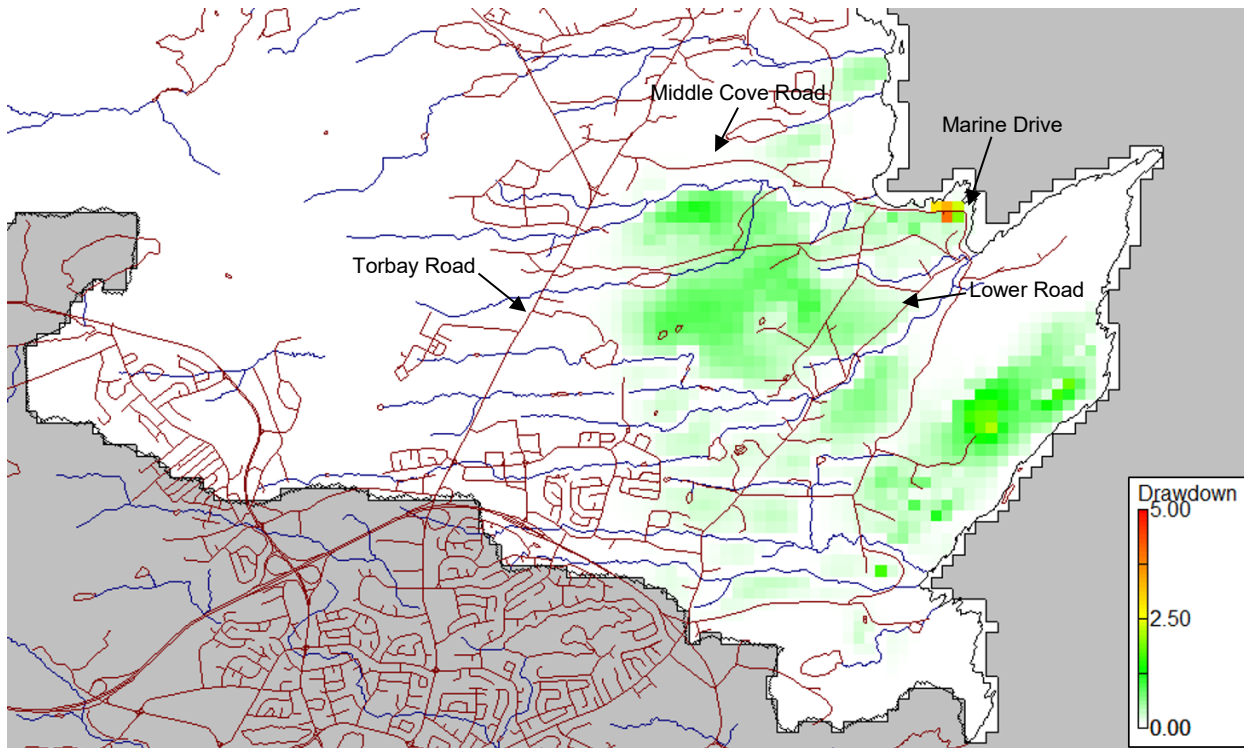
Figure 31 Scenario 4a: Development drawdown (in metres) in layer 4 relative to the base case. Recharge in future development areas scaled to 95% of base value.
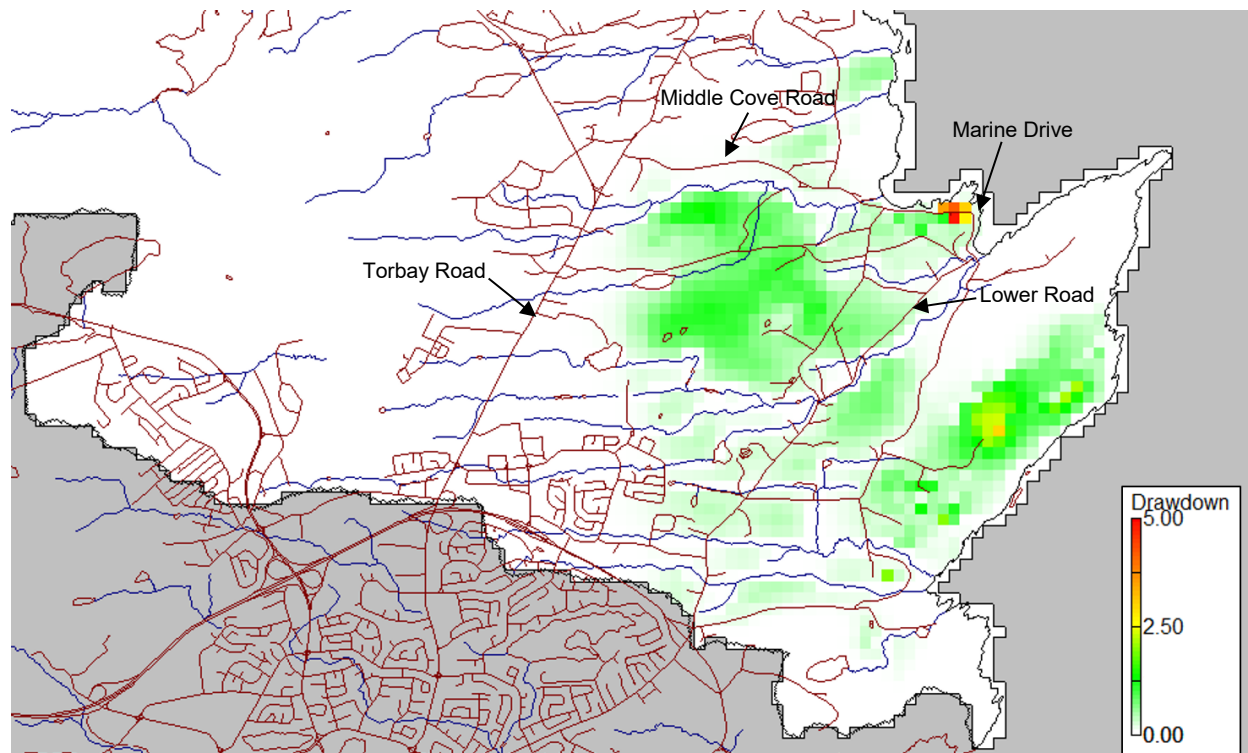
Figure 32   Scenario 4b: Development drawdown (in metres) in layer 4 relative to the base case. Recharge in future development areas scaled to 90% of base value.

6.2.7   Baseflow Predictions

Baseflow was simulated for each sub-basin of the LSA to quantify the predicted change in baseflow for each scenario. As expected, most sub-basins experience a slight loss in baseflow contribution to streams as water is removed from the ground by an increasing number of wells. The largest impacts predicted by the model for all scenarios is Area 2. As previously discussed, flow residuals were particularly high in Area 2 due to the relatively small catchment size. The predicted changes in baseflow in Area 2 are therefore likely to be exaggerated.   Predicted changes in baseflow for each scenario are shown in Table 6.1.

Table 6.1   Change in baseflow for each scenario in each sub-basin (refer to Figure 3)

Predictive Scenario	Change in baseflow				
	Area 1	Area 2	Area 3	Area 7	Area 8
Scenario 1	-0.5%	-17.2%	0.0%	-5.1%	-6.9%
Scenario 2	-0.5%	-14.8%	0.0%	-5.1%	-6.8%

Table 6.1   Change in baseflow for each scenario in each sub-basin (refer to Figure 3)

Predictive Scenario	Change in baseflow				
Scenario 3a	-0.5%	-15.6%	0.0%	-5.1%	-6.9%
Scenario 3b	-0.5%	-15.1%	0.0%	-5.1%	-6.8%
Scenario 4a	-0.7%	-19.3%	0.0%	-6.4%	-9.2%
Scenario 4b	-0.9%	-24.4%	0.0%	-7.8%	-11.9%

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 criteria used to define an adverse pumping condition does not act to incorporate discrete fracture features in the assessment of drawdown due to new residential development. Available drawdown statistics were derived from records for wells within the RSA. 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 identify the implications for baseflow impacts relevant to fisheries legislation. For example, while baseflow is estimated to exceed a 200% change (losing stream becomes a gaining stream in this case) 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 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 groundwater flow model was constructed using the MODFLOW software package; an international standard for simulating and predicting groundwater flow to evaluate potential impacts of residential development on groundwater availability in the Town of Logy Bay–Middle Cove–Outer Cove The model was calibrated by comparing model-predicted groundwater levels to recorded groundwater levels in a number of water wells in the vicinity of the Town. Once the model was considered calibrated to baseline conditions, it was used to predict groundwater level changes and changes to river and stream flow caused by additional groundwater demand from proposed residential subdivisions as per a development plan provided by the Town.

The model was calibrated by comparing model-predicted groundwater levels to recorded groundwater levels in selected water wells in the vicinity of the Town. Once the model was considered calibrated to baseline conditions, it was used to predict groundwater level changes and changes to river and stream flow resulting from additional groundwater withdrawal from further residential expansion as per a development plan provided by the Town.

The model indicates that water levels in the underlying aquifer are not expected to drop below acceptable levels if the current policy of the development of residential lots with a minimum area of one acre is maintained by the Town. Although predicted groundwater level changes are considered acceptable, larger lot sizes (2 to 5 acres) may be more appropriate in more sensitive areas (*i.e.*, along Marine Drive near Stack's Point, north of the Marine Drive-Middle Cove Road intersection, the end of Doran's Lane, and the area southeast of Cobbler Crescent) to further minimize changes in groundwater levels. The overall impact on stream baseflow was predicted to be with acceptable limits in the scenarios that were modelled. Changes to stream baseflow are limited to the immediate vicinity of new development and are generally most pronounced in smaller tributaries to larger watercourses. The presence of wetland areas within the Town should be preserved to maintain baseflow conditions in local streams and recharge to the underlying aquifer.

Future development in unserviced areas of the Town are still subject to existing Newfoundland Department of Municipal Affairs and Environment (NLDMAE) guidance for groundwater extraction. This includes the restriction on the use of open loop geothermal systems. Further, re-evaluation of the model predictions may be required if the Town's development plan changes (e.g. commercial development, development outside the currently proposed areas, etc.).

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, aquifer tests, and the direct

observation of changes due to development within the Town should be used to update the model to refine the tool for this purpose.

9.0 CLOSURE

This report has been prepared for the sole benefit of the Town of Logy Bay-Middle Cove-Outer Cove. 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 Logy Bay-Middle Cove-Outer Cove.

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.

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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