Groundwater Investigation for Jack Byrne Regional Sports and Entertainment Centre Expansion, Torbay, NL

Hydrogeological Site
Characterization and
Groundwater Flow Modelling

Prepared for:
Northeast Avalon Arena
Regional Board

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Sign-off Sheet

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1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) was retained by the Northeast Avalon Arena Regional Board to evaluate the groundwater supply potential for servicing a proposed expansion to the Jack Byrne Regional Sports & Recreation Centre (Arena). This report provides an evaluation of the groundwater resources for the site based on a review of the available relevant water resource, geological, geotechnical, and hydrogeological data maps and reports, a comparison of water consumption estimates for the arena and groundwater availability calculations using both analytical and numerical models.

2.0 BACKGROUND

It is our understanding that the Northeast Avalon Arena Regional Board is evaluating the possibility of expanding the Arena by adding a second ice surface to the existing facility that includes a 200' x 85' ice surface with a permitted maximum occupancy of 800 people. The existing facility currently obtains its supply of water primarily from a drilled bedrock water well (Well No. 2) located on the property, with lesser supplemental supply from another drilled bedrock water well (Well No. 1) on the property. Combined these two water wells meet the current ice-making and potable water demand for the arena. It is anticipated that the water demand for the arena facility will increase with the addition of the second ice surface and increased maximum occupancy of 2,780 (1980 existing plus 800 for expansion). Therefore, the Northeast Avalon Arena Regional Board has identified a need to assess the water supply potential in the area to support the development of the proposed addition to the Arena and to determine if the drilling of an additional well(s) will be required.

3.0 INFORMATION REVIEW

3.1 SITE CONDITIONS, GEOLOGY AND HYDROGEOLOGY

The Arena is located at 7 Kennedys Brook Drive of Torbay Road in the Town of Torbay, NL, and is situated at an elevation of approximately 90 metres above sea level (masl) as shown on Drawing No. 121414343-EE-01 in Appendix A. The pre-construction ground surface on the northern half of the site sloped to the northeast; the southern half to the southwest. The paved surface now slopes to the east with stormwater flowing to the ditch running along the eastern boundary of the property toward the water retention system located at the northeast corner of the site.

The natural overburden material in the area consists of an approximately 5 m thick layer of loose to compact brownish grey sand and gravel glacial till with varying amounts of silt and cobbles. The underling bedrock is comprised of grey-green siliceous siltstone and sandstone of the Drook Formation of the Late Proterozoic Conception Group (King, 1990). Numerous tectonic events have resulted in a complex large-scale doubly plunging anticlinal structure referred to as the Torbay Dome. The Site is located on the eastern limb of this feature that extends northwards towards Torbay, and the fractures controlling groundwater flow are expected to be sub-vertical. The orientation of bedrock structural features, including faults, in this area are dominantly northeast. Bedrock structure controls the surface topography and the orientation of surface water features, including Kennedy's Brook located along the north boundary of the Site.

Stantec (2015) conceptualized the subsurface geology in the area of the site as three distinct units that are relevant to groundwater flow (referred to as hydrostratigraphic units). The uppermost unit is the overburden, which ranges regionally in thickness up to 14 m, averaging 4.3 m. The bedrock is comprised of an upper weathered zone assumed to be 3 m thick, underlain by a competent bedrock zone. Regional-scale groundwater flow modeling (Stantec, 2015) yielded the following estimates of hydraulic conductivity for each unit: overburden = 4.32 m/day (horizontal = vertical), weather bedrock = 0.34 m/day (horizontal) and 0.588 m/day (vertical), and competent bedrock = 7.5×10^{-3} m/day (horizontal) and 0.126 m/day (vertical). The higher vertical hydraulic conductivity compared to horizontal in the competent bedrock is consistent with an aquifer that has inclined fractures controlling groundwater flow.

Groundwater recharge is defined as the quantity of infiltrating precipitation that reaches the water table. It is expected that the shallow groundwater system at the Site will be largely controlled by recharge from surface water and local groundwater recharge. The deeper groundwater system is expected to be recharged from more distant sources at higher elevations. Groundwater discharges locally to streams and ponds, and to the Atlantic Ocean on the regional scale. The Stantec (2015) groundwater flow model estimated that groundwater recharge is 20% of total annual precipitation on the regional scale. Most importantly, the modelling results suggest that only 10% of this recharge reaches the competent bedrock, while a combined 86.2% resides only a short time in the overburden before discharging to streams and ponds. Total annual precipitation for the St. John's airport averages 1,534 mm/year (Environment Canada, 2016).

Groundwater levels in the area are generally assumed to be a subdued reflection of topography. Local groundwater at the Site is expected to flow to the northeast. Based on a review of static water levels recorded on well logs for the area, water table elevation is approximately 96.7% of the ground surface elevation.

Groundwater in the underlying bedrock occurs primarily in secondary openings, such as fractures and joints and therefore the yield of a water well depends on the number of water-bearing fractures it intersects and how interconnected the fracture network is. As stated previously, fractures in bedrock in the area are expected to be sub-vertical. Therefore, well yields in the area can be variable since the likelihood of intersecting vertical fractures is reduced when drilling a vertical well. The Newfoundland and Labrador Department of Environment and Conservation (NLDEC) (2014) reports that the average well yield in the Torbay area is 14.42 L/min, which is considered low to moderate. A review of well logs shows that depth of wells supporting these yields range from 15 m to 170 m, averaging 87 m.

3.2 EXISTING SITE WELLS

In 2007, two (2) water wells (Well No. 1 and Well No. 2 were drilled at the site, separated by a distance of approximately 100 m, to provide a supply of water for the arena facility (see Drawing No. 121414343-EE-02, Appendix A). A summary of the well construction details for each well is provided in Table 1. Hydro-fracturing was performed in both wells to increase the yield by enhancing the connectivity of the intersecting fracture network. Aquifer testing was completed on both wells and the results are summarized in a report prepared by Jacques Whitford Limited (JWL, 2007). It was noted during the testing program that the hydro-fracturing was most successful on Well No. 2. A constant rate pumping test was not performed on Well No. 1 due to the low yield observed during drilling and the preliminary results of the step drawdown test. Drawdown in Well No. 2 approached steady-state conditions towards the end of the 72-hour constant rate pumping test, suggesting the well was being recharged from a constant hydraulic head boundary, such as from Kennedy's Brook. However, the potential diversion of groundwater discharge to Kennedy's Brook due to well operation was not investigated. The mitigation of baseflow reduction can be important for maintaining the ecological function of the brook.

Table 1 Existing Arena Water Supply Well Construction

Well	Total Depth (m)	Depth to Bedrock (m)	Casing Length (m)	Casing Diameter (mm)	Static Water Level (m)	Driller's Estimated Safe Yield (L/min)	1-Day Safe Yield (L/min)[1]	100-Day Safe Yield (L/min)[1]
Well No. 1	182.8	5.1	12.8	150	2.7	15.1	-	-
Well No. 2	213.0	3.8	6.9	150	artesian	-	112.6	69.0

Note:
1. Based on aquifer properties interpreted after hydro-fracturing

Based on the results of the 72-hour constant rate pumping test completed in Well No. 2, JWL (2007) provided an analysis of safe yield using the modified Cooper-Jacob equation, written as:

$$Q_t = \frac{0.7 \times T \times s}{0.183 \times \log t} \qquad \text{Equation 1}$$

where: Q_t is the continuous pumping rate for a given time (m^3/s)
s is the available drawdown (m)
T is the aquifer transmissivity (m^2/s)
t is time (min)

Using a transmissivity of 1.55×10^{-5} m^2/sec (interpreted from the 72-hour constant rate pumping test at Well No. 2) and an available drawdown of 100 m (half well depth) a summary of the calculated safe yield for different time periods is provided in Table 2. A 100-day safe yield of 69.0 L/min (i.e., 15.2 Igpm) was considered to be a representative estimate of the long-term safe yield potential of Well No. 2. The 100-day safe yield estimated for Well No. 2 was considered to be relatively high, especially compared to the average well yield for the Torbay area of 14.42 L/min. The driller's estimated well yield for Well No. 1 (15.1 L/min) aligns more closely with what can be expected on average for this area.

Note that the 100-day safe yield calculation is an over-conservative approach to estimating how to operate a well since it assumes that the well is being continuously pumped over a period of 100 days with no precipitation and/or groundwater recharge (e.g., a dry summer). It also incorporates a safety factor of 0.7 to account for uncertainties in the interpreted transmissivity and storativity, the presence of undetected boundary conditions, seasonal water level fluctuations and borehole head losses. However, the actual operating pumping rate could be higher if the climatic conditions allow, the maximum available drawdown is not exceeded and monitoring data demonstrates that there is minimal (or no adverse) interference effects with off-site groundwater users.

Table 2 Well No. 2 Safe Yield for a Specified Time Period

Time Period	Q_t (L/sec)	Q_t (L/min)	Q_t (Igpm)	Specific Capacity (L/min/m)
1 hour	3.3	200.1	44.0	2.0
8 hours	2.2	132.7	29.2	1.3
1 day	1.9	112.6	24.8	1.1
30 days	1.3	76.7	16.9	0.8
100 days	1.1	69.0	15.2	0.7
1 year	1.0	62.1	13.7	0.6
20 years	0.8	50.7	11.1	0.5

Stantec (2013) conducted a Level II Groundwater Supply Assessment for the proposed Pine Ridge Valley sub-division located to the east of the arena site (Drawing No. 121414343-EE-02 and 121414343-EE-03, Appendix A). As part of this investigation, two wells (Well 2 and Well 3) were drilled to a depth of 42.67 m and 85.34 m, respectively, to assess the potential well yield in this area. The 100-day safe yield for these two wells were determined to be 17.4 L/min (Well 2) and 1.8 L/min (Well 3). These values are typical of the range well yields that can be expected in this area. It is not known if the well yields would improve if the wells were deeper and/or subjected to hydro-fracturing

3.3 ARENA WATER DEMAND

An estimate of the water demand for the expanded arena facility is required to determine if there is a sufficient groundwater resource in the area to supply the demand. Given the seasonal and irregular use of the facility, the actual demand is difficult to determine and varied estimates have been provided over the past several years. JWL (2007) estimated the demand for the existing facility would be approximately 90 L/min based on design requirements for septic outflow. In 2008, Fracflow Consultants Inc. completed a hydrogeological assessment for a proposed commercial/residential development adjacent to the arena facility (Fracflow, 2008). As part of the assessment Fracflow estimated the water demand for the arena to be in the range of 30 to 35 L/min based on monitoring and assessment of a similar arena facility in Nova Scotia. Stantec (2015) represented the water demand for the arena in the regional groundwater flow model exercise at 23.8 L/min. In 2016, monitoring of water consumption over a 200-day period starting in July indicated that the average demand for the Arena was 75 L/min (Tucker, 2017).

Fracflow (2008) evaluated that ice making comprises 68% of water demand based on the example of a similar facility in Nova Scotia. Therefore, assuming the distribution of water use would be the same for the arena and the actual water demand for the existing facility is 75 L/min, then 51 L/min would be used for ice making and the remaining portion (24 L/min) would be used to service the existing building capacity of 1,980 people (0.012 L/min/person). Assuming the water demand for ice making (51 L/min) would be the same for the additional ice surface, then the additional non-ice making demand for the additional 800-person capacity would be 10 L/min. The estimated water demand is summarized in Table 3 below. These estimates do not account for any additional water saving measures that could be implemented at either facility.

The total demand for the expanded facility of 136 L/min exceeds the estimated 100-day safe yield for the current well (Well No. 2), which is the main water supply for the facility by a factor of 2. Considering the safe yield of Well No. 2, and the driller's estimated low well yield for Well No. 1, the additional demand for the expanded facility would likely have to be supplemented by an additional well(s) and/or storage.

Table 3 Estimated Arena Water Consumption

Scenario	Capacity (people)	Water Demand (L/min)		
		Ice Making	Non Ice Making	Total
Existing Arena Facility	1,980	51	24	75
Proposed Expansion	800	51	10	61
Total for New Facility	2,780	102	34	136

4.0 ANALYSIS AND RESULTS

The following outlines the various analytical and numerical approaches that were used to determine if the underlying bedrock aquifer in the area surrounding the arena site has the potential to supply the water demand required to service the proposed expansion.

4.1 WATER BALANCE APPROACH

A simplified water balance can be used to evaluate the suitability of a groundwater supply in a proposed development area. While the availability of groundwater supply is difficult to predict in fractured bedrock because of the complexities presented by fracture network connectivity, these calculations are based on conservative estimates of groundwater recharge and are meant to be used in conjunction with other lines of evidence, such as safe yield and drawdown interference calculations, to evaluate the sustainability of on-site well supplies at proposed developments.

The availability of groundwater supply is based on the theoretical maximum water demand, estimated natural groundwater recharge, and total lot area, and can be expressed using the following suitability index:

$$SI = \frac{A \times R \times FS}{N \times D} \qquad \text{Equation 2}$$

where: SI = suitability index

A = total catchment area (m^2)

R = estimated annual rate of groundwater recharge (m/year)

FS = factory of safety (assigned to be 80%)

N = number of proposed lots

D = annual water demand (m^3/year)

Calculated SI values greater than 1 indicate that the recharge area will theoretically receive more input to the groundwater system from recharge than is being removed by the development. It therefore indicates that water usage will not exceed the available groundwater resource. The calculated SI values for a residential sub-division would normally be calculated for an individual lot, for the whole subdivision, and again for the larger surface water catchment area, which could include houses from other developments. The calculated SI values for the existing arena lot, local drainage sub-catchment area, and well capture zone are provided in Table 4. An outline of the areas used in each calculation is shown on Drawing No. 121414343-EE-04, Appendix A. The SI results show that the Arena water use is not likely supported or sustainable by recharge on the lot alone, but may be accommodated within the larger context of the local sub-catchment scale or well capture zones (SI greater than one). For the larger capture areas, the demand for the expanded Arena was converted to an equivalent number of residential lots by dividing the total estimated water demand for the Arena by the average demand for a residential lot.

The application of the SI calculation to the existing lot development is somewhat constrained since the site consists of only one building on a large lot, with the majority of the area covered by impervious (or semi-impervious) roof and parking lot surfaces. Precipitation that falls on the site is routed to the ditch on the eastern boundary where it likely infiltrates into the shallow subsurface and then discharges into Kennedy's Brook.

Table 4 Summary of Suitability Index calculations

Area	Area (A), m²	Recharge Rate (R), m/year	Factory of Safety (FS)	Number of Lots (N)	Demand (D), m³/year/lot	Suitability Index (SI)
Existing Arena lot (with expansion demand)	30,000 [1]	0.307 [4]	0.8	1	71,531 [7]	0.10
Local drainage sub-catchment area	868,022 [2]			305 [4]	497 [8]	1.41
Well Capture Zone	1,017,925 [3]			308 [6]	497 [8]	1.63

Notes:
1. existing arena lot (refer to Figure A-8, Appendix A)
2. based on topographic mapping analysis (refer to Figure A-8, Appendix A)
3. from numerical model reverse particle tracking analysis (Case A) presented below (see Figure A-3 and A-8, Appendix A)
4. 20% of total annual precipitation of 1534 mm/year, based on precipitation data from Environment Canada Climate Normals, 1981-2010 and Stantec (2015) general groundwater recharge
5. a total of 135 existing residential lots are identified within the local drainage area, each is expected to use 497 m³/year; as well as the expanded arena (estimated demand of 84,680 m³/year is equivalent to 170 residential lots); combined total of 305 equivalent residential lots
6. a total of 138 existing residential lots identified within the capture zone, each is expected to use 497 m³/year; as well as the expanded area (estimated demand of 84,680 m³/year is equivalent to 170 residential lots); combined total of 308 equivalent residential lots
7. Assumed water demand for expanded Arena (136 L/min)
8. Expected four-person household daily use expected to be 1,360 L/day (497 m³/year)

4.2 WELL INTERFERENCE

Well interference is the change in the water level (drawdown) that occurs in one well as a result of pumping in another adjacent well. This induced drawdown can reduce the capacity of the well due to the reduction in available drawdown in the affected well. The theoretical drawdown at a radial distance from a pumping well can be evaluated using the hydraulic properties of the aquifer with the Theis equation, written as:

$$s = \frac{Q \times W(u)}{4\pi T} \qquad \text{Equation 3}$$

where: s is drawdown at a given radial distance (m)
Q is the pumping rate (m³/day)
T is the aquifer transmissivity (m²/day)
$W(u)$ is the well function of u
u is equal to $r^2S/4Tt$ (-)
r is the radial distance (m)
S is aquifer storativity (-)
t is time (days)

An evaluation of potential well interference on neighbouring wells resulting from pumping Well No 2 at the Arena site was carried out using the Theis equation. For this evaluation, the aquifer transmissivity value was set to 1.55×10^{-5} m^2/s (interpreted from the 72-hour constant rate pumping test at Well No. 2). The aquifer storativity was assumed to be 1×10^{-4}, based on expected values from the literature (Freeze and Cherry, 1979). Table 5 provides a summary of the theoretical drawdown that would occur at a well located at a radial distance of 100 m from the pumping well for the different safe yields and time period previously reported in Table 2 with an available drawdown of 100 m in the pumping well. In addition, the potential well interference was also calculated for pumping rates of 75 L/min (existing demand) and 136 L/min (expanded facility) for a 30-day period. This approach does not consider the input of water from recharge or boundary conditions, both of which will act to mitigate the drawdown.

Table 5 Theoretical Radial Drawdown from Well No. 2.

Time Period	Q_t (L/min)	Drawdown at r = 100 m (m)
1 hour	200.1	0.07
8 hours	132.7	2.5
1 day	112.6	12
30 days	76.7	30
100 days	69.0	33
1 year	62.1	37
20 years	50.7	43
30 days	75	29
30 days	136	53

The predicted drawdown at 100 m radial distance from Well No. 2 was relatively large (e.g., nearly 30 m at a pumping rate of 76.7 L/min for 30 days). A well interference of this amount could result in some off-site wells in close proximity of "going dry", depending on the well depth and elevation of the water-bearing zone. However, these are theoretical predictions of well interference only and there has been no known reports of adverse effects in other wells in the area with an average well depth of 87 m. The available data makes it difficult to determine the effect at the scale of an individual well. A groundwater monitoring program would help quantify the spatiotemporal distribution of drawdown and the potential for adverse off-site impacts.

4.3 NUMERICAL MODELING

4.3.1 Background

Stantec (2015) developed a numerical groundwater flow model for the Town of Torbay to assist in the evaluation of the effects of future development on existing groundwater conditions and well users in the municipality. This model considers a combined surface watershed area that is bigger than the municipal boundaries of Torbay. The steady-state flow model that was developed is comprised of four layers representing the overburden soil, a shallow bedrock zone with enhanced permeability due to glaciation, deeper competent bedrock above the typical bottom of casing elevation, and the deeper bedrock zone below the bottom of casing that the local domestic water wells access for water supply. The property zones that describe the hydraulic properties of the geologic materials and recharge to the groundwater system were derived by constraining their values during the calibration process using appropriate bounds based on data from hydraulic testing reports for subdivision development within the study area, water well head data from the provincial well record database, and estimated baseflow to streams using baseflow separation methods on provincial hydrometric data.

Appropriate boundary conditions were applied to represent the ponds, brooks (see Drawing No. 121414343-EE-05, Appendix A), domestic wells, and the Atlantic Ocean. The quality of the calibration achieved indicates that the model does a good job of simulating existing conditions and should therefore be able to predict the response to changes in site conditions, provided the expected influence is on the same scale as the historical conditions. For example, numerous subdivisions have been built around the existing Arena, all of which are implemented in the model as pumping cells in the deeper bedrock layer (layer 4) that extract the equivalent flowrate of groundwater as the number of houses over the same area they represent. Each house was expected to have a daily water need of 1,360 L (497 m^3/year) and the Arena was simulated to use 34,200 L/day (23.8 L/min).

Note that this model best describes the bulk, larger-scale subsurface properties and the long-term average behavior of the groundwater system hydraulics. It does not incorporate the highly heterogeneous properties that bedrock aquifers often present where groundwater only flows through preferential pathways such as fractures and faults. However, the evaluation of an adverse impact on groundwater users due to a new pumping condition used by Stantec (2015) does consider the available drawdown calculated as the distance between the static water level and the uppermost water-bearing zone, based on the available provincial water well records. An adverse condition was defined as one where pumping induced a drawdown of the water table that exceeds the available drawdown in 5% of existing wells. This corresponds to a drawdown of 15 m. It must be re-iterated that this approach is based on averages and does not describe site-specific conditions or potential impacts on individual wells. In addition, the elevation of where the water-bearing fractures intersect wells is highly variable due to their near-vertical orientation.

4.3.2 Predictive Simulation Cases

Numerical modeling was used to simulate existing conditions (Base Case) and test the response of aquifer water levels under different future development scenarios. The objective was to be conservative by considering development that has been proposed and are likely to be present near the Arena by the time the expansion is completed. It is assumed that the additional ice surface will be constructed on the current property, with parking being displaced onto adjacent lands to the east. The predictive simulations outlined below were used to evaluate the effects of the existing and expanded facility on groundwater baseflow to brooks and drawdown at off-site groundwater user locations. In addition, the potential impact of the co-development of the proposed adjacent commercial land and Pine Ridge Valley residential sub-division along with the

Arena expansion was also modeled since timing of these developments is not known and the cumulative effects on the groundwater resources in the area should be considered.

BASE CASE.

The Stantec (2015) Predictive Scenario 1 was used as the base case to compare the potential effects of each predictive simulation case. It includes consideration of the following features:

- Existing residential development in 2015 is included in the base case (Stantec, 2015),
- Total water demand for the Arena is simulated at a pumping at rate of 34.2 m^3/day (23.8 L/min); and.
- Completion of all existing sub-divisions, including parts of Scenic View and Eagle Nest Ridge, Forest Landing, Pine Ridge and Logy Bay.

The following outlines the additional considerations made for each predictive case:

CASE A: REVISED EXISTING JACK BYRNE ARENA PUMPING.

The pumping rate for Arena was increased to 108 m^3/day (75 L/min) to simulate actual consumption based on the results of monitoring completed in 2016.

CASE B: ADDITION OF PINE RIDGE VALLEY AND COMMERCIAL DEVELOPMENT.

The proposed Pine Ridge Valley sub-division and commercial land adjacent to the Arena were added to Case A. The Pine Ridge Valley residential development is proposed to contain 56 homes, each with an expected daily water need of 1,360 L/day. The proposed 35,000 m^2 commercial property along Torbay Road was previously simulated by Stantec (2015) as having the equivalent water consumption expected from six McDonald's restaurants. Based on a typical footprint of 4, 045 m^2 to 6070 m^2 for a restaurant in the St. John's area, the demand was spread across three model cells (i.e., 2 restaurants in each 100 m by 100 m cell). McDonald's reports that a typical restaurant uses 4,100 m^3 of water per year, with 2,255 m^3 going to sewer (McDonald's, 2017). This is the equivalent domestic water demand of approximately 8.3 household at 1,360 L/day. Thus, the water withdrawal at each model cell was set to 22.45 m^3/day.

CASE C: EXPANDED JACK BYRNE ARENA WITH PINE RIDGE VALLEY AND COMMERCIAL DEVELOPMENT.

This simulation is the same as Case B but also includes the addition of the proposed expansion to the Arena to an increased total demand of 195.8 m^3/day (136 L/min).

CASE D: EXPANDED JACK BYRNE ARENA ONLY.

This simulation considers the expansion of the Arena with the expected total demand of 195.8 m^3/day (136 L/min) as in Case C, but without the development of Pine Ridge Valley sub-division and the commercial property (Case B).

4.3.3 Metrics for Comparison of Model Results

The results from each simulation were then compared to the base case scenario (existing conditions). The key metrics of the comparisons were:

1. On-site and off-site drawdown and the corresponding water levels in wells, and
2. The change in baseflow to Kennedy's Brook.

A well boundary condition assigned to a cell in the model acts to remove water from a cell to simulate pumping from the well. However, the numerical groundwater model evaluates the average head in each cell and does not output what the actual drawdown would be in a well of finite diameter pumping at a given flow rate (Figure 1). Therefore, a correction factor has to be applied to predict the drawdown in an individual well. The following correction is based on the Theim solution:

$$h_w = h_* - \frac{Q}{2\pi T} \ln\left(\frac{r_e}{r_w}\right) \qquad \text{Equation 4}$$

where h_w is the head in the pumping well, h_* is head in the model 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 0.198Δx (Peaceman 1983) where Δx is the length dimension of a cell (assuming the cells are square in plan view).

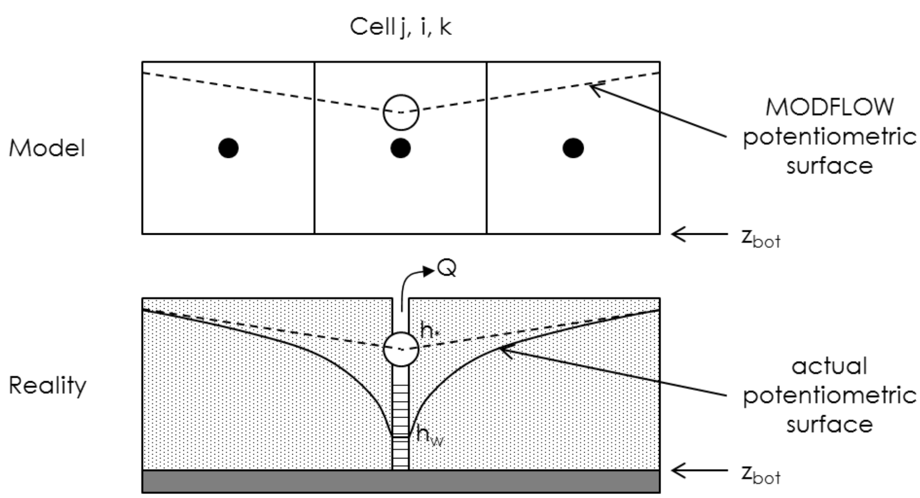

Figure 1 An Example of the Difference Between Model Results and the Actual Potentiometric Surface in a Pumping Well.

The second term on the right-hand side of Equation 4 represents the head correction in the well. Using inputs of T = 1.19 m^2/d and Δx = 100 m from the numerical model, the correction factor for a standard residential 150 mm diameter well (r_w = 0.0762 m) with an average demand of Q = 1.36 m^3/d the head correction would equal approximately 1 m.

For the purposes of groundwater numerical modelling and predicting drawdown in on-site water wells on the Arena property, the total water demand for the Arena is conservatively simulated as being derived entirely from Well No. 2. The Arena well and commercial wells will have the same radius as the residential wells, but the pumping rate are different in each scenario and therefore would have a different correction factor. A summary of the pumping rate and corresponding head correction for these wells is provided in Table 6.

Table 6 Summary of Pumping Well Head Correction.

Case	Type	Water Demand (m^3/day)	Head Correction (m)
Base	Residential Arena	1.36 34.2	1.0 25.4
A	Residential Arena	1.36 108	1.0 80.3
B	Residential Arena Commercial	1.36 108 11.2	1.0 80.3 8.3
C	Residential Arena Commercial	1.36 195.8 11.2	1.0 145.6 8.3
D	Residential Arena	1.36 195.8	1.0 145.6

4.3.4 Results

4.3.4.1 Base Case

Drawing No. 121414343-EE-06, Appendix A shows the distribution of groundwater elevation (hydraulic head) contours in the area around the Arena property for the Base Case simulation scenario. Simulated discharge from the groundwater system to the various reaches of Kennedy's Brook (i.e., baseflow) is summarized in Table 7. These are the hydraulic heads and discharge values that will be used for comparison with the other simulated scenarios.

Table 7 Summary of Baseflow to Kennedy's Brook with Percent Change from Base Case Provided in Parentheses.

Kennedy's Brook Segment	Base Case	Case A (m^3/day)	Case B (m^3/day)	Case C (m^3/day)	Case D (m^3/day)
Reach 103	795	795 (0%)	795 (0%)	795 (0%)	795 (0%)
Reach 105	863	794 (-8.0%)	783 (-9.3%)	701 (-18.8%)	711 (-17.6%)
Reach 201	3,952	3,947 (-0.1%)	3,817 (-3.4%)	3,811 (-3.6%)	3,942 (-0.7%)
Combined	5,610	5,536 (-1.3%)	5,395 (-3.8%)	5,307 (-5.4%)	5,448 (-2.9%)

The surface elevation in the model cell that represent Well No. 2 at the Arena is 77.4 m above mean sea level (ASL) and the simulated average head in cell without pumping is 77.8 masl (static is slightly artesian). When Well No. 2 is simulated at a pumping rate of 34.2 m^3/day the average cell head is 76.8 masl. The application of the head correction results in an estimated Well No. 2 pumping level elevation of 51.4 masl, which corresponds to a total drawdown from static in the well of 26.4 m.

Reverse particle tracking was implemented in the numerical model to determine the origin of the groundwater being captured by the well. Each flowline was tracked back along the velocity field to its starting point at the water table and the combination of flowlines define the recharge capture zone of the well Drawing No. 121414343-EE-06, Appendix A. The simulated capture zone is 1,017,925 m^2 and extends to the west under and beyond the Pine Ridge and Quarry Road sub-divisions and along the first-order segments of Kennedy's Brook.

4.3.4.2 Case A

Drawing No. 121414343-EE-07, Appendix A shows the distribution of the drawdown for Case A compared to the base case which simulated pumping Well No. 2 at the actual rate of consumption based on monitoring in 2016 of the rate of 108 m^3/day (75 L/min). The maximum aquifer drawdown of 2.3 m occurs at the Arena well where the average simulated head in the Arena pumping cell is 74.5 masl. This represents an average decrease of 2.3 m from the base case. The application of the head correction results in an estimated pumping level elevation of -5.8 masl, which corresponds to a total drawdown from static in the well of 83.2 m. This drawdown is less than the available drawdown of 100 m which indicates that Well No. 2 is likely operating within the constraints proposed by JWL (2007).

Off-site drawdown extends predominantly to the north towards Riverwood Place and Waverly Place (see Drawing No. 121414343-EE-07, Appendix A). Less than 0.20 m of drawdown is expected at the nearest properties on Riverwood Place located approximately 100 m away from the pumping well. The application of the head correction for domestic well results in a well drawdown of 1.2 m which is below the upset adverse drawdown condition of 15 m.

The simulated change in baseflow to Kennedy's Brook is -8% at Reach 105, but is less than -2% overall (see Table 7).

4.3.4.3 Case B

Drawing No. 121414343-EE-08, Appendix A shows the distribution of the drawdown for Case B with the addition of the Pine Ridge Valley and commercial developments to Case A compared to the base case. The maximum aquifer drawdown of 2.3 m at the Arena well is the same as was computed for Case A (i.e., the addition of the commercial and Pine Ridge Valley developments has no effect on the predicted drawdown in the Arena well for Case A). The simulated average head in the Arena pumping cell is 74.6 m, which corresponds to a corrected pumping level elevation of -5.7 masl for a total drawdown from static of 83.1 m. Therefore, the computed drawdown is less than the available drawdown in Well No. 2 of 100 m.

The distribution of off-site drawdown to the north is the same as was observed in Case A, but also extends to the west. The same well drawdown of 1.2 m at Riverwood Place is expected as in Case A (see Drawing No. 121414343-EE-08, Appendix A), which is below the upset adverse drawdown condition of 15 m.

The simulated change in baseflow to Kennedy's Brook is -9.3% at Reach 105, but remains less than -4% overall (Table 7).

4.3.4.4 Case C

Drawing No. 121414343-EE-09, Appendix A shows the distribution of the drawdown that would be expected with the addition of Pine Ridge Valley sub-division, commercial development and Arena expansion demand (136 L/min) compared to the base case. A maximum aquifer drawdown of 5.0 m occurs at the Arena well cell. The simulated average head in the Arena pumping cell is 72.2 masl, which corresponds to a corrected pumping level elevation of –73.4 masl (total drawdown of 150.6 m). This exceeds the available drawdown of 100 m.

The distribution of off-site drawdown is similar to Case B. The average drawdown in model cells underlying Riverwood Place is less than 1 m, corresponding to a pumping level change in domestic wells of less than 2 m. This is below the upset adverse drawdown condition of 15 m.

The simulated change in baseflow to Kennedy's Brook is -18.8% at Reach 105, and is -5.4% overall (Table 7).

4.3.4.5 Case D

Drawing No. 121414343-EE-08, Appendix A shows the distribution of the drawdown in hydraulic head with just the addition of the Arena expansion (no Pine Ridge Valley or commercial development) compared to the base case. In this simulation, a maximum aquifer drawdown of 4.9 m occurs at the Arena well cell (similar to Case C). The simulated average head in the Arena pumping cell is 72.2 masl, which corresponds to a corrected pumping level elevation of -73.4 masl (total drawdown of 150.6 m). This exceeds the available drawdown of 100 m.

The distribution of off-site drawdown is similar to Case A, but extends out further in all directions. The average drawdown in model cells underlying Riverwood Place is less than 1 m (see Drawing No. 121414343-EE-08, Appendix A) corresponding to a pumping level change in domestic wells of less than 2 m. This is below the upset adverse drawdown condition of 15 m.

The simulated change in baseflow to Kennedy's Brook is -17.6% at Reach 105, and is -2.9% overall (Table 7).

5.0 DISCUSSION

The analysis and results presented in Section 4.0 shows the potential and cumulative impact of the Arena expansion and commercial and residential co-developments on the groundwater supply resources in the area. The following provides a discussion of these findings and highlights the need to pursue an additional well(s) to provide a supplemental source of potable water to the expanded facility. This discussion is divided into sub-sections outlining the capacity of Well No. 2 and the groundwater system to support the additional demand.

5.1 WELL NO. 2

The analytical modeling presented in Section 4.2 indicates that pumping rates of 75 L/min (existing) and 136 L/min (expansion) will result in 30-day drawdowns in the well that would be greater than the available drawdown in Well No. 2 of 100 m. Similarly, numerical groundwater flow simulations for the arena expansion scenarios at 136 L/min (Case C and Case D) indicate that the pumping level in Well No. 2 would be below the available drawdown of 100 m. Therefore, it is unlikely that Well No. 2 alone could support the additional demand required of the expanded facility. However, water level data from Well No. 2 is not currently available for comparison to verify these drawdown estimates.

Based on the additional requirement of 61 L/min and the average well yield for the Torbay area of 14.42 L/min, the number of supplemental wells required to meet the demand for the expanded facility could be on the order of four. These wells would likely need to be located off of the existing property in a configuration that minimizes the pumping interference within the well field and with off-site groundwater users.

5.2 GROUNDWATER SYSTEM

The water balance presented in Section 4.1 suggests that the extraction of groundwater by the expanded Arena and neighbouring residential and commercial properties is less than the groundwater recharge at the scale of the sub-catchment area and capture zone, which indicates that there is a suitable groundwater water supply for development. Numerical modeling presented in Section 4.3 demonstrates that there will not likely be any adverse well interference issues at neighbouring domestic wells due to the extraction of water at Well No. 2. This is based on the expected drawdown at these off-site properties being less than the 15 m used as the adverse condition criterion.

The Theis analytical solution (Section 4.2) indicate that pumping in Well No. 2 at a rate of 75 L/min would result in a drawdown of over 15 m in a water well located at a distance of 100 m from the pumping well (i.e., at Riverwood Place). This could be problematic if these wells are shallow and have shallower water-bearing zones. However, this method used to predict radial drawdown does not account for recharge, boundary conditions, and the layered geology that numerical model does (i.e., it is a simplified approach). There is insufficient information on water level fluctuations in surrounding water wells to confirm this potential effects, but there has been no indication that any wells have gone dry under existing operating conditions.

One of the limitations of the numerical model is that it is steady-state and thus cannot account for transient effects induced by pumping. The water level in a well will rise and fall as the pump cycles on and off to fill the pressure tank (and storage tank, if present). Domestic, commercial, and industrial water use generally creates times of peak demand corresponding to consumption, such as the high domestic use of water in the morning to flush toilets, shower, wash dishes, wash laundry, etc. The Arena likely achieves peak demand when ice is being made and during hockey tournaments. The worst-case scenario for well interference would be when these peak use events are simultaneous. Fully quantifying the actual impact imposed by these events would require detailed pumping and water level monitoring data from the extraction wells and from a number of monitoring wells located in the aquifer.

Both the JWL (2007) field test results from Well No. 2 and the numerical modeling results presented in Section 4.3 and Table 7 show that the Arena well has the potential to divert groundwater that would otherwise discharge to ponds and brooks (i.e., baseflow). In some predictive modeling cases the reduction in baseflow in Reach 105 of the simulated Kennedy's Brook is up to approximately 19%. However, the overall baseflow to the larger connected Kennedy's Brook network is much less variable.

6.0 CONCLUSIONS

Based on the review, results, and discussion, Stantec provides the following conclusions:

1. Twinning the ice surface will likely increase the maximum occupancy by 800 persons, which could see the total water demand for the expanded Arena facility go to more than 136 L/min. The estimated demand is equivalent to the resources required to supply 144 residential house lots. Therefore, the demand required for the expanded Arena is more than that required to satisfy the future development of the commercial property and Pine Ridge Valley developments. Thus, the Arena expansion will have the largest impact on the future cumulative effect on the groundwater system in the area.
2. There is currently no monitoring data to better constrain the on-site and off-site effects of existing Arena water use and assist in the prediction of changes to groundwater conditions that may result from expansion in the area.
3. The groundwater system can likely support the Arena expansion and the development of the neighbouring commercial and residential properties. Off-site drawdown effects are expected to be minimal. Baseflow to portions of Kennedy's Brook may be reduced as groundwater is diverted to pumping wells. The potential implications of this reduction on aquatic habitat has not been determined.
4. Well No. 2 (with lesser supplemental supply from Well No. 1) is capable for supplying the 75 L/min demand for the existing Arena, though the actual pumping level during operations has not been monitored to know if the available drawdown is being exceeded. An additional

water well(s) would be needed in any expansion scenario since Well No. 2 and Well No. 1 are likely operating at close to peak capacity. Well No. 2 has a relatively high yield compare to other wells in the Torbay area, and it is likely that more than one well would be required to meet the demand of the expanded arena facility.

7.0 REFERENCES

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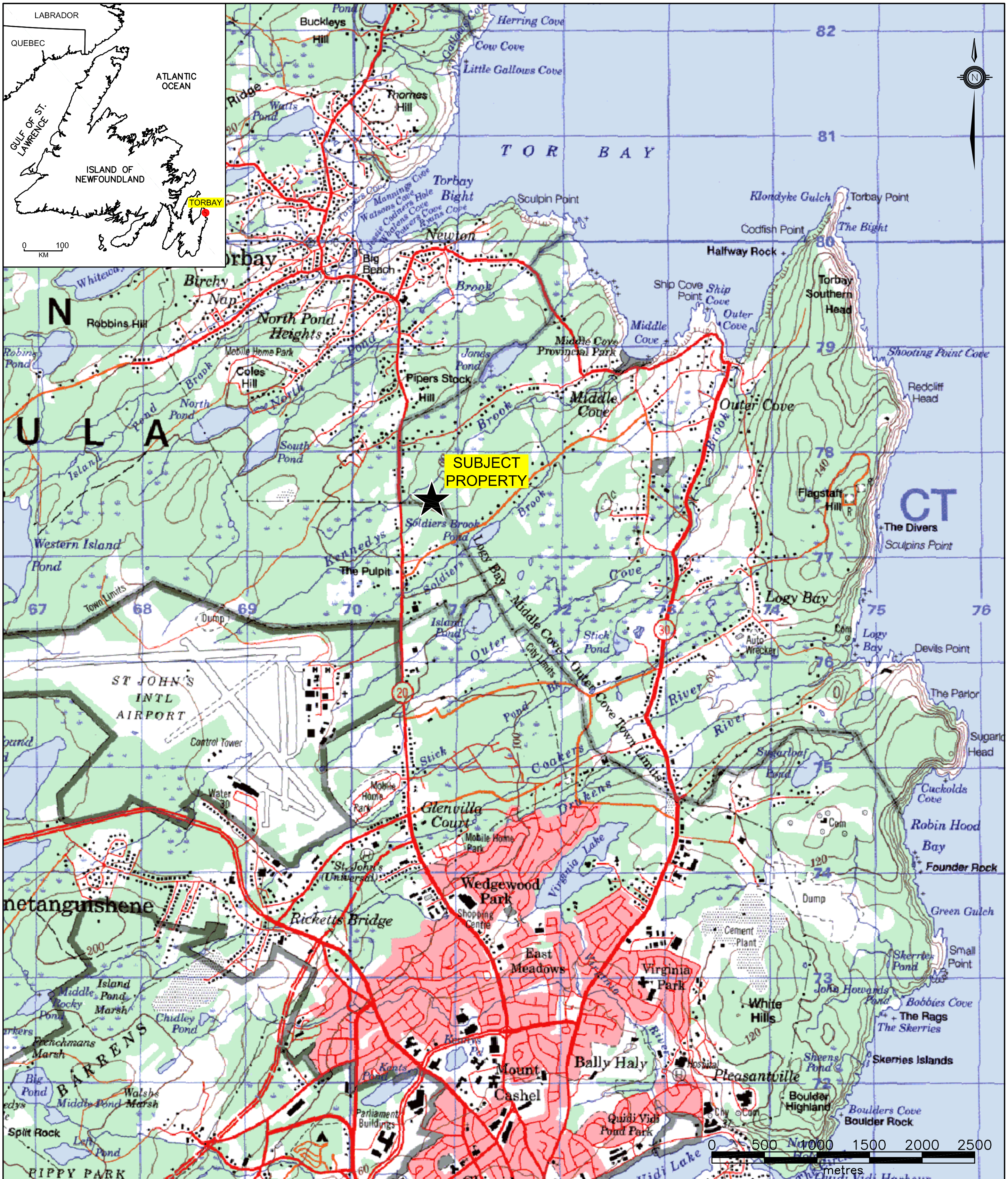
APPENDIX A

Drawings



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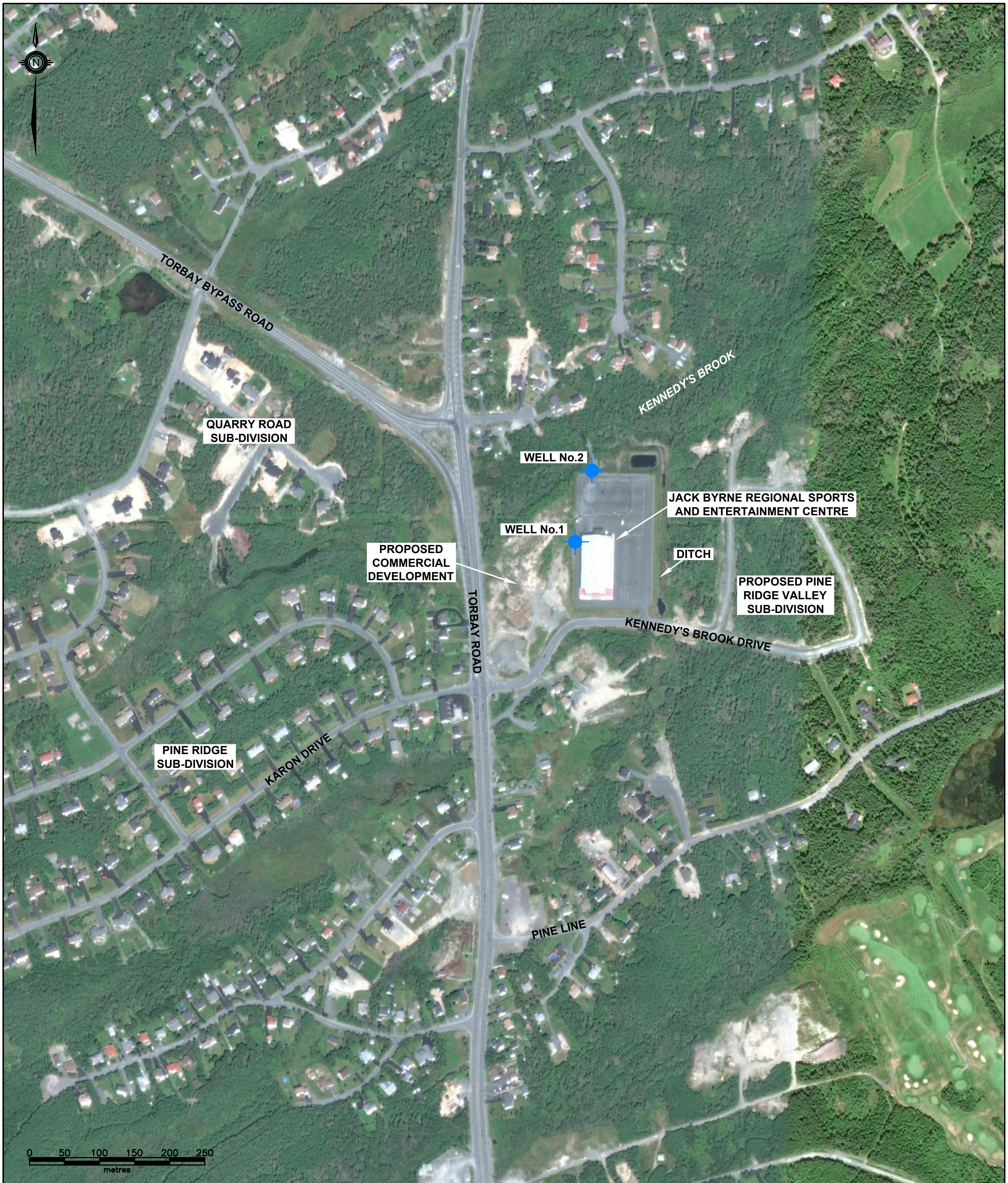

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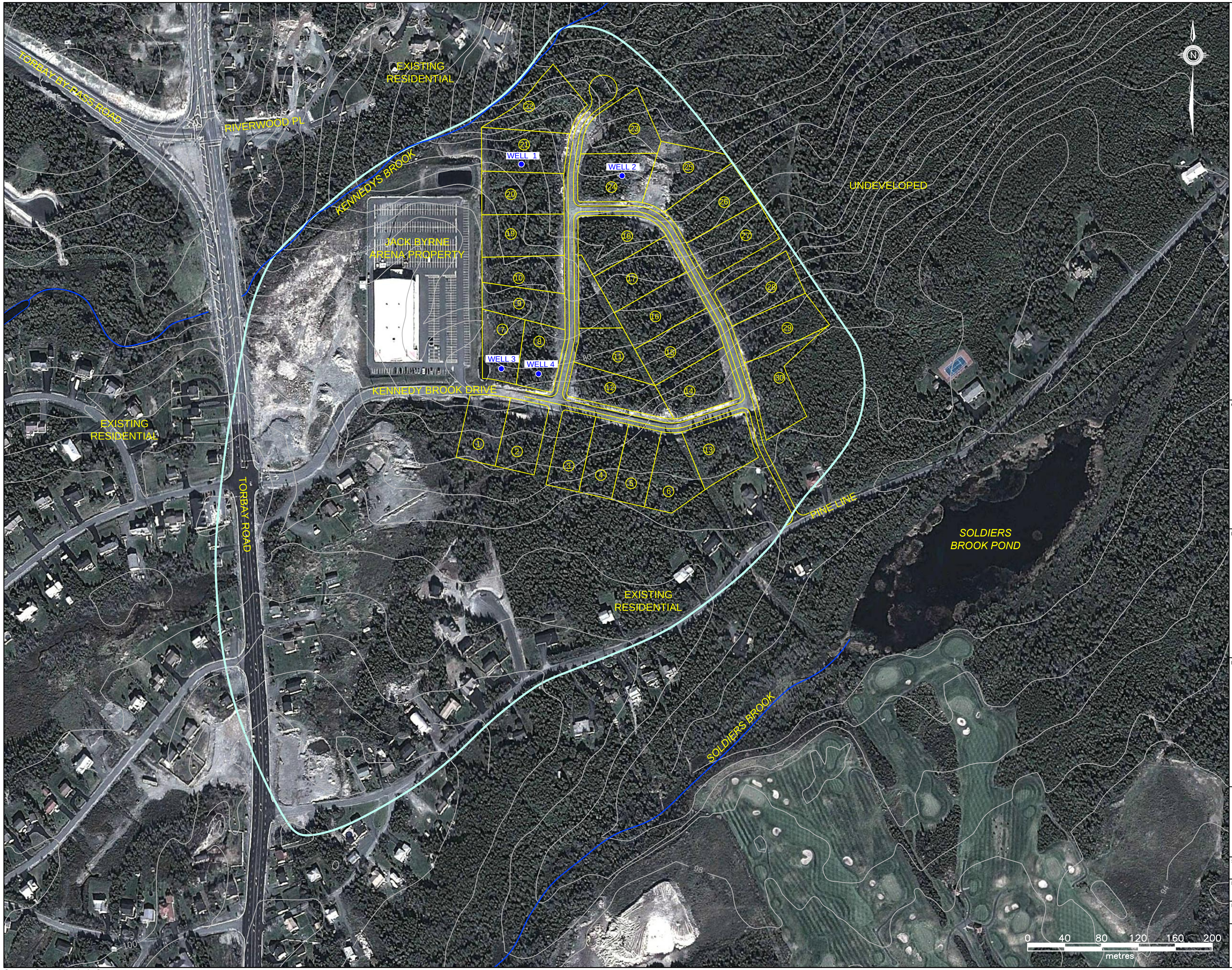

LEGEND

LOCAL DRAINAGE CATCHMENT AREA

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NORTHEAST AVALON ARENA REGIONAL BOARD

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GROUNDWATER INVESTIGATION FOR JACK BYRNE REGIONAL SPORTS AND ENTERTAINMENT CENTRE EXPANSION, TORBAY, NL

DRAWING TITLE:

SITE PLAN SHOWING ADJACENT PROPOSED PINE RIDGE VALLEY SUBDIVISION

Stantec Consulting Ltd.

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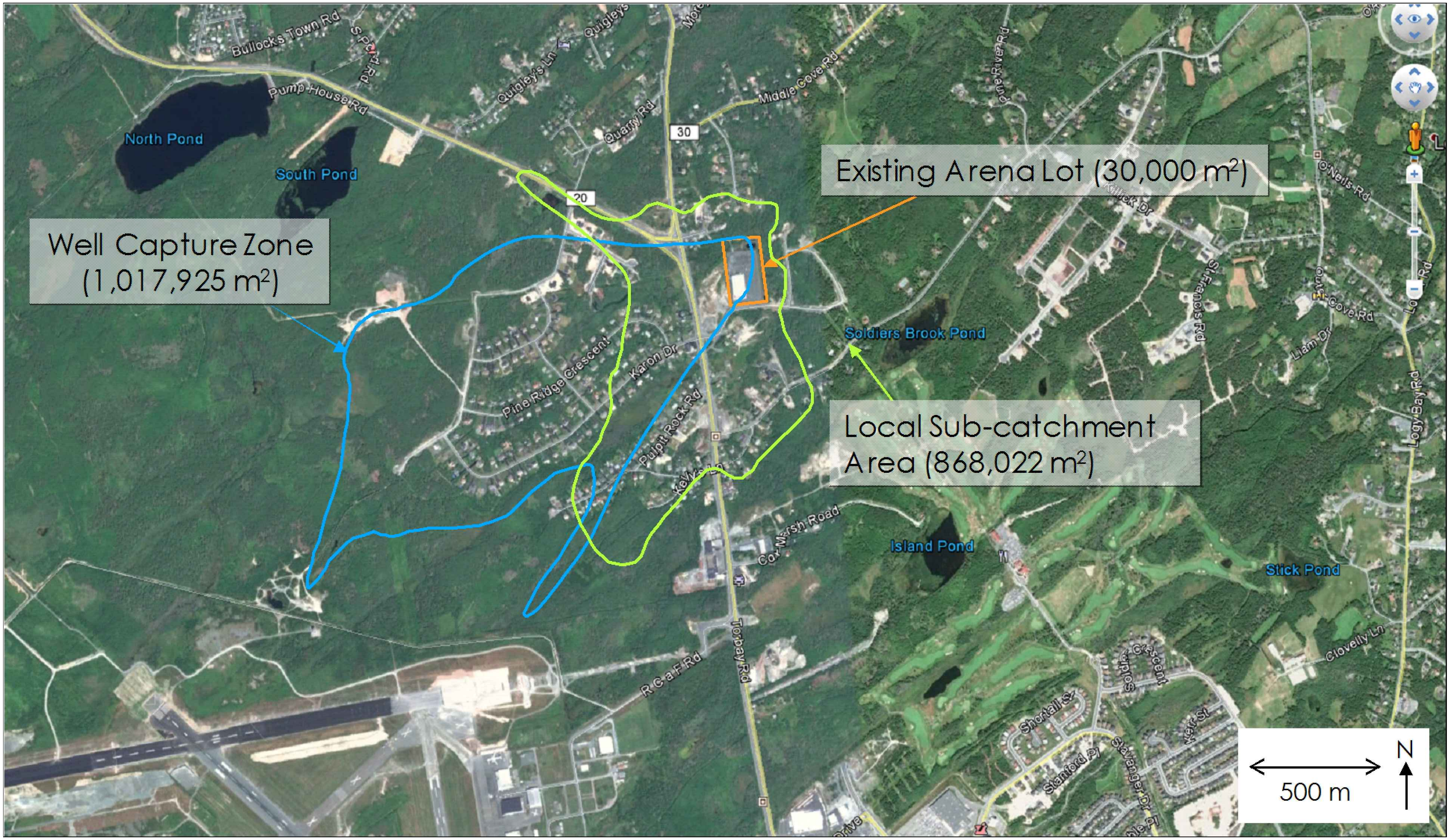

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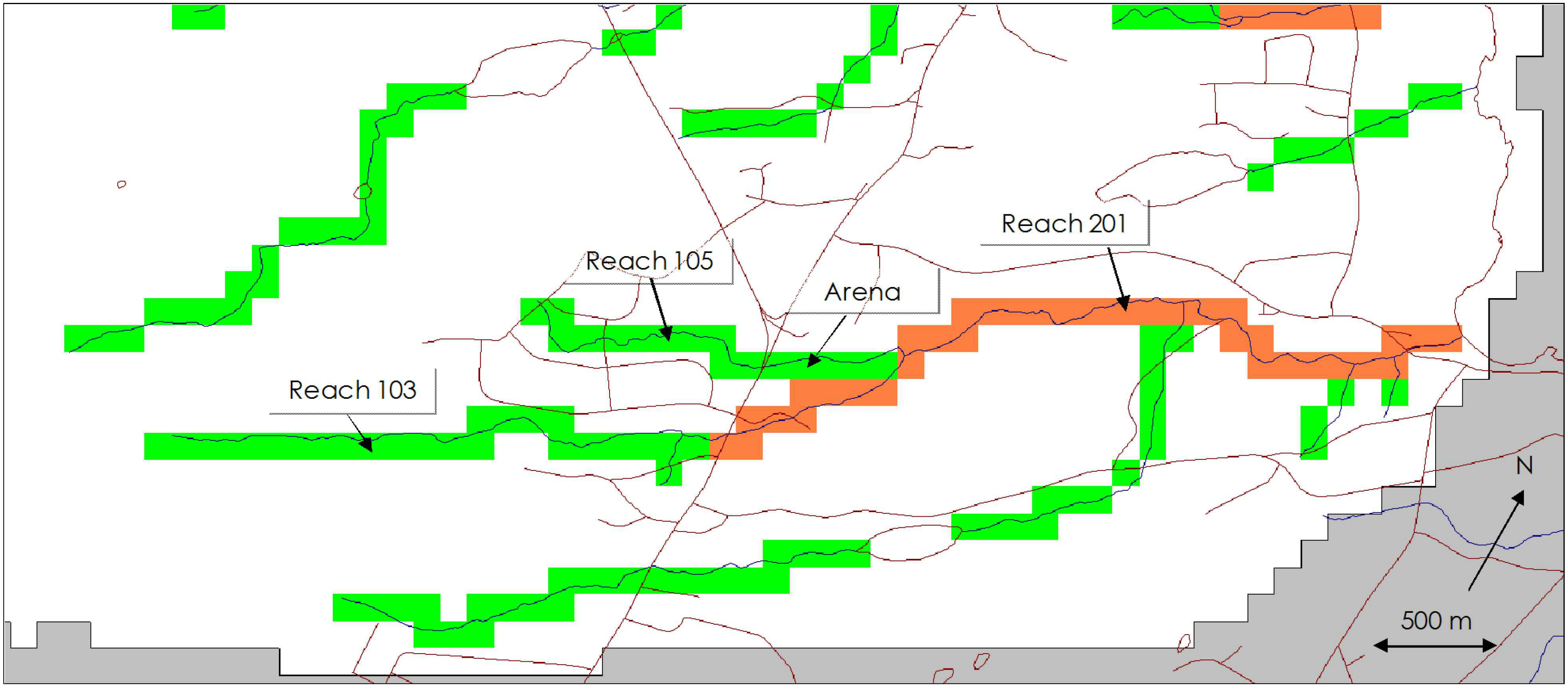

NOTE: RIVER BOUNDARY CELLS IN GREEN AND ORANGE REPRESENTING EXISTING BROOKS
(REACHES 103, 105 AND 201 ARE PART OF KENNEDY'S BROOK)

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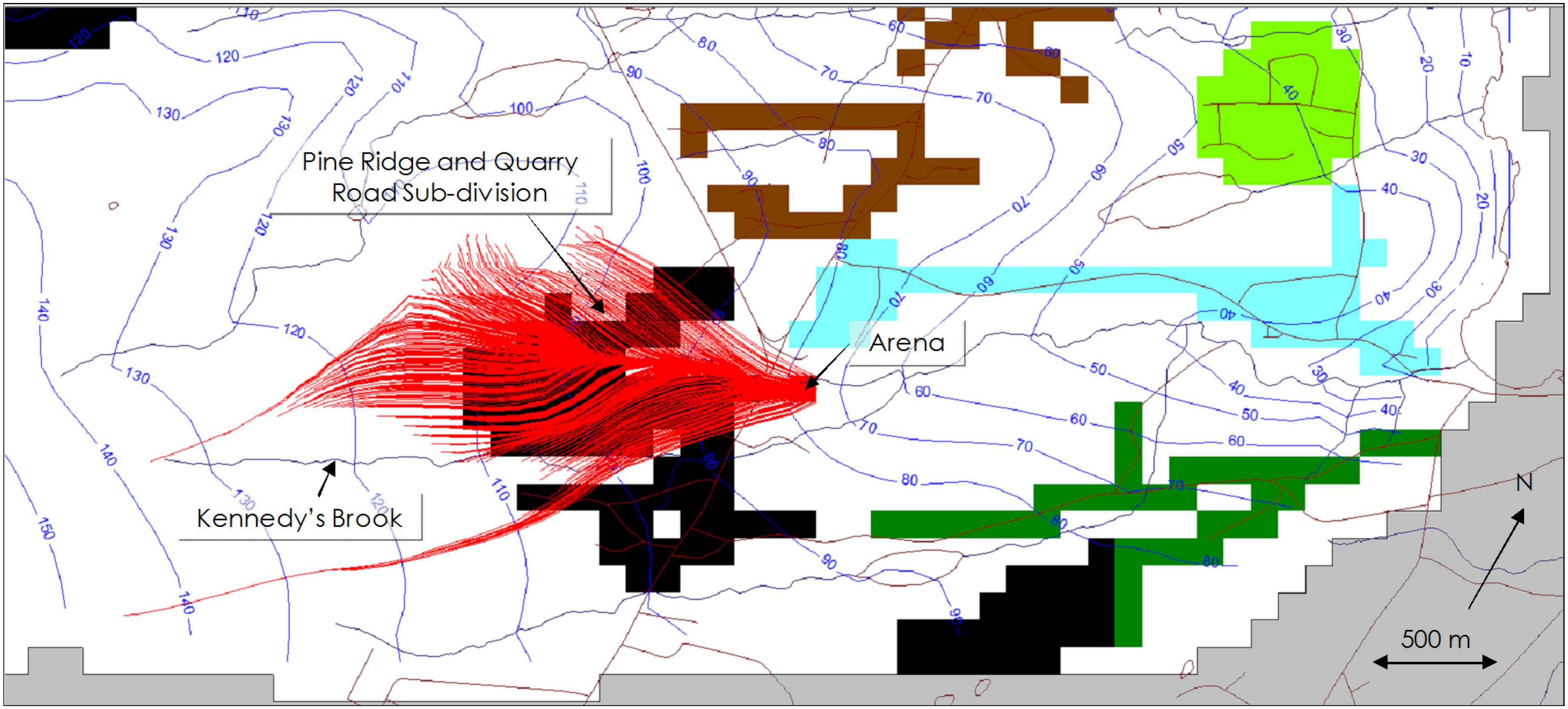

NOTE: GROUNDWATER ELEVATION CONTOURS (m ASL) AND FLOWLINES (RED) TO THE ARENA PROPERTY.
THE BLOCKS OF COLOURS REPRESENT GROUNDWATER PUMPING AT OTHER SUB-DIVISIONS

CLIENT: NORTHEAST AVALON ARENA REGIONAL BOARD	SCALE: AS SHOWN	DATE: APR. 27, 2017	REV. No. 0
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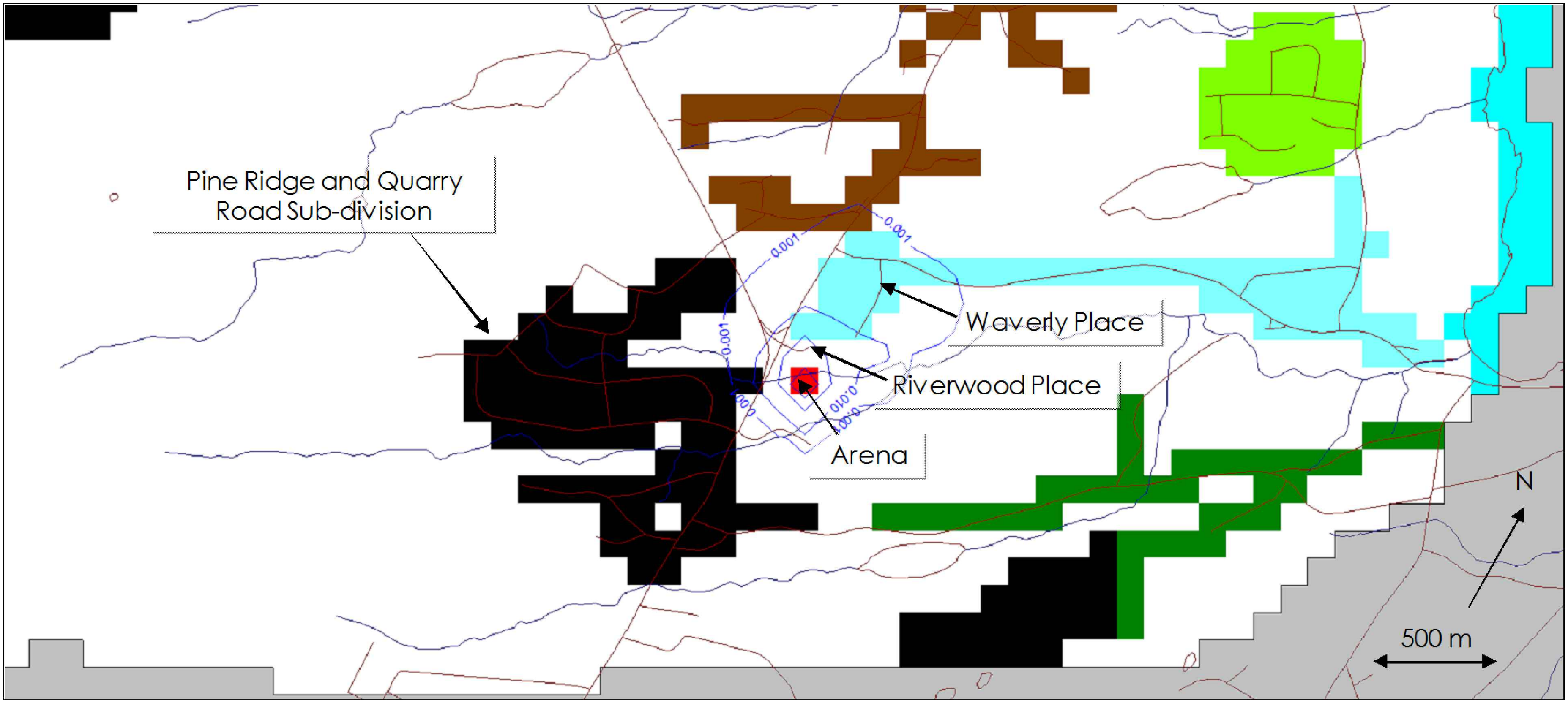

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NORTHEAST AVALON ARENA REGIONAL BOARD

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GROUNDWATER INVESTIGATION FOR JACK BYRNE REGIONAL
SPORTS AND ENTERTAINMENT CENTRE EXPANSION, TORBAY, NL

DRAWING TITLE:
CASE A DRAWDOWN CONTOURS (m) DUE TO INCREASED PUMPING AT THE ARENA PROPERTY

SCALE: AS SHOWN	DATE: APR. 27, 2017	REV. No. 0
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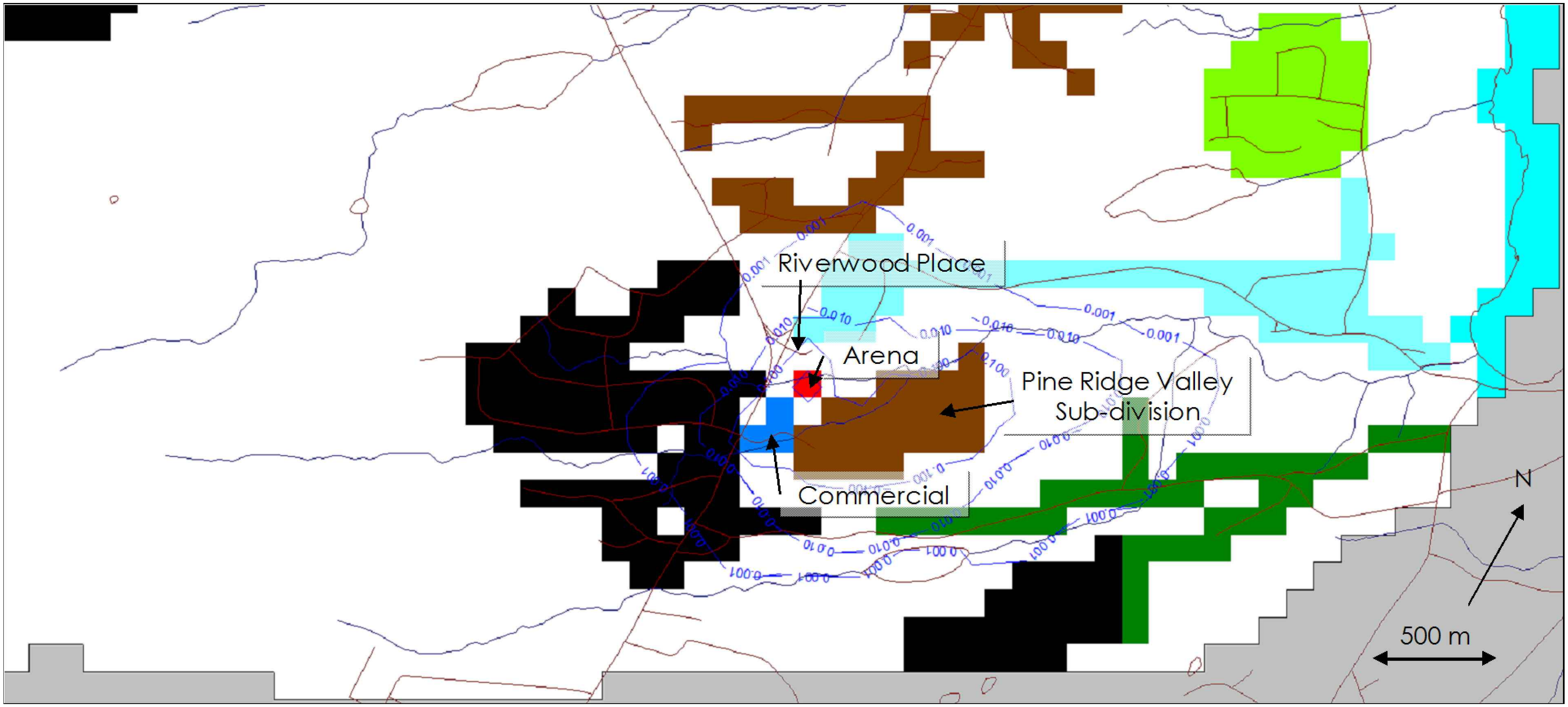

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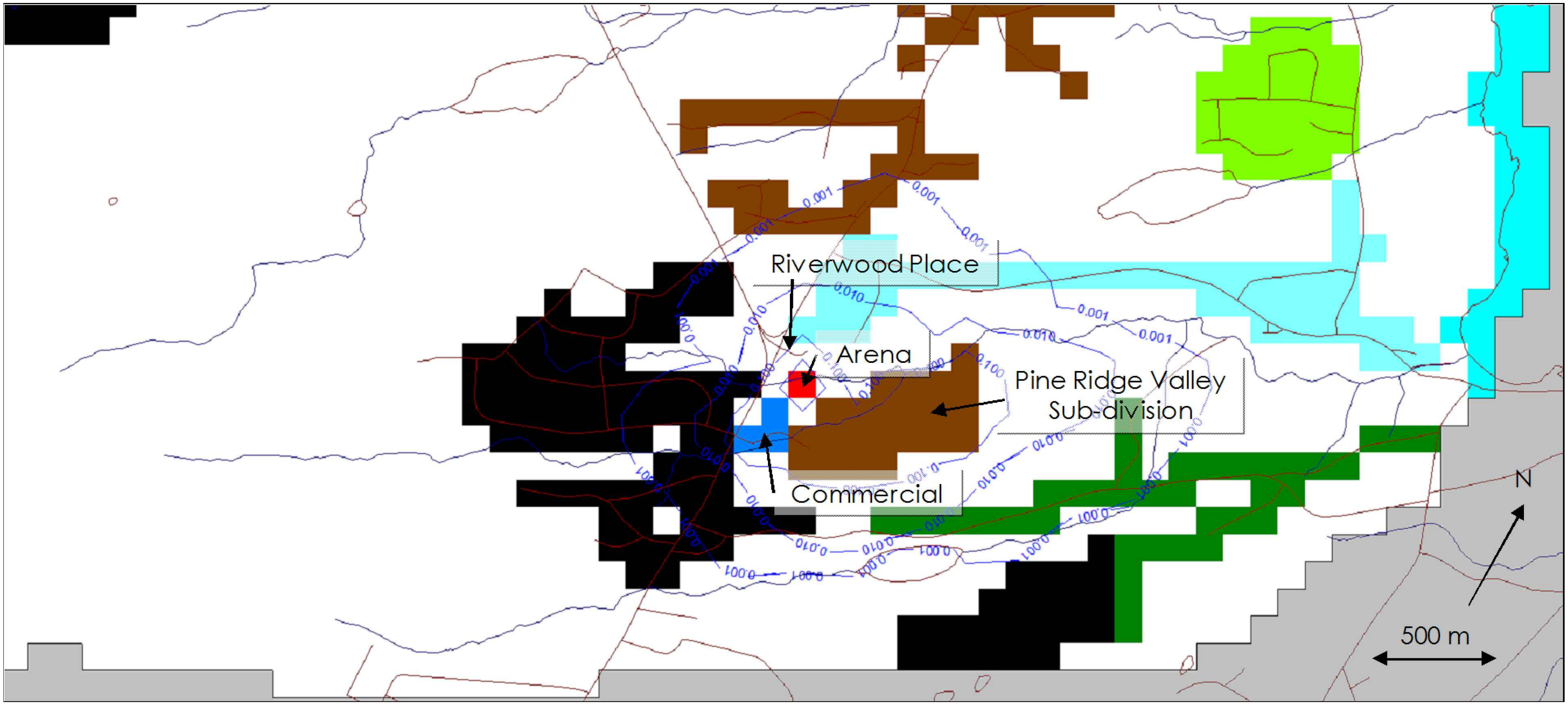

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CLIENT: NORTHEAST AVALON ARENA REGIONAL BOARD

PROJECT TITLE: GROUNDWATER INVESTIGATION FOR JACK BYRNE REGIONAL SPORTS AND ENTERTAINMENT CENTRE EXPANSION, TORBAY, NL

DRAWING TITLE: CASE C DRAWDOWN CONTOURS (m) DUE TO ARENA EXPANSION AND THE DEVELOPMENT OF COMMERCIAL PROPERTY AND THE PINE RIDGE VALLEY SUB-DIVISION

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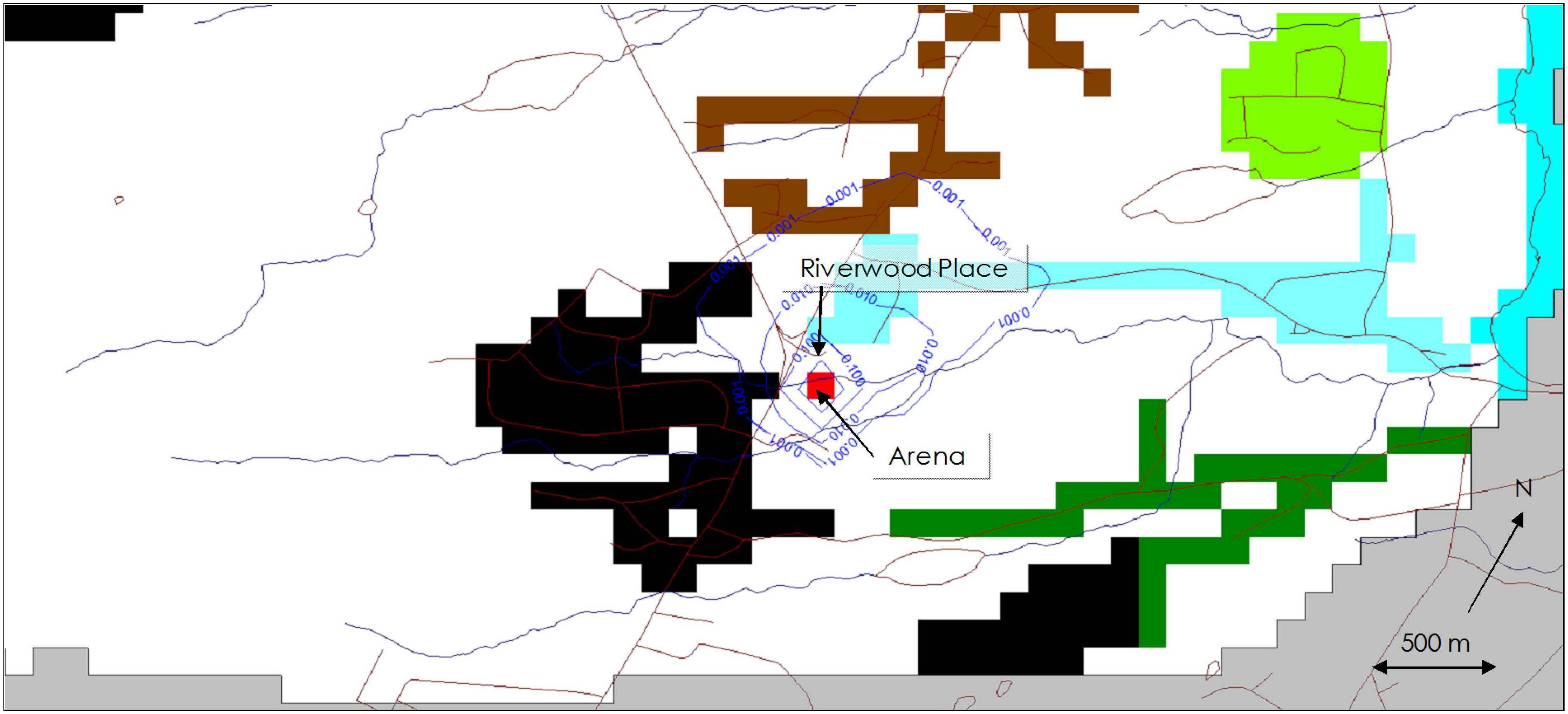

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Stantec