Minimizing water residence time in Quebec City's main distribution network using hybrid discrete dynamically dimensioned search (HD-DDS): Part II

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ABSTRACT

This paper presents a methodology to assess and reduce water residence times in a water distribution system in order to improve water quality. The methodology was developed and validated on Quebec City's main distribution system. A tracer study was conducted to evaluate real residence times and results are presented in the 'companion paper' (Part I) in this issue. A hydraulic model was then built to simulate the mean residence times (MRT) and develop strategies to reduce them. An optimization algorithm (hybrid discrete dynamically dimensioned search, HD-DDS) was used to calibrate the model using flows and pressures measured in the distribution system. Results show that the suggested methodology can lead to significant reductions in MRT (25.6%) in parts of the distribution system, but could also lead to significant loss in pressure, which should be monitored closely.

Key words | HD-DDS, hydraulic modelling, optimization, water distribution systems, water quality, water residence time

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ABBREVIATIONS

DS	water distribution system
DBP	disinfection by-product
GIS	geographic information system
HD-DDS	hybrid discrete dynamically dimensioned search
MAE	mean absolute error
MRT	mean residence time
RMSE	root mean square error
S1	Sillery
UL	Université Laval
UT	Upper Town
WTP	water treatment plant

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INTRODUCTION

In the literature, there is a consensus that water quality in a water distribution system (DS) varies over time, with shorter water residence times in a system being associated with high concentrations of free residual chlorine and low concentrations of disinfection by-products (DBPs) (Rodriguez *et al.* 2007). Knowledge of water residence times contributes to a better understanding of the biological and chemical phenomena at work inside a system. Hydraulic modelling of DS enables the simulation of water residence times in an entire system, allowing operators, managers and engineers to take decisions to improve water quality where needed. This information can be useful to determine locations for re-chlorination stations, sampling points, etc. (DiGiano *et al.* 2005). Uncertainties about water demands, their variations in time and the lack of information about the real physical characteristics of the pipe (roughness, effective diameter, etc.) are the main sources of error when modelling and simulating the behaviour of a DS (Ormsbee 1989; Ormsbee & Lingireddy 1997; Al-Omari & Abdulla 2009).

Calibration of a model is non-negligible to ensure its accuracy for pressure, flow and water quality simulations (Rodriguez *et al.* 2004; Al-Omari & Jamrah 2005; DiGiano *et al.* 2005; Jonkergouw *et al.* 2008). Calibration involves changing physical characteristics (such as pipe diameters and their roughness coefficients, water demands and their daily patterns) and modelling parameters to reduce the difference between simulation results and observations. These adjustments can be done manually, but this can be a difficult and time-consuming task for distribution systems of significant size. An optimization algorithm is, thus, very useful to calibrate a model.

Using a well-calibrated model, it is possible to verify strategies to reduce water residence times. In general, this consists of changing some elements of the model to force water to take different paths in order to reduce mean residence times (MRTs), thus promoting higher levels of residual chlorine and reducing the formation of DBPs (Simard *et al.* 2011). One strategy is to test the impact of closing valves on large diameter pipes to force water flow in smaller diameter pipes. This results in an increase in water velocities since the flow area is reduced, thus reducing MRTs. However, this strategy increases head losses, so monitoring pressures is required.

Modulating valves may also be used. This strategy also forces water into smaller diameter pipes without being as restrictive as fully closing valves. These modulating valves can control flow circulating in a specific pipe in real time. When the water demand is lower (e.g., night, weekend, winter season), water could be diverted from the water mains to smaller diameter local pipes to increase velocities and thus reduce MRTs. During times of higher water demand (e.g., mornings, summer season), directing water flows to larger diameter water mains would be a better choice to avoid excessive head losses. Evaluating such a strategy would quantify its benefits and also allow managers to plan, during rehabilitation works on the system, the set-up of modulating valves on water main access chambers (Prasad & Walters 2006).

This study was designed to develop a methodology to reduce water residence times in a DS in order to ensure greater concentrations of residual chlorine and reduce the formation of DBPs. To achieve this objective, experimental MRTs were first evaluated using a tracer study, which is presented in the 'companion paper' (Part I) in this issue (Delisle *et al.* 2015). Then, a hydraulic model was built to simulate these MRTs. The model, calibrated using observations (measured flows and pressures) and validated with the measured MRTs, was used to verify hydraulic strategies to reduce MRTs. The methodology is developed and verified on the main drinking water DS in Quebec City (Canada). The study also highlights the inevitable conflict between MRT and hydraulic performance in a real-life DS.

LITERATURE REVIEW

First, DS hydraulic model calibration and the criteria used for testing the quality of model calibration are presented. Then, different optimization algorithms used in the field of drinking DSs are revised.

Calibration

This research used an implicit model for calibration in which an optimization program is linked with a hydraulic model to optimize hydraulic model parameters. Ormsbee (1989), among many others, provides guidelines on conducting implicit DS model calibration.

Data collected during fire flow tests were the most valuable for calibrating the roughness values in pipes because they generate the highest head losses (Walski 2000). The advent of real-time monitoring and wireless reporting technologies has created expansive databases that are available for calibration; however, a large proportion of DS data are not suitable for calibration (Walski 2000) due to measurement inaccuracy. As pressure monitoring accuracy improves and more real-time data are collected, the importance and potential of utilizing many types of data in model calibration is increasingly evident. However, rigorous investigations assessing the effect of various field data on model calibration are lacking (Ostfeld *et al.* 2012). A review of DS calibration emphasized that investigations of model prediction uncertainty are particularly scant (Savic *et al.* 2009).

Jonkergouw *et al.* (2008) studied the use of residual chlorine concentrations to calibrate a DS model, whose demands are unknown or uncertain, by adjusting the consumption factors, as well as adjusting, by sector, the decay constant of chlorine linked to reactions with pipe walls and, finally, the concentration of chlorine at the source. They used a modified version of the shuffled complex evolution algorithm, developed by Duan *et al.* (1993). Their methodology was tested on three variants of a DS model previously studied by Vasconcelos *et al.* (1997). A maximum relative error of 2.2% was obtained for the consumption factors, 2.3% for the decay constant of chlorine linked to pipe walls and 1.3% for the chlorine concentration at the source. The authors found that, to achieve good results in terms of simulation of water quality, the model has to accurately simulate average flows and that daily variations have little influence.

There are several criteria to check the quality of a calibration. The most commonly used are the mean absolute error (MAE) (Maidment 1993; Jonkergouw *et al.* 2008) and the root mean square error (RMSE) (Maidment 1993). These criteria, largely used in the field of hydraulics and hydrology, are respectively, defined as follows:

$$\mathrm{MAE} = \frac{1}{N}\sum_{i=1}^{N} |y_i - \tilde{y}_i| \tag{1}$$

$$\mathrm{RMSE} = \sqrt{\frac{1}{N}\sum_{i=1}^{N} (y_i - \tilde{y}_i)^2} \tag{2}$$

where y_i is an observation and $\tilde{y}_i$ is an estimation and N is the total number of observations.

In any case, for the same data set, the MAE is always smaller or equal to the RMSE. The RMSE criterion is significantly influenced by large errors, due to the squared power. For both criteria, a perfect adjustment generates a value of zero.

Optimization

In the field of drinking water distribution, optimization is mainly used to reduce costs, often those associated with the choice of pipe diameters, while maximizing efficiency (often pressures) when designing new distribution systems or for rehabilitation works. For water distribution hydraulic models, heuristic optimization methods are most often used. Noteworthy, heuristic optimization methods are genetic algorithms (Simpson *et al.* 1994; Savic & Walters 1997; Wu & Simpson 2001; Tolson *et al.* 2004; Dandy & Engelhardt 2006), simulated annealing (Cunha & Sousa 2001), ant colony optimization (Maier *et al.* 2003; Zecchin *et al.* 2006, 2007), shuffled frog leaping algorithm (Eusuff & Lansey 2003) and particle swarm optimization (Suribabu & Neelakantan 2006), among others. Although, all of these algorithms attain a good solution, sometimes after some tweaking, they require a large number of parameters to control their search strategy. The choice of these parameters can have a significant impact on their performance.

Tolson *et al.* (2009) presented the hybrid discrete dynamically dimensioned search (HD-DDS) algorithm. HD-DDS is a single-objective optimization algorithm for solving problems with discrete decision variables. It was originally developed for solving water distribution network design problems. However, as shown in Matott *et al.* (2012), HD-DDS can efficiently solve other discrete problems such as sorptive barrier design problems. For DS problems, it is linked to EPANET for resolving hydraulics and water quality. Unlike conventional algorithms, it does not require parameter adjustments. A test bench on three case studies was conducted by the authors to compare the performance of HD-DDS with those of ant colony optimization, genetic algorithms and particle swarm optimization. Results indicate that, in all cases, a better solution was found using HD-DDS, with only 50% of the computing time required by the others. This time saving is mainly due to the fact that HD-DDS does not necessarily perform a hydraulic simulation (EPANET) at each evaluation of the objective function. For one of the studied cases, the worst result obtained by HD-DDS was better than the best result obtained by genetic algorithms, although the latter used 100 times more evaluations of the objective function. These encouraging results led us to choose HD-DDS to do the calibration of our hydraulic model from measurements of pressures and flows carried out in the field.

METHODOLOGY

The methodology developed in this research has been tested on one of the drinking DS in Quebec City. The city is supplied with drinking water through many independent distribution systems. However, interconnections can be opened during periods of high demand or when work is being done on one of the networks to ensure adequate supply and pressures. The study area corresponds to several sectors of the main DS and is presented in Figure 1 of the 'companion paper' (Part I) in this issue (Delisle *et al.* 2015). A hydraulic model of the study area was built and data collected during sampling campaigns were used to calibrate the model. Strategies to reduce MRTs were evaluated using the calibrated model. Model development followed the steps described in Figure 1. These steps are adapted from those suggested by Ormsbee & Lingireddy (1997).

Building the model

There are several hydraulic models available to perform hydraulic calculations for pressurized flows. EPANET (V2.00.12) was chosen for the current study (Rossman 2000). This model, widely used in the scientific community, is available from the US Environmental Protection Agency. A hydraulic model of the water mains was made available to our research team and was completed by the addition of all the local pipes and by associating appropriate water demands at the different model nodes. The choice of the initial physical characteristics and simulation parameters (e.g., diameter, roughness, elevation, water demand) was based on the data available at the time in the city and our research group databases: distribution network plans, flows and pressures acquired by telemetry, 'Real Estate Assessment Roles' (Gouvernement du Québec 2003), 'Drinking Water Operation Scheme' (Girard *et al.* 2008), etc.

Based on the location of valve chambers equipped with flowmeters, the total water consumption in a sector (Q_{cons}) can be estimated. In addition, one city database lists the average daily and annual consumption for important drinking water users. Average water demands from

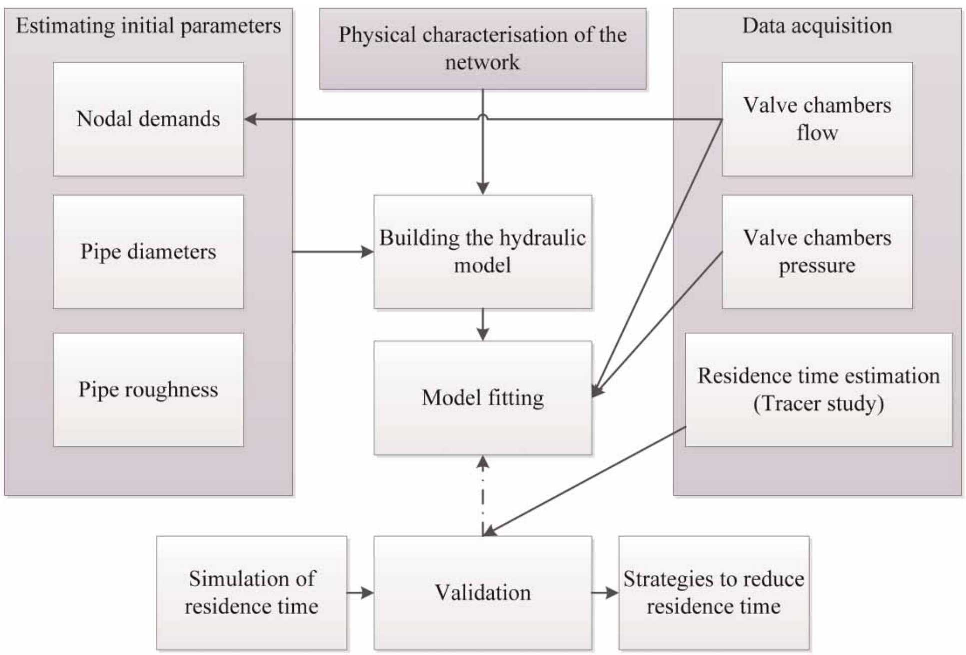

Figure 1 | Methodology.

these industries, businesses and institutions can, therefore, be associated with a particular node of the hydraulic model. Given that there are very few residential water meters, water demands at the nodes of the model were estimated using information from the 'Real Estate Assessment Roles' database (Gouvernement du Québec 2003). Since nodes are geo-referenced, it was possible to link all the buildings in the study area, whose centroid coordinates are contained in the database, to the closest (in Euclidean distance) node, using a geographic information system. The shortest Euclidean distance facilitates this operation, but may result in some association errors. Thus, land uses that generate larger water demands should be validated. This quick association method, despite its drawbacks, is more accurate than other techniques (e.g., even distribution of consumption at each node).

Flow and pressure data

Flow and pressure data, necessary for model calibration, were obtained in two steps: (1) sampling campaigns to assess MRTs in the DS and to identify the limits of the DS (presented in the 'companion paper'; Delisle *et al.* 2015); and (2) from the permanent meters installed in valve chambers. Valve chambers are usually located where water mains supply the various neighbourhoods (Figure 2). The data are transmitted by telemetry and stored in one of the city databases.

Model calibration

In general, calibration in a steady state (average water demand) is more responsive to changes in pipe diameter or roughness, while calibration in a non-steady state is more responsive to changes in peak factors associated with water demands at nodes (Ormsbee 1989). Calibration is often performed in a two-step process. In this case study, local variation of water demand (in local pipes or linked to residences) was not available, so calibration was only performed in a steady state, although simulation tests were realized in a non-steady state with various hourly demand patterns.

The steady-state calibration was performed using two sets of calibration parameters for comparison purposes. First, diameters were adjusted (a decrease in value to represent smaller effective diameters with time) to fit simulation results with average observed pressures and flows. Second, roughness coefficients (a decrease in Hazen–Williams coefficients to represent increased pipe wall roughness with time) were adjusted. Tests were also realized in a non-steady state by modifying peak factors

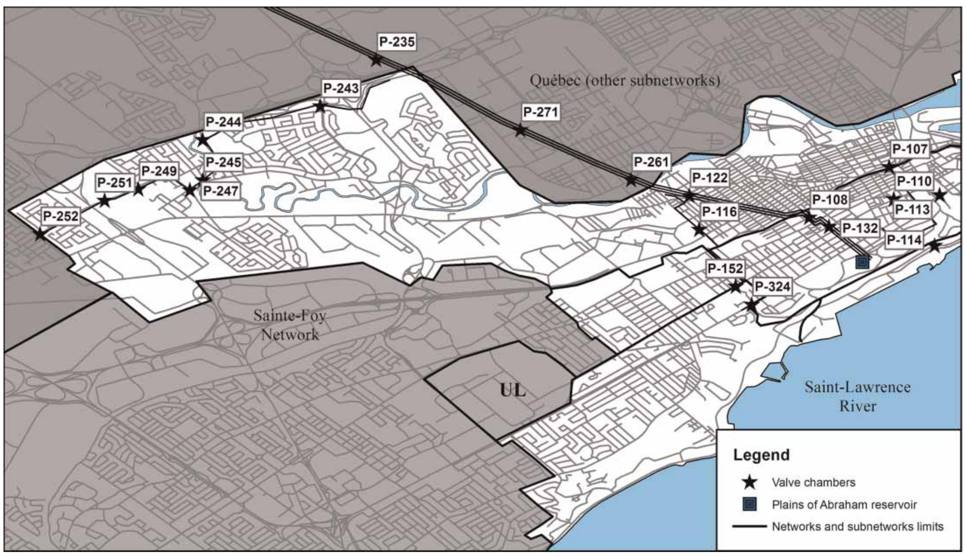

Figure 2 | Location of valve chambers.

associated with water demands at each node to fit variations in flows and pressures and tracer concentrations over time. The HD-DDS algorithm for water DS design optimization (Tolson *et al.* 2009) was adapted to perform the steady-state calibration. The algorithm seeks to minimize the following objective function:

$$f(x) = \sum_{d=1}^{D} |\tilde{v}_d(x) - v_d| \quad \forall d = 1, 2, \ldots, D \tag{3}$$

where x is the suggested solution, $\tilde{v}_d$ a simulated result based on the suggested solution, v_d an observation made in the field, d a control point where observations were made and D the total number of control points.

In an iterative process, the algorithm seeks to generate scenario x, which corresponds to a combination of diameters (or roughness coefficients, the calibration parameters) that minimizes the sum of errors between simulated and observed pressures and flow rates (the criteria to meet).

All pipe diameters (or roughness coefficients) can be changed globally or independently of each other. It is also possible to group pipes by class and change parameters by class. Each pipe parameter to be changed is assigned an upper bound, corresponding to the initial and maximum theoretical value of the parameter, and a lower bound, corresponding to a fraction of the initial theoretical value, between which the algorithm chooses the new value for the parameter. As mentioned previously, the effective pipe diameter decreases over time and the roughness coefficient increases, due to the formation of tubercles on cast iron pipe walls, for example.

Once the parameters are changed, the criteria for pressure and/or flow are checked. In theory, it is possible to combine these two criteria to formulate the objective function. However, it is suggested to check a single criterion at a time because, in order to combine them, it is necessary to standardize the data, since these are different physical measures. This heuristic optimization algorithm was developed to find good overall solutions rather than an optimal solution, like most other algorithms. It was designed to adjust its search horizon throughout the process, starting with overall changes then gradually moving to local changes, mimicking the steps of manual calibration. The number of decision variables therefore decreases as the maximum number of iterations is reached.

This optimization is performed under certain constraints. The solution scenario suggested by the algorithm must meet the minimal and maximal pressures (P) defined by sector or globally. The suggested solution ($\tilde{P}$) is rejected otherwise. These constraints are formulated as follows:

$$P_i^{\min} \leq \tilde{P}_i(x) \leq P_i^{\max} \quad \forall i = 1, 2, \ldots, N \tag{4}$$

where P_i is the pressure at node i and N the total number of nodes where the constraints (pressures) must be respected. The iterative process ends when the goal $f(x) = 0$ or the maximum number of iterations defined beforehand is reached. It is unlikely that the objective function reaches 0 in a complex optimization problem. It is therefore very important to define a maximum number of iterations that must not be exceeded, since the calculation time can become very high.

Finally, it is important to conduct a validation of the calibrated hydraulic model. A series of validation data was used to assign new water demands to nodes and perform a new hydraulic balance. Success of the validation is checked using the quality adjustment criteria (MAE and RMSE), defined above, in terms of flow and pressure.

Hourly fluctuations in water demands can influence not only the water paths within the network and therefore the tracer concentrations, but also the MRT. Since the only flow data available were those obtained at the valve chambers supplying entire neighbourhoods (where the observed flow rate variations are much less important than locally), tests were realized with demand patterns constructed following trends commonly observed (important peaks in the morning and/or evening) to assess the impact on MRTs.

Observed and simulated MRTs

Observed MRTs at different locations in the case study area were obtained with a tracer study. The methodology and results of this study are presented in the 'companion paper' (Part I) in this issue (Delisle *et al.* 2015). Tracer studies can also be useful in identifying the limits of areas supplied by more than one source ('companion paper' (Delisle *et al.* 2015) and Simard *et al.* 2009).

EPANET has a simulation module for water quality, including simulation tools for various types of tracers and to calculate water age (time spent in the network from the source which can be the exit of the treatment plant or a reservoir). According to Rossman (2000), these calculations, established on the principles of mass and energy conservation, use a time-based Lagrangian approach to track the movement of each water element through the pipes, including mixing at nodes. In addition to the transport of chemicals by advection, dispersion and diffusion can also be considered. It is also possible to take into account reaction kinetics (increasing or decreasing), of first- or second-order, between the various water constituents (contaminants, tracer, etc.), and also with the pipe walls. Therefore, it is possible to validate the simulated MRTs based on a simulated tracer injection and the techniques described above or by directly using the tool to calculate the water age in EPANET.

RESULTS AND DISCUSSION

Model calibration

The model behaviour is first checked based on readings of pressure and flow at various valve chambers located on the outskirts of, and inside, the study area (Figure 2). The hydraulic head at the water treatment plant (WTP) from the 'Drinking Water Operation Scheme' (Girard *et al.* 2008) was set in the model at 150 m. Flow rates and pressures obtained with the uncalibrated model differed from measured values depending on the valve chambers. Simulated and observed flow rates are shown in Figure 3, while simulated and observed pressures are shown in Figure 4. There is no pressure measurement at the Venturi. A major discrepancy is observed for flow rates in the three Venturi water mains (750, 1,015 and 1,070 mm) as well as at valve chambers P-107 and P-113 (Figure 3). At the Venturi, flow rates predicted by the model are much higher than those observed in the field. Since, flow rates passing through the Venturi are equal to the one exiting the WTP (no water demand between these two points) and water demands at all nodes are equal to the water production at the WTP, water produced in excess, in the model, can only go to a reservoir, the one on the Plains of Abraham in this case. The reservoir actually fills up much more than it empties, resulting in excessive water production at the WTP. This is due to poor modelling of the reservoir behaviour in the uncalibrated model. Simulated flow rates for valve chamber P-107 located downstream of the reservoir and P-113 are much lower than the observed flows. Valve chamber P-113 represents a connection between the UT1 and UT2 sectors; a local modelling problem is suspected in this case.

The lack of data on the Venturi pressures may be largely responsible for the poor pressure simulation at all valve

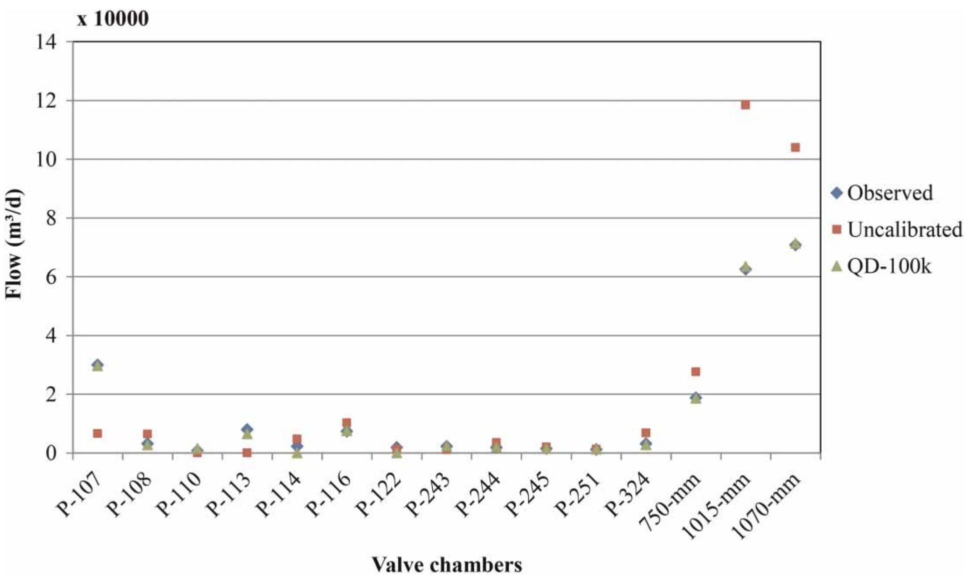

Figure 3 | Observed, uncalibrated model and QD-100 k model flow rates.

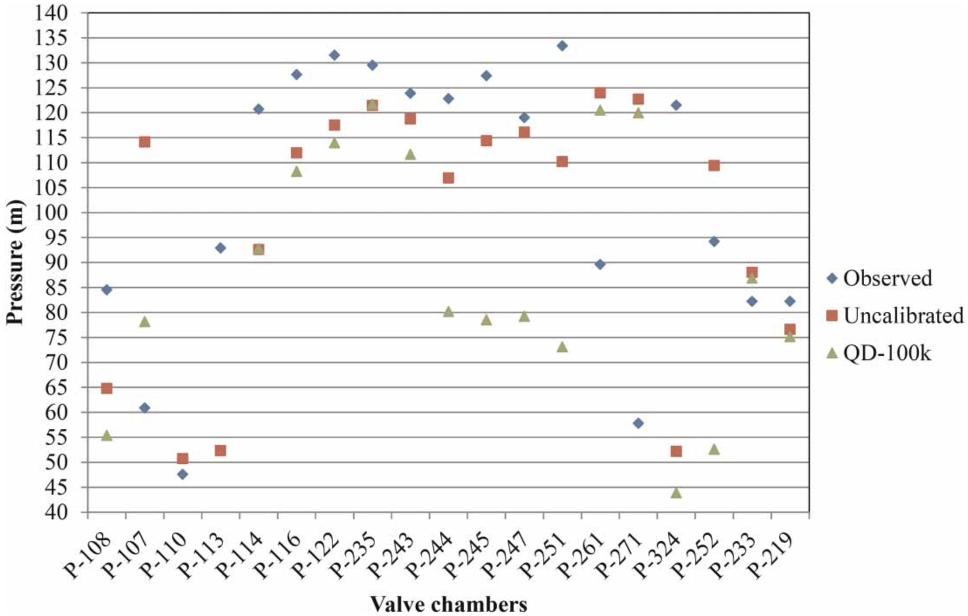

Figure 4 | Observed, uncalibrated model and QD-100 k model pressures.

chambers shown in Figure 4. Pressures were underestimated by the uncalibrated model in most valves chambers, except for chambers P-107, P-261 and P-271, where they were greatly overestimated. It could be that the hydraulic head at the WTP was not set correctly in the model. Different hydraulic heads were tested but no major improvement was noted. The facts that water is distributed by three pipes with different diameters, that these pipes supply sectors in different proportions and that there are connections between the pipes, make it difficult to adjust the model properly. To improve the hydraulic balance of flows and pressures in the model, a micro-calibration, using the HD-DDS optimization algorithm, was made. Different combinations of criteria (pressure and flow) and calibration parameters (diameter and roughness coefficient) resulted in the five scenarios shown in Table 1. Because the model has to accurately simulate the injection of fluoride in the DS, the model was calibrated using the data set from 9 May 2008. Once the optimization was done, water demands at nodes were replaced by those from 24 and 25 May 2008 (the fluoride sampling campaign) for validation.

Table 1 | Calibration scenarios

Criteria	Calibration parameter	Scenario
Flow (Q)	Diameter (D)	QD
	Roughness coefficient (C_{HW})	QC
Pressure (P)	Diameter (D)	PD
	Roughness coefficient (C_{HW})	PC
	Uncalibrated	UC

Hydraulic balances of flows and pressures, for the five scenarios presented in Table 1, were tested using the two error criteria (MAE and RMSE) described before. Results are shown in Table 2 for all scenarios using 10,000 (10 k), 100 k and 1,000 k iterations as a stopping criterion for the optimization algorithm. Hydraulic balances of flows are always improved by changing either diameters (QD) or roughness coefficients (QC) compared to the uncalibrated model (UC) but, with these two scenarios, all three error criteria for pressures deteriorate compared to the UC model.

Pressure measurements used for optimizing the DS are obtained upstream of pressure reducing valves and are mainly dependent on the pressure at the WTP and head losses in the water mains. Pressure measurements immediately downstream of pressure reducing valves are also stored in the database, but these pressures depend largely on the actual calibration of the valves, not on the head losses in local pipes. Local head losses could be estimated if pressure readings were performed within the

Table 2 | Appreciation of calibration results on flow or pressure through two adjustment criteria

		Flow		Pressure	
Number of iterations	Scenario	MAE (m³/d)	RMSE (m³/d)	MAE (m)	RMSE (m)
10 k	PD	3,734	5,150	21.1	28.7
	PC	4,162	6,620	20.5	27.8
	QD	1,800	2,956	31.5	36.9
	QC	1,467	2,411	29.7	34.9
100 k	PD	3,635	5,081	20.9	28.6
	PC	3,618	6,066	20.4	27.8
	QD	645	922	32.3	38.5
	QC	1,392	2,447	29.7	34.8
1,000 k	PD	3,155	4,852	21.0	28.6
	PC	3,766	5,766	20.7	27.9
	QD	1,406	2,343	32.0	37.1
	QC	1,290	2,189	30.6	35.7
	UC	9,762	18,182	23.0	30.5

neighbourhoods (downstream of the valves) but no such readings were available for the case study. Since flow rates in valve chambers are considered more precise than pressure measurements, and even if the pressure error criteria in the QC and QD scenarios deteriorate compared to the uncalibrated model, these scenarios were considered more reliable than the PD and PC scenarios.

Of all the QC and QD scenarios, the MAE and RMSE criteria are minimized with the QD-100 k scenario (Table 2). For an average flow from the WTP equal to 151,941 m^3/d, a MAE of 645 m^3/d (0.4%) and a RMSE of 922 m^3/d (0.6%) are very good. Pipe diameters found by the optimization algorithm for scenario QD-100 k are on average 25% smaller than in the uncalibrated model. Such a diameter reduction represents a cross-section area reduction of 44%, which has been observed in the field for pipes from that period.

It may seem surprising that scenarios with 100,000 iterations (QD-100 k) are showing better results than those with 1,000,000 iterations (QD-1,000k). However, the algorithm randomly generates an initial scenario, better than the uncalibrated scenario, on which the optimization process is applied. The initial scenario for QD-100 k was probably much better than the initial scenario for QD-1,000 k, which explains the better results with less iterations. Figures 3 and 4 allow the visual assessment of the improvements resulting from the optimization for scenario QD-100 k. A much better fit is observed at the five sites that were off in the uncalibrated model: Venturi's 750, 1,015 and 1,070 mm water mains and at P-107 and P-113. Pressures do not fit well for the reasons mentioned above.

Estimation of mean residence times

To evaluate MRTs, the tracer must reach, at the sampling point, at least 50% of the maximum concentration measured at the exit of the WTP. In addition, mixing water from the Plains' reservoir or the neighbouring DS has the effect of diluting the fluoride concentrations of water from the WTP. It has therefore only been possible to assess the residence times using the fluoride concentrations of samples collected in the field at points strictly and directly supplied by the WTP. The observed MRT isogram created by the natural neighbour tool in ArcMap10.0 is presented in Figure 7 of the 'companion paper' (Part I) in this issue (Delisle *et al.* 2015).

The simulation of residence times is strongly influenced by the 'Times Options' selected by the user in EPANET. The 'Total Duration and Quality Time Step' parameters must therefore be chosen carefully, especially when it comes to evaluating MRTs using the AGE tool or assessing MRTs from fluoride injection simulations. Tests were performed on the QD-100 k scenario to determine the best combination of time parameters to obtain the smallest simulation errors for MRTs. A total duration of 100 hours and a quality time step of 25 minutes gave the best results (MAE of 2.9 hours and RMSE of 4.3 hours) using the AGE tool. For the fluoride injection simulation, a total duration of 38 hours and a quality time step of 11 minutes yielded the best results with a MAE of 2.8 hours and RMSE of 4.2 hours. Even though the time parameters are very different, the AGE tool and fluoride injection simulation gave results whose quality adjustment criteria are practically the same. DiGiano *et al.* (2005) also observed that the MRT estimation made from simulated tracer concentrations correctly approximates the MRT estimated by the AGE tool in EPANET.

A final test was performed by attributing a consumption pattern to residential nodes. Since, no local consumption pattern is available, one was created with an evening peak with a factor of 1.4, which somewhat improves the quality criteria for MRT (MAE of 2.7 hours and RMSE of 4.1 hours).

The simulated MRT isogram is shown in Figure 5. Compared with observed MRTs (Figure 7 of the 'companion paper'; Delisle *et al.* 2015), the QD-100 k model correctly simulates low and high MRT areas. However, the model does not reproduce MRTs well for some isolated points (sites 19, 42, 69 and 79), where observed MRTs are high. Sites 19, 42 and 79 are located in residential areas supplied by local pipes with low flow rates, thus MRTs are expected to be high. As for site 69, it is located in the middle of the industrial area where water consumption may vary significantly depending on the day. The model, as constructed, may have difficulty considering these factors, resulting in an underestimation of simulated MRTs.

Evaluation of hydraulic strategies

The strategy of closing valves on large diameter pipes was tested in sector S1, where simulated MRTs are the highest (21.3 hours on average), while MRTs in most other sectors are generally under 15 hours. The HD-DDS algorithm was used to find a combination of opening and closing of valves to reduce MRTs with the QD-100 k model. At first, the large-diameter pipes on which the valves could be closed were identified: only the ones with an effective diameter (diameter decreased with calibration) greater than 275 mm were selected (73 pipes). The algorithm seeks to minimize the following objective function:

$$f(x) = \frac{\sum_{d=1}^{D} \mathrm{MRT}_d}{D} \quad \forall d = 1, 2, \ldots, D \tag{5}$$

where x is the suggested solution, MRT_d the simulated mean residence time, d a model node where we wish to reduce the MRT, and D the total number of nodes where we wish to reduce MRTs. In an iterative process, the algorithm seeks to generate a scenario corresponding to a combination of opening and closing valves that minimizes MRT for the nodes of interest. The algorithm converged to a MRT of 15.8 hours in the S1 sector, a reduction of 5.5 hours compared to the reference model (21.3 hours), by closing the seven pipes shown in Figure 6.

The variation of MRTs was calculated for each of the 268 nodes of this sector and reduction intervals were determined. Table 3 shows the number of nodes corresponding to the different intervals. This classification allows us to assess the large number of nodes that undergo an important MRT

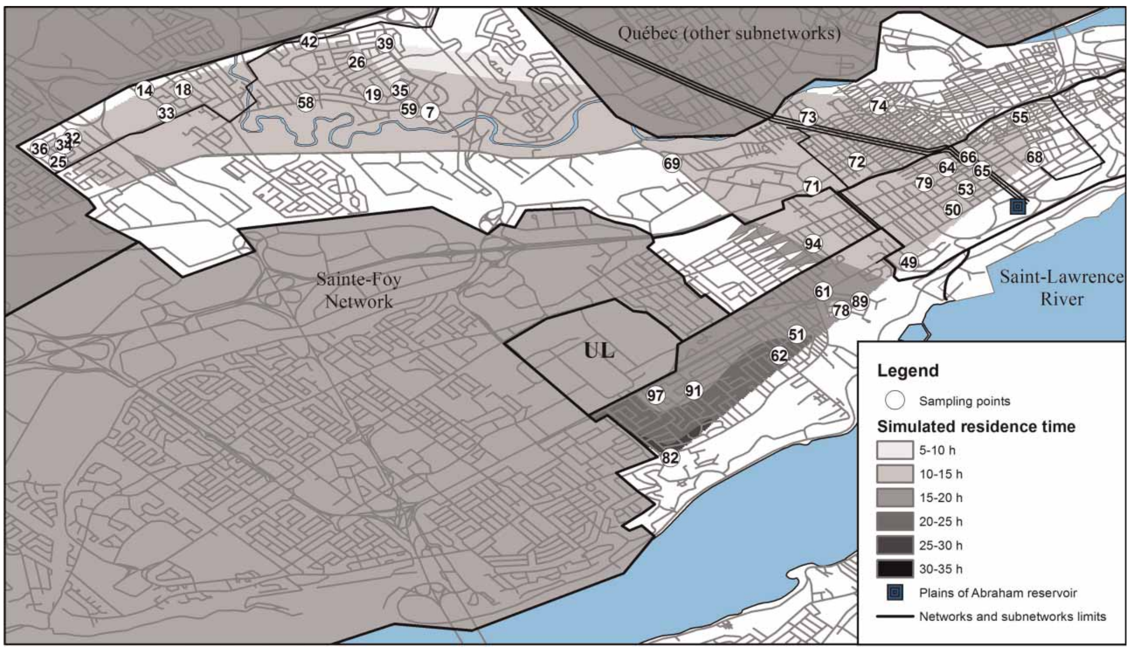

Figure 5 | Simulated MRTs.

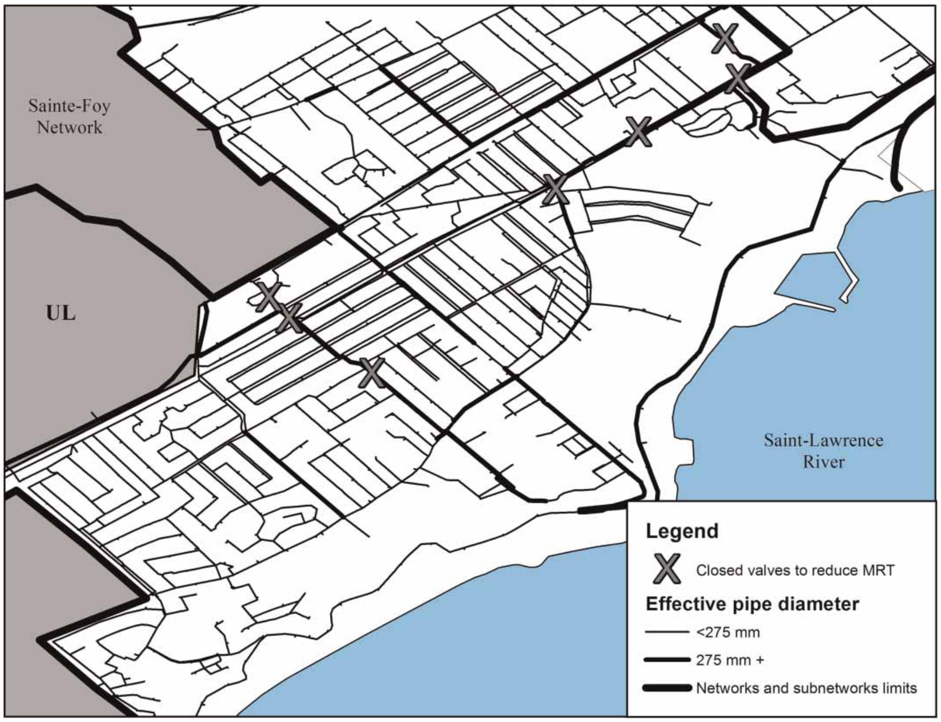

Figure 6 | Location of targeted valve closings.

Table 3 | Variation of simulated MRTs for the targeted large-diameter valve closings in sub-sector S1 using the QD-100 k model

Intervals of MRT variation (h)	Number of hydraulic nodes
[−20.0; −15.0]	2
[−15.0; −12.5]	0
[−12.5; −10.0]	9
[−10.0; −7.5]	44
[−7.5; −5.0]	101
[−5.0; −2.5]	77
[−2.5; 0.0]	31
[0.0; 2.5]	1
[2.5; 5.0]	2

Table 4 | Variation of simulated pressures for the targeted large-diameter valve closings in sub-sector S1 for a non-steady state scenario using the QD-100 k model

Pressure	Intervals of pressure variation (m)	Number of hydraulic nodes
Minimum	[−12; −11]	22
	[−11; −10]	199
	[−10; −9]	28
	[−9; −8]	18
	[−8; −7]	1
Average	[−7; −6]	9
	[−6; −5]	223
	[−5; −4]	35
	[−4; −3]	1
Maximum	[−3; −2]	0
	[−2; −1]	50
	[−1; 0]	218

reduction. The mean reduction of 5.5 hours corresponds to a mean reduction of 25.6%. Only three nodes saw their MRTs increase.

Changes in pressures, under this strategy, were verified at each hydraulic time step of the simulation. Minimum, average and maximum pressures were verified at each node as the analysis is done in non-steady state. Table 4 shows the number of nodes corresponding to the different pressure intervals. As expected, there were only pressure reductions due to increased head losses from flow being diverted to smaller diameter pipes. Minimum pressures decreased on average by 10.2 m, average pressures and maximum pressures decreased by 5.4 m and by 0.9 m, respectively. With the minimum operating pressure being about 30 m in the calibrated QD-100 k model, these variations are significant enough to raise a concern. Minimum pressures are observed during periods of high consumption, and thus, closed pipes could be opened to reduce head losses (and thus increase pressure) in areas most affected by the strategy.

However, this operation is not necessarily easy to do on an older system, given the physical condition of the valves (e.g., rust). The addition of new valves, ideally remotely controlled, at targeted locations, to open/close when and where desired, could be planned along with rehabilitation/renewal works. These new valves would allow water to be diverted from water mains to smaller-diameter local pipes in low consumption periods (night, weekend, winter season, etc.), to increase velocities and thus reduce MRTs. During periods of high consumption (day, week, summer season, etc.), water flow in water mains would be encouraged to avoid excessive head losses. A more holistic economic approach, including a life-cycle analysis taking into account all hydraulic and water quality aspects, would be needed to determine the cost–benefit ratio of introducing automatic valves in a real DS.

CONCLUSION

Different sampling campaigns were carried out on the studied drinking water DS in order to determine the residence times and validate the origin of the water at different points in the studied system. The results and the protocols of these sampling campaigns are presented in the 'companion paper' (Part I) in this issue (Delisle *et al.* 2015). The hydraulic model was then calibrated using flow rate and pressure measurements made at various valve chambers. This task was made possible by using the HD-DDS optimization algorithm.

Different calibration scenarios were tested and the one with the best results is a 25% reduction in pipe diameters so that the simulated flow rates are consistent with those observed at the various valve chambers. This optimization required 100,000 iterations to obtain an MAE of 645 m^3/d and a RMSE of 922 m^3/d, for a total flow rate leaving the plant of 151,941 m^3/d. The adjustment of pressures is not as conclusive since pressure measurements are made upstream of pressure reducing valves. Adding both flowmeters and pressure gauges on local pipes would be very useful to improve water quality management with regards to MRTs.

Fluoride concentrations obtained from the tracer campaign were used to assess MRTs and identify areas of high and low MRTs. The hydraulic model was used to simulate MRTs by using EPANET'S AGE tool and by simulating the tracer injection. This latter technique gave better results, with a MAE of 2.8 hours and a RMSE of 4.2 hours, for the steady-state simulation. Adding a local water consumption pattern in a non-steady-state simulation did not have much impact (MAE of 2.7 hours and a RMSE of 4.1 hours).

Finally, a strategy to reduce MRTs was evaluated using the QD-100 k model in the S1 sector where MRTs are the highest. The HD-DDS algorithm was used to check the effect on MRTs of closing valves on large-diameter pipes. This strategy resulted in an average reduction of 5.5 hours (25.6%) for all points in the sector. Such a MRT reduction is significant in terms of chlorine decay, simultaneously reducing the formation of DBPs. Less chlorine would need to be injected for the same protection and this would be even more beneficial for a system that had chlorine booster stations.

However, this significant decrease in MRTs generated a considerable decrease in minimum pressures (10.2 m on average), compared to a minimum operating pressure of about 30 m. The methodology developed during this research showed the potential for real-time hydraulic control strategies to reduce residence times in distribution systems, but also the challenge of managing pressures in all places at all times. It is clear that the main objective should be to have adequate pressures but that water quality, as it changes in distribution systems, should also be a concern for water managers. Tools, as developed in this research, are needed for the integrated management of drinking water.

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