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OPTIMISATION DU DIMENSIONNEMENT, DE LA LOCALISATION ET DE LA GESTION DES RÉSERVOIRS D'EAU POTABLE SUR LES RÉSEAUX DE DISTRIBUTION

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MÉMOIRE PRÉSENTÉ EN VUE DE L'OBTENTION

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Ce mémoire intitulé :

OPTIMISATION DU DIMENSIONNEMENT, DE LA LOCALISATION

ET DE LA GESTION DES RÉSERVOIRS D'EAU POTABLE

SUR LES RÉSEAUX DE DISTRIBUTION

Présenté par : BASILE Nicolas

en vue de l'obtention du diplôme de: Maîtrise ès sciences appliquées

a été dûment accepté par le jury d'examen constitué de:

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RÉSUMÉ

Les réservoirs d'eau potable sont une composante essentielle des systèmes de distribution d'eau. La conception des réservoirs des réseaux de distribution d'eau potable est complexe du fait qu'elle implique l'intervention de plusieurs facteurs: la performance hydraulique, le coût des installations, la consommation d'énergie, la qualité de l'eau distribuée, les emplacements disponibles sur le réseau existant en plus de certains compromis à faire en général pour que la solution finale reste malgré tout fiable et efficace économiquement et du point de vue de la qualité de l'eau.

Actuellement, au Québec, l'approche utilisée pour la conception des réservoirs est celle basée sur les règlements dictés par les municipalités, les compagnies d'assurances et le ministère de l'Environnement qui fixent les grandes lignes à respecter en tenant compte de la fonctionnalité des réservoirs. La tendance générale dans cette approche est de recourir à des réservoirs de grandes dimensions supposés hydrauliquement sécuritaires pour les réseaux mais qui peuvent se révéler problématiques en ce qui concerne la qualité de l'eau distribuée (long temps de résidence de l'eau dans le réservoir).

Le projet de recherche a pour but de définir une nouvelle méthodologie de conception se basant sur un modèle d'optimisation des dimensions, des opérations d'approvisionnement et de localisation des réservoirs en tenant compte de l'aspect hydraulique et de la qualité de l'eau distribuée.

L'application de cette méthodologie a abouti au développement d'un algorithme de calcul. Cet algorithme, qui a été développé pour fonctionner en parallèle avec le logiciel

de modélisation hydraulique EPANET, permet de générer une série de possibilités pour l'installation d'un réservoir enterré ou surélevé (type château d'eau). L'analyse et le tri des solutions selon un certain nombre de critères importants pour les gestionnaires de réseaux (retournement du débit dans les conduites, qualité de l'eau distribuée, pressions suffisantes, etc.) permet de déterminer la solution la plus « optimale » des solutions déterminées.

Cette méthodologie a été appliquée sur deux études de cas réels pour valider l'algorithme développé. Le projet visait à étudier la fiabilité des réseaux de distribution dont les réservoirs ont été optimisés : des gains en termes d'économie de coût de construction ont été obtenus tout en améliorant la qualité de l'eau distribuée.

ABSTRACT

Water tanks are essential components of water distribution systems. The design of tanks for water distribution systems is more complex than it looks due to the influence of several parameters: the hydraulic performance, the costs of the installations, the energy consumption, the water quality and the potential locations on the existing network. The result must take into account all those parameters to produce an efficient and reliable solution.

Nowadays in Quebec, the design approach of water tanks is based on policies from cities, insurance companies and the environment ministry which give the main rules to fulfill. The general tendency in this approach tends to implement large water tanks which are supposed hydraulically safe for water distribution systems. Those large tanks can however be a source of trouble regarding the quality of the water provided to the citizens (long hydraulic residence time in the tank).

The research project aims to define a new design methodology based on an optimization model of the dimensions, the supply operations and the localization of the tanks. This methodology takes into account hydraulic and water quality aspects.

The application of the optimization model leads to the realization of a computing algorithm. This algorithm is developed to run in parallel with the EPANET hydraulic modeling software and allows generating a number of solutions for the set up of ground or elevated tanks. The analysis and sorting of the solutions following important criteria for water distribution systems managers (inverse-flow in pipes, quality of the water supply, pressure constraints, etc.) gives an “optimal” solution from the batch of solutions.

This methodology is applied to two real case study to validate the developed algorithm. This project studies the reliability of the water distribution systems where tanks have be optimized: improvement in terms of construction costs have obtained whereas the water quality is improved.

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AHP: analytic hierarchy process

ACO: ant colony optimization

ANN: artificial neural network

EPS: Extended period simulation

DBP: disinfectant by-products

MDDEP: Ministère du Développement Durable, de l'Environnement et des Parcs

WQ : water quality

WDS : water distribution system

GA : genetic algorithm

SA: simulated annealing

TS: tabu search

HS: harmony search

MLD: 1000 habitants

WSM: weighted-sum method

WTP: water treatment plant

USGPM: US gallon per minute

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INTRODUCTION

Cadre théorique

Depuis des millénaires, l'Homme a créé et amélioré des systèmes d'adduction d'eau pour desservir la population dans les zones les plus densément peuplées. C'est un peu avant l'époque romaine que sont apparus les premiers aqueducs utilisant principalement la simple force gravitationnelle: il suffisait d'incliner légèrement les conduites pour conduire l'eau à sa destination. La simplicité de ce dispositif avait quelques inconvénients : pour traverser une colline, il fallait soit la contourner, soit creuser un tunnel. Inversement, pour passer une vallée, il fallait construire un pont ou utiliser un siphon. L'utilisation de ce système permettait d'acheminer l'eau d'un point A, un fleuve ou une rivière, à un point B, un réservoir d'eau par exemple. L'adduction d'eau aux populations a donc longtemps été l'objet de considération hydraulique pour faire face aux contraintes physiques du terrain.

C'est seulement depuis le 19ème siècle que la qualité de l'eau a été étudiée, du à de nombreuses épidémies en Europe. Depuis le début du 20ème siècle, de nombreux progrès ont été accomplis dans les deux domaines : hydraulique et traitement de l'eau. L'alimentation en eau des populations fait maintenant appel à un certain nombre de systèmes et composantes complexes - usines de purification d'eau, réseaux de distribution complexes, stations de surpressions, réservoirs d'eau potable – pour satisfaire les besoins en eau des citoyens et des industries.

De nouveaux défis se posent maintenant aux concepteurs et gestionnaires des réseaux d'eau potable dans nos sociétés industrialisés : la construction et l'entretien des réseaux de distribution d'eau coûtent cher. Pour preuve, les coûts liés uniquement à l'approvisionnement (englobant le traitement, l'entreposage, le pompage et le transport) en eau dépassent le milliard de dollars par année depuis 1991 au Canada (InfraGuide, 2005). Les autres coûts proviennent de la maintenance de l'infrastructure et de l'administration. Les réseaux d'eau sont énergivores. Ils consomment dans certains pays industrialisés jusqu'à 20 % de l'électricité utilisée par les collectivités publiques (InfraGuide, 2005).

Dans le même temps, il est nécessaire d'adapter ces infrastructures aux besoins en alimentation actuels et à venir. Pour ce faire, l'installation de nouveaux équipements «lourds» comme des réservoirs d'eau potables sont nécessaires. Les réservoirs, y compris ceux situés aux stations de purification d'eau, ont de multiples usages :

- ils permettent d'assurer un fonctionnement uniforme du processus de traitement, indépendamment de la demande en eau du réseau : en période de pointe, les réservoirs assurent l'approvisionnement en eau que le processus de traitement ne peut délivrer tandis qu'en période de faible demande, ils emmagasinent l'eau traitée par les stations de purification ;

- sur les réseaux, ils limitent les variations de pression et de débit par la multiplication des sources d'approvisionnement ;

- ils fournissent les volumes d'eau nécessaires en cas d'incendie ;

- ils assurent l'approvisionnement en eau dans un premier temps pendant les périodes d'entretiens des processus de traitement et en cas de défaillance du système (bris de conduites, panne d'électricité) ;

- ils sont, dans le cas des réservoirs à l'usine, une partie intégrante du processus de désinfection de l'eau avant sa mise en distribution.

Face à l'accroissement des coûts marginaux liés à l'approvisionnement en eau potable et des coûts engendrés par la réhabilitation des installations ayant atteint leur durée de vie, il importe donc de repenser la manière traditionnelle de conception conservatrice de l'ensemble des composantes des réseaux de distribution.

Actuellement, au Québec, l'approche utilisée pour la conception des réservoirs d'eau potable est celle basée sur les règlements dictés par les compagnies d'assurances et le ministère de l'Environnement qui fixent les grandes lignes à respecter en tenant compte de la fonctionnalité des réservoirs (MDDEP, 2002). La tendance générale de cette approche est de recourir à des réservoirs de grandes dimensions supposés hydrauliquement sécuritaires pour les réseaux. Mais ceux-ci peuvent se révéler problématiques en ce qui concerne la qualité de l'eau distribuée (long temps de résidence de l'eau dans le réservoir). Toutefois, il n'est pas rare que l'introduction de réservoirs d'eau potable s'accompagne de problèmes liés à la qualité de l'eau. La chaire en eau potable de l'École Polytechnique a d'ailleurs travaillé et étudié à plusieurs reprises les problèmes de qualité d'eau dans les réservoirs existants (Chevalier, 2001; Gauthier, Besner, Barbeau, Millette, & Prevost, 2000).

Une optimisation de ces ouvrages devrait permettre d'améliorer la qualité de l'eau et de réaliser des économies substantielles au niveau du coût global de conception et d'exploitation. Grâce aux outils informatiques performants disponibles actuellement, il est actuellement possible de développer des modèles numériques fiables de dimensionnement et de gestion de l'eau. Les réservoirs d'eau potable sont conçus de nos jours pour répondre à différents critères (qualité de l'eau, performance hydraulique, coût de construction et d'exploitation) mais leur localisation et le choix du type de réservoir dépendent le plus souvent d'un compromis entre la disponibilité des sites et l'esthétique.

Problématique

Du point de vue qualitatif, les réservoirs doivent être conçus de manière à éviter les zones mortes et les écoulements préférentiels et à assurer un mélange continu de l'eau. Les bassins de contact à proximité des usines de traitement devront assurer un temps de contact et le maintien d'un résiduel d'oxydant suffisant pour rencontrer les objectifs de désinfection fixés par le concepteur. Une longue période de stockage d'eau dans les grands réservoirs peut favoriser un mélange incomplet de l'eau, sa stratification, la perte du chlore résiduel libre, la croissance microbienne, etc.

Du point de vue quantitatif, les volumes des réservoirs doivent être suffisants pour répondre aux besoins des réserves d'équilibre (ou opération) et d'incendie. La réserve d'équilibre est estimée en relation avec la valeur du débit journalier maximal qui n'est pas une donnée statique. Une estimation révisée de cette valeur basée sur des données historiques pourrait avoir des répercussions sur les dimensions des réservoirs. Au Québec, le volume d'incendie est estimé selon les règles techniques à un débit d'incendie minimum de 2000 L/minute pour une durée d'au moins une heure, soit 120m^3 (Brière, 2007). Un recours aux sources d'approvisionnement alternatives pourrait aider à réduire le débit d'incendie requis, et par conséquent les dimensions des réservoirs.

Réduire les volumes des réservoirs ne suffit pas, encore faut-il localiser ces réservoirs aux meilleurs endroits. Pour éviter de surestimer les diamètres des conduites à installer et réduire les pertes de charge, il est recommandé de localiser les réservoirs à proximité des secteurs à desservir où la consommation est importante. Le choix du lieu reste toutefois lié à des compromis à faire en rapport avec des facteurs comme la disponibilité des sites, la question de l'esthétique ou encore la fiabilité du réseau à implanter ou réhabiliter.

Du point de vue de la performance hydraulique, il est permis de penser que plus larges sont les réservoirs, plus stables seront les pressions dans le réseau malgré les fluctuations dues aux régimes transitoires (les réservoirs se comportant comme des absorbeurs des coups de bélier). Des économies substantielles pourraient être réalisées avec des réservoirs aux dimensions optimales ayant fait l'objet d'une étude préalable d'optimisation.

C'est la nouvelle approche que nous allons étudier dans ce projet de recherche en répondant à la problématique suivante : est-il possible d'optimiser le dimensionnement, la localisation et l'exploitation des réservoirs d'eau potable au sein des réseaux de distribution tout en assurant une bonne qualité de l'eau, une meilleure performance hydraulique et la fiabilité du réseau?

Hypothèses

Afin d'évaluer la pertinence du travail effectué, nous veillerons à répondre à l'hypothèse suivante : l'utilisation d'une méthode d'optimisation permet de réduire la dimension des réservoirs et de déterminer les moments propices de l'approvisionnement en eau et de choisir l'endroit idéal pour l'implantation des réservoirs. Ce projet est original en ce sens que la dimension des réservoirs, leur localisation n'ont jamais été des variables indépendantes lors de l'optimisation de la gestion des réseaux de distribution d'eau. L'hypothèse sera réfutée si les résultats de l'optimisation des réservoirs ne réduisent pas les coûts de construction et d'exploitation des réseaux d'eau potable.

Objectifs de l'étude

L'objectif principal de ce projet de recherche consiste à établir un modèle d'optimisation des dimensions, des opérations d'approvisionnement et de localisation des réservoirs et d'étudier la performance et la fiabilité des réseaux de distributions en terme de coûts et de qualité de l'eau dont les réservoirs ont été optimisés.

Les objectifs spécifiques de ce programme de recherche sont de (i) développer théoriquement et numériquement un modèle d'optimisation des dimensions, des opérations d'approvisionnement et de localisation du réservoir, (ii) de valider ce modèle par l'application de deux études de cas réels et (iii) d'évaluer la performance hydraulique et la fiabilité du réseau réduit.

Revue critique de la littérature

La première étape de ce travail de recherche consiste à établir la revue critique de la littérature, sous forme d'article (article 1) pour faire l'état de la problématique de l'optimisation des réseaux d'eau potable. Cette question n'est pas un sujet nouveau : de nombreuses recherches ont été menées pour optimiser la configuration des réseaux d'eau potable et en particulier la taille des différentes conduites. En effet, le coût d'installation est proportionnel à la taille des conduites, d'où l'intérêt de déterminer leurs tailles optimales.

CHAPITRE 1 : ARTICLE 1: OPTIMIZATION OF WATER DISTRIBUTION SYSTEMS DESIGN AND OPERATIONS: A CRITICAL REVIEW OF LITERATURE WITH RESPECT TO TANK LOCATION AND STORAGE CAPACITY

The following is a critical review of the literature regarding water distribution system design and operation optimization with respect to tanks. The lack of tank management in existing networks leads to quality problems. Until now, the implementation and choice of tank capacity is carried out following a traditional engineering approach, but at the same time, the optimization of water distribution systems has followed two paths: design optimization (pipe diameter) and operation optimization (pump schedule) sometimes taking into account tank size and location. This document investigates the different models applied to optimizing the water network: the "traditional" ones (linear programming, nonlinear programming, dynamic programming, etc.), the evolutionary algorithms (genetics algorithms, simulated annealing, etc.), and the multi-objective optimization. Advantages and disadvantages of each model are presented and the need for new studies regarding tank location, size, and water quality management in tanks is emphasized.

1.1 Introduction

The impact of storage tanks on drinking water quality is a relatively recent concern and has gained interest, in part, due to the availability of modern hydraulic software enabling efficient water storage tank management. Improper storage tank management may lead to microbial regrowth and water quality (WQ) degradation (Martel et al., 2002). Excluding site specific physical problems linked to the configuration of storage tanks (mixing problems, sediment build-up, intrusions, cross-connections, temperature stratification, etc.), potential water quality degradations may be categorized as either chemical or microbiological (Mays, 1999).

From a water utility perspective, the loss of disinfectant residual is the most common chemical water quality issue. This situation can lead to microbiological concerns: bacterial regrowth can occur (Evison & Sunna, 2001; Gauthier et al., 2000), sometimes leading to the occurrence of total coliforms, which are regulated under most North American legislations. It has been proven that poor mixing and underutilization of the facilities, which leads to long residence time, are the main reasons for the observed loss of disinfectant residual and the deterioration of water quality inside reservoirs (Grayman et al., 2004; Mahmood, Pimblett, Grace, & Grayman, 2005). Long residence time can also lead to excessive formation of chlorinated disinfection by-products. Before modern hydraulic softwares were available, distribution system managers and hydraulic engineers had few tools with which to determine optimal residence time in storage tanks in order to prevent WQ degradation in existing or planned networks.

Traditionally, storage facilities have been designed and operated to meet hydraulic requirements: providing emergency storage, equalizing pressure, and balancing daily water demand. However, their impacts on water quality should be considered in designing, operating, and maintaining distribution system tanks and reservoirs. Over the years, there has been little changes in the methods used for determining the size and location of these facilities so as to provide equalization and emergency storage: water tanks are often located (i) according to hydraulic constraints, (ii) next to the area to be served, and (iii) according to social impacts (aesthetics, available land, etc.).

Optimization of water distribution systems (WDS) has advanced in two main areas:

- Optimizing the design (diameters of pipes, location of chlorine booster, etc.)
- Optimizing the operations (pump schedules, chlorine booster schedules, etc.)

Design and operational conditions are interdependent; yet, it is possible to solve each problem by fixing the dependent parameters (fixing design parameters to optimize operations, for example). The development of new optimization techniques and the increase in computational speed now allow more complex problems to be solved in a reasonable amount of time. This paper presents a critique of the literature regarding the various optimization techniques proposed to simultaneously optimize (i) storage facility operational conditions (including water quality), (ii) capacity, and (iii) physical location.

1.2 Current practices in storage tank design for networks

Water distribution storage is provided to ensure the reliability of supply, maintain pressure, equalize pumping and treatment rates, reduce the size of transmission mains, and improve operational flexibility and efficiency (Mays, 1999).

Tank dimensions are driven by standards (each state or province in each country has its own standards for tank size) which take into account the storage required for each of the recognized purposes: equalization, fire protection and emergencies other than fires. The requirements of these national/local rules and guidelines are conservative and it has been proven that the storage capacity and, consequently, the economy of the system can be reduced (Piqueiro, Tentugal-Valente, & Tomas, 1991). At the same time, traditional design practice suggests locating reservoirs close to the area of highest demand (Mays, 1999). Compared to pipe construction / rehabilitation / replacement costs and pump operational costs, storage tanks represent a small fraction of the entire cost of a WDS. Yet, the influence of a poorly located storage tank may significantly increase pipe design costs, operational and design pumping costs, and more importantly, reduce the quality of the supplied water.

The development of numerical methods, such as the Newton method, has paved the way for the creation of powerful hydraulic solvers, such as EPANET (Rossman, 2000), which are able to simulate the behaviour of large sized, looped networks over extended period simulation with the integration of pumps and tanks. Commercial solvers now available allow designers to evaluate and test their networks before implementation in the field. Simulation should allow the designer to go from the simple application of design rules to the development of optimization methods.

1.3 Optimization of water distribution systems

The use of optimization methods for WDS has been largely discussed in the literature of the last 25 years. As previously mentioned, optimizing WDS involves the resolution of two issues: design and operations. The optimization of the location, size, and operation of storage tanks is a universal problem related to both of the aforementioned WDS optimization issues.

1.3.1 Review of traditional optimization approaches

WDS optimization would determine the minimum objective function, the cost function in general, which is subjected to constraints: the mass conservation equation, the energy conservation law, pressure boundaries at each node of the network, etc.

1.3.1.1 WDS design optimization

The main output variable of interest for design optimization is the choice of pipe diameter. The diameter must be chosen from the available commercial diameters, which is a discontinuous function. Approaches using explicit or implicit enumeration (such as dynamic programming) have been attempted but their application for large-scale problems is limited, due to the exponential growth of computational time (Gessler, 1982, 1985; Loubser & Gessler, 1990). Several proposed models have been linearized to facilitate the use of linear programming (Alperovits & Shamir, 1977; Jacoby, 1968; Rasmusen, 1976; Shamir & Howard, 1968; Watanatada, 1973). Nonlinear programming was also used to determine an optimal design for WDS.

The Alperovits & Shamir and the Lansey & Mays approaches will be presented in the following sections.

The Alperovits & Shamir Model: The most popular linear model is the linear programming gradient proposed by Alperovits & Shamir (1977), which relies on a linear programming gradient technique. This is an iterative method where a fixed set of demands is tested iteratively and every pipe is divided into segments, each with a different diameter.

For a given network, each end of a pipe is represented by two nodes, *i* and *j*. The length of the pipe connecting *i* and *j* is L_{ij} while x_{ijm} is the length of the pipe segment of the *m*th diameter in the link connecting nodes *i* and *j* so that:

$$\sum_{m} x_{ijm} = L_{ij}$$

Équation 1.1 - Alperovits & Shamir equation #1

The decision variables are the lengths of each segment: instead of selecting pipe diameters, the procedure allows a set of “possible diameters” in each link. The use of this procedure reduces the problem to a linear equation. The cost function, subject to constraints (minimal and maximal head at each node, considering head loss in pipes) and non-negativity requirements (Alperovits & Shamir, 1977) is given by Eq. 2:

$$\text{Min} \sum_{i,j} \sum_{m} c_{ijm} x_{ijm}$$

Équation 1.2 - Alperovits & Shamir equation #2

where c_{ijm} is the linear cost for a given diameter

Several authors followed the same line of thought with the introduction of various improvements:

-correcting the procedure for gradient computing so as to improve the performance of the linear programming gradient method (Quindry, Brill, Liebman, & Robinson, 1979)

-Quindry (1981) has added the gradient step to another formulation

-the linear programming gradient to the layout optimization was extended (Goulter & Morgan, 1985)

-Goulter & Morgan (1986) have studied the interdependence between the final flow and the pressure-defining constraints and showed the "branched" nature of the optimized networks

-a modified linear programming gradient was proposed by Fujiwara (1987). He modified the search direction and the step size. The quasi-Newton search direction is proposed instead of the steepest descent direction, with the step size determined by backtracking the line search method instead of using a predetermined step size. A numerical example showed an improvement of the results in comparison to the original method.

-Fujiwara (1990; 1991) proposed a two-phase decomposition method for the optimal design of new looped water distribution networks and the expansion of existing ones. The first phase uses the gradient approach to produce a local optimal solution. The second phase uses this solution to determine link flows and pumping heads. These are then used to restart the first phase and obtain an improved local optimal solution. The whole procedure is repeated until no further improvement can be achieved.

-a matrix reformulation of the linear gradient programming using well known graph theory matrices was proposed (Kessler & Shamir, 1989).

-Sonak (1993) proved that none of the current methods provides a global optimum solution for looped WDS.

The main disadvantage of these methods relates to the simplification and the resolution method. It is not possible to apply this technique to a large-scale network. In this approach, the storage tanks are not considered as decision variables: they are at a fixed location, the head is fixed and the water level in the storage tank is determined solely by the total cost of the infrastructure.

The Lansey & Mays Model: At the end of the 1980's, Lansey and Mays (1990) developed a new methodology for determining the optimal WDS design. The mass and energy conservation equations are highly nonlinear, which would make considering multiple demand patterns desirable, since the number of nonlinear constraints can become extremely large. To overcome this, the formula is simplified: the set of equations is divided into subsets corresponding to each loading condition. Each subset is then solved by the use of a hydraulic model. Using this approach, the number of constraints is reduced significantly. The pressure heads, H, are defined as the state variables and are expressed as functions of the design parameters, D, which are the control variables. The optimization formula is:

$$\text{Min Cost} \qquad f[H(D), D]$$

Équation 1.3 - Lansey & Mays objective function

Subject to:

Head boundaries $$\underline{H} \leq H(D) \leq \overline{H}$$

Équation 1.4 - Lansey & Mays head boundaries

Design constraints $$\underline{j(D)} \leq j(D) \leq \overline{j(D)}$$

Équation 1.5 - Lansey & Mays design constraints

General constraints $$\underline{w[H(D), D]} \leq w[H(D), D] \leq \overline{w[H(D), D]}$$

Équation 1.6 - Lansey & Mays general constraints

Where $\overline{H}$, $\underline{H}$, $\overline{j(D)}$, $\underline{j(D)}$, $\overline{w[H(D),D]}$, $\underline{w[H(D),D]}$ are the upper and lower boundaries for the pressure heads, design constraints, and general constraints, respectively.

The Lansey & Mays' model has several advantages:

- pumps and tanks can be sized and pressure-reducing valve allocations can be controlled
- due to the use of an augmented Lagrangian approach – a nonlinear programming optimization tool – the procedure can now deal with nonlinear equations which help to avoid over-simplification
- the problem includes a network simulator to implicitly solve the mass and energy conservation equations

This technique does not consider the discrete nature of some variables (pipe diameter for example), the system of equations being nonlinear rather than combinatorial solutions of linear equations. Because the model must be based on a continuous diameter approach, the conversion of the optimal output variable to the nearest discrete commercial diameter may not guarantee a feasible solution. In addition, a nonlinear optimization technique does not guarantee a global optimum value, which is a typical drawback compared to linear optimization techniques.

The size of storage tanks is calculated by setting the floor elevation and no upper limit on the water level in the tank. The location of tanks is also considered: the riser diameter (the riser is the connecting pipe between the network and the tank) is a decision variable which could be set to zero. Setting this diameter to zero is equivalent to eliminating the presence of a storage tank in the WDS under evaluation. Their study introduced the optimization of operations linked to the optimization of design.

1.3.1.2 Optimization of WDS operations

Models Considering the Costs and the Total Pump Operation Time: The optimization of WDS operation and design has evolved in parallel to one another. With respect to operations, the approach was the same for optimization of pump schedules as it was for WDS design. Ormsbee & Lansey (1994) discussed the numerous methodologies that have been proposed for developing optimal scheduling algorithms for water-supply pumping systems, along with the advantages and disadvantages of each approach. The difference in methodologies is mainly the control algorithms: linear programming, linear quadratic programming, nonlinear programming, dynamic programming, etc. These authors have identified three main systems:

- single and multiple pump stations with no tank
- single tank with single and multiple pump stations
- multiple tanks and multiple source systems

It is important to note that pumps and storage tanks are linked variables: if the network has no tanks, pumps will have to be switched to meet demand. Small networks are able to work using this control philosophy. The storage capacity will therefore influence pump scheduling.

Several aspects of operation optimization have been studied over the years. Lansey & Awumah (1994) developed a methodology to determine optimal pumping schedules for minimizing energy costs and included a constraint to limit the number of pump cycles. Ostfeld and Shamir (1993a; 1993b) developed a model for optimal operation of a multi-quality water-supply system to minimize the total cost of water treatment and energy by using an approximation of the equation for water quality in pipes.

Models Considering Water Quality : Some models have considered water quality in the optimization process. Boccelli et al. (1998) formulated a linear optimization model for booster chlorination stations in order to minimize the total chlorine dose required to satisfy disinfectant residual constraints.

Sakarya (1998) optimized pump operations while considering the chlorine concentration in the pipes. He formulated a new objective function, different from the previous objective function. The latter was further tested (Sakarya, Goldman, & Mays, 1998; Sakarya & Mays, 2000). The purpose of this new objective function is to minimize the deviations of the actual concentrations of a targeted chemical (generally chlorine) from the desired concentrations. For N nodes and T period of time, the objective function is

$$\text{Min} \sum_{n=1}^{N}\sum_{t=1}^{T}\min(0,\min(C_{nt}-\underline{C_{nt}},\overline{C_{nt}}-C_{nt}))^2$$

Équation 1.7 - Sakarya & Mays objective function

Where C_{nt} is the concentration of a particular compound and $\underline{C_{nt}}$ and $\overline{C_{nt}}$ are the lower and upper boundaries of the substance concentration at node *n* and time *t*. Moreover, new constraints were introduced to take into account the principle of mass conservation. The water quality constraint within each pipe *m* connecting nodes *i* and *j* in the set of all pipes *M* is

$$\frac{\partial(C_{ij})_t}{\partial t} = -\frac{(q_{ij})_t}{A_{ij}}\frac{\partial(C_{ij})_t}{\partial x_{ij}} + \theta(C_{ij})_t \qquad \forall i,j \in M \text{ and } t=1,\ldots,T$$

Équation 1.8 - Water quality constraint

with A_{ij} the cross-sectional area of pipe *m* connecting nodes *i* and *j*, x_{ij} distance along the pipe m, q_{ij} the flow rate between *i* and *j* at time *t* and rate of reaction of constituent within pipe *m* connecting nodes *i* and *j* at time *t*.

For the previous objective function, he has also introduced new water quality constraints. The new constraints are formulated by

$$\underline{C_{nt}} \leq C_{nt} \leq \overline{C_{nt}} \text{ with } n=1,\ldots,N \text{ and } t=1,\ldots,T$$

Équation 1.9 - New water quality constraints

where $\underline{C_{nt}}$ and $\overline{C_{nt}}$ are the lower and upper bounds of substance concentration at node n and time t.

Sakarya (1998; 2000) has compared the results of the different approaches: objective function that considers water quality versus objective function that considers costs with water quality constraints. The results showed the model's ability to solve both problems. Costs were significantly increased when the objective function considered water quality, but no violation of concentration constraints appeared for the objective function considering costs. This showed that water quality was seen as a constraint.

This study was an improvement considering optimization and water quality, but the main disadvantage is that the optimization is based on GRG2, a nonlinear algorithm that does not guarantee the solution to be optimal. Moreover, tank location is not considered. Considering storage tanks came with the introduction of evolutionary algorithms, which is discussed in the next section.

1.3.2 Heuristic optimization methods

The application of traditional optimization methods – linear, dynamic, or nonlinear programming – has been attempted with models considerably simplified to reduce computing time. In recent years, the use of heuristic methods (combined with increasing computing performance of computers) was a significant step forward in the search for the most suitable method for solving least-cost design and optimal WDS operations.

The heuristic models are derived from nature: the main characteristics of these algorithms are that they try to imitate the natural process involved in finding an optimal solution. Due to the stochastic nature of evolutionary algorithms, there is no guarantee that the global optimum will be found using those methods, although the number of applications in different fields suggests a desirable rate of success in identifying viable solutions. Tabu search, simulated annealing, harmony search, and evolutionary algorithms are subsets of heuristic methods. The main advantages of using heuristic methods are:

- They use continuous and discrete variables, which allow to create models closer to reality

- The mathematical dimensions of the solution domain are searched using a random number generator. Even if a general optimum cannot be guaranteed, a good feasible solution is generally found.

Tabu search and simulated annealing are basic heuristic methods compared to genetic algorithms and ant colony, which belong to the evolutionary algorithms subset. For most of heuristic methods applied to WDS, the optimization process is done in four steps:

- Step 1: setting up the problem

- Step 2: programming the optimization problem (following a heuristic method) and realizing the communication between the optimizer and the hydraulic solver i.e. EPANET (or programming a new hydraulic solver with Newton method for example).

- Step 3: running the optimizer and the hydraulic solver with the data. In fact, once launched, the optimizer generates solutions and the hydraulic solver returns the new values of different parameters (i.e. pressure, etc.).

- Step 4: retrieving the results and making the analysis

Another method has been applied to WDS optimization: the artificial neural network. Artificial neural network is much more of an artificial intelligence method than a heuristic one.

1.3.2.1 Genetic algorithms

The most popular evolutionary algorithms are probably genetic algorithms (GA). GA use techniques inspired by evolutionary biology such as inheritance, mutation, selection, and crossover. Candidate solutions are represented by strings of numbers (chromosomes) using a binary or non-binary alphabet. Most of the GA devised for pipe network optimization problems use such coding (Dandy, Simpson, & Murphy, 1996; Savic & Walters, 1997; Simpson, Dandy, & Murphy, 1994). GAs respect the following procedure:

- The initialization: a set of solutions, the population, is randomly generated. The fitness of each individual in the population is evaluated.

- The selection: a proportion of the existing population is selected to breed a new generation. Individual solutions are selected through a "fitness-based" process, where fitter solutions (measured by a fitness function) are typically more likely to be selected.

- The reproduction: the next step is to generate a second generation population of solutions from those selected through genetic operators: crossover and/or mutation. For each new solution to be produced, a pair of "parent" solutions is selected for breeding from the pool selected previously. By producing a "child" solution using the above methods of crossover and mutation, a new solution is created which typically shares many of the characteristics of its "parents". New parents are selected for each child, and the process continues until a new population of solutions of appropriate size is generated. These processes ultimately result in the next generation population of chromosomes being different from the initial generation. Generally, the average fitness of the population will have increased with this procedure, since only the best organisms from the first generation are selected for breeding, along with a small proportion of less fit solutions.

- The termination: the generation (selection + reproduction) is repeated until one or several termination conditions are achieved: the number of iteration is reached, the highest ranking solution's fitness has reached a plateau such that successive iterations no longer produce better results, etc.

A large number of GA applied to WDS optimization exists with many modifications. GAs have the advantage of allowing a search of the entire field and identifying several solutions from which the decision maker can select, rather than a single optimum. Design and operation optimization were considered separately or at the same time.

In 1994, tank location and tank storage decision variables were included in a single-objective GA model (Murphy, Dandy, & Simpson, 1994). In this model, tank locations and the fractions of additional storage needed at each tank site are considered as independent decision variables, determined by the GA. A 6h time step is used for daily operation simulation. Any mismatches are treated as penalties to the objective function. In practice, tank levels are determined and emergency volumes are added manually after the optimization process.

The use of GA is not that straightforward due to the fully parametrable generation. Once the initialization is carried out, the selection and reproduction are defined by the programmer and at the same time, the new population can represent unrealistic solutions.

1.3.2.2 Simulated annealing

Simulated annealing (SA) is another EA applied to WDS optimization. SA is based on an analogy with the physical annealing process. In this process, the temperature of a material is increased to give mobility to the molecules, followed by a slow cooling: those molecules will form a crystalline structure. The high mobility of molecules in the "hot zone" enables them to reach different physical states. If cooled properly, they will reach the minimum energy state which corresponds to an ordered crystalline configuration.

By analogy, each step of the SA algorithm replaces the current solution by a random "nearby" solution, chosen with a probability that depends on the difference between the corresponding function values and a global parameter T (the *temperature*), that is gradually decreased during the process. The interdependence is such that the current solution changes almost randomly when *T* is high, but increasingly "downhill" as *T* approaches the final temperature. The allowance for "uphill" moves saves the method from becoming stuck at local minima.

The optimization process for an SA optimization is:

- Step 1: choose an initial WDS configuration
- Step 2: choose an initial temperature
- Step 3: choose a final temperature
- Step 4: choose a new configuration in the neighbourhood of the initial one
- Step 5: choose a random number p between 0 and 1
- Step 6: save the new one if the Metropolis criterion is satisfied (Cunha & Sousa, 1999)
- Step 7: choose a new temperature inferior to the initial one
- Step 8: repeat step 4 to 7 until the temperature falls under the stopping temperature

SA was initially proposed by Metropolis (Metropolis, Rosenbluth, Rosenbluth, Teller, & Teller, 1953) and has since been applied to WDS design and operation optimization by several other authors (Costa, De Medeiros, & Pessoa, 2000; Cunha & Sousa, 1999; Goldman, 1998; McCormick & Powell, 2004; Sakarya, 1998; Sousa, Da Conceicao Cunha, & Sa Marques, 2002). Results are generally similar to those from the GA optimization (Cunha & Sousa, 1999). SA is easy to use and can quickly provide high quality solutions. However, most of the studies applying SA to WDS optimization are limited and do not take into account tank sizing and location.

1.3.2.3 Tabu search

Tabu Search (TS) was introduced for WDS optimization design by Lippai (Lippai, Heaney, & Laguna, 1999) and applied later by Cunha (Cunha & Ribeiro, 2004). Tabu search (TS) is a global optimization heuristic method based on the human memory process (Glover & Laguna, 1997). The implementation of this method relies on the exploration of the neighbourhood of the current solution. A procedure using long-term memory function will ensure the exploration of solutions not previously tested. To avoid cycling around a local optimum, some moves are forbidden and listed in the "tabu list". Tabu moves are not permanent: they stay in the tabu list for a certain number of iterations (defined by the tabu tenure parameter) and a short-term memory function removes them from the list. An aspiration criterion – similar to crossover for GA – allows certain moves on the tabu list to overcome any tabu status. This will occur whenever some moves lead to finding a solution corresponding to a predefined criterion (in general, a better solution).

Costs obtained with TS were found to be superior (Lippai et al., 1999) or similar (Cunha & Ribeiro, 2004) to those found with previous methods (GA, SA, etc.). Like SA, TS present the advantage of being easy to build and calibrate. But the implementation of TS for WDS optimization was limited to WDS pipe design without considering tanks. Further studies are necessary to validate the use of TS as a suitable method.

1.3.2.4 Harmony search

The harmony search (HS) algorithm was conceptualized based on searching for a "perfect state" of musical harmony, such as jazz improvisation (Geem, 2006). Jazz improvisation seeks a best state (fantastic harmony) determined by an aesthetic estimation, just as the optimization algorithm seeks a best state (global optimum) determined by evaluating the objective function. Aesthetic estimation is performed by the set of pitches played by each instrument, just as the objective function evaluation is performed by the set of values assigned by each decision variable. The harmony quality is enhanced practice after practice, just as the solution quality is enhanced iteration by iteration.

Geem applied HS to WDS optimization design. A harmony is a vector composed by the pipe diameters chosen from possible candidate diameters in a process similar to the one used with GAs. For each iteration, a new harmony is generated and the updated process includes the new and excludes the worst harmony from the batch of valuable solutions. For more information, readers are referred to Geem's essay (2006).

The resulting costs obtained by the HS for different WDS were either the same or 0-10 % lower than those of competitive meta-heuristic algorithms, such as the GA, SA, and TS. The advantage of using HS is the HS-based model can suggest alternative solutions. But the optimization achieved using HS was limited to benchmark case studies and did not include pump schedules, etc. Moreover, the study did not consider storage tank allocation.

1.3.2.5 The ant colony

A new development in EAs applied to WDS optimization seems to be the introduction of the ant colony optimization (ACO). ACO is a heuristic derived from nature based on the foraging behaviour of ants and was first proposed by Dorigo (Dorigo, Maniezzo, & Colorni, 1996). Over a period of time, an ant colony will be able to determine the shortest path from its nest to a food source. This perceived "swarm intelligence" is achieved via an indirect form of communication between the colony members that involves them depositing and following a decaying trail of chemical substance, called pheromone, on the paths they travel. Over time, shorter paths are reinforced with increased levels of pheromone, thus becoming the dominant paths for the colony.

ACO is based on this analogy of the incremental learning of a colony by an iterative trial and error process. In the ACO algorithm, artificial ants construct solutions to the underlying combinatorial problem by probabilistically selecting options at each decision point. The probabilistic decision policy is governed by two weighting factors: one is the pheromone intensity (to learn information) and the other one is desirability (to avoid high cost options). At each generation of a new set of solutions by the colony, information from the previous iteration is used to alter the pheromone values and hopefully increase the probability of the optimum solution being found. In order to effectively use more recent information, the pheromone values are decayed with time (mimicking the evaporation of their real life counterpart), thus placing more of an emphasis on recent information.

For WDS optimization design, it is important to identify an appropriate representation of the problem in terms of a graph G = (D, L, C) where D is a set of points at which decisions have to be made, L is the set of options available at each decision point, and C is the set of costs associated with options L. For WDS optimization design problems, the graph G = (D, L, C) must be as shown in Figure 1. One decision point is associated with each pipe. In the example in Figure 1, there are five pipes and hence five decisions points (d_1, d_2,...,d_5). At each decision point, there are a number of options corresponding to the available pipe diameters (Φj). In figure 1, there are eight possible pipe diameters (Φ_1, Φ_2,..., Φ_8) corresponding to eight choices at each decision point ($l_{i(1)}$, $l_{i(2)}$,..., $l_{i(8)}$, i =1,2,...,5). The costs corresponding to each of these choices ($c_{i(1)}$, $c_{i(1)}$,..., $c_{i(1)}$, i=1,2,...,5) are the product of the unit cost per meter of each of the pipe diameters ($UC_{\Phi j}$) and the length of the pipe segment under consideration (LE_i). The cost associated with a particular trial solution (f (φ)) is therefore given by

$$f(\varphi) = \sum_{i=1}^{n} UC_{\Phi j} \times LE_i$$

Équation 1.10 - Cost function for ACO

The formula required for the optimization of WDS is different from that used for other combination optimization problems in its constraints. The feasibility of a particular trial solution can only be assessed after it has been constructed in its entirety, and consequently, the constraints cannot be taken into account explicitly during the construction of trial solutions. To deal with this problem, most authors give negative reinforcement to pipe diameter options that result in solutions that violate the pressure constraints

$$\Delta\tau_{i(j)}^{k} = \begin{cases} \dfrac{R}{f(\varphi)^{k}} - P_{pher} \times \Delta H_{max} & \text{If option } l_{i(j)} \text{ is chosen at cycle k} \\ 0 & \end{cases}$$

Équation 1.11 - Pheromone concentration

where P_{pher} = pheromone penalty factor and ΔH_{max} = maximum pressure deficit in the WDS, which is obtained using a hydraulic solver for each trial solution (combination of pipes) generated. In order to calculate the change in pheromone concentration $\Delta\tau_{i(j)}$, f (φ) is modified as follows in order to determine which cycle results in the best solution

$$f(\varphi) = \begin{cases} \sum_{i=1}^{n} UC_{\Phi j} \times LE_{i} + PC \times \Delta H_{max} & \text{If any of the pressure constraints are violated} \\ \sum_{i=1}^{n} UC_{\Phi j} \times LE_{i} & \end{cases}$$

Équation 1.12 - Trial solution

where PC = penalty cost multiplier ($/m head violated). For more information, readers are referred to Maier's study (2003).

To exploit information about the current global-best solution, another model was developed to keep the best ants (Bullnheimer, Hartl, & Strauss, 1999). To overcome the problem of premature convergence while still allowing for exploitation, a max-min ant system was developed (Stuetzle & Hoos, 2000). The different algorithms have been recently applied to WDS optimization and have been found to perform very competitively for WDS optimization design (Afshar, 2006; Maier et al., 2003; Zecchin, Maier, Simpson, Leonard, & Nixon, 2007).

Results from the optimization with the use of ACO were similar to those obtained with previous methods (GA, etc.). However, the complexity of implanting and calibrating this method on a simple WDS (no pumps, no tanks, etc.) makes it tedious to use.

Even if ACO present results similar to other optimization techniques, the main disadvantage is the complexity of setting the optimization process. ACO requires a good knowledge of the graph theory to allow the application of the ACO to WDS design optimization. Moreover, in that optimization, operations and tanks are not considered, which limits the application of such a method.

1.3.2.6 Artificial neural network

The use of the Artificial Neural Network (ANN) is another metamodeling approach applied to WDS optimization. An ANN is an adaptive system that changes its structure based on external or internal information that flows through the network. The advantage of ANN is that it can represent complex nonlinear functions without the need to predetermine the form of the model (contrary to regression models) and it has the ability to be used as an arbitrary function approximation mechanism which 'learns' from observed data.

The implementation of ANNs is, in practice, quite different from the traditional WDS optimization methods. The procedure involves:

- Step 1: setting up the problem by establishing the objective function, the constraints for a chosen WDS to optimize

- Step 2: programming and coding the optimization problem. This step involves the selection of the best ANN architecture and the programming of the training process as well.

- Step 3: generating training data. There are two possible methods for obtaining data to train the ANN. One method is to generate data initially across the entire search space with a simulation model and then train the ANN (Johnson & Rogers, 2000), while another method is to train the ANN while running a GA (or an SA, etc.) and use the solutions the GA obtains to periodically recalibrate the ANN (Lingireddy & Ormsbee, 1998). Both methods are acceptable but the first one was used by Broad (2005) considering the efficiency of the simulation model to generate valuable data.

- Step 4: training the ANN with the data generated in Step 3.

- Step 5: running the optimizer and the ANN. The hydraulic solver is substituted for the ANN.

- Step 6: retrieving the results and making the analysis.

The initial purpose of using a meta-model such as ANN was to reduce the computer time (Broad et al., 2005). The efficiency of using an ANN was proven, despite the technique being very data intensive. However, the main disadvantage is the time and effort required to generate correct data, the time to create the ANN. In practice, the availability of affordable, reliable, and robust tools, such as EPANET, renders the use of ANN unnecessary for hydraulic solving.

Several techniques were applied to the WDS design or operations optimization. The interest of using theses different techniques is to allow a combination of variables to be considered, such as pipes diameters, pump schedules, etc. Tank locations and sizing can then be considered more precisely but until now, most cases did not.

Although interesting, the results were hard to implement. In fact, cost was minimized by reducing the capacity and robustness of the network (Walski, 2001). It has been shown that not enough consideration has been given to the reliability of the network: in fact, the "minimum cost function" of the optimizer tends to "delete" some pipes (diameters equal to zero) to reduce the total cost and lead to a tree-shaped network. Multi-objective optimization does not overcome this problem, but establishes other decision factors.

1.3.3 The multi-objective optimization of WDS operations and design

The purpose of multi-objective optimization of WDS is not necessarily to determine the best solution, but a set of feasible solutions – the set of Pareto, which is the expression for the trade-off between two decision factors: the total cost versus the reliability of the network.

To estimate the reliability of a network and to avoid the "tree-shaped network" problem, the concept of resilience has been introduced (Todini, 2000). The resilience of a WDS is literally its ability to overcome stress or failure conditions. Looped topology of WDS allows for redundancy, which assists in ensuring that there is sufficient capability in the system to overcome local failures and guarantee the distribution of water to users. The resilience index I_r of a looped network is defined as:

$$Ir = 1 - \left(\frac{P_{\text{int}}}{P_{\max,\text{int}}} \right)$$

Équation 1.13 - Resilience index

Where P_{int} is the amount of power dissipated in the network to satisfy the total demand and $P_{max,int}$ is the maximum power that would be dissipated internally in order to satisfy demand.

To evaluate the coupling between the costs and reliability, multi-criteria decision-making methodologies are applied and multi-objective evolutionary algorithms – mainly population-based evolutionary algorithms – are used. The multi-objective optimization process accomplishes the design and operation optimization at the same time. The output is a set of optimized WDS, which is a trade-off between cost and reliability (Farmani, Savic, & Walters, 2004; Farmani, Walters, & Savic, 2005). They offer a less subjective approach for finding many Pareto-optimal solutions in a single run.

Walters & al. (1999) developed a network design optimization model including location, volume, and water levels as decision variables for tanks using a structured messy GA and multi-objective optimization. They then used two different approaches for modeling tank variables. The first approach is similar to the one proposed by Murphy (Murphy et al., 1994): tank location and tank storage are design variables, the maximum and minimum operating levels are treated as dependent variables, whose values are determined by the pressure required to provide the network inflows and outflows necessary for filling and emptying the tanks. In the second method, location, volume, maximum operating level, minimum operating level, and diameter of the riser are treated as independent design variables, thereby completely defining storage design. With this approach, storage will not initially match the requirements of the network. To prevent reservoirs overfilling or emptying below safe operating levels, it is necessary to include a penalty function to drive the GA towards fully feasible and balanced solutions.

The most tank-centered multi-objective optimization was probably carried out by Vamvakeridou-Lyroudia & al. (2007). The author pointed out the gap (already outlined by Walski in 2001) between the tank simulation needed for optimization and the current traditional engineering practices. Their study proposed a new mathematical model based on two decision variables: capacity and minimum normal operation level. Assessment of tank performance is carried out by four criteria for the normal daily operational cycle, differentiating between operational and filling capacity, as well as two more criteria for emergency flows. The original design and operational mathematical assumptions are implemented in a fuzzy multi-objective GA model. This work reinforces the idea that WDS optimization must also consider storage tank capacity and location.

1.4 Future developments and conclusion

Storage tanks have a significant impact on water quality. Long residence times result in the loss of disinfectant residuals, often associated with increased microbial growth and disinfection by-product formation.

On one hand, studies considering water quality in tanks did not consider the size and location of the tank itself on the WDS. This is due to the fact that most studies of storage tanks affected by WQ issues were dealing with already existing tanks and were naturally more focused on the means to minimize their impacts.

On the other hand, the optimization of WDS design and operation have been focused on sizing pipes and on pump schedules without much attention given to storage tank location and capacity. This was originally due to the limits of the traditional methods of optimization (linear programming, nonlinear programming, dynamic programming, etc.) which had difficulty dealing with combination problems. The introduction of heuristics methods to achieve WDS optimization allows tank location and dimensions to be considered. GA, SA, TS, etc. were proven to be efficient in determining good solutions in the domain of available solutions. The use of these combinatorial techniques allows one to consider new and more realistic parameters. The recent developments in multi-objective optimization of WDS based on GA now allow design and operation optimization at the same time and find sets of solutions that consider cost and network reliability. The number of recent papers dealing with GAs may be indicative of researcher preference for GAs.

The use of multi-objective optimization for WDS better answers the concerns of managers dealing with WDS: instead of a single solution, it may be better to choose one from a set of solutions, which is the current trend in research. Another trend in research is the transfer from the existing research results to practical cases.

Today, urban water systems must meet new challenges and future research should take these needs into account. For example, most cities are managing an already existing network and most of the time, new developments lead people to reconsider the design and the operation of the WDS. This reengineering has to deal with traditional hydraulics and reliability constraints (i) but includes severe water quality constraints and a special attention should now be paid to tank sizing and location. The traditional way to locate tanks ("next to the area to be served") (ii) must now be reconsidered with water quality, while keeping in mind the social impact (iii). Future studies should be adapted to these needs.

1.5 Figures

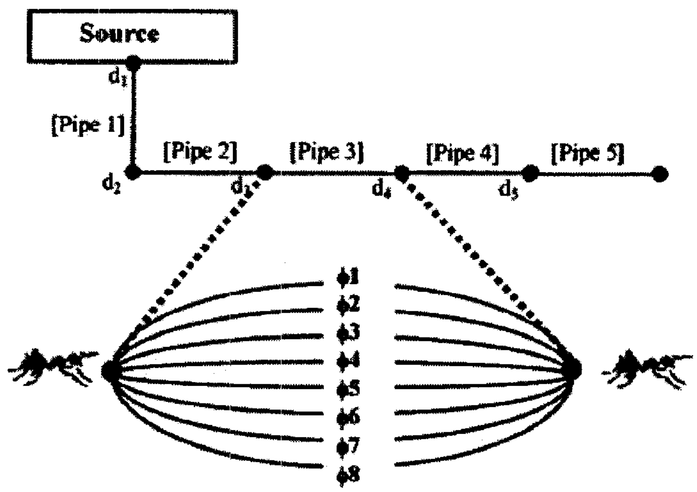

Figure 1.1 - Representation of water distribution system optimization problems in terms of graph (Maieir et al., 2003)

CHAPITRE 2 : DÉMARCHE DE L'ENSEMBLE DU TRAVAIL DE RECHERCHE ET ORGANISATION DU DOCUMENT INDIQUANT LA COHERENCE DES ARTICLES PAR RAPPORT AUX OBJECTIFS DE LA RECHERCHE

La première étape consista à réaliser la revue critique de littérature (article 1). Elle a montré que l'optimisation de réseaux d'eau potable devrait s'étendre à la conception et à la localisation des réservoirs d'eau potable tout en prenant en compte (i) la qualité de l'eau et (ii) l'aspect économique. Le but de ce projet de recherche consiste donc à développer un modèle d'optimisation des dimensions, des opérations d'approvisionnement et de localisation des réservoirs et étudier la performance et la fiabilité des réseaux de distributions dont les réservoirs ont été optimisés.

Étude des réservoirs existants au Québec

Ce constat énoncé, la seconde étape de l'étude fut la caractérisation des réservoirs existants au Québec. En effet, le projet vise à développer une méthodologie applicable au Québec, et si possible au Canada et à l'international. Durant cette étape, le but a été de récupérer des données quant au nombre et à la taille des réservoirs présents sur les réseaux.

Les données recueillies auprès des municipalités ont permis d'élaborer les quelques caractéristiques générales liant la taille de la ville et la présence de réservoirs. Les villes de petite taille (avec une population inférieure à 30 000 habitants) ont en général un seul réservoir d'eau potable : le réservoir situé à l'usine de purification d'eau. Les villes de taille moyenne (avec une population comprise en 30 000 et 100 000 habitants) ont deux réservoirs en général, dont un réservoir en usine. Quant aux grandes villes (population de plus de 100 000 habitants), l'analyse des résultats ne fût pas concluante : en effet, pour ces grandes métropoles, la topographie (collines, lacs, rivières et fleuves) et l'historique du développement urbain jouent un rôle prépondérant dans la configuration du réseau de distribution d'eau et il est impossible de donner des caractéristiques générales, tant les configurations sont différentes les unes des autres.

De cette étude, il résulte que de nombreuses villes moyennes font ou vont faire face dans les prochaines années à un accroissement de leur population et que l'extension de leur réseau de distribution d'eau potable va nécessiter l'implantation d'un ou plusieurs nouveaux réservoirs sur les réseaux existants.

En outre, de nombreux problèmes de qualité d'eau ont déjà été observés dans des réservoirs d'eau potable au Québec (Chevalier, 2001; Gauthier et al., 2000). La méthodologie utilisée dans cette étude est principalement axée sur deux aspects :

- L'optimisation du point de vue hydraulique et de la qualité de l'eau ;
- La possibilité d'appliquer cette méthodologie aussi bien pour un réservoir existant (déterminer les niveaux d'opérations) que pour la conception (dimensions) d'un futur réservoir à un cas réel.

Les objectifs sont de développer théoriquement et numériquement un modèle d'optimisation des dimensions, des opérations d'approvisionnement et de localisation du réservoir, d'évaluer la performance hydraulique et la fiabilité du réseau réduit.

L'article 2 décrit de façon détaillée la méthodologie, l'algorithme de calcul développé et qui est utilisé par le modèle d'optimisation ainsi que les méthodes d'analyse de résultats.

L'article 3 montre comment l'algorithme développé peut être appliqué. Deux cas réels sont alors utilisés. Les résultats obtenus sont discutés et analysés.

La discussion générale (chapitre 6) permet de reprendre l'ensemble de l'étude et d'éclairer certains points avant la conclusion finale et les recommandations.

CHAPITRE 3 : ARTICLE 2: OPTIMIZATION OF THE DESIGN, LOCATION AND MANAGEMENT OF WATER TANK IN WATER DISTRIBUTION SYSTEMS – METHODOLOGY

Water distribution pipes are designed to provide adequate pressure at distribution nodes and reasonable velocities in pipes. Water distribution system (WDS) design has been the subject of minimal improvements over the last decade, despite the fact that computer models accessibility has increased. The design is usually achieved using a hydraulic solver coupled with the engineering expertise of the designer. Storage tanks are designed according to standard requirements considering minimal requirements provided by local guidelines. Storage tank allocation is generally done without taking into account the network capacity and robustness. Tanks are located next to the distribution area with highest demand and considering other site-specific constraints (topology, multiple pressure-zone systems, land availability, etc.). Optimal network configuration, in terms of hydraulic efficiency and water quality, is rarely considered. Long residence time results in the loss of disinfectant residual and favors water quality degradation. Up to now, the optimization of the WDS design and operations have mostly focused on pipe sizing and pumping schedules without taking into account storage tank locations, storage capacity and water quality.

The purpose of this paper is to present a new methodology to optimize water storage tank volume and location. This methodology will take into account hydraulic requirements as well as water quality requirements (by minimizing the residence time in order to reduce disinfectant decay and disinfection by-product formation).

3.1 Introduction

3.1.1 Current practices in storage tank design for networks and water quality problems

Water distribution storage has traditionally been designed to ensure the reliability of supply, maintain pressure, equalize pumping and treatment rates, reduce the size of transmission mains and improve operational flexibility and efficiency (Mays, 1999). Tank dimensions are driven by standards (most states or provinces in the US or Canada have their own standards or guidelines for tank size) which take into account the storage required for each of the objectives described earlier: equalization, fire protection and emergencies other than fires. At the same time, traditional design practice suggests locating reservoirs close to the area of highest demand (Mays, 1999). From an hydraulic point of view, the requirements of these national/local rules and guidelines are conservative and it has been proven that the storage capacity and, consequently, the costs of the system can be reduced (Piqueiro et al., 1991).

From a water utility perspective, the loss of disinfectant residual is the most common water quality issue. This situation can lead to microbiological concerns. Bacterial regrowth can take place (Evison & Sunna, 2001; Gauthier et al., 2000), sometimes leading to the occurrence of total coliforms which are regulated under most North American legislations. It has been proven that poor mixing and underutilization of the facilities, which leads to long residence time, are the main reasons for the observed loss of disinfectant residual and the deterioration of water quality inside reservoirs

(Grayman et al., 2004; Mahmood et al., 2005). Long residence time can also lead to excessive formation of chlorinated disinfection by-products (CBP). To reflect this issue, CBP regulations in North America are verified at sampling locations on the water distribution system (WDS). The longest residence time in the WDS will usually dictate the compliance to CBP regulation, which reemphasizes the importance of adequate storage tank design and management.

Before modern hydraulic software were available, distribution system managers and hydraulic engineers had few tools at their disposal to determine optimal residence time in storage tanks in order to prevent water quality degradation in existing or planned networks. The development of numerical methods, such as the Newton method, have paved the way for the creation of powerful hydraulic and water quality solvers, such as EPANET (Rossman, 2000). These programs are able to simulate the behavior of large looped networks over several days with the integration of the different component of a typical network: pipes, valves, pumps, reservoirs and tanks. Moreover, some water quality aspects – water age, wall and bulk Cl_2 decay reactions – can now be simulated. In contrast with the simple application of design rules, simulation allows the designer to simulate the future behavior of the network and to apply optimization methods.

3.1.2 Optimization of water tank sizing and location

The use of optimization methods for WDS has been largely discussed over the last 25 years. Optimizing WDS involves the resolution of two issues: design and operations. The optimization of location, size, and operation of storage tanks is a universal problem related to both of the aforementioned WDS optimization issues and has never been studied before. The WDS design optimization traditionally deals with pipes sizes while WDS operation optimization takes into account pump schedules. Compared to pipe construction/rehabilitation/replacement costs and pump operational costs, storage tanks represent a small fraction of the entire cost of a WDS. Yet, the influence of a poorly located storage tank may significantly increase pipe design costs, operational and design pumping costs, and more importantly, reduce the quality of the supplied water.

The objective of this paper is to present a new methodology to deal with the issue of optimizing the location and size of tanks on WDS. Pipe sizes and pump schedules were deliberately not considered as variables of optimization but rather as fixed parameters. Figure 1 summarizes the principle of the proposed optimization. It starts from the optimization process. The process, coded in C for instance, and the hydraulic and water quality solver, such as EPANET, feeds information to each other: the optimization process checks the results of the solver and provides it with new inputs (figure 3.1).

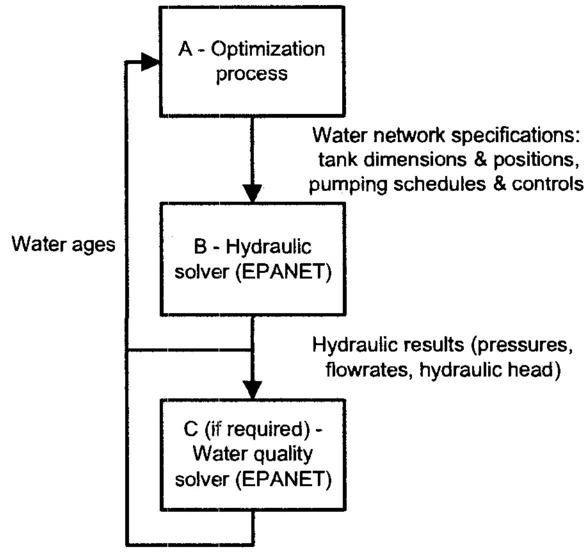

Figure 3.1 - Solver/optimization process information transmission

This methodology aims to answer a common problem that Quebec's middle-sized municipalities (around 100 MLD) have to deal with. As their WDS is expanding, the existing storage tank (typically locate at the water treatment plant) becomes insufficient. This situation leads to the need to introduce a new storage tank in the WDS. The layout and pipe sizes are known for the existing networks and for the extension. However, the optimal location of a new storage tank remains to be identified.

To determine the best location on a new tank and its best size, a methodology following different steps has been developed. The first steps deal with the identification of the need for a new tank and with the determination of the kind of tank (elevated vs. underground) according to the specifications of the WDS. Then the parameters, variables, demand scenarios and pumping schedules must be set: for instance, if the tank does not exist, it is interesting to optimize the size of the tank for forecasted demands whereas the optimization of the operation levels (minimum and maximum levels) is more valuable in the case of an existing tank considering up-to-date pattern demand. Once this elements set, the optimization process is applied to determine a batch of solutions. Finally, a multi-criteria decision tool allows determining the "optimal" for requested constraints.

3.2 Methodology

The methodology developed to achieve tank optimization follows different steps detailed in figure 3.1.

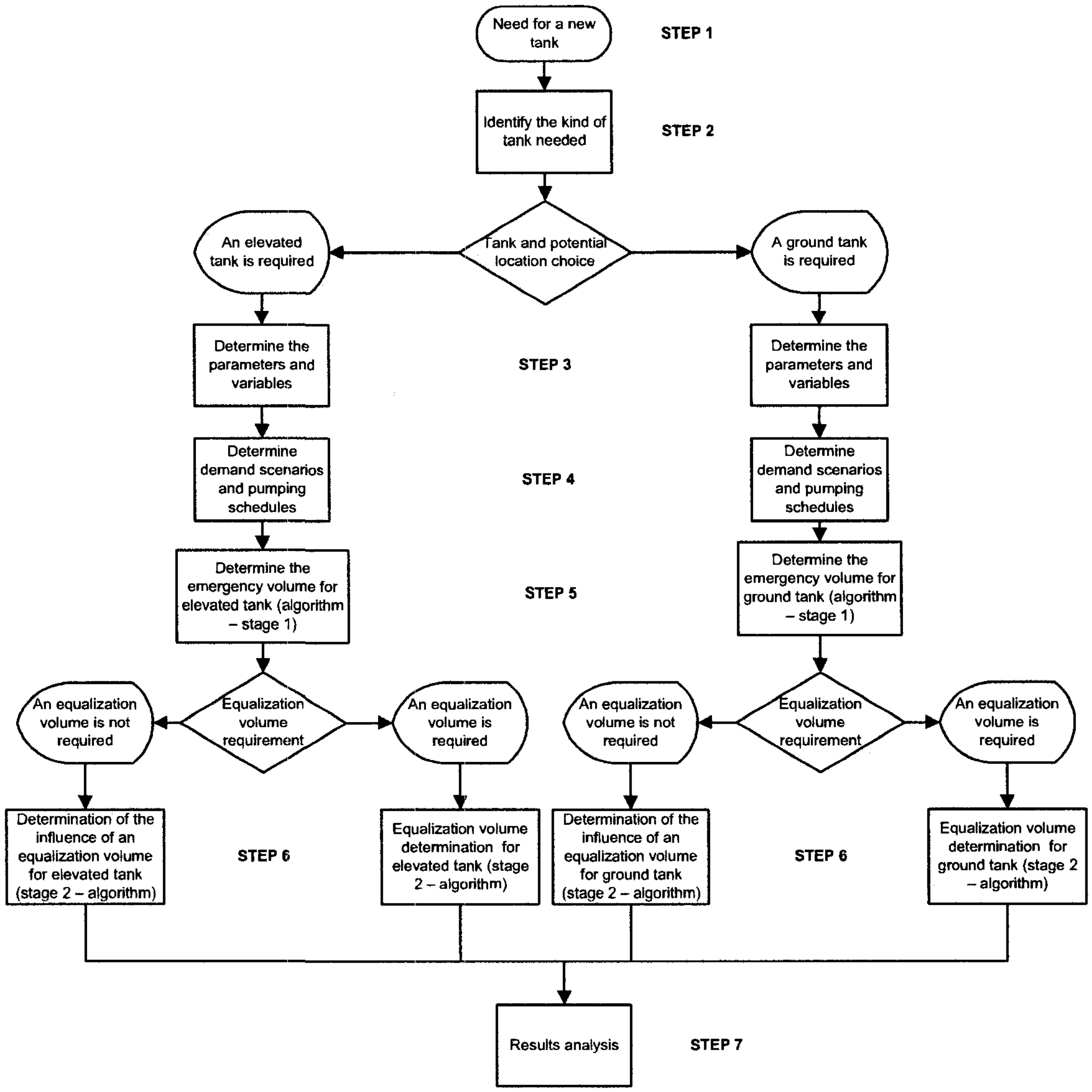

Figure 3.2 - Summary of the proposed methodology

Step 1: Identify the need for a new tank

The introduction of a new storage tank on a network can solve several problems and therefore several types of storage (fire, equalization, emergency) can be identified.

The fire storage was considered as including the emergency storage since the risk of facing a fire and a major pipe break simultaneously is very low.

In the case of a new extension to an existing WDS, the need for a storage tank generally occurs following hydraulic simulations of future water demand scenarios using the extended WDS. The lack of capacity generally appears during peak demands, or in certain conditions of peak demand combined with fire flow. It is essential to study the behavior of the extended network under peak demand scenarios in order to evaluate the need for new storage.

Hydraulic solvers available now allow to perform extended period simulation (EPS). EPS provides good indications on the behavior of the WDS.

Step 2: Selecting the type of tank (underground vs elevated) and its potential locations

Essentially, three types of tanks can be found on water distribution systems: elevated tanks, ground tanks and hydro-pneumatic tanks. Elevated tanks are "floating-on-the-system" water storage facilities. For this type of facilities, the hydraulic grade line is the same as the water level in the tank. The water can freely enter and exit the tank. An elevated tank provides in general the smoothest pressure changes due to the absence of pumps. For equivalent volume, elevated tanks are usually more expensive to build than ground tanks but they can be an interesting option to avoid pumping.

Ground tanks refer to tanks built at the level of the ground or nearly (buried or semi-buried). The filling is generally done by an important water conveyance pipe whereas the emptying is performed by pumping equipments. Ground tank are interesting in Quebec as they are least susceptible to freezing.

Hydro-pneumatic tanks have an hydraulic grade line higher than the water surface in the reservoir. To allow the water level in the tank to fluctuate (so that the tank is not simply a wide spot in the pipe), air is forced in the tank to expand or compress as the volume of water in the tank changes. Their use is limited to small tanks that provide limited equalization and emergency storage and virtually no fire protection (Mays, 1999). Due to those limitations, the optimization of hydro-pneumatic tanks will not be studied here.

Deciding between an elevated tank and a ground tank depends on the restrictions on availability, terrain, aesthetics and the pressure characteristic of the potential locations. If the pressure variation at a potential node is not large enough, the set up of an elevated tank may reveal impossible: the tank would not be able to fill or empty properly. All those considerations will drive the choice of the type of tank for the potential locations.

Step 3: Determining the parameters and variables

To realize an appropriate optimization, it is necessary to determine carefully the different parameters and variables involved.

Some characteristics are fixed parameters: it means that those parameters will not change at all during the entire optimization process. Considering a network and its extension, pipe characteristics have been chosen as fixed (diameters, length, Hazen-Williams coefficient): old pipes have existing characteristics – the challenge is to have the budget to have them measured - and new pipes are designed by the engineer in charge of the WDS development considering the future infrastructures and the characteristics of the materials.

Other characteristics are adjustment parameters: it means that those parameters have to keep their value once the optimization process is launched but they can easily be change if necessary. Pumping schedules and water demands are the parameters of the optimization: they will be changed to evaluate several possibilities. Elevation bound, pressure bound the diameter/height ratio of the tank can also be adjusted.

Variables are the characteristics that have to be determined by the optimization process: the dimensions (diameter, height, maximum and minimum level) of the tank are generally the variables of the algorithm.

It is important to correctly determine which characteristic will be a fixed parameters, a adjustment parameters or a variable.

Step 4: Determine demand and pumping schedules scenarios

An algorithm has been developed to calculate the tank volume. The purpose of the algorithm is to determine for each possible location a number of possibilities for the set up of the tank. It is a two-stage algorithm: the first stage (stage 1) deals with the determination of the emergency storage volume whereas the second stage allows setting up the required equalization volume of the tank.

The design must be realized under extreme conditions. These scenarios should include peak demand, several critical fires with normal flow demands and the related pumping schedule. The scenarios must be created case by case considering the specification of each WDS. We propose that two scenarios be considered: the fire scenario which will dictate the volume of emergency storage and the peak demand scenario which will be used to set the equalization storage volume needed.

Fire scenario and emergency volume

The fire scenario is realized first as it is the basis needed to determine the emergency volume and the minimal level. Fire flows should be determined following the municipal and/or provincial rules applicable generally considering the worst possible case.

Peak or normal demand scenario and equalization volume

Following the fire/emergency analysis, a scenario for the determination of an equalization volume is determined. The equalization volume can be either *required* or *recommended*.

A required volume means that without the introduction of an equalization volume, the WDS would not be able to provide the flows and pressures needed for peak demands. An equalization volume is required if the simulation with a peak demand show that the WDS is not able to satisfy the minimum pressure. For the extension of an existing network, it is anticipated that this case is seldom encountered. In fact, the decision of adding a new tank on a WDS is generally led by the impossibility of the system to meet the fire flows. If the equalization volume is required, the scenario uses the peak demand to determine the equalization volume.

A recommended volume means that from an hydraulic standpoint, a tank is not necessary to provide water in peak demands. However, in order to maintain water quality, it is recommended to provide an equalization volume in order to renew the water of the tank and therefore reduce the residence time. For this condition, the normal demand is used to determine the equalization volume.

Pumping schedules for each scenario

The electric price setting is very different from one location to another: the price rate in Europe and in the US varies on an hourly basis in order to reduce peak demand. However, in Quebec, the energy cost is constant throughout the day. Simultaneously, the pumping schedule must minimize the number of pump switches to reduce pipe breaks and leakage in the WDS due to the transient pressures produced by pump starts/stops.

Once the demands are determined, the pumping schedules for each scenario are adjusted manually to take into account several parameters: the filling/emptying cycle, the cost of energy, the need to minimize the number of pump switches.

The pumping schedules can be adjusted manually following two main ways: control rules or a fixed schedule depending on the case. Control rules allow to start and stop pumps based on the water level in a tank or the pressure at a node. A fixed schedule starts and stops pumps following a user-defined time pattern.

The purpose of the determined schedules is not to establish the future operation schedules but to provide realistic, nearly optimal schedules for the design of the infrastructure. Optimizing the operations occurs once the design is fixed: the recent developments in the control of operations with the use of variable-speed pumps coupled with fixed pumps now makes it possible to operate with the best energy use (avoiding the recycling loop in the pumping station) corresponding to the demand.

Step 5: Determine the emergency volume (stage 1 of the algorithm)

Stage 1 of the algorithm aims to determine the emergency volume using the fire scenario. In order to do so, distinct algorithms were developed for ground tanks (figure 3.3) and elevated tanks (figure 3.4). The algorithm is successively applied at each tank potential location of the WDS. Initially, both the elevation and the volume are set equal to their upper bound values. The tank is only used during the fire period: at the beginning of the fire, the riser is opened and then closed when the fire is put out. Pumps are switched on during that period. This stage is decisive and has to be completed first: it allows fixing the minimal operating level of the tank for each configuration tested (elevation, node). This information is required to continue to the second stage of the algorithm.

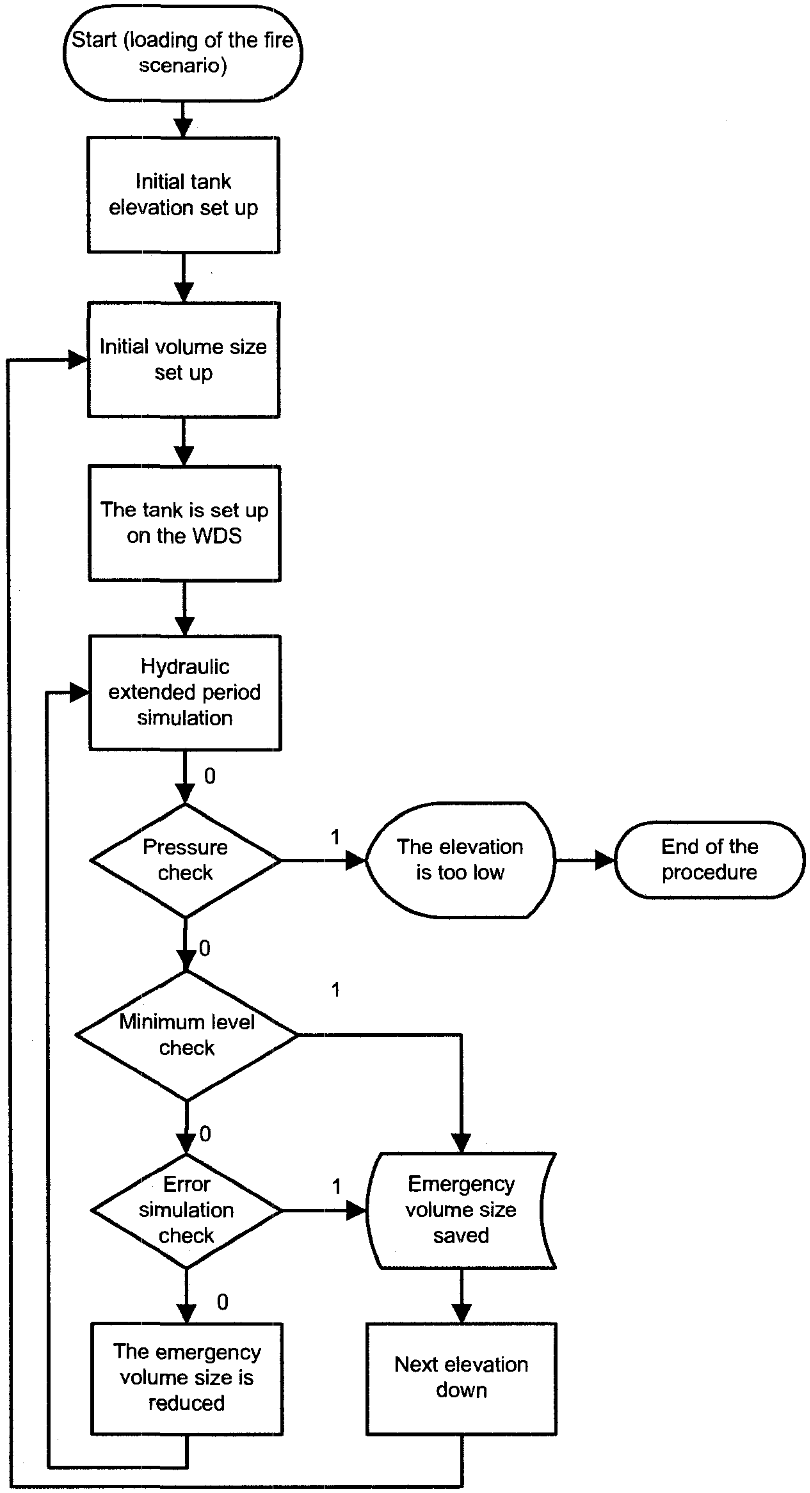

Figure 3.3 - Emergency volume determination for elevation tank

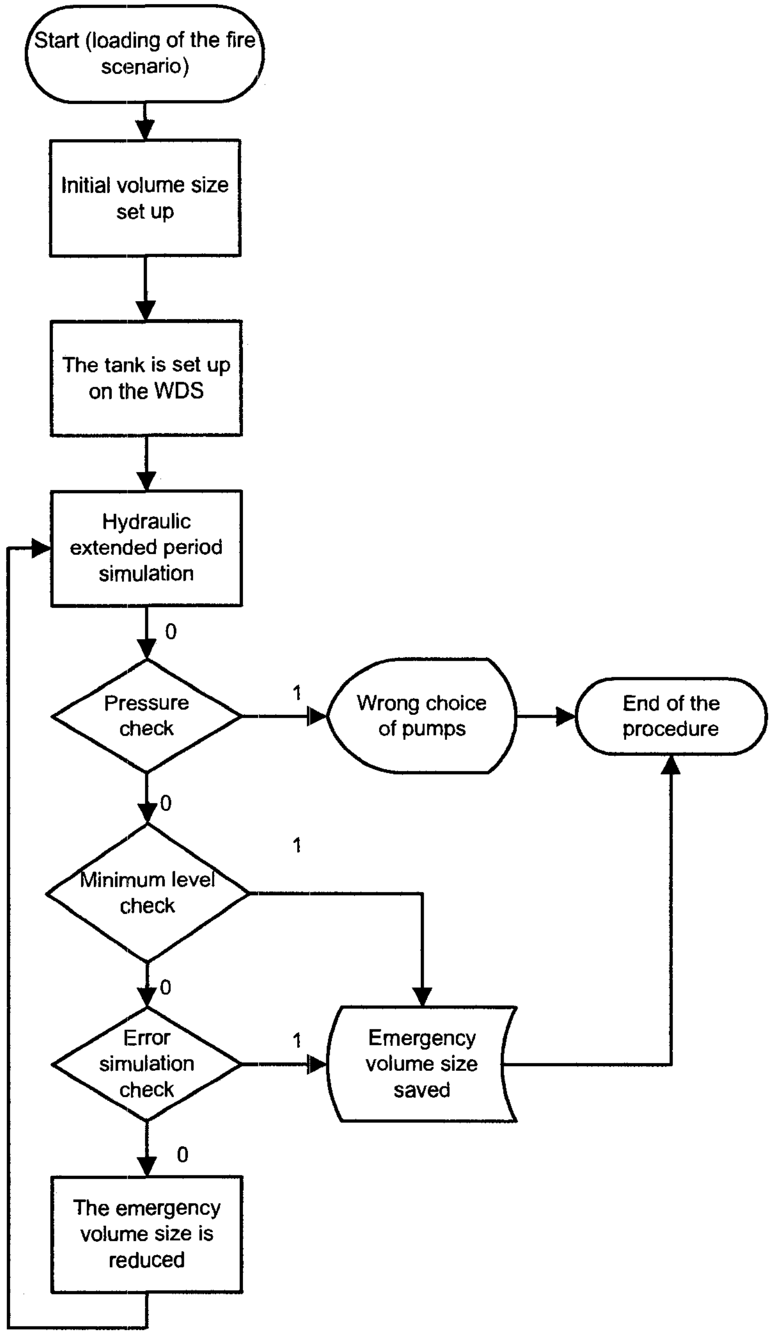

Figure 3.4 - Emergency volume determination for a ground tank

Several checks are performed during the determination of the emergency volume:

- **Pressure check:** the pressures of several key nodes are controlled over the simulation. Those key nodes are the nodes where the pressure could possibly be the lowest on the WDS. The pressure check toggles an indicator to 1 if one of those pressures is lowered below a minimum pressure.
- **Minimum level check:** the level in the tank is checked during the simulation period. If the level goes below the minimum level for a certain period, an indicator will report the time during which the tank was empty. It also toggles a Boolean value to 1. For instance the elevation of the output pipe is generally the minimum level.
- **Error simulation check:** if an error occurs during the simulation process, the check toggles an indicator to 1. Negative pressures, pump overflow, syntax error are common simulation or configuration errors.

Step 6 : Determine the equalization volume (stage 2 of the algorithm)

The first stage allowed setting up the required storage tank emergency volume. The second stage deals with the determination of the equalization volume. Depending on the type of tank (elevated versus buried) and the "status" of the equalization volume (*required* or *recommended*), the algorithm applied will be different (4 possibilities – figure 3.5 to 3.8).

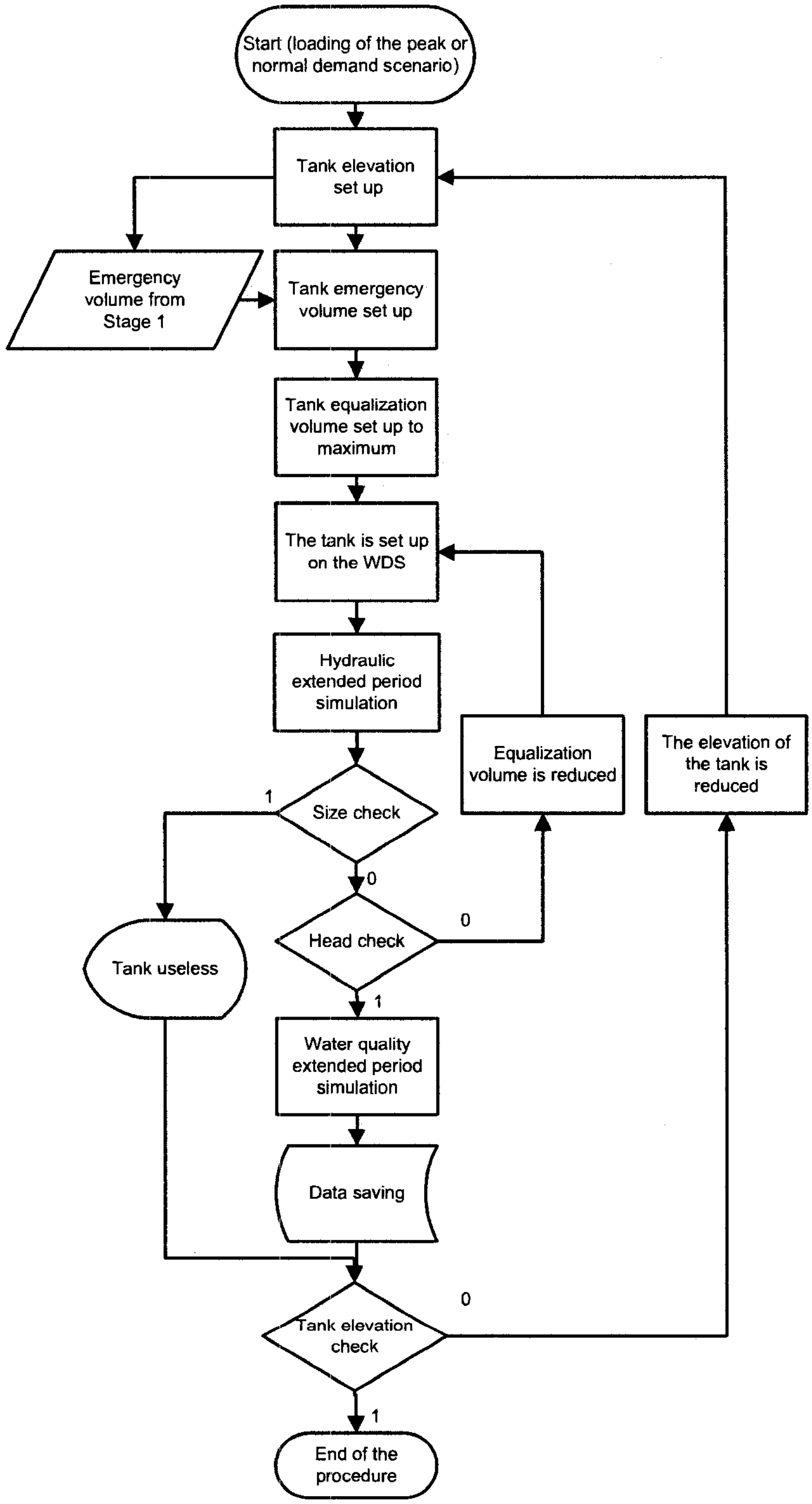

Figure 3.5 - Equalization volume determination for an elevated tank

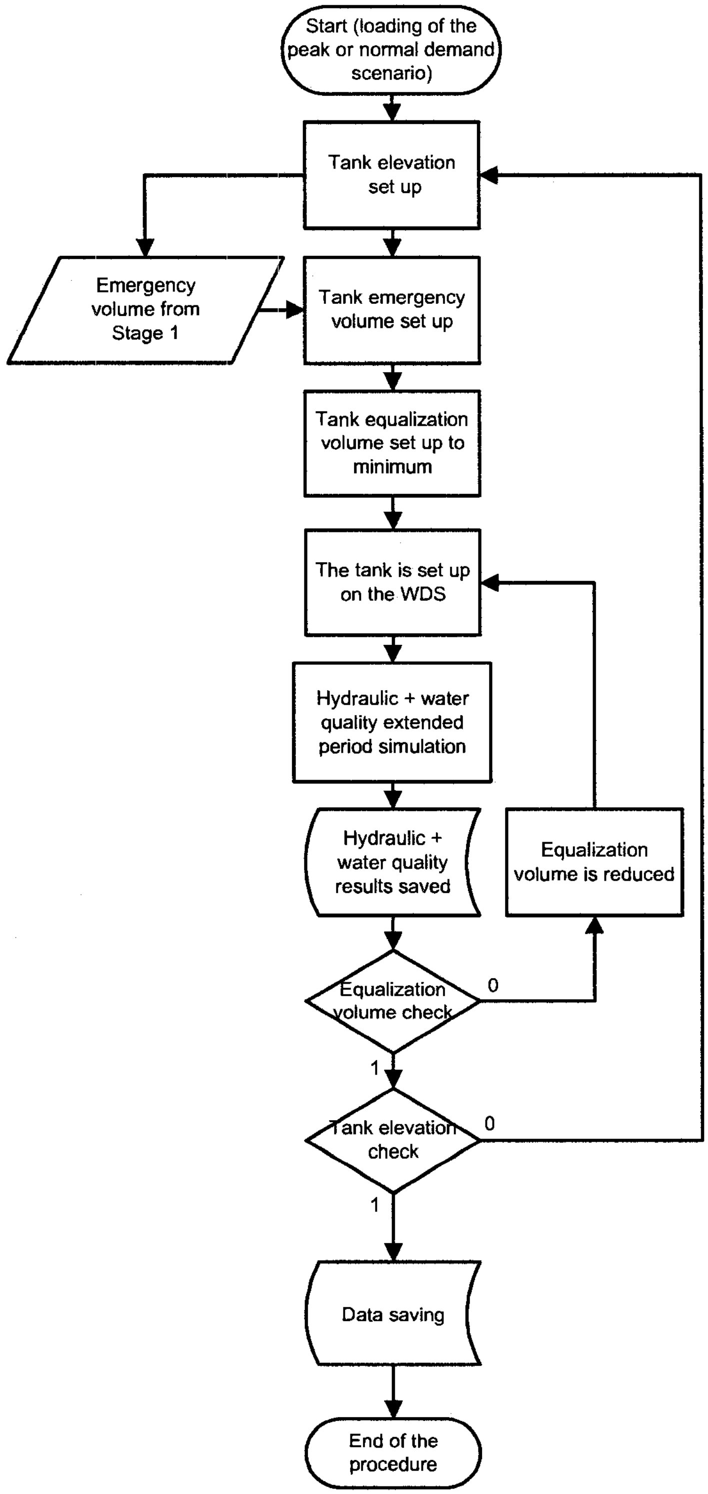

Figure 3.6 - Determination of the influence of the equalization volume for an elevated tank

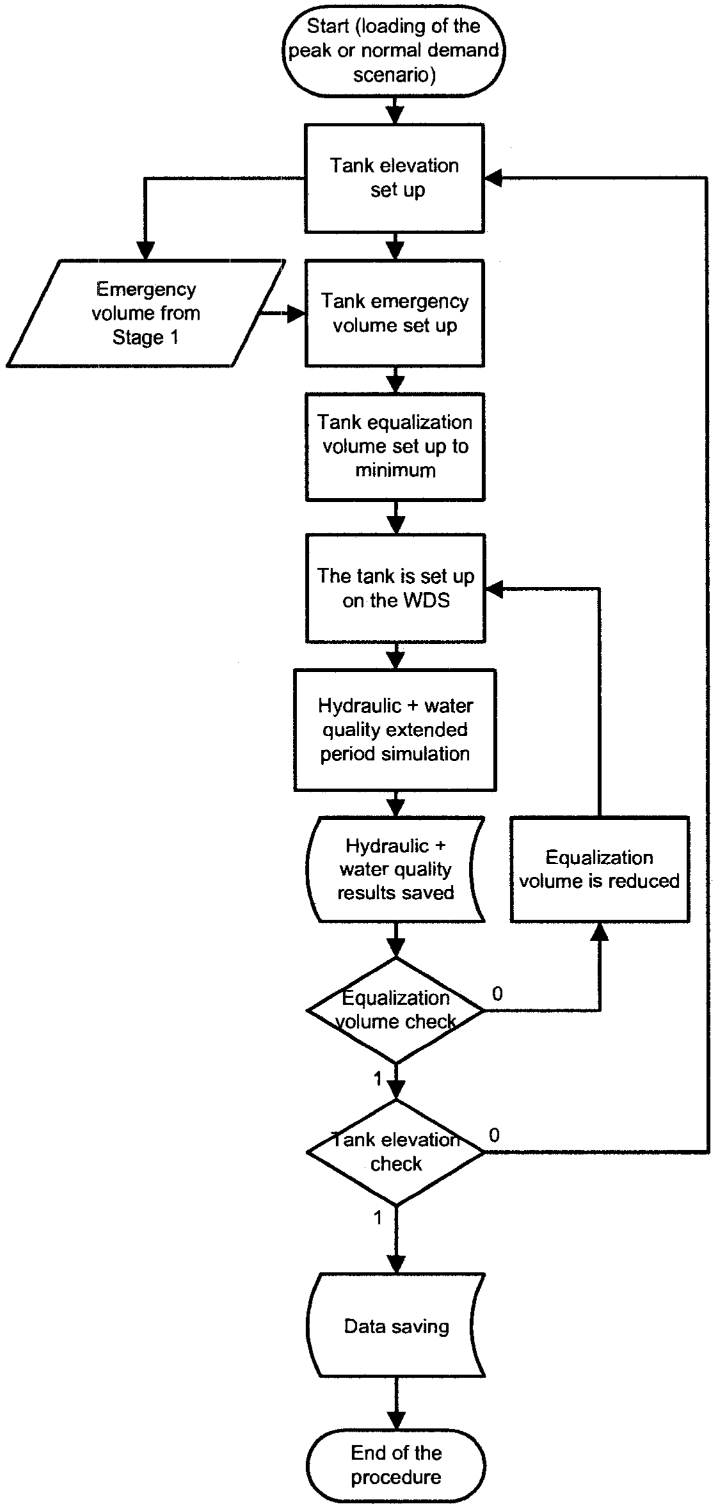

Figure 3.7 - Equalization volume determination for a ground tank

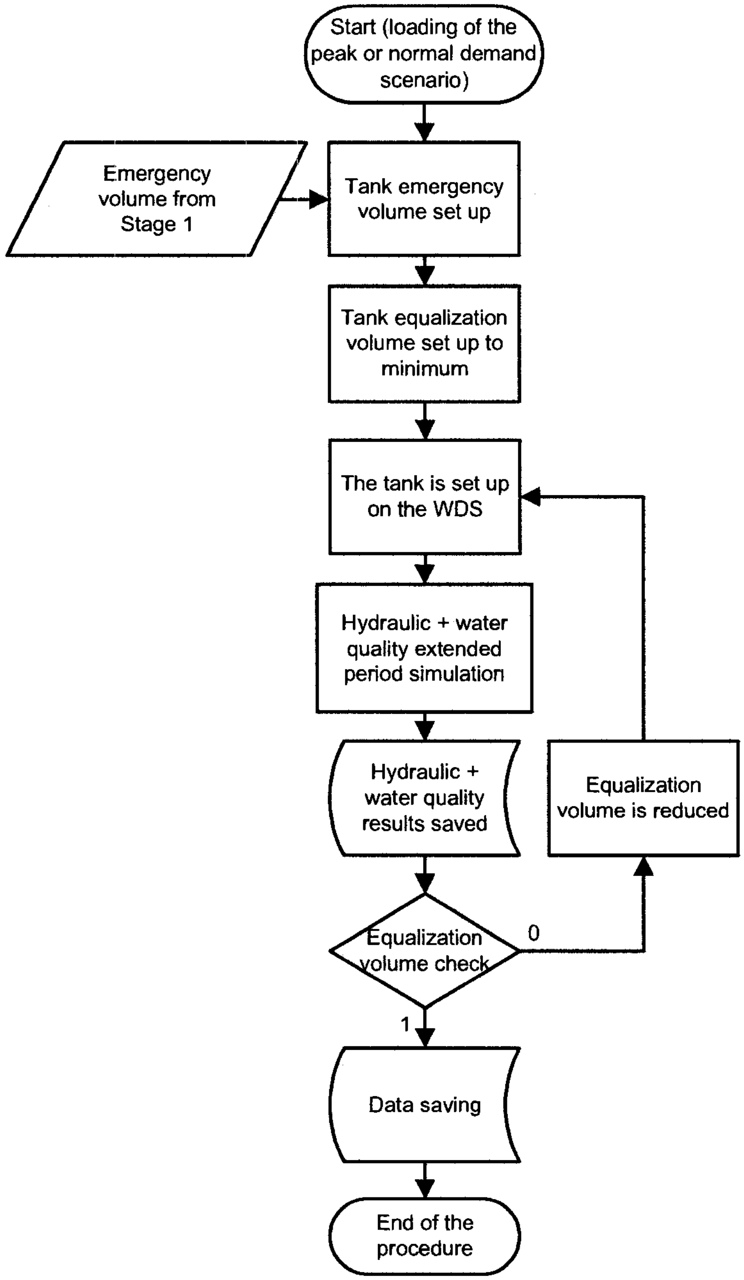

Figure 3.8 - Determination of the influence of the equalization volume for a ground tank

Several checks are performed during both tests:

- **Head check:** the head of the tank is checked during the period of simulation. If the head is equal to the maximum level for a certain period, an indicator will report the time during which the tank was full. It also toggles a Boolean to 1.
- **Size check:** the size check toggles an indicator to 1 if the size is maximal when the check is performed.
- **Tank elevation check:** the tank elevation check toggles a Boolean indicator to 1 if the elevation is at the minimal value when the check is performed.
- **Equalization volume check:** the equalization volume check toggles a Boolean indicator to 1 if the predefined range of volumes has been simulated. The range of volume has been defined at step 3.

Step 7: Analyzing the results

Once the optimization has been completed, the program generates a data file including several types of information:

- **Design data:** the diameter and height of the tank
- **Operational data for the tank:** the minimal and maximum levels of operation in the tank, the maximum water age in the tank over the period, the electrical consumption and peak over the period and the number of pump on/off cycles over the period.
- **Operational data for the network:** the critical (minimum and maximum) pressure on the network over the period and the ID of the associated node,

the maximum water age on the network over the period and the ID of the associated node

- **An inverse-flow penalty constraint:** this penalty has been introduced to deal with an operational issue. WDS managers who operate old networks do not want changing the direction of the flow in their old pipes to avoid water quality problem such as the introduction of particles. This penalty takes into account the length (Li), the diameter (Di) of each pipes i where flows are reversed during the simulation. The equation 3.1 gives the expression of the inverse-flow penalty, where Ri is a Boolean indicator moving from 0 to 1 if the flow in the pipe i change direction.

$$P = \sum_{i=0}^{n} R_i \times L_i \times D_i^{1.3}$$

Équation 3.1 - Inverse-flow penalty constraint

At this last step, construction costs are also calculated. If necessary, depending on the results, a multi-criteria method can be applied to identify the optimal location. Selecting a multi-criteria decision making tool amongst the large panel of existing methods is left to the discretion of the designer. A few of these methods are: the Kepner-Tregoe Matrix, the Weighted-Sum Method (WSM), the Analytic Hierarchy Process (AHP), the Multi Attribute Utility Theory, the Electre family of methods, and goal programming. In WDS optimization, it is important to consider all criteria of a solution: water quality, costs, etc. In the previous list, two methods are particularly attractive:

- The WSM is interesting for (i) the high penalty applied to bad solutions for one criteria, (ii) its simplicity of implementation and (iii) its ability to quickly provide good results.
- If the solutions are very different, the application of the AHP can help the manager to establish a comprehensive and rational framework for structuring the problem and evaluating the batch of solutions.

3.3 Conclusion

The optimization of water tanks considering hydraulic and water quality aspects has never been done. This paper describes a new methodology proposed for optimizing tank location and sizing on an existing WDS. The methodology was developed more specifically for the common situation of the extension of an existing WDS due to the growth of population and industries.

The methodology includes a two-stage algorithm. The purpose of the first stage is to determine the emergency volume whereas the second stage deals with the determination of the equalization volume. The equalization volume is not hydraulically required in general but it is recommended to avoid water quality problems. The methodology was refined to account for elevated or buried storage tanks.

The methodology should now be tested on case studies to verify its capability to solve tank design optimization. It is important to mention that the optimization for in-ground tank is limited by the fact that the pumps must still be chosen manually. In addition, the current methodology is only applicable for the addition of one tank on a WDS. The next development steps would be to (i) adapt the current algorithm to the more complex situation of multiple tanks and (ii) develop a user-friendly interface. Such program would allow the engineer to optimize the design of new tanks on WDS, determine the different operation level for a new or an existing facility and to improve the management of facilities already in operation.

CHAPITRE 4 : ARTICLE 3: OPTIMIZATION OF THE DESIGN, LOCATION AND MANAGEMENT OF WATER DISTRIBUTION SYSTEMS – CASE STUDIES

A new methodology to optimize the water storage tank volume, location and management has been developed. This methodology takes into account hydraulic requirements – pressures, velocities in the pipes, etc. - as well as water quality requirements – water ages. It has been developed to be applied in the case of the extension of existing water distribution systems (WDS) due to the growth of the population and industries. It includes a two-stage algorithm: the purpose of the first stage is to determine the emergency volume whereas the second stage deals with the determination of the equalization volume. The equalization volume is not hydraulically required in general but it is recommended to avoid water quality problems. This situation has led to the development of two ways of determining the equalization volume for each kind of tank.

The purpose of this paper is to present the feasibility of the developed methodology on two case studies. The case studies deals with the introduction of a tank on a university campus and in a middle-size city (65 000 inhabitants).

4.1 Introduction: summary of the methodology

The proposed methodology deals with the issue of optimizing the location and size of tanks on drinking water distribution systems. Pipe sizes and pump schedules were deliberately not considered as optimization variables but only as parameters. In summary, the optimization process and the hydraulic (and water quality) solver feed iteratively information one to another: the optimization process evaluates the validity of the solver output and, if needed, modifies the inputs before restarting the process. Figure 4.1 summarizes the general process of the hydraulic software interacting with the optimization procedure.

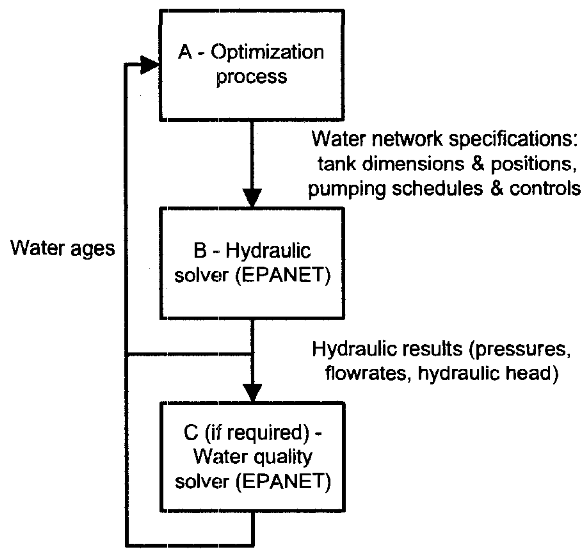

Figure 4.1 - Solver/optimization process information transmission

The first part of the methodology determines the need of a new tank, the choice of the kind of tank, the determination of the potential locations, the parameters, the variables, the demands and the pumping schedules. The second part of the methodology is the application of an algorithm to determine the emergency and the equalization volumes.

This methodology presents the solution to a problem that Quebec's middle-sized cities (around 80-100 MLD) have to deal with: the expansion of their WDS requires the use of a tank at the water treatment plant (WTP) as buffer insufficient and leads to the introduction of water tanks on the WDS. The layout and pipe characteristics (length, diameter, Hazen-Williams coefficient) are known for the existing networks and for the extension.

4.2 Case study #1: a university campus

Our first example will be based on the case of a small private distribution system managed by a university and supplying several buildings on the campus. The original network includes two pumps and no storage tank. The network is illustrated in figure 4.2. The source water is feed by the municipal WDS at a relatively constant pressure of about 60 psi (414 kPa). The pumps are rarely operated: a fire flow is required for them to be switched on.

Due to the construction of a new building on the campus, the baseline demand and fire flow will be significantly increased as the new building will nearly double the land use on the campus. The analysis of the new configuration indicates that the municipal WDS will still be able to provide the normal baseline demand. However, the existing pumps are not able to provide the required fire flow.

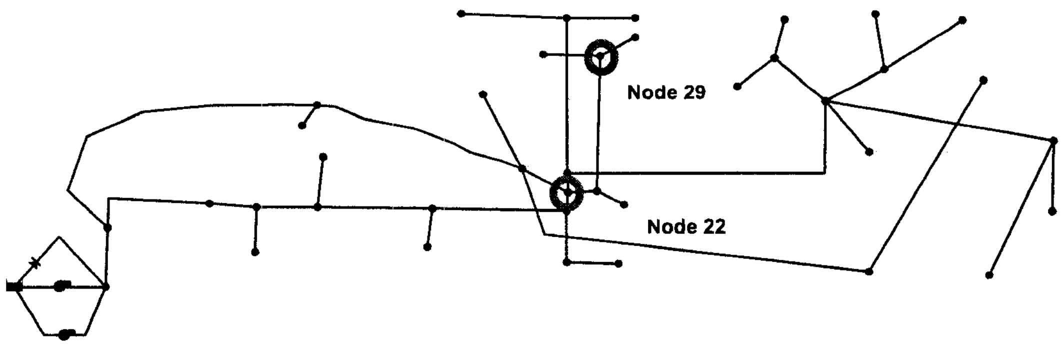

Figure 4.2 - Network layout

Three scenarios to meet the baseline demand and the fire flow can be evaluated:

- **Adding an elevated tank at one of the two marked nodes:** The tank will supply fire flow conditions and would be filled by the two existing pumps. This solution offers the following advantages: (i) increased pumps operation, (ii) improved reliability of the WDS and (iii) avoidance of the pumping station refurbishment. Two nodes have been identified for the introduction of a new tank: node 22 and node 29. These nodes have been chosen for their proximity to the fire hydrant and their central position on the network.
- **Replacing the existing pumps**: This option has the advantage of reducing the operational costs.
- **Adding a ground storage tank coupled with a new pump:** This option was rejected from the start as it provided of the highest initial capital costs.

The first scenario is studied in details and results are compared to the cost of adding new pumps. In order to size the storage tank volume and select its location amongst the two potential sites, the developed methodology, summarized in figure 4.3, is applied.

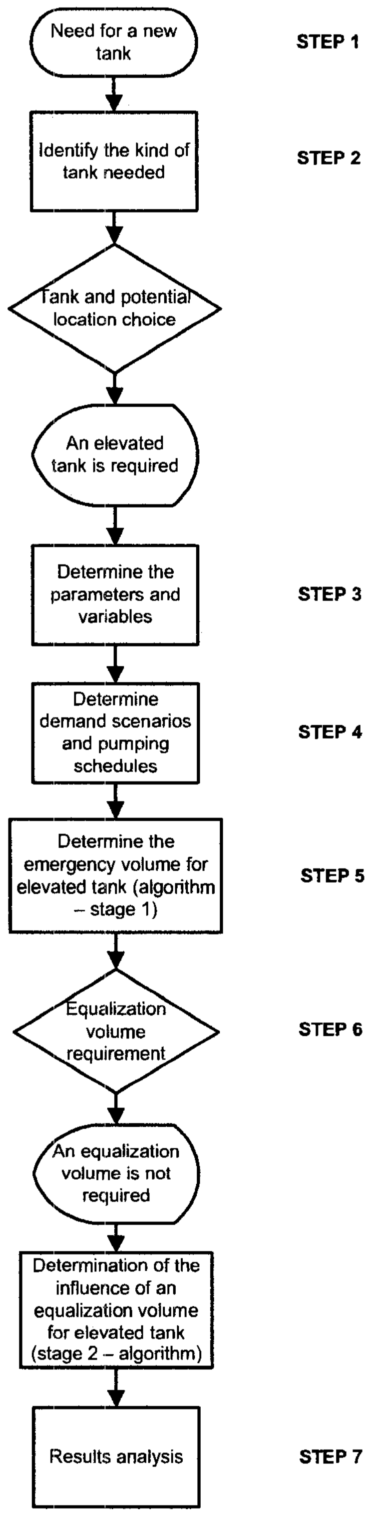

Figure 4.3 - Summary of the proposed methodology for sizing a storage tank

Step 1: Identify the need for a new tank

The baseline water demand (and the seasonal/daily patterns) of the existing buildings is known from a monitored water meter (24h/7d) locate at the entry of the university WDS. The future demand is forecasted using the historical water consumption data. Both existing pumps have a maximum flow rate of 525 USGPM (33 L/s). In this case study, adding a storage tank in the WDS mainly solves the fire flow requirements and improves the reliability of the WDS.

Step 2: Selecting the kind of tank and potential locations

The existing WDS has already two existing pumps designed to meet the past fire flows requirements. As the future fire flows are more important, redesigning the pumps would be necessary if a ground tank was selected as opposed to an elevated tank. In order to avoid the need to both construct a new tank and install new pumps, the alternative to use an elevated tank is selected.

Two nodes have been initially identified as good candidates for the introduction of a tank: node 22 and node 29. As discussed earlier, these nodes have been chosen for their proximity to the fire hydrant and their central position on the network.

Step 3: Determining the parameters and variables

As this project involves an existing network, the operational parameters (pump schedules, etc.) and the variables (dimensions of the tank) are given. The minimum pressure targeted is set at 20 psi (138 kPa), the minimal pressure recommended for a fire hydrant (Brière, 2007). The aspect ratio (diameter/height) is set at 1:1.1 and the upper elevation bound at 250 ft (76 m). This value is also used as the initial bound in the iteration process.

The emergency volume in Quebec must be at least equal to the fire volume (Brière, 2007). The fire volume is equal to the fire flowrate (1 000 USGPM – 63 L/s - in this case) multiplied by the fire duration (2 hrs), which yields an emergency volume of 120 000 US gallons (454 m^3).

The equalization volume will be found by the optimization process. It is included between 3 000 (12 m^3) and 80 000 US gallons (300 m^3): On one hand, the maximum volume allows to avoid excessive pump startups. On the other hand, the minimum volume limits the size of the equalization volume. It was selected in order to impose a minimum of one daily startup. The bound values have been computed from previous tests on the WDS.

Step 4: Determining demand and pumping schedules scenarios

For each node, a baseline demand is provided which must be multiplied by a coefficient shown in the demand pattern represented on figure 4.4.

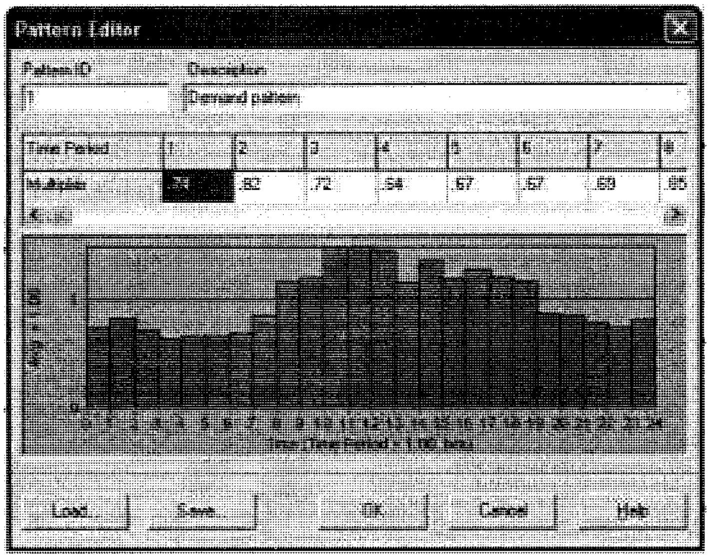

Figure 4.4 - Demand pattern

Two scenarios must be analyzed: the fire scenario and the baseline demand scenario.

Fire scenario and emergency volume

For the fire scenario, the fire flow is 1000 USGPM at the fire hydrant for a period of 2 hours during the peak demand. The pumping schedule is simple: the pumps are switched on during the fire period. Two pump configurations are studied: both pumps are switched on or only one of them.

Normal demand scenario and equalization volume

The simulation for peak demand showed that an equalization volume is not *necessary* – pressures are higher than 20 psi (138 kPa) - on the WDS but rather *recommended* for water quality issue. The determination of an equalization volume is based on the baseline demand (+ pattern) without the fire flow. Water quality (*i.e.* residence time) is computed for a 5-day simulation. For cost determination, if two pumps are switched on, an additional cost for the addition of a new pump will be considered since Quebec's legislation imposes a minimal redundancy of $n + 1$ (MDDEP, 2002).

In summary, an equalization volume is not hydraulically required but is recommended to provide a good water quality. Considering the fire volume (120 000 US gallons) described earlier, the optimal equalization volume and operational costs are determined using the baseline demand scenario, as described in step 5.

Step 5: Determining the emergency volume (stage 1 of the algorithm)

For both candidate nodes, the stage 1 algorithm (cf figure 4 of Part 1 - (Basile, 2008)), is applied to the two different pump configurations. An emergency volume is then determined.

Step 6: Determine the equalization volume (stage 2 of the algorithm)

An equalization volume is *recommended* here for water quality issue. The stage 2 algorithm (cf figure 7 of Part 1 - (Basile, 2008)) is applied. An equalization volume is then determined.

Step 7: Analyzing the results

The algorithm is coded in C language, using the EPAnet toolkit v2.00.12 (Rossman, 2000) recompiled with minor modifications: the modifications have been realized to retrieve already-computed costs by EPAnet. Using a 2.4 GHz Intel Duo Core 2 processor and 2GB 800MHZ DDR2 memory, the computing time is about 20 minutes for a 25-nodes network with two possible tank locations.

Once the optimization is realized, the program generates a data file (figure 14). The file presents several "hydraulically" feasible solutions. The tank dimensions (minimum and maximum level, diameter, elevation) are also provided. Several operational characteristics can also be retrieved: minimum and maximum pressure during normal scenario demand, highest residence time at each node, highest residence time in the storage tank, electrical demand (average and peak) and an inverse-flow penalty (cf section 2 – Step 8 of Part 1 - (Basile, 2008)).

Costs were calculated using the following hypothesis: an annual interest rate of 5%, an amortization period of 20 years, a capital cost of 3.78$/USGal for the storage tank, a fixed cost for the pumping station (if 2 pumps are used) of 400 000$, a monthly electrical cost of 15.54$ per kW of peak power plus a tarification of 0.0448 $ per kW-h,. These hypotheses are in line with current (2008) costs for projects in the region of Montreal and the Hydro-Québec industrial electricity rates (Hydro-Québec, 2008).

The costs have been computed using the current value (VPN – equation 2) reported at year 20. The investment costs have been updated from year 0 to year 20 whereas the operational costs have been updated year-by-year.

$$VPN = \sum_{t=0}^{n} \frac{C_t}{(1+r)^t}$$

Équation 4.1 - Current Value

Where C_t represented the value for the t period, *r* is the annual interest rate, *n* the number of period – 20 years and t the period.

Results (Fig. 4.5) indicate that the lowest costs are met when only one pump is switched. However, in order to minimize residence time (and therefore improve water quality), the use of two pumps is needed. As we have several simultaneous objectives, (minimum cost, minimum water age, minimum inverse-flow penalty and minimum of pump cycling), a weight sum method (WSM) was applied. In order to do so, some constraints were defined in agreement with the manager of the WDS:

- The water age limit is fixed to 24h: every configuration with a water age in the tank up to 24h gets a important weight (A) for the WSM.
- The number of pump switches is limited to 2 switches per hour: every configuration with more than 2 pump switches per hour gets an important weight (B) for the WSM.
- The inverse-flow penalty is computed (C) but is not used for the WSM: the manager of the WDS was not concerned about flow inversion in the system.
- The costs are directly used in the WSM (D).

The optimal solutions are the ones which provide the lowest sum A + B + D. From the WSM analysis, the best solutions are encountered with the tank located at node 22 or 29 with 1 pump switched on. Optimal solutions are presented in Table 4.1.

Elevation (ft)	Node	Tank volume (USgal)	Water quality in the tank (h)	On a 20-year amortization period		
				Electrical costs	Construction costs	Total costs
224	22	105840	18.81	522 331 $	1 061 519 $	1 583 850 $
225	22	105840	18.69	536 506 $	1 061 519 $	1 598 025 $
222	29	106956	22.04	534 432 $	1 072 719 $	1 607 152 $
223	22	109215	19.32	512 101 $	1 095 376 $	1 607 477 $
223	29	105840	21.77	547 198 $	1 061 519 $	1 608 717 $
226	22	105840	18.69	550 655 $	1 061 519 $	1 612 174 $
224	29	105840	21.67	561 267 $	1 061 519 $	1 622 786 $
227	22	105840	18.76	564 786 $	1 061 519 $	1 626 305 $
224	22	110962	18.28	519 627 $	1 112 895 $	1 632 523 $
225	29	105840	21.82	575 344 $	1 061 519 $	1 636 863 $
228	22	105840	18.69	578 935 $	1 061 519 $	1 640 454 $
221	29	111508	22.99	525 576 $	1 118 375 $	1 643 951 $
222	22	113836	20.08	503 209 $	1 141 719 $	1 644 929 $
225	22	110962	18.29	533 599 $	1 112 895 $	1 646 494 $
226	29	105840	21.79	589 422 $	1 061 519 $	1 650 941 $
229	22	105840	18.63	593 057 $	1 061 519 $	1 654 576 $
222	29	112079	20.15	531 666 $	1 124 096 $	1 655 762 $
223	22	114338	18.86	509 610 $	1 146 752 $	1 656 362 $
223	29	110962	19.99	544 246 $	1 112 895 $	1 657 141 $
226	22	110962	18.3	547 579 $	1 112 895 $	1 660 474 $
227	29	105840	21.87	603 491 $	1 061 519 $	1 665 010 $
228	29	104731	21.47	616 274 $	1 050 403 $	1 666 678 $
230	22	105840	18.73	607 179 $	1 061 519 $	1 668 698 $
221	22	117392	20.66	493 262 $	1 177 388 $	1 670 651 $
224	29	110962	19.97	558 164 $	1 112 895 $	1 671 059 $
227	22	110962	18.29	561 550 $	1 112 895 $	1 674 446 $
224	22	116085	18.57	516 941 $	1 164 271 $	1 681 213 $
225	29	110962	19.9	572 064 $	1 112 895 $	1 684 960 $
228	22	110962	18.27	575 530 $	1 112 895 $	1 688 426 $
221	29	116631	20.91	523 014 $	1 169 751 $	1 692 765 $

Tableau 4.1 - Summary of optimal solutions according to the WSM

From the two possible locations, one appears to offer a better alternative. Positioning the tank at node 22 with an elevation of 224 ft (68m) and a volume of 105 840 US gallons ($400m^3$) provides the best solution with respect to the predefined constraints. Even if the choice of the manager was to rebuild the pumping station (the installation of a new pumping station is less expensive than building a tank), this case study offered a good example of the interest of the proposed methodology. The next step was to test the methodology on a larger WDS. This is the objective of the following section.

4.3 Case study #2: the city of Saint-Jérôme

The city of Saint-Jérôme (Qc, Canada) is a typical middle-size municipality of 65 000 inhabitants, exhibiting a significant growth of around 1.4 % per year. The WDS, a publicly owned and operated entity, is fed by one water treatment plant (WTP). Most of the treated water is gravity fed from the WTP to the lower pressure zones, exception made of a few areas (representing about 10% of the city), which are re-pressurised on the WDS. Due to its topography, the WDS is divided in 6 different pressures zones with the use of 48 control valves (figure 4.5). The network has about 1500 nodes and 280km of pipes.

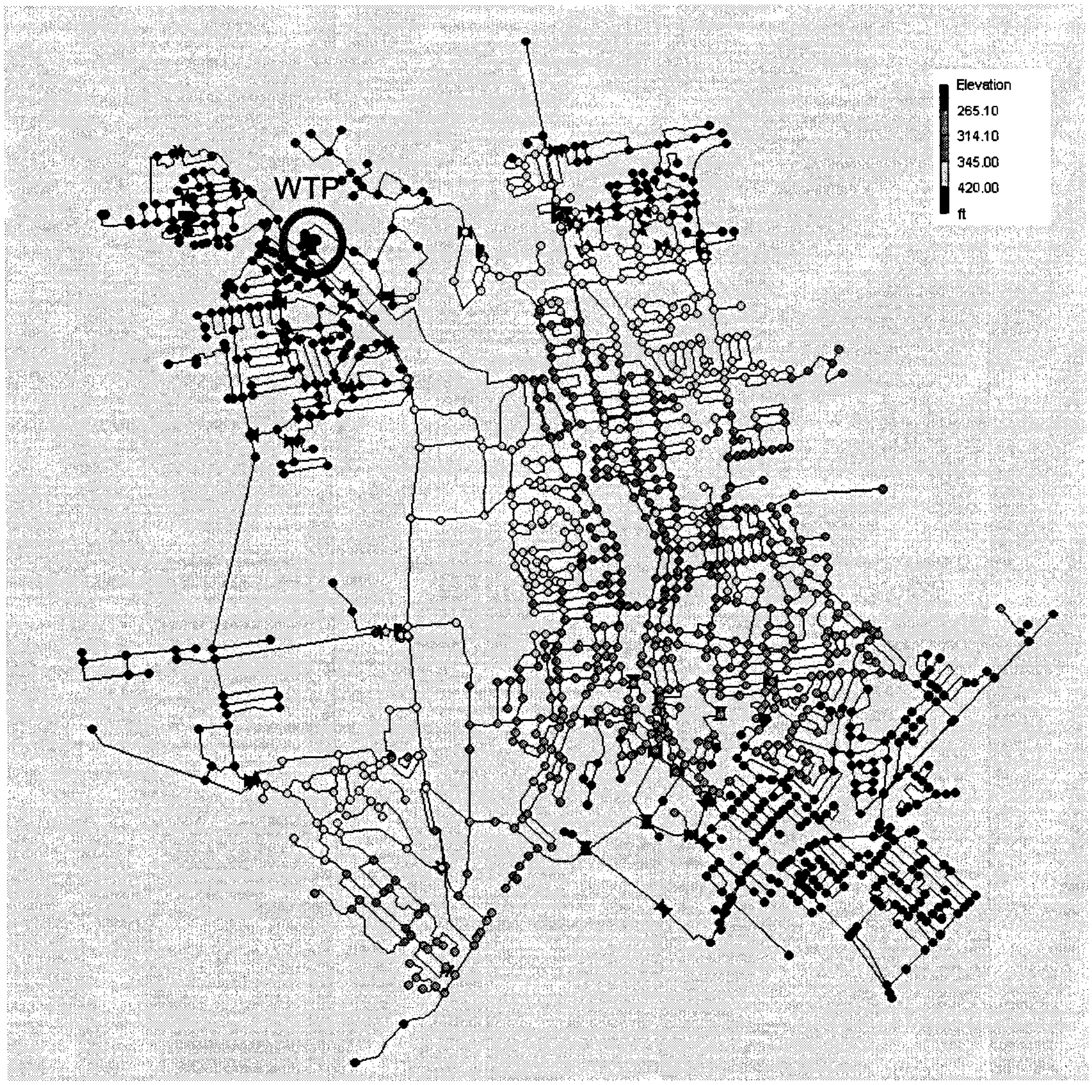

Figure 4.5 - Saint-Jérôme WDS layout and elevations

A forecast study realised in 2001 suggests a lack of capacity to supply the future demand under several configurations:

- During the summer, the forecasted peak demand will be larger than the WTP capacity

- The WDS will not be able to meet fire flow conditions for the south-east portion of the city.

Following those considerations, the WDS manager decided to install two new tanks:

- A new tank located at the WTP was built in 2008.This storage, with a capacity of 635 000 US gallon, increases the WTP equalization volume to 10 millions US gallon. The new installation will provide sufficient storage for the forecasted peak demand.
- A new tank, designed to provide the fire flow for the buildings located in the south-east bound area, will be constructed. Our case study is focused on the installation of this tank in the south-east part of town, where the elevations are the lowest.

Step 1: Identify the need for a new tank

A fire flow of 5764 USGPM for a period of 5 hours (Pagé, 2005) was identified as a requirement for the south-east suburb. This flow is split amongst 5 nodes around the building that was identified as the critical design location. With the existing configuration, the WDS is not able to provide the required fire flow. The installation of a new tank is therefore considered to achieve this objective.

Step 2: Selection of the type of tank and potential locations

Before determining the type of tank to be used, it is important to primarily determine its the potential localizations. Due to its topography, the WDS of Saint-Jérôme have several pressure zones. In order to be effective, the storage tank should be within the south-east pressure zone. Two potential locations have been considered (figure 4.6): The first one is just next to the critical building, at the edge of the WDS, on an available land space. The second one is farther located on a main pipe and at a higher elevation.

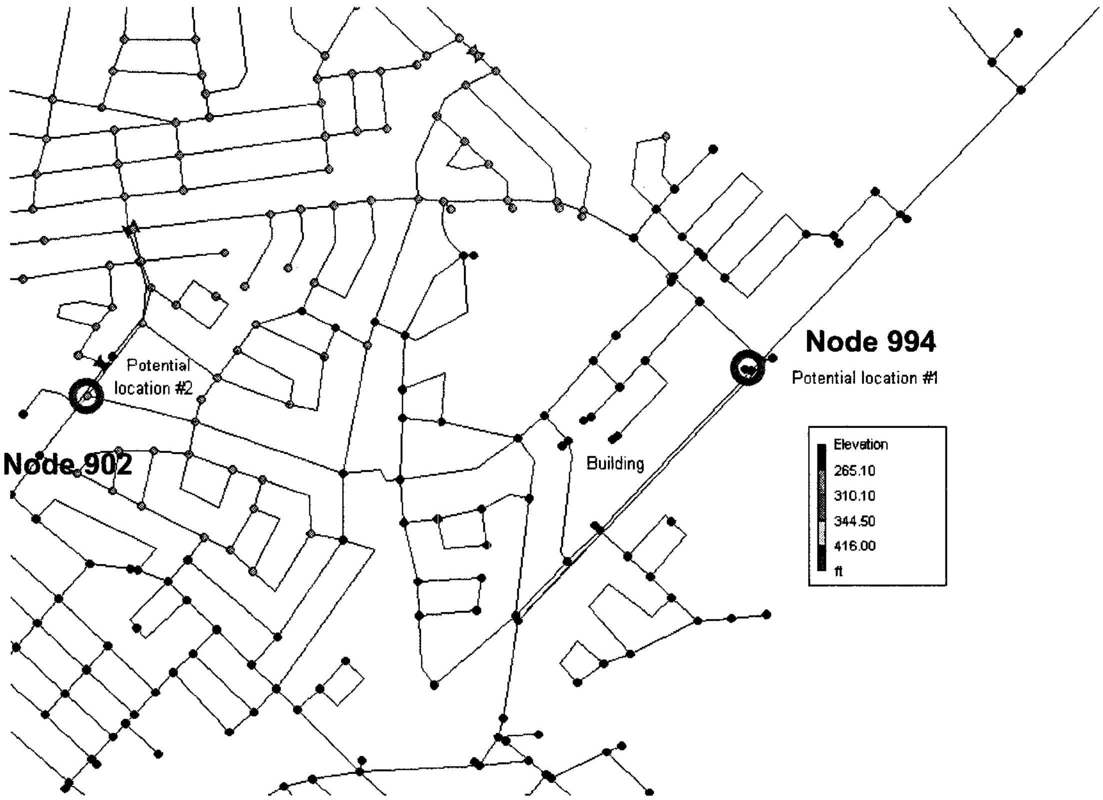

Figure 4.6 - Potential locations for the installation of a new tank

Secondly; the type of tank has to be determined. For normal flow condition, the pressure variation at either candidate sites is very low (figure 4.7). The pressure is approximately 48 psi at node 902 (goes 45-50 psi (310-345 kPa). For node 994, the pressure is approximately 52 psi (358 kPa). Under this condition, the installation of an elevated tank is not recommended: the tank would not be able to fill/empty in normal condition, which would, in turn, yield excessive residence time.

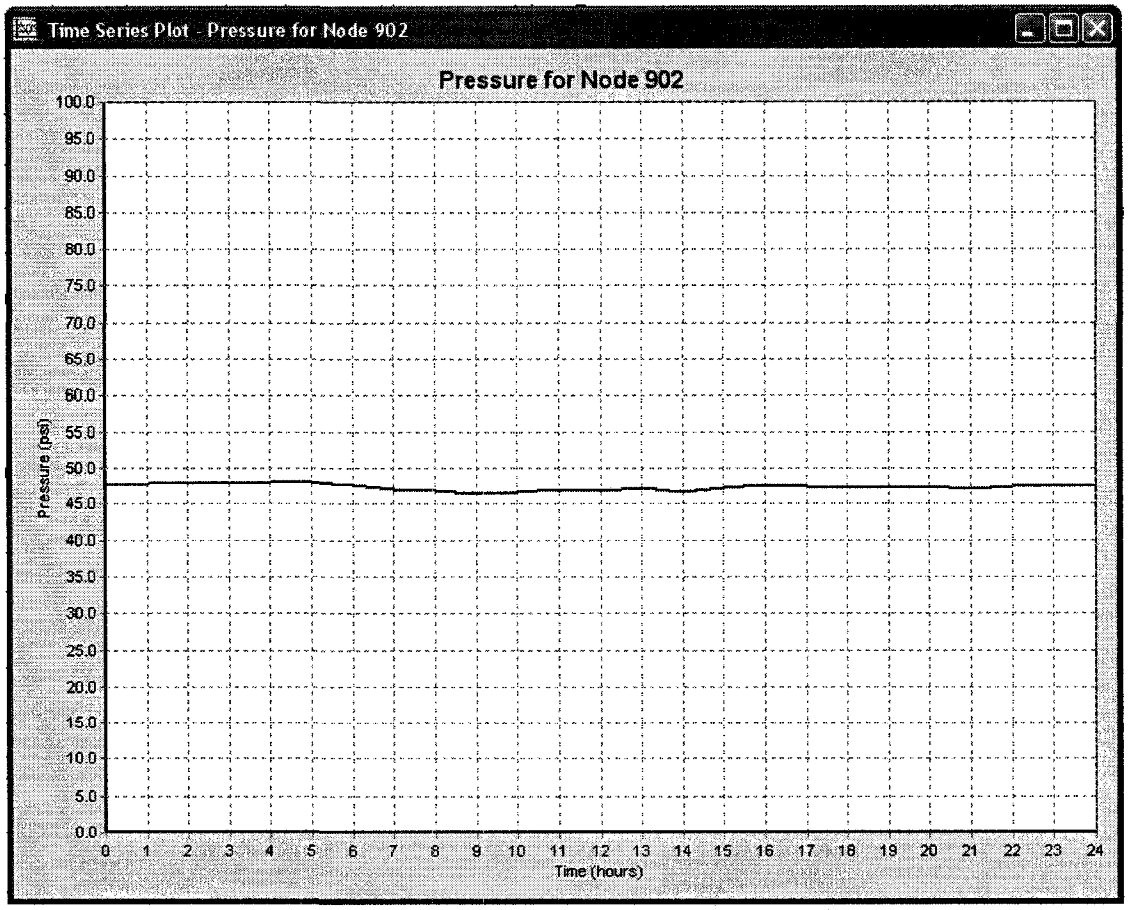

Figure 4.7 - Pressure variations at a potential tank location during normal flow condition simulation

With those considerations in mind, a ground tank with pumps would be the best configuration. It is however important to mention that pumps were chosen manually considering the pressure and the flow needed. In addition, the pumps characteristics (size, curve, speed, number, type) were not subjected to any king of optimization in this study. The two pumps were designed to provide either the emergency flow or to empty the tank equalization volume (the tank is a ground tank and the hydraulic head is smaller than the pressure at the connection node). This situation results in the introduction of two different pumps: the pump for the fire flow being significantly more powerful.

Step 3: Determining the parameters and variables

The WDS model has been created using the EPANET software (Rossman, 2000). The tank elevation is set equal to the connecting node elevation. The tank diameter was fixed at 250 ft (75 m).

The optimization variables are the tank minimum and maximum level. The upper bound is fixed at 20 ft. Choosing the minimum and maximum level of the tank as variables (compared to the diameter and height of the tank) aims to reflect the reality of the construction process: the construction will probably be designed and built to fulfill the requirements of the next 25 years, whereas the operation criteria must achieve the current requirements.

Step 4: Determining demand and pumping schedules scenarios

Two scenarios are applied: the fire scenario and the baseline demand scenario. The fire scenario allows determining the emergency volume whereas the baseline demand scenario provides the equalization volume.

Fire scenario and emergency volume

The fire flow, 5764 USGPM for a period of 5 hour, is split on 5 nodes around the critical building. It is added to the baseline demand. The fire pump is only switched on during the identified duration of the critical fire (5 hrs). Following the methodology previously proposed, the emergency volume is determined.

Normal demand scenario and equalization volume

The determination of the equalization volume uses the same baseline demand pattern without the fire flow. The water quality is computed for a 240-h simulation using the baseline demand to provide stable solutions – a small simulation time would not let the system reach stability. The pump starts when the water level goes under the minimum level and stops when it goes up to the maximum level.

The simulation for peak demand – the peak factor is 2.0 – showed the pressures in this part of the WDS where up to 20 psi (138 kPa). Once the fire volume is determined, the normal demand scenario will be used to determine a good equalization volume and operational costs by following a different procedure.

Step 5: Determine the emergency volume (stage 1 of the algorithm)

Stage 1 of the algorithm (cf figure 3.4 of Part 1 - (Basile, 2008)) is applied at two different locations.

Step 6: Determine the equalization volume (stage 2 of the algorithm)

An equalization volume is not required but is recommended to provide a good water quality. Stage 2 of the algorithm (cf figure 3.8 of Part 1 - (Basile, 2008)) is applied at two different locations.

The algorithm is coded in C language, using the EPAnet toolkit v2.00.12 recompiled with minor modifications and applied to the network. Using a 2.4Ghz Intel Duo Core 2 processor and 2GB 800MHZ DDR2 memory, the computing time is about 50 minutes for a 1500 nodes and 2000 pipes network with two possible tank locations.

Step 7: Analyzing the results

The tank will be constructed in the oldest part of the city of Saint-Jérôme. In this section, the WDS is mainly composed of unlined ductile and grey cast iron pipes. Contrarily to the previous case study, it appears prudent to avoid solutions which promote flow reversals in order to reduce red water events. This issue is dealt with by introducing an 'inverse flow penalty', which takes into account the number, length and diameter of pipes where the flow has been reversed during the simulation (Basile, 2008). In addition, the water age should also be minimized in order to reduce chlorine decay and DBP formation.

The node 994 configuration provides lower water age in the tank whereas the node 902 configurations less flow reversals (figure 19 & 20).

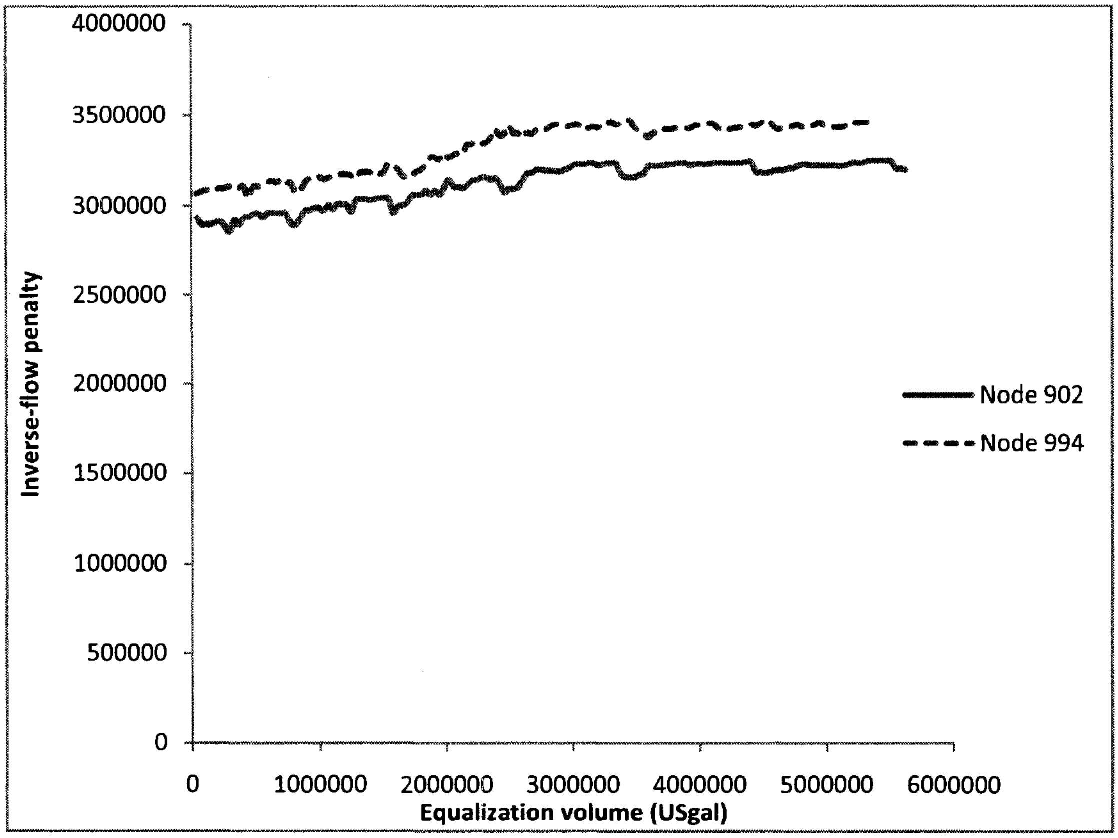

Figure 4.8 - Equalization volume vs. inverse-flow penalty in the tank for both configurations (nodes 902 & 994)

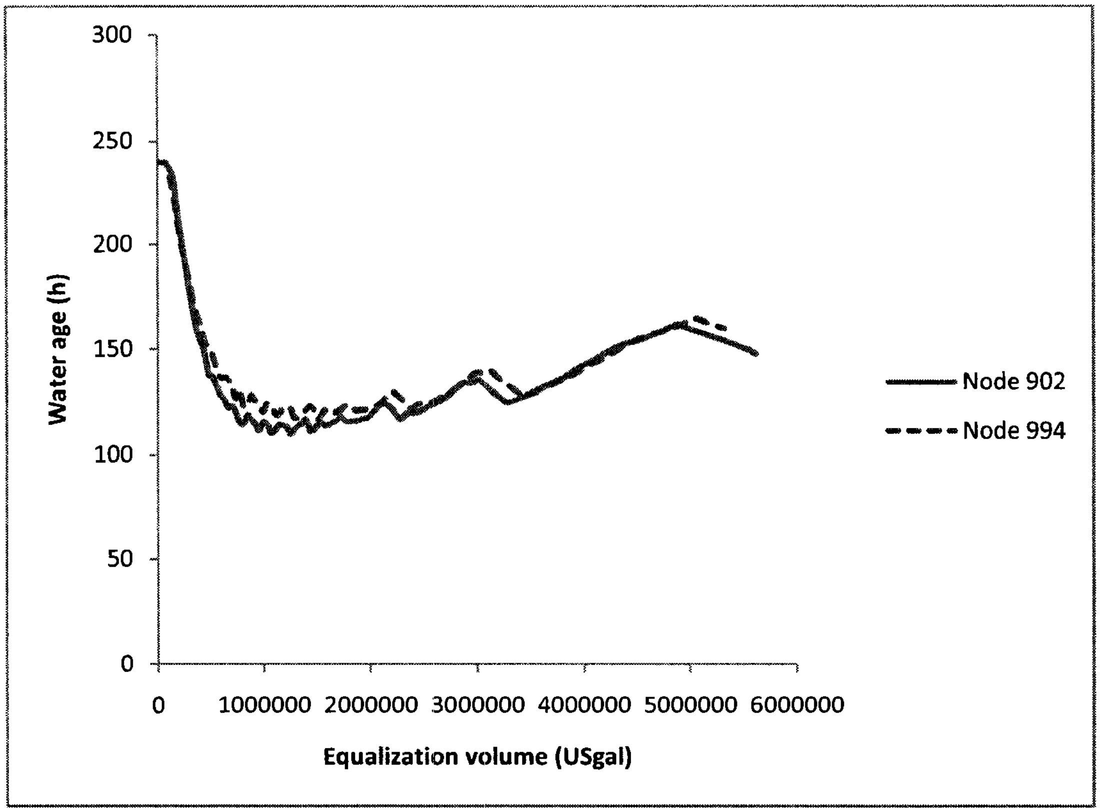

Figure 4.9 - Equalization volume vs. water age in the tank for both configurations (nodes 902 & 994)

These solutions were analyzed using a multi-criteria decision tool. It is interesting to point out that the operation (pumping) and construction (tanks) costs were not the main criteria in this case study. The objectives targeted were to determine tank volume and its hydraulic optimal location considering the water quality. Therefore, the two main objectives, established with the WDS manager, are:

- To maximize the water quality in the tank (minimize the water age in the tank)
- To minimize the inverse-flow penalty

A Pareto front was determined using the entire offset of potential solutions (figure 4.10). From this analysis, six good solutions were extracted. Amongst these, the three solutions with water ages higher than 120 hr were discarded even though they provided less flow reversals. The three remaining solutions yielded very similar configurations: the tank diameter is 250 ft (76m), the minimum height (emergency volume) is 4.6 ft (1.4m) and the maximum height is around 8 ft (2.44m). The emergency volume is 1.6 million US gallons (6000 m^3), the approximate equalization volume is about 1.4 million US gallons (5300 m^3). Under these design conditions, the water age, or hydraulic residence time, is around 110 hr (4.5 days). This value is quite high and may lead to undesirable water quality degradations. This result is due to the important fire flow (5764 USGPM – 363 L/s - for a period of 5 hours) which makes the emergency volume very important.

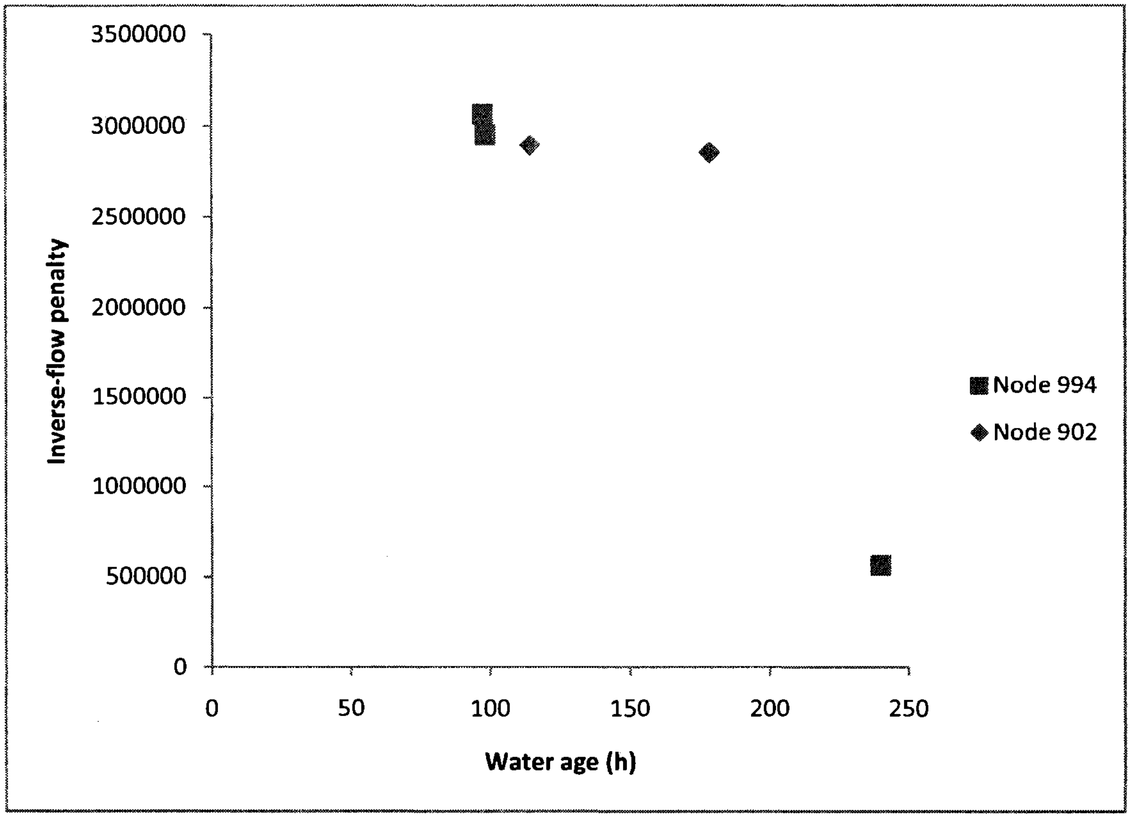

Figure 4.10 - Equalization volume vs. water age in the tank for both configurations (nodes 902 & 994)

It would probably be useful to reconsider more carefully the value of the fire flow. Following the Fire Underwriters Survey of Canada (Brière, 2007), a city must be able to provide the fire flow for a minimum period of 2 hours. If the fire flow is maintained at 5764 USGPM but the duration is reduced from 5 to 2 hr, the same analysis provides a 900 000 US gallons (3400 m^3) emergency volume and a 900 000 US gallons (3400 m^3) equalization volume. With such volume, the diameter is about 180 ft (55m), the height is 9.6 ft (2.9m) with a minimal level of 4.8 ft (1.45m). The water quality is then slightly improved with a hydraulic residence time of 93 hour.

Finally, another solution would be to set up a tank disconnected from the WDS to provide the fire flow (with a rechlorination station) and recommend a boiling water advice when this reservoir is put on line.

4.4 Conclusion

A new methodology was developed for optimizing tank design on WDS. The methodology includes a two-stage algorithm. The purpose of the first stage is to determine the emergency volume whereas the second stage deals with the determination of the equalization volume. The equalization volume is not hydraulically required in general but it is recommended to avoid water quality problems. Depending on the type of tank (underground or elevated), two different approaches were proposed.

This methodology was applied in the case (i) of the extension of a small private network and (ii) of the extension of an existing medium-size municipal WDS. For both case studies, it is interesting to point out that the fire flow was the main factor leading to the introduction of a new tank on the network. The methodology was successfully applied and the analysis of the results allowed designing an "optimal" tank in both cases.

Future refinements of this methodology would include the development of a user-friendly interface for the current algorithm. Such program would (i) facilitate the analysis of different operation levels for a new or an existing facility, (ii) help to identify the best tank location and (iii) improve the management of facilities already in operation.

CHAPITRE 5 : DISCUSSION GÉNÉRALE

La revue critique de la littérature (article 1) nous a montré que comparativement à ce qui a déjà été réalisé en terme d'optimisation hydraulique, l'aspect innovant du projet tient dans le fait que nous n'avons pas voulu optimiser l'ensemble des paramètres d'un seul coup (taille de conduites, horaires des pompes) mais que l'étude s'est limitée à l'optimisation des réservoirs.

En terme de qualité d'eau, l'étude a un caractère proactif : il est en effet plus judicieux de prendre en compte la qualité de l'eau dès la conception, que de résoudre les problèmes une fois les infrastructures réalisées. Plusieurs paramètres ont été envisagés pour caractériser la qualité de l'eau : la re-croissance de micro-organismes, la décroissance du chlore résiduel et le temps de résidence hydraulique. En modélisation numérique, la plupart des modèles de qualité d'eau utilisent le temps de résidence hydraulique pour calculer les concentrations de différents composés. L'hypothèse faite dans cette étude est que plus le temps de séjour est court, meilleure est la qualité de l'eau.

Concernant les résultats, il est maintenant question d'étudier si les solutions proposées sont réellement implantables sur le terrain. L'objectif du projet consiste à établir un modèle d'optimisation des dimensions, des opérations d'approvisionnement et de localisation des réservoirs et d'étudier la performance et la fiabilité des réseaux de distribution en terme de coûts et de qualité de l'eau dont les réservoirs ont été optimisés. La méthodologie développée (article 2) a été appliquée sur deux études de cas réels (article 3):

- l'ajout d'un réservoir sur un campus universitaire pour faire face à l'augmentation du nombre de bâtiments (l'espace occupé par les nouveaux bâtiments représentant une surface équivalente à celle utilisée par les bâtiments existants), le campus étant alimenté en eau par le réseau municipal.
- l'ajout d'un réservoir pour une ville de taille moyenne (65 000 habitants) pour satisfaire le débit incendie (5800 USGPM pour une période de 5 heures) préconisé par le plan directeur de la ville (Pagé, 2005)

Les résultats proposés sont fournis à partir d'un modèle de simulation numérique (Rossman, 2000) couplé à l'algorithme développé. L'étude de cas du campus universitaire a montré que des deux positions d'implantation possibles, l'une fournit de meilleurs résultats en termes de coûts et de qualité d'eau. L'ajout du réservoir ne fut pas l'option choisie par les gestionnaires du campus universitaire mais la méthodologie a prouvé sa capacité à fournir des solutions satisfaisantes. L'étude de cas de Saint-Jérôme a montré que le débit de conception était surestimé et que l'installation d'un réservoir d'eau potable pour satisfaire le débit incendie aurait pour conséquence une qualité d'eau médiocre. Des solutions alternatives ont été préconisées.

Au Québec, l'ajout d'un réservoir sur un réseau était jusque là dicté par des règles empiriques qui ont été obtenues par la collecte des informations sur les réseaux existants. L'une de ces règles "du pouce" tend à dire qu'en dessous de 15 000 habitants, c'est le débit d'incendie qui devient le critère majeur pour le dimensionnement du réseau. Au delà, le débit de pointe horaire devient alors le critère de conception.

L'étude du campus universitaire semble d'ailleurs confirmer ce que la « règle du pouce » nous suggère : c'est bien le débit incendie qui a été utilisé dans la conception du réservoir.

Dans le cas de Saint-Jérôme, il semblerait que les résultats trouvés par le modèle de simulation tendent à dire le contraire: le débit d'incendie a été utilisé alors que le secteur sud-ouest a une population d'environ 10 000 habitants. Comment expliquer cette situation ?

En dialoguant avec les responsables de la Ville de Saint-Jérôme, un nouvel élément a été apporté à notre attention : l'existence d'un plan d'intervention contre les incendies. Selon le plan d'intervention, les équipes de sapeurs-pompiers sont capables d'intervenir sur un feu en 4 minutes sur le lieu où les incendies ont été simulés. De ce fait, les débits estimés sont bien supérieurs aux valeurs de débit réellement nécessaires : le volume calculé par le guide de conception impose un débit unique (prenant en compte que l'ensemble des installations visées sont touchées par l'incendie) pour une durée d'une heure.

L'installation d'un réservoir est planifiée à l'avenir mais comme les simulations l'ont montré, ce réservoir n'est pas nécessaire à l'heure actuelle pour subvenir aux besoins en heure de pointe de cette partie de la ville.

L'aspect financier est aussi à prendre en compte : en effet, est-il financièrement intéressant de construire un réservoir d'eau potable pour alimenter le débit incendie de quelques bâtiments industriels ?

CONCLUSION, CONTRIBUTION ET RECOMMANDATIONS

Le but de l'étude consistait à déterminer une méthodologie pour l'optimisation des réservoirs d'eau potable au Québec en tenant compte des critères hydrauliques et de la qualité de l'eau distribuée. Les travaux réalisés ont permis de dégager les conclusions suivantes :

- L'étude des ouvrages existants a été une étape déterminante quant à la détermination des besoins. Elle nous a permis de nous focaliser sur l'ajout de réservoirs sur les réseaux existants lorsque ceux-ci s'agrandissent et sur l'optimisation des réservoirs existants. Elle a permis d'éviter l'utilisation de méthodes mathématiques très complexes qui n'auraient pas forcément fourni de meilleurs résultats.

- La méthodologie s'appuie sur un algorithme d'optimisation qui a été mis au point au cours de l'étude. Cet algorithme, au stade de logiciel pilote, a été testé sur deux études de cas réels et a prouvé son efficacité pour déterminer de bonnes solutions. Le modèle développé répond donc aux besoins en eau (tant du point de vue qualitatif que quantitatif) que certaines municipalités auront dans les prochaines années.

- Une analyse rigoureuse des résultats, à l'aide d'une méthode multicritères, permet de choisir une solution qui convient le mieux à chaque situation. Il est essentiel de bien prendre en compte à ce stage les attentes des gestionnaires en termes de qualité d'eau, de pressions minimales, de coûts d'infrastructures et d'opérations pour proposer une solution qui répondent parfaitement à leurs besoins.

Néanmoins, l'étude réalisée n'est qu'au stade de recherche et n'a pas fait l'objet de tests « grandeur nature ». Il est permis de penser que l'application de cette méthodologie à des projets « pilotes » permettrait de recueillir des données avant et après l'installation d'un réservoir optimisé afin de valider le modèle de manière efficace. Plusieurs éléments restreignent l'utilisation du modèle dans sa phase actuelle :

- D'une part, l'application de l'algorithme a nécessité la création du programme sous forme de code dont l'utilisation est un processus lourd et complexe : le développement d'un programme complet, doté d'une interface élaborée permettrait de faciliter l'utilisation de ce dernier par les concepteurs de réseaux de distribution d'eau, en particulier ceux qui doivent implanter des réservoirs sur les réseaux. Cette interface permettrait à l'étude de passer du stade de recherche à un stade d'utilisation courante.

- D'autre part, une étude complémentaire sur l'impact des phénomènes transitoires (coup de bélier) sur les réservoirs optimisés serait nécessaire pour assurer la fiabilité de la réponse du réseau.

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ANNEXES

lol_1pompe_noeud4.txt - Bloc-notes

Fichier Edition Format Affichage ?

Tank 41 sur le noeud 4

Elévation	Diamètre	Niveau Min	Niveau Max	Volume	Pression Min	Noeud	Pression Max	Noeud	Qualité Age	Noeud	Qualité tank Age	Consommation (KWh/d)	Pointe (Kw)
250.00	26.45	24.05	24.05	98834.27	55.06	16	93.61	34	120.00	31	120.00	0.00	0.00
250.00	26.55	23.87	24.13	99918.77	80.12	16	118.74	34	120.00	31	25.60	925.92	15.80
250.00	26.64	23.70	24.22	101003.29	80.05	16	118.78	34	120.00	32	18.92	924.30	15.83
250.00	26.74	23.53	24.31	102087.80	79.97	16	118.82	34	120.00	32	17.28	923.12	15.85
250.00	26.83	23.37	24.39	103172.30	79.90	16	118.86	34	120.00	32	16.67	923.20	15.87
250.00	26.93	23.20	24.48	104256.81	79.83	16	118.89	34	120.00	31	16.30	920.66	15.89
250.00	27.02	23.04	24.56	105341.33	79.76	16	118.93	34	120.00	32	16.30	919.52	15.91
250.00	27.11	22.89	24.65	106425.84	79.69	16	118.97	34	120.00	32	16.29	919.14	15.94
250.00	27.20	22.73	24.73	107510.34	79.63	16	119.00	34	120.00	32	16.25	917.21	15.96
250.00	27.29	22.58	24.81	108594.86	79.56	16	119.04	34	120.00	32	16.32	917.38	15.98
250.00	27.38	22.43	24.89	109679.37	79.50	16	119.07	34	120.00	31	16.34	919.03	16.00
250.00	27.47	22.29	24.98	110763.88	79.43	16	119.11	34	120.00	32	16.49	918.90	16.02
250.00	27.56	22.14	25.06	111848.38	79.41	16	119.14	34	120.00	31	16.25	914.06	16.04
250.00	27.65	22.00	25.14	112932.90	79.31	16	119.18	34	120.00	31	16.45	914.27	16.06
250.00	27.74	21.86	25.22	114017.41	79.32	16	119.21	34	120.00	32	16.66	916.99	16.08
250.00	27.83	21.72	25.30	115101.91	79.19	16	119.24	34	120.00	32	16.81	910.14	16.10
250.00	27.92	21.59	25.38	116186.42	79.13	16	119.28	34	120.00	32	16.83	909.26	16.12
250.00	28.00	21.45	25.46	117270.94	79.07	16	119.32	34	120.00	32	16.81	907.57	16.14
250.00	28.09	21.32	25.53	118355.45	79.05	16	119.35	34	120.00	32	17.20	911.96	16.16
250.00	28.17	21.19	25.61	119439.95	78.99	16	119.38	34	120.00	32	17.31	912.15	16.18
250.00	28.26	21.07	25.69	120524.47	78.91	16	119.42	34	120.00	32	17.12	914.16	16.20
250.00	28.34	20.94	25.77	121608.98	78.85	16	119.45	34	120.00	31	17.30	905.25	16.22
250.00	28.43	20.82	25.84	122693.48	78.80	16	119.48	34	120.00	32	17.41	906.97	16.24
250.00	28.51	20.70	25.92	123777.99	78.74	16	119.52	34	120.00	32	17.39	911.77	16.26
250.00	28.59	20.58	25.99	124862.51	78.69	16	119.55	34	120.00	32	17.57	905.01	16.28
250.00	28.68	20.46	26.07	125947.02	78.64	16	119.58	34	120.00	32	17.61	913.73	16.30
250.00	28.76	20.34	26.14	127031.52	78.62	16	119.61	34	120.00	32	17.69	907.54	16.32
250.00	28.84	20.23	26.22	128116.03	78.54	16	119.65	34	120.00	32	17.96	901.28	16.34
250.00	28.92	20.11	26.29	129200.55	78.53	16	119.68	34	120.00	32	18.15	900.68	16.36
250.00	29.00	20.00	26.37	130285.05	78.44	16	119.71	34	120.00	31	18.34	899.47	16.37
250.00	29.08	19.89	26.44	131369.56	78.40	16	119.74	34	120.00	32	18.50	899.11	16.39
250.00	29.16	19.78	26.51	132454.08	78.48	16	119.77	34	120.00	32	18.31	911.69	16.41
250.00	29.24	19.67	26.58	133538.59	78.34	16	119.80	34	120.00	31	18.75	909.45	16.43
250.00	29.32	19.57	26.65	134623.09	78.29	16	119.84	34	120.00	32	18.63	901.11	16.45
250.00	29.40	19.46	26.73	135707.61	78.25	16	119.86	34	120.00	32	18.76	897.62	16.47
250.00	29.48	19.36	26.80	136792.11	78.17	16	119.90	34	120.00	32	18.95	911.06	16.49
250.00	29.55	19.26	26.87	137876.63	78.16	16	119.93	34	120.00	32	19.16	899.91	16.51
250.00	29.63	19.16	26.94	138961.13	78.11	16	119.96	34	120.00	32	19.21	898.54	16.52
250.00	29.71	19.06	27.01	140045.64	78.70	16	119.99	34	120.00	32	19.71	902.35	16.54
250.00	29.78	18.96	27.08	141130.16	78.03	16	120.02	34	120.00	32	19.57	889.40	16.56
250.00	29.86	18.87	27.15	142214.66	77.95	16	120.04	34	120.00	32	19.52	892.84	16.58
250.00	29.94	18.77	27.22	143299.17	77.91	16	120.08	34	120.00	32	19.95	893.21	16.59
250.00	30.01	18.68	27.28	144383.69	77.87	16	120.11	34	120.00	32	20.22	892.61	16.61
250.00	30.09	18.58	27.35	145468.19	77.83	16	120.14	34	120.00	32	20.49	896.33	16.62
250.00	30.16	18.49	27.42	146552.70	77.79	16	120.17	34	120.00	31	20.92	893.33	16.64
250.00	30.24	18.40	27.49	147637.22	77.78	16	120.20	34	120.00	32	20.68	899.65	16.67
250.00	30.31	18.31	27.55	148721.72	77.71	16	120.23	34	120.00	32	20.77	883.81	16.68
250.00	30.38	18.22	27.62	149806.23	77.71	16	120.25	34	120.00	32	21.54	907.36	16.70
250.00	30.46	18.14	27.69	150890.75	77.67	16	120.28	34	120.00	32	21.33	882.50	16.71
250.00	30.53	18.05	27.75	151975.25	77.63	16	120.31	34	120.00	32	21.07	883.51	16.73
250.00	30.60	17.96	27.82	153059.77	77.60	16	120.34	34	120.00	32	20.82	884.93	16.75
250.00	30.67	17.88	27.89	154144.28	77.52	16	120.37	34	120.00	32	20.61	888.30	16.76
250.00	30.75	17.80	27.95	155228.78	77.49	16	120.39	34	120.00	32	20.49	891.16	16.78
250.00	30.82	17.71	28.02	156313.30	77.45	16	120.42	34	120.00	32	20.22	893.72	16.80
250.00	30.89	17.63	28.08	157397.81	77.42	16	120.44	34	120.00	32	20.66	907.94	16.82
250.00	30.96	17.55	28.14	158482.31	77.42	16	120.48	34	120.00	32	21.43	899.11	16.83
250.00	31.03	17.47	28.21	159566.83	77.38	16	120.51	34	120.00	32	21.19	905.09	16.85
250.00	31.10	17.39	28.27	160651.34	77.35	16	120.53	34	120.00	32	21.14	889.47	16.86
250.00	31.17	17.32	28.34	161735.84	77.31	16	120.56	34	120.00	32	21.46	889.22	16.88
250.00	31.24	17.24	28.40	162820.36	77.28	16	120.59	34	120.00	32	21.85	899.50	16.90

Annexe A - Screenshot of the data file

Annexe B - Results: Costs vs. Tank water age