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Methodologies for Ensuring Quality Requirements in Water Distribution Networks

by

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Declaration of Authorship

I hereby declare that I am the sole author of this thesis. I take full responsibility of this work and declare that the results of this study are original and based on my own work. All scientific sources used in this thesis are fully addressed and referenced.

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Abstract

Water utilities worldwide are required by government regulations to supply water with acceptable quality standards. Among the quality parameters of the water intended for human consumption, a minimum disinfectant concentration (e.g., 0.2 mg/L) should be guaranteed at all consumers' tap.

However, by the time water reaches the extremities of the water distribution network (WDN), sufficient disinfectant residuals may not be guaranteed in all the nodes. The reason for this lies in the long residence times associated with storage and transport of water in the network and the reactivity of disinfectant within pipes. As for all chemical and microbial reactions, temperature plays a key role. In fact, high water temperature enhances the growth of microorganisms and so, the disinfectant decay. Hence, the importance of monitoring the water temperature through the WDN to avoid overcoming of the limit of 25°C at the tap, as recommended by the World Health Organization (WHO).

To extend the delivery of disinfectant residuals and to ensure that a minimum target is achieved at all the users' tap, the addition of disinfectant at the source(s) of WDN can be combined with some operating measures to be implemented in the network. The choice of a technical measure depends on the network's behavior to the injection of disinfectant at source(s). For example, some WDNs can experience low residuals at the peripheral nodes of the network (also known as dead-end nodes) due to high residence times and low flow conditions that occur in these zones. In other WDNs the problem of low residuals can affect both the dead-ends and some large areas of the network.

For the prediction of residuals and the implementation of technical solutions in WDNs, various pieces of hydraulic/water quality software can be used, such as EPANET (a public domain piece of software available on the United States Environmental Protection Agency website). These mathematical models allow simulation of the hydraulics of the network and of the disinfectant decay. The EPANET software is based on the Advection-Reaction (AR) transport model that neglects the dispersion as a solute transport mechanism. However, it has been showed by previous researchers that this simplification can compromise the accuracy of water quality simulations at dead-end branches where laminar flow conditions prevail and, hence, the effect of dispersion. Besides the dispersion effect, the temporal averaging of the water demand can affect the water quality results. Temporal averaging of the water demands involves using demand patterns at an hourly resolution. Such averaging masks the actual flow patterns in WDN happening at the sub-hourly level.

Therefore, the above considerations have motivated the present research activity, which was carried out to i) understand the influence of temperature on water quality and practices adopted by some water companies to monitor water temperature in WDN, ii) propose technical solutions to the problem of low residuals in WDNs and iii) evaluate the effects of considering the dispersion transport and the pulsed nature of demand on chlorine residuals at dead-ends of WDN.

On the first point, an exhaustive state-of-the-art review on drinking water temperature was carried out, combined with the findings from a survey of international stakeholders in order to understand legislation and local practices adopted to monitor water temperature in WDN. Based on the information obtained, water temperature is often monitored at water sources and treatment plants, while there is a limited monitoring through WDN, from source to tap, despite a known effect on physical, chemical, and microbial reactions which impact water quality.

On the second point, new methodologies have been numerically analyzed to guarantee high standards of water quality in WDNs, with special focus on dead-end sections. Technical solutions to the problem of low disinfectant concentrations were implemented for two case studies, a medium-size WDN and a large-scale WDN. Specifically, a heuristic procedure to modulate nodal outflows was carried out with the objective to prevent low disinfectant concentrations at dead-end nodes in the medium-size WDN. The methodology used the EPANET software to simulate the hydraulics and chlorine decay in the network, identifying a list of critical nodes, in correspondence to which nodal outflows can be slightly increased to reduce disinfectant decay. This increase can be obtained through the opening of blowoffs all day long in proximity of dead-end nodes. In the optimization problem, the concentration of chlorine on supply and the values of emitter coefficients at critical nodes (associated with faucet/blowoff openings) were used as decisional variables. Two objective functions were considered, namely the total volume *Vol* of water delivered, and the total mass *W* of chlorine injected into the network, while meeting the minimum value of disinfectant at all nodes of 0.2 mg/L. As the objectives clearly compete against each other, the output of the optimization consists of a set of trade-off solutions (Pareto front). The post-processing of these solutions in the WDN considered proved the economic profitability of increasing nodal outflows for solving problems of low disinfectant concentrations at dead-end nodes. The lowest operational costs for the WDN were obtained using lower chlorine doses at the source and larger nodal outflows. Furthermore, the percentage of leakage in the WDN only slightly increased with the implementation of blowoffs.

In the second case study considered, a large-scale WDN, the problem of low concentrations occurs not only at dead-end nodes but also in some large (internal) areas of the network. To address this combination of problems, the modulation of nodal outflows at dead-ends was combined with the operation of booster stations in suffering bulk areas in the case of chlorine injection at WDN sources, to guarantee a minimum residual of 0.2 mg/L. Afterwards the comparison between continuous and intermittent outflows was performed in the chlorinated network. The water volume being the same, water was provided by blowoff for 24-hrs with lower water discharges or for limited durations with larger water discharges, respectively. The study ended with the investigation of switching chlorine with chloramine in combination with the continuous blowoff solution to meet the residual target at dead-ends. The comparison between the different techniques in the WDN considered was made by using the EPANET software and its Multi-Species eXtension EPANET-MSX to simulate the chlorine and chloramine decay, respectively. Results showed that both booster stations in suffering bulk areas and nodal blowoffs at dead end nodes are effective to tackle the problem of low disinfectant residuals in WDN. Intermittent blowoffs have a similar performance to the continuous blowoffs except for a minimum percentage of violation occurred for few hours per day. The use of chloramine as a possible alternative to chlorine led to an overall increase in residuals throughout the WDN and consequently to a decrease in the number of blowoffs to open and in blowoff outflows.

Ultimately, the effects of considering a) the dispersion transport and b) the pulsed nature of demand on water quality and chlorine residuals at dead-end nodes of WDN were investigated. The methodology used a public domain software, WUDESIM, based on an Advection-Dispersion-Reaction (ADR) model which includes the dispersion into the solute transport for water quality simulations at dead-ends, and a stochastic demand generator, the cor-PRP model, to simulate demand pulses at a fine time scale (5 min in the study) in the network. The chlorine outputs predicted by the dispersion model (WUDESIM) and the dispersion coupled with the demand pulses model (WUDESIM + Cor-PRP) were compared with those obtained by EPANET that neglects the effects of dispersion and of pulsed nature of demands. It was proved that neither dispersion nor the temporal demand variation has relevant effect on the chlorine residuals at dead-ends operated under turbulent flow regime, leading the three models to give similar results. Otherwise, both dispersion and demand pulses have effect on the chlorine residuals at dead-ends operated under laminar flow regime, leading to an overall, although not particularly relevant, increase in the residuals compared to ones obtained by neglecting them.

Keywords: Water distribution network, Drinking water temperature, Disinfectant residual, Modeling, Dispersion, Demand pulses.

Table of Contents

List of Abbreviation and Symbols

ADR	Advection-Dispersion-Reaction
AR	Advection-Reaction
C_{ch}	Chloramine concentration at the source/node (mg/L)
C_{cl}	Chlorine concentration at the source/node (mg/L)
c_{min}	Minimum disinfectant concentration (mg/L)
DBPs	Disinfection-By-Products
e	Emitter coefficient (L/s/m$^{1/2}$)
E	Longitudinal dispersion coefficient (m^2/s)
GDWQ	Guidelines for Drinking-Water Quality
HAAs	Haloacetics acids
HOCl	Hypochlorous acid (free chlorine)
k_b	bulk decay constant (d^{-1})
k_w	wall decay constant (d^{-1})
NCl_3	Trichloramine
NH_2Cl	Monochloramine
NH_3	Ammonia
$NHCl_2$	Dichloramine
NOM	Natural Organic Matter
PRP	Poisson Rectangular Pulse
q	Average outflow of blowoff (L/s)
Re	Reynolds number
THMs	Trihalomethanes
TOC	Total Organic Carbon
u	Average flow velocity in the pipe (m/s)

Vol	Total volume of water delivered in the network (m^3)
W	Total mass of disinfectant injected at the source(s) (kg)
WDN	Water Distribution Network
WHO	World Health Organization

List of Figures

List of Tables

Chapter 1

Introduction

1.1. Background and Gaps

A water distribution network (WDN) is an integral part of a water supply network comprising pipelines, storage facilities and associated assets to carry potable water from treatment plant(s) to final users in order to satisfy residential, commercial, industrial, and firefighting requirements.

One of the most difficult, yet critical, roles of WDN operation is maintaining microbiological safety for the protection of public health. To guarantee good standards of water quality supply at the end point of the WDN, many countries maintain a disinfection residual (commonly chlorine) within treated water during distribution (Agudelo-Vera et al. 2020). However, when disinfectant is provided at WDN source(s), it will be hardly maintained throughout the system. In fact, disinfectant reacts with organic material in the bulk water and/or with the biofilm on the surface of the pipes, resulting in a fast decay.

As for most chemical and microbial reactions, temperature plays a key role. In fact, high water temperature enhances the growth of microorganisms and so, the disinfectant decay. Hence, the importance of monitoring the water temperature through the WDN to avoid overcoming of the limit of 25°C at the tap, as recommended by the World Health Organization (WHO). The Italian regulation D. Lgs 31/2001 «Implementation of Council Directive 98/83/EC on the quality of water intended for human consumption» states that “drinking water must be fresh, acceptable to consumers and with no abnormal change”. Further investigation on drinking water temperature is essential since the water quality deterioration depends on it. On this point, the present work was carried out to address the research gap related to drinking water temperature.

On the other hand, some operational practices to ensure the safety of drinking water can be adopted. Walski (2019) provided an overview of the range of options available to maintain a disinfectant residual.

The common one is to inject large doses of disinfectant (typically chlorine) at the entry of WDN. However, even increasing the chlorine dose at source(s), the satisfaction of a minimum chlorine residual 0.2 mg/L required by regulations [40 CFR 141.72 (CFR 2000); D. Lgs 31/ 2001; WHO 2017] may not be guaranteed in all network nodes. Furthermore, it may lead to excessive chlorine concentrations near the feeding points, resulting in taste and odour problems, as well as the formation of Disinfection-By-Products (DBPs), considered harmful for public health (Clark 1998; Boccelli et al. 2003).

Among the solutions proposed in the literature, the improvement of chlorine residuals in WDNs was typically achieved by the implementation of booster stations alone (Boccelli et al. 1998; Tryby et al. 2002; Prasad et al. 2004; Lansey et al. 2007; Meng et al. 2013; Goyal and Patel 2017; Abokifa et al. 2019) or in combination with other measures, e.g. with the optimal pump (Ostfeld and Salomons 2006) and valve operation (Kang et al. 2010). Booster stations reapply chlorine at strategic locations within the distribution system to compensate for the losses that occur as it decays over time (Tryby et al. 2002). Most of these studies formulated the problem of booster stations using different optimization techniques and searching for solutions in terms of optimal injection scheduling, operation, and locations of booster stations.

Though booster stations are effective in providing a more uniform distribution of chlorine within the system, they may fail to solve the problem at the peripheral zones of the WDN, commonly known as the dead-ends. Distribution dead-end mains are characterized by intermittent low flow velocities and frequent stagnation times. They are well known problematic locations for the long and excessive residence times, leading to rapid water quality deterioration, high potential for bacterial regrowth and disinfectant residual disappearance (Abokifa et al. 2016). So far, less attention was given to solve the problem of low chlorine residuals in these zones, which often comprise 25% or more of the total infrastructure in a distribution system and tend to service a high percentage of the residential consumer base (Tzatchkov et al. 2002).

Water flushing can be a good management practice for improving water quality in the WDN (Antoun et al. 1997; Friedman et al. 2002; Kirmeyer et al. 2002; Barbeau et al. 2005). Many U.S. utilities have regularly scheduled flushing programs, while others flushed on an as-needed basis (Friedman et al. 2003).

Flushing activity involves moving water through the distribution system and discharging it through flushing devices, hydrants or blowoff ports. They can be operated in two ways i.e., continuously through a manual flusher or at intermittent times by an automated one. A blowoff is a flushing device that allows to obtain a continuous flow at low rate at dead-end node causing fewer undesired effects in terms of service pressure decrease, compared to the typical intense flushing.

The use of flushing strategies has been proposed in the scientific literature mainly 1) as a tool for the removal of deposits and contamination from water supply pipes (Carrière et al. 2005; Deuerlein et al. 2014; van Bel et al. 2019) and 2) as a first hydraulic response to contamination (Baranowski and Walski 2009; Poulin et al. 2010; Alonso et al. 2010). Few studies investigated the use of water flushing for the maintenance of chlorine residuals in WDN (Walski and Draus 1996; Xie et al. 2014). Generally, these studies used intensive flushes to obtain an overall coverage of chlorine in the network pointing out potential pressure deficiency in the WDN, as a result of the intensive flushing. To the best of our knowledge, little attention has been dedicated to solving the problem of low residuals in dead-end nodes for which low continuous flows may be more effective. On this point, the present work was carried out to address this research gap.

Regarding the disinfectant typically used in treatment plants, chlorine has been used since the 20th century to disinfect drinking water because it is simple to use, economic etc. However, chlorine has disadvantages, including the high reactivity of chlorine with Natural Organic Matter (NOM) and the production of DBPs, some of which are likely human carcinogens (Duirk and Valentine 2007; Zhang et al. 2017). This led some water utilities throughout the North America to convert to chloramine for a more stability, taste, odor, or DBPs control (Kirmeyer et al. 1993; USEPA 2012).

As mentioned, there have been various works dedicated to use operational techniques (i.e., booster stations or flushing) in chlorinated WDNs but, to the best of our knowledge, none have explored the effect of these measures on the increase in chloramine residuals in WDNs. Furthermore, little attention has been dedicated to the analysis and comparison of the effects of the various solutions, implemented alone or in a combined way, in a large-scale WDN in the case of injection at source(s) of a) chlorine and b) chloramine. On this point, the present work was carried out to bridge this research gap.

The prediction of chlorine or chloramine residuals in WDNs can be made by using hydraulic/water quality software, such as EPANET (Rossman 2000) and its extension EPANET-MSX (Shang et al. 2007). The EPANET software is based on the Advection-Reaction (AR) transport model that neglects the dispersion as a solute transport mechanism. However, it has been demonstrated that this simplification can compromise the accuracy of water quality simulations at dead-end branches where laminar flow conditions prevail and, hence, the effect of dispersion (Tzatchkov et al. 2002; Abokifa et al. 2016).

Beside the dispersion effect, the temporal averaging of the water demand can affect the water quality results. Temporal averaging of the water demands involves using demand patterns at an hourly resolution. Such averaging masks the actual flow patterns in WDN happening at the sub-hourly level.

In this view, the present work used an Advection-Dispersion-Reaction (ADR) model (Abokifa et al. 2016) coupled with a stochastic demand generator, the Cor-PRP (Creaco et al. 2015; Creaco et al. 2016), to evaluate the effects of both dispersion and demand pulses on chlorine concentrations in dead-end branches of WDN.

1.2. Aim of the Research

Keeping in mind the overview in the previous subsection, the main aim of the present work is to solve the problem of low residuals in WDNs by implementing different technical measures (booster stations and flushing blowoffs) alone or in a combined way. Furthermore, the effects of dispersion and demand pulses on the residuals were investigated.

Additionally, an exhaustive state of art of drinking water temperature and its potential impact on water quality is reported, as result of the research conducted within the “WAT-QUAL: Water Quality in drinking water distribution systems” European project. The literature review was combined with the findings from a survey of international stakeholders in order to understand legislation and local practices adopted to monitor water temperature in WDN.

1.3. Outline of the Thesis

The remaining of the thesis is outlined as follows:

- Chapter 2: presents an overview of drinking water quality in WDNs. First, a summary of the state of art of drinking water temperature, its impact on water quality and the current practices adopted to monitor it is presented followed by a review on the disinfection practice and its aspects related.
- Chapter 3: describes the water quality models used in this work for the prediction of disinfectant residuals in WDNs.
- Chapter 4: presents a methodology for the implementation of nodal blowoffs in a medium-size chlorinated WDN to solve the problem of low residuals at critical dead-end nodes. The methodology is based on the combined use of optimization and flow routing/water quality modelling of WDNs.
- Chapter 5: provides a numerical comparison between the different techniques in a large-scale WDN in the cases of injection at sources of 1) chlorine and 2) chloramine.

- Chapter 6: investigates the effects of dispersion and demand pulses on chlorine residuals in a medium-size WDN.
- Chapter 7: summarizes the main conclusions and contributions of this work.

Chapter 2

Literature Review

This chapter presents a review of the state of art related to drinking water quality in WDNs. First, a summary of the current knowledge and policies regarding drinking water temperature in WDNs is presented, in the framework of the research conducted within the “WAT-QUAL: Water Quality in drinking water distribution systems” European project. The study was presented to the scientific community in the paper "Drinking Water Temperature around the Globe: Understanding, Policies, Challenges and Opportunities", authored by C. Agudelo-Vera, S. Avvedimento, J. Boxall, E. Creaco et al. and published in Water, 12 (4) - April 2020. Afterward, a review on the disinfection practice and its aspects related is followed.

2.1. Overview

Safe drinking water is paramount for the health and wellbeing of all human populations. Water is extracted from surface and groundwater sources, treated, and then circulated through the WDN.

Distribution networks are complicated civil infrastructures, comprised of hundreds of kilometers of pipes, storage tanks, pumps, valves, and other important assets for the operation of the network system. A primary function of a WDN is to provide adequate amounts of drinking water while meeting demands and pressure requirements in the system; in this view the network must be reliable so that the required quantity of water is continuously available (Male and Walski 1990). Furthermore, it must be designed and operated to provide water of acceptable quality for human consumption (National Research Council 2006).

Current government regulations (USEPA 2010; WHO 2017) set a list of indicator parameters that drinking water must comply with. Among these parameters a detectable disinfectant residual should be guaranteed at all points of consumption (i.e., 0.2 mg/L in the case of free chlorine).

A residual disinfectant can control the regrowth of microorganisms that remain after the treatment plant and minimize the microbial interactions with pipe wall biofilms (Haas 1999a; LeChevallier 1999). A residual is also assumed to inactivate pathogens that might enter a WDN and perhaps prevent diseases (Propato and Uber 2004).

Several documented outbreaks have led to severe health consequences, including hospitalization and death. Among these frequent contaminations of *Escherichia coli* and the outbreak of *Salmonella* occurred as in MO, USA (Geldreich 1996).

Most of these systems used water with no residual, and researchers have argued that the outbreak could have been prevented or minimized by the presence of a residual (Propato and Uber 2004).

Under normal conditions, water quality gradually deteriorates due to a complex group of physical, chemical, and biological interactions occurring both with the NOM present in water and with the pipe wall leading.

As for most chemical and microbial reactions, temperature plays a key role. In fact, high water temperature enhances the growth of microorganisms and so, the disinfectant decay. Hence, the importance of monitoring the water temperature through the WDN, to prevent overcoming of the limit of 25°C at the tap, as recommended by WHO. Further investigation on drinking water temperature is essential since the water quality deterioration depends on it. In the scientific literature little attention has been paid to evaluate the impact of drinking water temperature on water quality. Furthermore, no investigation has been carried out in an international context to identify local experiences, issues, and current knowledge on drinking water temperature. On this point, the first part of the present Chapter was carried out to address this research gap.

On the other hand, the water quality may be affected by possible deficiencies in the network. These include cross-connection and back-siphonage, contamination while in storage, contamination during construction/repair, broken and leaking main, hydraulic transients such as negative pressures and subsequent intrusion of soil water (Propato and Uber 2004). In addition to these failures during normal operating conditions, the occurrence of natural disasters (e.g., floods or earthquakes) or terroristic attacks may affect the water quality integrity (Tsitsifli and Kanakoudis 2018). In these situations, bacteria may enter in the system and be transported with the bulk water.

It is clear that in all the cases mentioned above, to guarantee a disinfectant residual is essential for the water's safety and human health.

However, mainly in large networks, due to the long transport times between the source and the end users -commonly known as the "water age"- a longer time is available for the above-mentioned interactions to occur (Masters et al. 2015) thus leading to a disinfectant decay. When the disinfectant residual is below the minimum required by national standard guidelines due to the decay processes, its bactericidal function within the WDN may not be guaranteed. Therefore, the management of disinfectant concentrations within defined limits in drinking water systems is a major concern for utilities.

As highlighted in the previous Chapter, the main aim of the present work is to solve the problem of low residuals in WDNs by implementing different technical measures (booster stations and flushing blowoffs) alone or in a combined way.

In fact, in the scientific literature, little attention has been dedicated to the analysis and comparison of the effects of the operational measures, implemented alone or in a combined way, in real WDNs when a type of disinfectant is injected at WDN source(s). The most prevalent disinfectants in WDNs are the chlorine-based ones.

Prior to evaluate the effectiveness of booster stations and/or flushing activities in real WDNs, which will be illustrated in the next Chapters (Chapters 4 and 5), it is necessary to provide an overview of the disinfection practice and its related aspects. In fact, the maintenance of disinfectant residuals is also dependent on the type of disinfectant used in WDNs. On this point, the second part of the present

Chapter provides an overview of the main disinfectant used in real WDNs, the decay of disinfectant, the disinfection practices, and regulations.

2.2. Drinking Water Temperature

Whether or not a disinfectant residual is present, a variety of water quality reactions takes place between microorganisms (present in biofilms, sediments and free-floating in the water column), inorganic contaminants, such as corrosion byproducts, and nutrients. These complex reactions are influenced by source water quality (after treatment), hydraulic conditions in the WDNs (driven by customer demands), nature and condition of the infrastructure and temperature (Douterelo et al. 2019).

Water quality and hydraulics in the WDNs have been extensively studied (Douterelo et al. 2013; Liu et al. 2013; Prest et al. 2016; Blokker et al. 2017; Sharpe et al. 2019). Although little is known in practice, research has been conducted to model temperature changes in the WDNs and to determine delivered water temperature at the customer (Blokker et al. 2012; van der Zwan et al. 2012; Blokker and Pieterse-Quirijns 2013; De Pasquale et al. 2017; Zlatanovic et al. 2017). Temperature is an important determinant of water quality, since it influences physical, chemical, and biological processes, such as absorption of chemicals, chlorine decay (Monteiro et al. 2017) and microbial growth and competition processes (Prest et al. 2016). Specifically, it influences the survival and growth conditions of microorganisms and the kinetics of many chemical reactions. Temperature can influence the dynamics of microorganisms in the WDNs promoting the role of biofilms as a reservoir of opportunistic pathogens and their release into the bulk drinking water (Ingerson-Mahar and Reid 2013). Many water treatment processes (e.g., clariflocculation, filtration, ozonation) are influenced by water temperature. However, the applied hydraulic and quality models in the literature usually consider a constant temperature (DiGiano and Zhang 2004; Fisher et al. 2012). Machell and Boxall (2014) highlight the complex interaction of hydraulics (specifically water age), infrastructure conditions and water quality. They specifically show the heating effect of water during its transit through the WDNs during summer months in the UK and the route-specific nature of this. Blokker et al. (2013) also analyzed this complex interaction when studying the potential to extract thermal energy from drinking water.

Drinking water temperature can significantly increase or decrease during distribution from the source to the customer. This change is strongly influenced by the weather, the depth of installation of transport and distribution pipes, the soil type, ground water levels, presence of anthropogenic heat sources and hydraulic residence times (Blokker and Pieterse-Quirijns 2013; Agudelo-Vera et al. 2017). At the building level, drinking water temperature can also be affected by the layout of the hot water installations (Zlatanovic et al. 2017).

The Netherlands is one of the few countries with a specific regulation regarding temperature: the Drinking Water Standards (Drinking Water Directive) states that the temperature of drinking water at the customers' tap should not exceed 25°C. Within the regular tap sampling program of the Dutch utilities, in the relatively warm year of 2006, it was reported that 0.1% of samples exceeded the 25 °C limit. With global warming and increasing urbanization, it is expected that the quantity of samples that exceed the temperature limit will increase.

Over the last decade, Dutch drinking water companies have been researching the impact of drinking water temperature in their WDNs to guarantee high drinking water quality and to prepare the infrastructure for the challenges that climate change may pose. Despite its importance, according to our best knowledge, only a few researchers (Blokker and Pieterse-Quirijns 2013; Piller and Tavard 2014) have developed and published a validated model about how the drinking water temperature changes in the distribution network. In The Netherlands, it was shown that the water temperature at the tap approaches the temperature of the soil that surrounds the distribution mains (pipes with a diameter of 60–200 mm, typical residence times of 48 h or more and located at a depth of 1 m) (Blokker and Pieterse-Quirijns 2013). In the urban environment, temperatures easily approach the 25°C limit during a warmer than average summer. Locally, under the influence of anthropogenic heat sources such as district heating pipes or electric cables, the temperature in the WDNs can temporally and locally be higher than 25°C (Agudelo et al. 2017). Yet, there remains a paucity of information regarding drinking water temperature in the WDNs, especially in countries where temperature limits are not enforced.

The following two sections provide a comprehensive summary of the current knowledge about drinking water temperature from source to tap, as well as a comparison between the policies and practices adopted in a number of countries. Challenges for drinking water companies and policy makers are formulated, resulting in identification of future research directions.

Two methods were used to gather data. A survey was performed to identify local experiences, issues, and current knowledge. A questionnaire was sent to 18 participants of the European Project WatQual (www.sheffield.ac.uk/civil/wat-qual) in August 2018. Participants were researchers from universities or employees of water utilities. The questionnaire contained twelve open questions regarding legislation, practices, knowledge, and data about drinking water temperature in the WDNs. A literature review was conducted to determine the current scientific knowledge about the potential impact of water temperature on the WDNs.

2.3. Drinking Water Temperature from Source to Tap

Monitoring Practices

Most of the surveyed water companies systematically monitor source water temperature, and/or the temperature of the treated water (Table 2.1), as an operational parameter. However, the temperature from source to tap is not systematically monitored in most of the surveyed countries. In the countries where the temperature is monitored, the results are often from discrete samples, resulting in data as shown in Table 2.1.

A few countries monitor water temperatures at the tap. From the surveyed countries, Czech Republic, France, The Netherlands, Serbia, and the UK monitor the tap water temperature. This monitoring is usually random, and a standard thermometer is used. These samples collected at customers’ taps are discrete data sets and are very temporally and spatially sparse.

Table 2.1. Overview of recorded temperatures SW = surface water, GW = ground water, MW = Mix of GW and SW, RTD: Random Day Time.

Country	Source	Water treatment plant	At the customer
Colombia [a]	13 – 28 °C	16 – 26 °C	25 – 28 °C
Czech Republic	GW: 6 – 15 °C [b]	SW: 4 – 11 °C [c]	MW: 2 – 24 °C [d]
France		GW: 12 °C [e]	RDT: 10 - <25 °C [f]
Italy		6 – 15°C [g]	
Netherlands [h]		SW: 2 – 23 °C GW: 12 – 13°C	RDT: 4 – 25 °C
Serbia		9 – 16 °C [i] SW: 6 – 27 °C [j] GW: 12 – 18 °C [j]	5 – 18 °C [i]
South Africa		10 – 28 °C [k]	20.5 – 24.5 C [l]
Spain		10 – 29 °C [m]	
United Kingdom	SW: 1-21°C [n]	SW: 2 – 26 °C [o] GW: 10 – 18°C [o] MW: 2 – 23 °C [o] SW: 3 – 24 °C [p] GW: 11 – 12°C [p] MW: 6 – 22 °C [p]	SW: 3 – 25 °C [o] GW: 4 – 27°C [o] MW: 4 – 26 °C [o]

[a] City of Cali – At the source and water treatment plant: daily measurements, years: 2017-2018, at the tap: nine water samples collected in different days (Montoya-Pachongo et al. 2018)

[b] City of Vsetín, Czech Republic – ground water source, bank infiltration from Bečva river, year 2018 - 2019

[c] City of Vsetín, Czech Republic – WTP from valley reservoir Karolinka, years 2018 – 2019

[d] City of Vsetín, Czech Republic – costumer's tap in the city center, years 2018 – 2019

[e] At Strasbourg – ground water. For other location, it can excess 25°C in some situations.

[f] ARS 2020 http://www.eaupotable.sante.gouv.fr. Exceedances of the reference temperature (25°C) on the water of the distribution networks are frequent in the summer period (2017 results: 138 non-compliant values out of 800 samples taken in June, July, and August and 3500 during the year; source ARS)

[g] Campania, Southern Italy

[h] Rotterdam, tap samples - RDT, years 2008 - 2012

[i] Measurements in the city of Pancevo, Serbia, between Feb. 2017 – Jan. 2018 at three locations: two at the city center and the third a village nearly 18 km from the WTP

[j] Belgrade, Serbia – Years 2013-2018

[k] Non systematically monitored.

[l] Jacobs et al. 2018.

[m] City of Murcia, Spain. Year 2009. Measurements in the water treatment plants and in the network.

[n] Dŵr Cymru Welsh Water – years 2010 - 2017

[o] Anglian Water – Years 2018

[p] Bristol Water – Years 2018 – daily measurements

Only in The Netherlands and the Czech Republic temperature is measured and recorded to comply with regulatory reporting requirements. In other countries, it is common that temperature is measured when discrete samples are collected at customer taps, for example, as part of chlorine residual measurements, but the values are not typically recorded or reported.

In The Netherlands, the reading is made from the closest tap to the water meter (usually in the kitchen sink on the ground floor). The stagnant water in the domestic installation is flushed; after flushing, the temperature stabilizes, and it is recorded. In the UK, the standard procedure for random day time sampling is to run the tap for one minute prior to sample collection. In France, The Regional Health Agency (French ARS) randomly checks water temperature at consumers' water taps, where number and frequency of measurements depend on the size of the water utility. In the Czech Republic, the analysis at the consumer's tap also includes measuring the water temperature. The results of the analyses are then electronically sent to the common national information system (IS PiVo). The IS PiVo database was created in 2004 as a tool of hygienic service for water quality monitoring in the Czech Republic. All operators of public water supply systems are obliged to monitor the quality of drinking water by law. The results are provided electronically and processed statistically on an annual basis (Novakova and Rucka 2019). Table 2.1 shows the range of measured drinking water temperatures in the surveyed countries.

Drinking Water Temperature at the Source

Source water temperatures have a limited impact on the temperature at the tap. Measurements in The Netherlands have shown that temperature at the customer's tap is mostly determined by the temperature of the soil around the distribution mains (typically at 1.0 m depth in The Netherlands and much of the world), independent of the water source type. Figure 2.1 shows two supply areas with different water sources: one with a ground water (GW) source, one with a surface water (SW) source; the temperature profiles are unique for each source. Water temperatures at the tap for these areas were also analyzed and showed similar temperatures with a seasonal pattern between the two different water sources. These results confirm that the water temperature at the tap is to a limited extent determined by the temperature at the source/outlet of water treatment plant (WTP).

Drinking water temperature at the point where source water (after treatment) enters the WDNs is determined by the type of source water (ground water or surface water) and the characteristics of the facilities where the water is treated, and treated water is stored. As a general rule, groundwater temperature is mainly stable over the year. For example, groundwater temperature in The Netherlands is around 12–13°C, but seasonal temperature variations can be higher if the source is close to a river and groundwater is influenced by riverbank filtration (see Table 2.1, data for ground water in Belgrade, Serbia). Meanwhile the surface water temperature has high seasonal variations, and its typical pattern is shown in Figure 2.1. Table 2.1 shows an overview of variations of water temperature after treatment from different sources, recorded in various countries.

<u>Drinking Water Temperature in the Transport and Distribution System</u>

The temperature gradient between soil surrounding the water main and water in the pipe drives temperature change in the WDNs. The temperature of the shallow underground soil (1–2 m depth), where drinking water mains are often installed, shows seasonal variations. The ‘frost depth’ is the depth to which the ground water in the soil is expected to freeze in subzero conditions, and it depends on climatic conditions. Frost depth is considered in many countries to determine the minimum installation depth of drinking water mains to avoid freezing of water in the pipes, or breaking pipes from freezing and thawing of the soil around the pipes. Typical installation depths in central Europe vary between 0.8 m and 1.5 m, whereas in countries such as Finland at higher latitudes, installation depths increase, up to 2.5 m. In other countries, where frost is not an issue, the minimum depth of the trenches is determined in such a way that the pipes are protected from traffic and external loads. In Cali (Colombia) an installation depth between 1.0 m and 1.5 m was reported. In Spain, for instance,the minimum depth will be such that the upper border of the pipeline is at least one meter from the surface; under sidewalks it should be a minimum of 0.60 m. In South Africa, the cover should be no less than 0.9 m (Standard 2011), although older South African standards stipulated 0.6 m minimum cover. Pipes in South Africa are typically installed at approximately 1.5 m from the surface. Water reticulation design guidelines provided by WaterCare in New Zealand suggest 1.0 m cover in roads and 0.75 m in berms and open country (Services 2013).

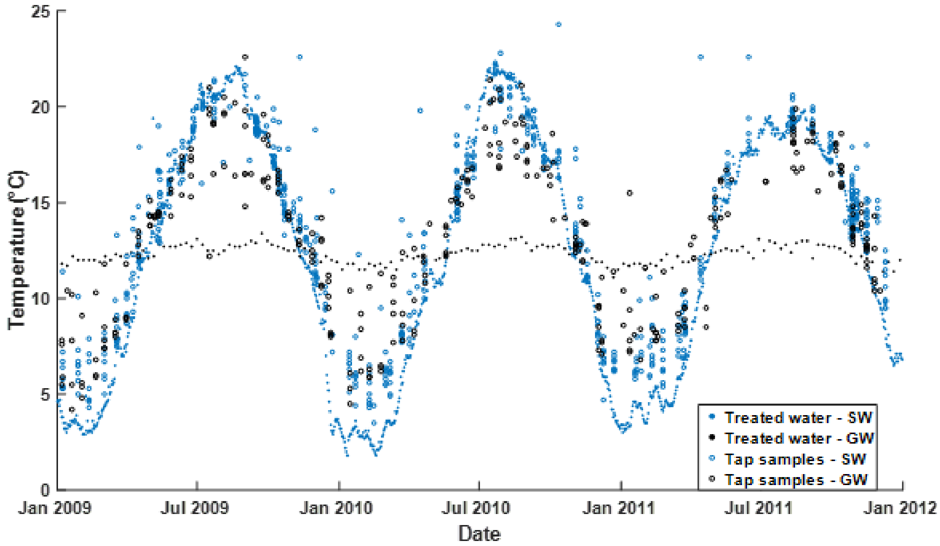


Figure 2.1. Measured water temperature at two pumping stations in the Netherlands one from surface water (SW) and the other from ground water (GW) and the respective temperatures at the tap measured at random locations in the separate WDNs (Agudelo-Vera et al. 2015).

Soil temperature is influenced by the weather (air temperature, solar radiation, etc.), the environment (rural vs. urban), land-cover (bitumen/tar vs. natural vegetation), soil type and conditions (sand vs. clay and moisture content), as shown below. The energy transfer rate from the soil to the inner pipe wall is determined by the conductivity of the pipe material and the thickness of the pipe wall. Subsequently, the energy is transferred from the inner wall to the flowing water. Within a few hours, drinking water reaches the surrounding soil temperature, depending on factors such as the pipe diameter, wall thickness and flow velocity. Based on the equations presented by Blokker and Pieterse-Quirijns (2013) it is possible to calculate the time needed to warm up the water contained in a pipe of a certain diameter, given an initial drinking water temperature and the soil temperature. Figure 2.2 shows the number of hours needed for drinking water in distribution pipes to heat up from 15°C to 25°C and number of minutes in connection pipes to warm up from 20°C to 25°C. Plastic and asbestos cement pipes are thermal insulators, and this means a relatively long heating time. Cast iron pipes, even with cement lining, show a much shorter time for the water to heat up from 15 to 25°C for the same diameters, e.g., less than 1 h for a 150 mm cast iron pipe with cement lining (Blokker and Pieterse-Quirijns 2013).

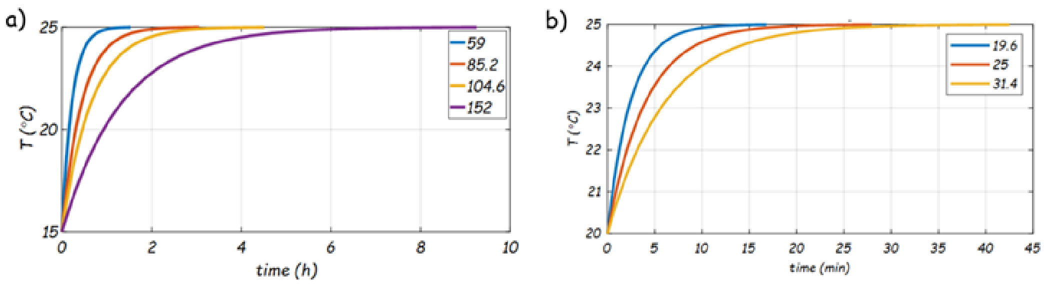


Figure 2.2. Heating up of the drinking water temperature in (a) PVC distribution pipes with inside diameters between 59 mm and 152 mm. Original water temperature is 15 °C and soil temperature is 25°C; (b) plastic connection pipes with inside diameters between 19.6 mm and 31.4 mm. Original water temperature is 20°C and soil temperature is 25°C.

The term "urban heat island" describes built up areas that are hotter than surrounding rural areas due to limited evapotranspiration, heat storage in buildings and urban surfaces, and anthropogenic heat sources. Sources of anthropogenic heat include cooling and heating of buildings, manufacturing, transportation, lighting, etc. (Mihalakakou 2002; Herb et al. 2008). Recently it was proven that the temperature of the shallow underground is also strongly influenced by anthropogenic heat sources such as district heating pipes, electricity cables, underground parking garages, etc. and it can lead to which is known as the 'subsurface heat island effect' (Menberg et al. 2013a; Menberg et al. 2013b; Muller et al. 2014). Analysis of German cities has shown that superposition of various heat sources leads to a significant local warming (Menberg et al. 2013a). Measurements of soil temperatures in The Netherlands have shown that soil temperatures at depth of 1.0 m in a warmer than average summer with a heat wave can reach very local up to 27°C and can heat up at a rate of 1°C per day, in so-called 'hot-spot' locations. Examples of 'hot-spot' locations are industrial areas with large anthropogenic heat sources, with no vegetation and good drainage that prevents infiltration and fully exposed to the sun radiation (Agudelo-Vera 2017).

Blokker et al. (2012) modelled drinking water temperature in the WDNs using EPANET-MSX (Shang and Uber, 2008). The use of EPANET-MSX facilitates the calculation of temperature at each node in the distribution network. The model was developed assuming a constant soil temperature over 24 h. Figure 2.3 shows that tap temperatures vary from 10°C close to the WTP to 25°C further downstream. Machell and Boxall (2014) reported measured temperatures in the networks and showed that temperature increases with increasing water age along flow routes. Figure 2.4 shows different pipe routes for a network with two Service Reservoirs (SRs) and demonstrates a range of temperature increases. Although several soil temperature models for rural areas have been proposed, little is known about the soil temperature profile in urban areas. A one-dimensional soil temperature model was developed by Blokker and Pieterse-Quirijns (2013) and extended by Agudelo-Vera et al. (2017) to include anthropogenic heat sources, as seen in Figure 2.5.

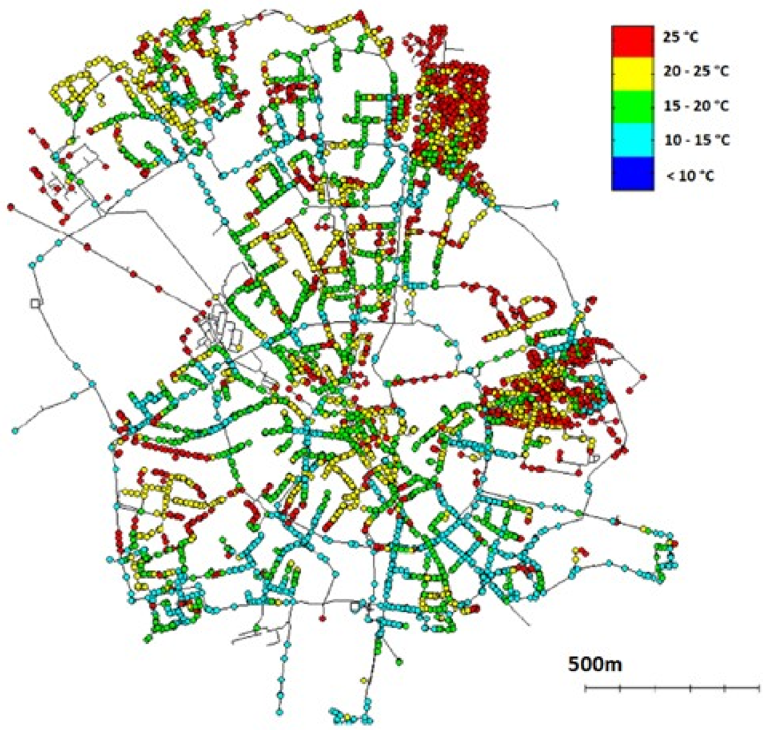


Figure 2.3. Example of simulation of the temperature in the WDNs (Blokker et al. 2012).

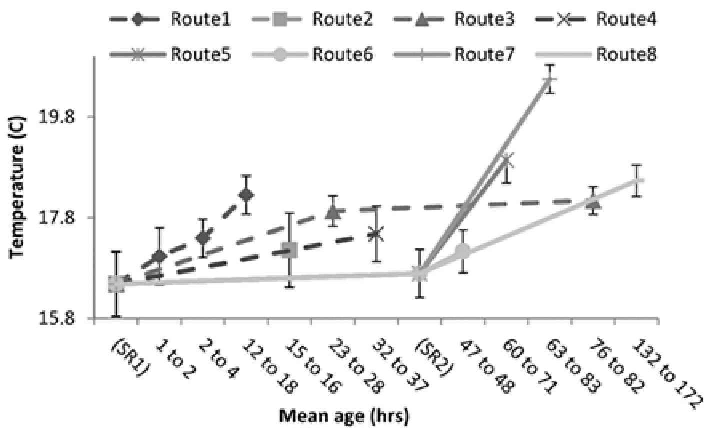


Figure 2.4. Measured temperature versus the calculated water mean age along flow routes for a network with two Service Reservoirs (SRs), with permission from the American Society of Civil Engineers (ASCE) (Machell and Boxall 2014).

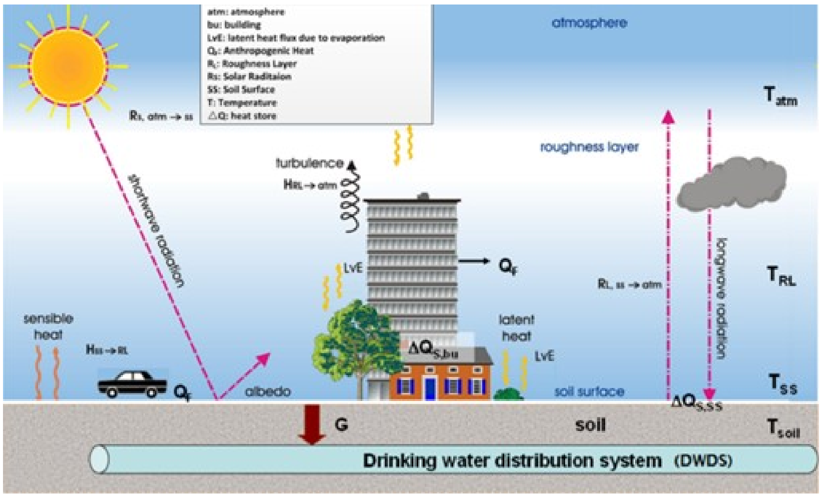


Figure 2.5. Schematic representation of the one-dimensional soil temperature model, with permission from Copernicus Publications (Agudelo-Vera et al. 2017).

<u>Drinking Water Temperature in the Domestic Drinking Water Installation and at the Tap</u>

WDNs are the final step in the supply of drinking water to consumers. Drinking water temperatures are generally higher in households and buildings than in the distribution system. Drinking water temperature in the domestic drinking water installation can increase due to pipes installed through heated rooms or nearby heat sources (Lautenschlager et al. 2010; Lipphaus et al. 2014). Zlatanovic et al. (2017) developed a model to simulate the temperature in WDNs. The model showed that inlet water temperature and ambient temperature both have a large effect on the water temperature at the household tap.

Drinking water temperature at the customer in tropical countries could be even higher than the limit of 25°C (see Table 2.1); nine water samples collected on different days in the city of Cali (Colombia) resulted in temperatures between 25°C and 28°C (Montoya-Pachongo et al. 2018). Measurements recorded at a particular bath outlet in Cape Town, South Africa and taken at 05h00, 13h00 and

15h00 each day showed that the ultimate afternoon reading of temperature at the cold water tap varied from one day to the next with a range of ± 6°C over a period of 10 days (Jacobs et al. 2018). Spot measurements made in summer with the cold water end-use temperature in one home peaked at 34°C in an afternoon on December 2018, after a few seconds of the tap running. Temperatures up to 41°C degrees have been measured during the first 10 s after opening the cold water tap during a very hot midsummer day in Cape Town (January 2020) with an outside air temperature of 42°C (Jacobs 2020). These relatively high values could be ascribed to the shallow buried plumbing pipe (300 mm ground cover) passing around the Northern side of the house in the full sun (Jacobs et al. 2018). Drinking water temperature without flushing in the WDNs can reach the indoor temperature, in countries where homes are typically not climate controlled, such as South Africa.

<u>Drinking Water from Source to Tap</u>

Water has a relatively large heat capacity; therefore, considerable amounts of energy are required to heat up water. Additionally, water has a relatively high heat transfer coefficient, so it takes some time for the water to heat up; note that the time required to reach a certain temperature is decreased by convection (i.e., flowing water enhances heat transfer). A heat transfer model can calculate that it takes tens of hours to heat up water in a reservoir or a transport main (pipe diameter 300–800 mm), a few hours in a distribution pipe (diameter 60–150 mm), and a few minutes in a property connection pipe (diameter 15–30 mm). This is shown in Figure 2.2 and in Blokker and Pieterse-Quirijns (2013). This simple heat transfer model assumes that the driving force is the temperature at the pipe wall, which is not affected by the temperature of the drinking water. This means that the temperature of the pipe wall can be assumed to be equal to the undisturbed soil temperature at installation depth. The undisturbed soil temperature can easily be determined by a one-dimensional micrometeorology model. However, there is a heat exchange between soil and drinking water.

However, as drinking water pipes distribute water of varying temperatures (5–25°C throughout the year due to seasonal variation), the soil temperature around the drinking water pipe is also affected by the drinking water temperature. As the pipes are installed for a long period of time, it can be expected that the soil temperature around the pipes is not always equal to the undisturbed soil temperature. Thus, the soil temperature around the drinking water pipe is also affected by the drinking water temperature. The interactions between and within the soil temperature and water temperature are complex. The effect of soil temperature on short and long wave radiation, surface convection, and heat transfer through the soil need to be considered in combination with the effect of drinking water temperature, which is difficult to model. The weather-related variables have a seasonal temporal resolution, whereas the drinking water temperature could change in a few hours depending on the flow rate of the water through the pipe.

Given the above and considering the typical residence times of water in the various parts of the network between source and tap, drinking water temperature at different locations between the source and a tap is estimated as follows:

Drinking water temperature at source or treatment plant (Table 2.1): Temperature is often measured here, and hence known. Ground water temperature at the source will be relatively stable (e.g., 12–13°C in The Netherlands and U.K./Bristol) yearround, and surface water source temperature can vary substantially between 2 and 27°C.

Drinking water temperature in the transport main: Typically almost equal to source/treatment plant temperature (difference of ± 1°C). Firstly, these mains have a large diameter and are usually short enough for the residence time to be much smaller than the heating time given in Figure 2.2. Secondly, these large mains substantially influence the surrounding soil temperature, which means there is a limited net heat exchange between the soil and water in the pipe. Furthermore, these mains are typically installed deeper than distribution mains, hence the soil temperature is less affected by the weather.

Drinking water in SRs/tanks: The large volume to surface area of most SRs compared to pipes leads to slower heating/cooling effects during the residence within these critical structures. However, they often have very long residence times. Figure 2.4 shows the relative impact of flow routes, including a second large SR to retard heating effects during the summer in the UK. It should be noted that this was for an underground tank in a hilly area. Underground tanks are affected by ground temperature as with the buried pipes. Where topology is flatter, such tanks are typically elevated above the ground. In above-ground reservoirs, heating and cooling effects can be very significant due to bigger and more rapid variations in air temperature than in soil temperature. Temperature in the reservoirs can be also affected by material and features of the insulating layer. However, there is not enough data to quantify the level of difference.

Drinking water temperature in the distribution mains: Typically quickly approaches the undisturbed soil temperatures at installation depth (i.e., 1.0 m).

These mains have a limited diameter, where the residence time is greater than the heating time from Figure 2.2. As these mains influence the surrounding soil temperature to a limited extent, the actual heating time may be longer than that shown in Figure 2.2, but the residence times have the same order of magnitude, so there is significant heat exchange. These mains are typically installed at a depth of 1 m, where the soil temperature is subjected to seasonal change.

Drinking water temperature in the connection water supply pipes: Typically almost equal to the temperature at the end of the distribution main (so soil temperature at depth of 1 m). Firstly, these small diameter mains have short lengths, where the residence time (during flow, the situation of stagnant water is kept out of the analysis) is much smaller than the heating time. These small mains hardly influence the surrounding soil temperature, and if they do, the equilibrium would be towards the temperature of the distribution mains. These pipes are typically installed at a shallower depth than distribution mains, so the soil temperature is more influenced by the weather.

Drinking water temperature in the premises plumbing pipes: Typically almost equal to the temperature at the end of the connection and thus of the distribution main (i.e., soil temperature at depth of 1 m). These small diameter mains have short lengths, hence their water residence time (again during flow) is much smaller than the heating time. These mains are not located in the soil, but in airshafts, and the air temperature is not affected by the drinking water temperature of these small-diameter pipes.

Drinking water temperature at the tap: Typically (during flow after flushing) equal to temperature at the end of the distribution mains (i.e., soil temperature at the depth of 1 m) when customers are directly connected to the network. For situations where storage occurs between the distribution network and the customer's tap, other temperatures apply depending on the type of storage (roof or underground), local climate and storage times. Stagnant water will reach the surrounding temperature. Consequently, it is clear that the soil temperature at the installation depth of the distribution mains is important to know. This temperature is determined by, on one hand, short and long wave radiation (including from above ground anthropogenic sources), surface convection, and heat transfer through the soil and, on the other hand, by the underground (anthropogenic) heat sources. As the anthropogenic sources can have a local effect, it is not easy to predict drinking water temperatures in the entire network. Tap temperatures are not typically measured (Table 2.1), and in the soil/ground water, only on a project basis. Therefore, the soil temperatures at installation depth are mostly unknown.

2.4. Consequences of Higher Temperatures and Legislation

The WHO guidelines recommend a maximum temperature limit of 25°C at the tap (WHO 2006): "Cool water is generally more palatable than warm water, and temperature will impact on the acceptability of a number of other inorganic constituents and chemical contaminants that may affect the taste. High water temperature enhances the growth of microorganisms and may increase taste, odor, color and corrosion problems". In a recent review the WHO reports that in a survey of 104 countries, 18 countries have a regulatory/guideline value of temperature (WHO 2018). This review also states that "None of the values for temperature were mandatory, being guiding levels or operational goals. None of the countries and territories' documents indicated what would happen if temperatures rose above the suggested value. In addition to those with numerical values, seven countries and territories had descriptive levels such as: 2.5°C above normal; "not objectionable"; "air temperature plus 3°C"; "acceptable"; and "ambient". No additional information about the countries or the type of standard is given. In the survey conducted for this work, a number of legal standards were identified, as summarized in Table 2.2.

Table 1.2. Legal standards and monitoring of the surveyed countries.

Country	**Legal standard for drinking water temperature**	**Legal standard for *Legionella***
Colombia	No legal standard	No legal standard
Czech Republic	Decree No. 252/2004 Coll. Decree laying down hygiene requirements for drinking and hot water and frequency and scope of drinking water control. The recommended temperature of drinking water at the costumer's tap is between 8 to 12°C.	Decree No. 252/2004 Coll. Decree laying down hygiene requirements for drinking and hot water and frequency and scope of drinking water control. This indicator is only set for hot water, where the limit of 100 HTP/100 ml is mandatory. This is the limit that applies to health and accommodation facilities, hot water supplied to showers of artificial or natural pools and drinking water used for hot water production; for other buildings, it is the recommended value to be sought through technical measures. The limit 0 HTP/100 ml as the highest limit value applies to wards of hospitals where immunocompromised patients are located
France	The temperature at the consumer tap should be less than 25°C (decree from 11 January 2017) in metropolitan France.	For water heating systems of public premises (hospital, hostel, camping, retreat houses, etc.) and cooling towers there is a regulation for environmental monitoring of *Legionella*. Since 1 August 2012, monitoring has been based on culture methods (as per Standard NF T90-431 "Detection and enumeration of *Legionella spp*. and of *Legionella pneumophila* by culture in agar media". However, there are several detection and enumeration methods for *Legionella* that are under development or that are currently in use to greater or lesser extents. Since January 1, 2012, monitoring is mandatory on hot water

		networks for establishments receiving the public ANSES (French Agency for Food, Environmental and Occupational Health & Safety).
Italy	No legal standard. However, it is recommended that temperature should range between 12°C to 25°C (Rapporti ISTISAN 97/9, Istituto Superiore della Sanità)	National guidelines from Conferenza Stato-Regioni del 07 maggio 2015. Drinking water temperature must be controlled to be outside of the critical range 20°C-50°C to prevent *Legionella* infections.
Netherlands	The Dutch Drinking Water Directive contains a maximum temperature limit of 25°C at the tap (Drink Water Directive).	National guidelines concerning prevention of *Legionella* infections that state the drinking water temperature in a building may not exceed 25°C, and hot water temperatures must be at least 55°C (Publicatie, 2012).
Serbia	Drinking water quality standards (Official gazette of FRYu, No. 42/98 and 44/99, Official gazette of RS No. 28/19), temperature at the consumer is not set, but there is a requirement that it shall not be higher than the temperature at the source.	No standards
Spain	No standards	There are two laws that establish some parameters related to *Legionella*: a) Royal Decree 140/2003 of February 7th (Real Decreto 140/2003, 2003) establishing the sanitary criteria for quality of water for human consumption. In this law there is no mention to temperature nor *Legionella* at all but fixes all the values applied to suitable drinking water. It also fixes that sampling protocols for every water company. b) Royal Decree 865/2003, of 4 July (Real Decreto 865/2003, 2003) establishes hygienic-sanitary criteria for the prevention and control of Legionnaires' disease. The aim of this law consists in preventing and controlling legionellosis by adopting hygienic and sanitary measures in those facilities where *Legionella* can proliferate and spread. In this sense, it focuses on hot water facilities inside the buildings. The Building Technical Standards (CTE from its initials in Spanish, for the design of plumbing installations inside

		buildings CTE-DB H4 are based on the aforementioned law. There is a non-mandatory recommendation for drinking water to be under 20°C where weather conditions allow.
South Africa	No standards	No standards around the presence of *Legionella* in drinking water. The National Institute for Communicable Diseases (Carrim et al. 2016) recommends: "The proper design, maintenance and temperature of potable water systems are the most important method for preventing the amplification of *Legionella*. Hot water should be stored above 60°C and delivered to taps above 50°C. Cold water should be stored below 20°C, and dead legs or low flow areas eliminated." Legionnaires disease is a notifiable health condition (compulsory notification) in South Africa.
United Kingdom	No standards. The Water Fittings Regulations Guidance book advises to try and keep water supplied to 20°C as a maximum.	Health and Safety England (HSE) have produced a document which is an "Approved Code of Practice" regarding controlling *Legionella* in water systems. The risk assessment, prevention and control of *Legionella* falls under the 1974 Health and Safety at Work Act (HSWA) and a framework for this assessment is covered by the Control of Substances Hazardous to Health Regulations 2002 (COSHH) (Britain, 2002). Guidelines suggest control measures of: • Cold water stored <20°C and distributed to all outlets at <20°C within two minutes of operation • Hot water stored at 60°C and distributed to outlets at > 50°C within 1 minute of operation

Factors such as nutrient concentration, temperature and pH determine microbial community structure and potential for regrowth within WDNs. Consequently, changes in temperature in WDNs can influence microbial community composition, promoting the presence of pathogens and the potential for microbial regrowth, particularly of biofilms in the pipe environment (Agudelo-Vera et al. 2015; Preciado et al. 2019). A temperature increase of drinking water can influence the microbial ecology of WDNs, affecting parameters such as potential growth (e.g., colony count at 22°C, bacteria of the coli group and Legionella) and the presence of undesirable microorganisms because of their possible role in disease (Agudelo-Vera et al. 2015). There is a difference in the effect of temperature on microorganisms depending on location, either as free-living planktonic organisms in the bulk-water, or as a community within a biofilm attached to the pipe wall. The effect of temperature may also depend on water quality (e.g., disinfectant residual, organic loading) and hydraulics. Some microorganisms have their optimal growth at 20°C, others at 25°C, and yet others at 30°C. Thus, the temperature will affect the composition of the biofilm. However, publications about microorganisms in water supplies in many cases do not provide accurate data on water temperature (van der Kooij and van der Wielen 2013). It has been shown in a chlorinated WDNs in the UK that a rise of temperature from the average 16°C in the warmer months to a temperature of 24°C promoted changes and loss in the complexity of microbial biofilm communities (Preciado et al. 2019).

The main concern regarding the impact of temperature increases in WDNs is the potential for the proliferation of pathogens such as Legionella spp. Legionellosis is a collection of infections that emerged in the second half of the 20^{th} century, and that are caused by Legionella pneumophila and related species of bacteria belonging to the genus Legionella. Water is the major natural reservoir for Legionellae, and these bacteria are found worldwide in many different natural and artificial aquatic environments, such as cooling towers, water systems in hotels, domestic water heating systems (Stone et al. 2019), ships and factories, respiratory therapy equipment, fountains, misting devices, and spa pools (WHO 2007). Whether or not disinfectant is used, controlling Legionella spp. in a drinking water installation can be problematic (van der Lugt et al. 2019). Temperature control is a known measure to prevent the proliferation of Legionella. The WHO states that to prevent Legionella infection, the recommended temperature for storage and distribution of cold water is below 25 °C, and ideally below 20°C. Table 2.2 shows that this recommendation has not been adopted everywhere. Table 2.2 also shows that temperature standards of building owners are not always matched with temperature standards for drinking water utilities. Laboratory studies of mutant Legionella strains show that the bacteria may grow below 20°C under certain conditions (Soderberg et al. 2004). Legionella will survive for long periods at low temperatures and then proliferate when the temperature increases if other conditions allow.

When temperatures remain below 25°C, it is expected that growth of Legionella pneumophila will not occur or will be limited, whereas at temperatures above 30°C, it is likely that growth of Legionella pneumophila will occur at significant levels, providing the biofilm concentration in the drinking water distribution system is high enough. Another prerequisite for the significant growth of Legionella pneumophila, is that the temperature has to be higher than 30°C for a prolonged period, reported as more than seven days (Agudelo-Vera et al. 2015).

The results of the survey conducted herein showed that seasonal increase of temperatures can cause unpleasant taste on the palate, which may be related to pipe material (e.g., black alkathane pipework, or lead plumbing pipes). Drinking water companies are generally aware that potential issues can include the occurrence of infections (such as Salmonella, Legionella, Mycobacterium), chlorine decay and formation of byproducts. As expressed in one survey response

“. . . it is known that increased water temperature leads to increased biofilm activity in distribution network”. Research in The Netherlands on the influence of temperature on discoloration risk, concludes that it is likely that higher temperatures in the WDNs can augment discoloration risk (Blokker and Schaap 2015). In a tropical WDNs in the city of Cali (Colombia), the formation of disinfection byproducts was clearly influenced by pH, temperature, chlorine dosage, and water age. The interactions observed between these parameters and Trihalomethanes (THMs), were also shaping the microbial characteristics of these systems (Montoya-Pachongo et al. 2018). Other studies regarding the effects of temperature in the WDNs are reported in Table 2.2.

Table 2.2. Scientific studies on the effects of temperature in the distribution network or at the tap.

Aspect	Location	Reference
Changes in bacterial dynamics	Network	[1,2-4]
Increased chlorine decay	Network	[5,1,6]
Increased discolouration risk	Network/tap	[1,7,8,9]
L. pneumophila & opportunistic pathogens		[10]
Seasonal shifts in bacterial communities	Effluents of treatment utilities	[11]
Trihalomethanes propagation in WDNs	Network	[12]

References: 1 = Machell and Boxall 2014; 2 = Francisque et al. 2009; 3 = Niquette et al. 2001; 4 = Vital et al. 2012; 5 = Monteiro et al. 2017; 6 = Li et al. 2003; 7 = Blokker and Schaap 2015; 8 = van Summeren et al. 2015; 9 = Sunny et al. 2018; 10 = van der Wielen and van der Kooij 2013; 11 = Pinto et al. 2012; 12 = Li and Zhao 2006.

Trends

Increasing urbanization and climate change seem to be the most important current trends affecting drinking water temperature. The ‘urban heat island’ has been an object of studies during the last decades, but only recently was it shown that it also affects the shallow subsurface, where WDNs pipes are located. In an urban environment with numerous anthropogenic heat sources, the ground is warmer than it is in a rural area. This also influences the temperature of drinking water and therefore the water quality. Although the biggest impacts of climate change will be felt many years from now, it is important to consider the long life-span of a water distribution network and the potential impacts on infrastructure integrity and water quality management. The replacement of water mains offers the opportunity to improve the network by, among other things, starting to take the impact of climate change into account now. For places where replacement is not feasible and considering that climate change and water shortages are likely to influence the way water is used and stored, it is important to understand the potential consequences of elevated temperature to manage their risks in alternative ways.

Currently, during hot summers, there are concerns when water temperatures exceed 20°C due to the increased risk of Legionella proliferating in premises water systems. With climate change and urbanization, it is expected that drinking water temperatures will rise (Levin Ronnie et al. 2002; Agudelo-Vera et al. 2015). As there is hardly any monitoring being done, it is not easy to actually see this trend occurring. The effect of higher water temperatures (on health, organoleptic parameters) is not known. In some countries, this means that legislation is on the "safe side" and limits the drinking water temperature to 25°C. However, it is not easy to guarantee water supply below this temperature. Firstly, there is no monitoring program, so compliance is largely unknown and hard to enforce. Secondly, when there is a noncompliance, there is no easy operational measure available to resolve the issue. Flushing can work locally, but at the network scale, the system may not have enough pressure capacity to drastically shorten the residence times (Blokker and Pieterse-Quirijns 2012) and it provides only short or very short-term amelioration. Forensics to quickly determine where the problem is introduced upstream do not exist and when the problem location is determined it may be expensive to solve, too late to react and difficult to determine where the liability lies. Thirdly, when there is a large noncompliance, i.e., the problem is not local but instead occurring in the whole network, there is no operational measure available at all to resolve the issue. The only solution would be to install pipes deeper or take other (large scale) design and installation measures to ensure less effect of climate change or urbanization on high soil temperatures. Alternatively, we could accept the inadequacy of WDNs and, for example, advocate solutions such as point-of-treatment via small packaged UV systems. Such systems are commercially available and, anecdotally, increasingly common in countries such as South Korea. However, the social, moral, and regulatory implications of such an approach are dramatic and far-reaching.

Another factor to consider is the increasing use of smart appliances and other water saving/demand management type technologies (e.g., rainwater harvesting, grey water recycling, smart meters, etc.). These technologies are likely to affect water temperature at different locations in the system, from property level to pipe network level. For example, increased use of rain or grey water may reduce potable water demand, increasing domestic plumbing and WDNs residence times, and increasing summer months' heating effects from the surrounding air and ground, respectively. The impact of these technologies on water temperature is not currently well understood. Greywater poses an increased risk as it originates from heated sources in the home such as the shower, bath or clothes washing machine, with a notably increased temperature of the reused greywater, often combined with relatively poor quality when compared to water from the WDNs (Nel and Jacobs 2019). Alternatively, smart appliances may be managed to use water at specific times and locations to limit residence times by managing the flow through WDNs and premises to avoid peak high temperatures.

Other future changes in the urban environment (e.g., wider use of geothermal energy, district heating systems, etc.) and related planning which is increasingly done in an integrated way, based on the principles of circular economy and water–energy nexus type thinking, may result in further alterations of water temperatures in the built environment and consequentially WDNs water quality as well. The impact of temperature and its link to these issues is not understood well.

2.5. The Role of Disinfection

The purpose of a disinfection process is to maintain the water quality achieved at the treatment plant throughout the distribution system up to the users' tap. The first continuous application of disinfection through chlorination for drinking water supplies was in Middlekerke (Belgium) in 1902.

Disinfectants, in addition to removing pathogens from drinking water, can prevent the biological re-growth if a minimum disinfectant residual is maintained throughout the WDN. They are also capable of inactivating any microorganisms that may enter the system through contamination (e.g., during pressure drops) (LeChevallier 1999).

The efficiency of disinfection is affected by the treatment plant operational conditions, such as pH and temperature, and the type and nature of microorganisms (Sadiq and Rodriguez 2004).

2.6. Chlorine-Based Disinfectants

Regarding the disinfectant typically used in treatment plants, chlorine products have been used since the 20th century to disinfect drinking water.

<u>Chlorine</u>

Chlorine is the most commonly used disinfectant because it is simple to apply, economical, efficient, and measurable. Three forms of chlorine are commonly used for disinfection: chlorine gas (Cl_2), sodium hypochlorite (liquid) (NaOCl), and calcium hypochlorite (tablet, granular, or powdered) [$Ca(OCl)_2$]. Chlorine inactivates all types of microorganisms: protozoa, bacteria, and viruses. An important advantage of chlorine as a disinfectant is that it remains in water and continues to protect against the effects of re-contamination (WHO Fact sheet 2.17). The persistence of chlorine in water after disinfection is referred to as chlorine residual or free chlorine. Free chlorine, the concentration of residual chlorine present in water as dissolved gas (Cl_2), hypochlorous acid (HOCl), and/or hypochlorite ion (OCl^-), participates in a variety of aquatic chemical reactions. With regard to commonly occurring constituents in drinking water sources, it is known to oxidize reduced iron, manganese, and sulfide. Free chlorine is also known to react with NOM and bromide to form halogenated organic compounds such as trihalomethanes (THMs), haloacetic acids (HAAs), and chlorophenols. These typically are referred as DBPs, some of which may put human health at risk (Duirk and Valentine 2007; Zhang et al. 2017).

Furthermore, the oxidized organic matter can serve as food for microorganisms growing in the distribution system.

<u>Chloramine</u>

Chloramines are formed by adding chlorine and ammonia to water. Chorine-to-ammonia ratios of 3:1 or 5:1 is commonly used (Haas, 1999b).

Hypochlorous acid from the chlorine reacts with ammonia to form inorganic chloramines in a series of competing reactions in aqueous solutions. In these reactions, monochloramine (NH_2Cl), dichloramine ($NHCl_2$), and nitrogen trichloride (also known as trichloramine) (NCl_3) are the species formed.

The relative amounts of each of these species are dependent on bulk water pH, temperature, contact time and the chlorine-to-ammonia ratio (Huang 2008). Monochloramine is the predominant species formed in the pH range 7.5-9 and when the applied chlorine-to-ammonia ratio is less than or equal to 5:1 by weight (Kirmeyer et al. 2004). As the chlorine concentration increases and pH decreases, dichloramines and trichloramines can form. Trichloramines are quite volatile and will usually dissipate, however their formation is typically kept to a minimum due to odour formation (Kirmeyer et al. 1993). Temperature and contact time also affect these reactions.

Chloramines were first used to control taste and odor problems in drinking water. Over time, they were recognized to be more stable, less reactive, and more persistent than free chlorine. Chloramines have the tendency to form significantly lower levels of THMs and HAAs (Kirmeyer et al. 2004).

In order to meet the stringent drinking water regulations on DBPs' control, many water utilities throughout North America have converted to chloramine.

Although chloramine is less reactive than free chlorine in producing regulated

DBPs in combination with organics, it still forms some DBPs (Ricca et al. 2019). In typical water distribution systems, monochloramine is the dominant and preferred species produced because of its biocidal properties, relative stability and relatively lower taste and odour properties (Kirmeyer et al. 2004).

2.7. The Decay of Disinfectant

The main managerial problem in the use of disinfectants is that they decay over time resulting in a decrease in the bactericidal function. When disinfectant is provided at WDN sources, it will be hardly maintained throughout the system. In fact, disinfectant interacts with the NOM in the bulk water and/or with the biofilm on the surface of the pipes, resulting in a fast decay (Figure 2.6)

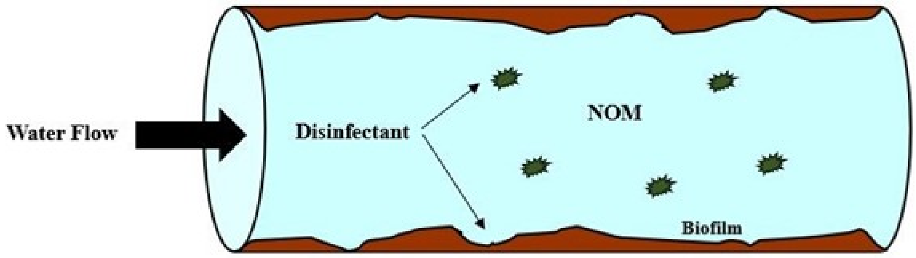


Figure 2.6. Interactions of the disinfectant inside the pipe.

Several kinetic models have been developed mainly in the chlorine case. They are based on laboratory data refined with field data. Their essential feature is simplification and idealization of the complex biochemical phenomena (Propato and Uber 2004). Kinetic models can be incorporated in software packages (see Chapter 3) for the estimation of disinfectant residuals in WDNs.

<u>Chlorine decay</u>

The chlorine decay rate in water can be described by an initial rate, which is relatively rapid, and a long-term decay rate, which is slower. The initial rate is attributed to substances in water that react rapidly with chlorine and are usually referred to as chlorine demand. Once this demand has been met, a more persistent residual is established with a slower decay rate (USEPA 2007).

A number of factors can affect the kinetics including the water temperature, the total organic carbon (TOC) concentration, initial chlorine concentration, biofilms, the rate of pipe corrosion and the presence of corrosion products (Vasconcelos et al. 1996). In general, chlorine decay kinetics increase as these factors increase.

Many models of the chlorine decay have been developed. They generally rely on the assumption that disinfectant consumption is due to reactions with organic and inorganic substances present in bulk water (bulk decay) and reactions that occur with pipe materials or biofilms on inner pipe walls (wall decay) (Lu et al. 1999). The bulk decay is frequently the predominant chlorine decay mechanism (Kiéné et al. 1998; Clark and Haught 2005).

The n^{th} order bulk decay kinetic model is described by

$$HOCl_t = HOCl_0{}^n \cdot e^{-k_b t} \tag{2.1}$$

where $HOCl_t$ = free chlorine concentration (mass/volume) at time t; $HOCl_0$ = initial free chlorine concentration (mass/volume) at time t_0; k_b = bulk decay rate (1/time); and n = order of the reaction.

The bulk constant k_b can be obtained by “bottle” or “jar” experiments in laboratories values. Several first-order k_b has been derived from literature, with values ranging from 0.12 to 17.7 d^{-1} (Vasconcelos and Boulos 1996; Vasconcelos et al. 1997; DiGiano et al. 2000; Rossman et al. 2001; Fisher et al. 2012) at temperatures ranging from 14 to 28 °C.

First-order bulk decay is a particular case of n^{th} order ($n=1$) kinetics and the simplest one, in which the rate of reaction is assumed proportional to the free chlorine concentration and other species with which chlorine react are not taken into account. It has been extensively used in the simulation of chlorine decay in water supply systems (Rossman et al. 1994; Vasconcelos et al. 1997; Powell et al. 2000).

Several studies have developed and demonstrated that a second order reaction can provide a more accurate prediction of chlorine concentrations (Clark 1998; Boccelli et al. 2003; Speight et al. 2009; Fisher et al. 2012) these models require an estimation of the concentration of reactant material.

Further studies analyzed the influence on chlorine bulk decay of several water quantity and quality parameters, such as flow velocity, temperature, initial chlorine dosage, pH, organic matter, and iron content (Kastl et al. 2003; Menaia et al. 2003; Vieira et al. 2004; Al-Jasser 2007; Nagatani et al. 2008).

For the wall decay in non-metallic pipes, a first order kinetic model is generally used (Rossman et al. 1994; Vasconcelos et al. 1997). It has been used by researchers to describe chlorine wall decay in pipes of low reactivity materials such as PVC and medium-density polyethylene (Hallam et al. 2002). The chlorine consumption due to the wall decay can be described as

$$HOCl_t = \frac{2k_w k_f}{R(k_w + k_f)} HOCl_0 \tag{2.2}$$

with k_w = wall decay constant (length/time); k_f = mass transfer coefficient (length/time); and R = pipe radius (length).

The amount of wall area available for reaction and the rate of mass transfer between the bulk fluid and the wall influence the overall rate of this reaction (Rossman 2000).

The wall decay coefficient k_w depends on temperature, as any kinetic constant, and has been correlated to pipe age and material. It is usually estimated through calibration procedures, comparing results obtained from simulations of the network model with the ones obtained from sampling. In PVC pipes, the k_w value can be equal to zero as was found by Clark et al. (2010; 2012).

<u>Chloramine decay</u>

While there is a wide available state-of-the-art on the chlorine kinetic models, the same cannot be said for the chloramine case.

The chloramine decay is more complex since more reacting species are involved in the decomposition process. The main product of these reactions is ammonia, which has the potential to promote nitrification reactions within distribution systems having higher residence times. Nitrification can have the adverse impact of promoting bacterial regrowth (Wilczak et al. 1996) and increasing the rate of decay. Furthermore, when it is accompanied by a decrease in pH, it can promote the corrosion of infrastructures (Zhang et al. 2008).

Chloramine decay has been modeled by Vikesland et al. (2001) and Duirk et al. (2005) shown in Table 2.4.

The model developed by the authors takes account of the auto decomposition of monochloramine to ammonia in the presence of NOM. The model contains 14 bulk species and no surface species and a second order rate expression. It involves both kinetic rate expressions and nonlinear equilibrium relationships.

The principal species are HOCl, hypochlorite ion (OCl^-), ammonia (NH_3), ammonium ion (NH_4^+), NH_2Cl, $NHCl_2$, an unidentified intermediate compound (I) and TOC.

More details on the operational conditions and parameters can be found in Vikesland et al. (2001) and Duirk et al. (2005).

Table 2.3. Chloramine auto decomposition based on Vikesland et al. (2001) and Duirk et al. (2005) models.

N.	Reaction Stoichiometry	Rate coefficient/Equilibrium constant
1	$HOCl + NH_3 \rightarrow NH_2Cl + H_2O$	$k_1 = 1.5\times10^{10}$ M^{-1} h^{-1}
2	$NH_2Cl + H_2O \rightarrow HOCl + NH_3$	$k_2 = 7.6\times10^{-2}$ h^{-1}
3	$HOCl + NH_2Cl \rightarrow NHCl_2 + H_2O$	$k_3 = 1.0\times10^{6}$ M^{-1} h^{-1}
4	$NHCl_2 + H_2O \rightarrow HOCl + NH_2Cl$	$k_4 = 2.3\times10^{-3}$ h^{-1}
5	$NH_2Cl + NH_2Cl \rightarrow NHCl_2 + NH_3$	$k_5 = 2.5\times10^{7}$ $[H^+] + 4.0\times10^{4}$ $[H_2CO_3]$ + 800 $[HCO_3^-]$ M^{-2} h^{-1}
6	$NHCl_2 + NH_3 \rightarrow NH_2Cl + NH_2Cl$	$k_6 = 2.2\times10^{8}$ M^{-2} h^{-1}
7	$NHCl_2 + H_2O \rightarrow I$	$k_7 = 4.0\times10^{5}$ M^{-1} h^{-1}
8	$I + NHCl_2 \rightarrow HOCl$ + products	$k_8 = 1.0\times10^{8}$ M^{-1} h^{-1}
9	$I + NH_2Cl \rightarrow$ products	$k_9 = 3.0\times10^{7}$ M^{-1} h^{-1}
10	$NH_2Cl + NHCL_2 \rightarrow$ products	$k_{10} = 55.0$ M^{-1} h^{-1}
11	$NH_2Cl + S_1{}^{a}\times TOC \rightarrow$ products	$k_{11} = 3.0\times10^{4}$ M^{-1} h^{-1} $S_1 = 0.01$
12	$HOCl + S_2{}^{b}\times TOC \rightarrow$ products	$k_{12} = 6.5\times10^{5}$ M^{-1} h^{-1} $S_2 = 0.42$
13	$HOCl \leftrightarrow H^+ + OCl^-$	$pKa_1 = 7.5$
14	$NH_4^+ \leftrightarrow NH_3 + H^+$	$pKa_2 = 9.3$
15	$H_2CO_3 \leftrightarrow HCO_3^- + H^+$	$pKa_3 = 6.3$
16	$HCO_3^- \leftrightarrow CO_3^{2-} + H^+$	$pKa_4 = 10.3$

Notes: [a] S_1 is the fast reactive fraction of TOC
[b] S_2 is the slow reactive fraction of TOC

2.8. Disinfection Practices and Regulations

The current approaches to disinfection in Europe are influenced by the wide diversity of water resources and supply infrastructures, as well as disinfection philosophy, so European countries vary considerably in their disinfection practices and use of disinfection.

A study conducted for the European Union in 1996 and 1997 (Premazzi et al. 1997a; 1997b and European Commission Directorate) documented the methods of disinfection used in different European countries (Table 2.5).

Focusing on chlorine and chloramine, in most southern European countries (e.g., Italy, Spain and Greece) and the United Kingdom (UK), chlorine is added for residual disinfection. Spain is one of few European countries that commonly use chloramines for residual disinfection in the distribution network. The UK also uses chloramines for disinfection occasionally. The use of chloramines in France is presently prohibited.

Table 2.4. Disinfection Practice in the European countries.

Country	Chlorine	Chloramines	Ozone	UV
United Kingdom	1	3	3	3
Italy	1	-	3	-
Spain	1	2	2	-
Greece	1	-	-	-
France	2	-	1	
Netherlands	-	-	1	2
Germany	-	-	2	3

1 = most commonly used; 2 = commonly used; 3 = occasionally used; “- “= seldom/not used

In the Netherlands and Germany, by contrast, a disinfectant residual is not used. In these countries water utilities rely instead on catchment protection, advanced treatment (via ozone or UV light) and good WDN design, operational and maintenance practices (i.e., monitoring, flushing, break repair etc.), which prevent contaminants entering the WDN.

The WHO regularly publish the Guidelines for Drinking-water Quality (GDWQ). The GDWQ has formed an authoritative basis for the setting of national regulations and standards for water safety in support of public health. They derive maximum and minimum concentration guideline values for the various microbial, chemical constituents that can be found in the drinking water.

In Europe, the Council Directive 98/83/EC (implemented in the Italian regulation D. Lgs 31/2001) on the quality of water intended for human consumption set a total of 48 microbiological, chemical and indicator parameters that must be monitored and tested regularly. The European guideline does not contain standards for chloramine. For the other drinking water quality parameters in general the GDWQ standards are used.

With regard to disinfectant residuals, the maximum values for free chlorine and monochloramine are set to 5 mg/L and 3 mg/L, respectively (WHO 2017).

The Centers for Disease Control and Prevention (CDC 2020) recommends that the limit of 2 mg/L should not be overcome for chlorine due to taste concerns.

At the point of delivery, a minimum residual concentration 0.2 mg/L of free chlorine should be maintained throughout the distribution system, while it is normal practice to supply water with a chloramine residual of 0.1-0.15 mg/L to act as a preservative during distribution (WHO 2017).

Chapter 3

Models for the Prediction of Disinfectant Residuals in WDNs

This chapter focuses on the water quality models used in the research activity for the prediction of disinfectant residuals in WDNs. A mathematical background of the governing equation implemented in each model for simulating the disinfectant transport and decay in WDNs is provided.

3.1. Overview

Disinfectant residuals within a WDN are not steady or uniform, instead they vary spatially and temporally. Due to their dynamic nature, sampling in the system provides only a partial picture of what occur in the pipes (Walski 2019).

A valuable alternative for predicting disinfectant residuals in a cost-effective manner is to use a mathematical water quality model of the WDN under study.

A water quality model is used to simulate the behavior of a disinfectant through the pipes, utilizing information produced by the hydraulic model of the investigated WDN, such as flow and velocity, and the kinetic reaction model of the disinfectant considered, as seen in Chapter 2.

These models have been extensively used in a variety of applications such as in the design and operation of WDNs (Mala-Jetmarova et al. 2017), and in the security and resilience of WDNs in the face of natural and man-made hazards (Ostfeld 2006).

The various simulation algorithms developed to represent water quality in WDNs are based on steady-state approaches (Males et al. 1985; Clark et al. 1986; Boulos and Altman 1993) or dynamic formulations (Grayman et al. 1988; Rossman et al. 1993; Boulos et al. 1995). Between the two kinds of approaches, dynamic modelling provides a more accurate and realistic representation of the actual operation of a system, because it allows determination of the spread of species under time-varying conditions. Rossman and Boulos (1996) categorized and compared various solutions techniques available for water quality modelling, classified spatially as either Eulerian or Lagrangian and temporally as time-driven or event-driven. A more detailed overview on water quality modelling in WDNs can be found in Grayman (2008).

Much of the research and development in WDN quality modelling dates to the early-1990s. The first version of the well-known software EPANET was released in 1993 (Rossman 1993).

EPANET is a public-domain software package developed by the US Environmental Protection Agency (USEPA) for simulating both the hydraulics and the water quality in pressurized pipes of WDNs (Rossman 2000). EPANET's hydraulic engine allows running extended period simulations, while its water quality module allows modelling single-specie reactions in the bulk flow and at the pipe wall (Rossman et al. 1994).

The development of EPANET not only became a valuable tool used by both engineering practitioners and researchers worldwide but even the basis for commercial models introduced later (e.g., WaterGEMS).

The most common water quality application of EPANET is to predict chlorine residuals using a first-order decay both in the bulk water and at the pipe wall.

Over the years several attempts have been made in order to increase the usability and extend the capabilities of EPANET. Shang and Uber (2008) released an extension of EPANET, denominated EPANET-MSX (Multi-Species eXtension), which allows modelling multi-species reactions in the bulk flow and at the pipe wall. The EPANET-MSX model has many applications that address real-world water quality problems that are generally complex. Examples include the chloramine decay, the bacterial regrowth with chlorine inhibition etc. (Shang et al. 2008).

Both EPANET and its extension EPANET-MSX models assume that the transport of disinfectants along the pipes of WDNs is solely driven by advective flow (Rossman et al. 1994). Hence, the outspread of disinfectants in the pipes by longitudinal dispersion is neglected. Otherwise, several studies have demonstrated the importance of including solute transport by dispersion, particularly in the low-flow pipes, the dead-end ones (Axworthy and Karney 1996; Tzatchkov et al. 2002; Li et al. 2006; Basha and Malaeb 2007; Tzatchkov et al. 2009; Abokifa et al. 2016).

Recently, Abokifa et al. (2016) developed a model named WUDESIM to simulate single-specie reactions in the dead-end branches of WDNs. In addition to advective transport, WUDESIM includes species transport by longitudinal dispersion.

For all the water quality models mentioned, the governing equations are based on the principles of mass conservation coupled with disinfectant reaction kinetics. These models also assume complete and instantaneous mixing of water at the nodes and junctions, and storage facilities.

In the following sections, a mathematical background of the three water quality models abovementioned and used in the research activity is provided.

3.2. EPANET

The water quality module of EPANET incorporates the 1-D Advection-Reaction (AR) equation (Rossman et al., 1993; Rossman and Boulos, 1996)

$$\frac{\partial C_i}{\partial t} = -u_i \frac{\partial C_i}{\partial x} + r(C_i) \tag{3.1}$$

where, C_i = concentration of the disinfectant (mass/volume) in pipe i as a function of distance x and time t; u_i = average flow velocity in pipe i (length/time), and r = rate of reaction (mass/volume/time) as a function of concentration. According to Eq. 3.1 the disinfectant will travel down the length of a pipe with the same average velocity as the carrier fluid while at the same time reacting (either growing or decaying) at some given rate. Longitudinal dispersion is not considered as a transport mechanism, meaning there is no intermixing of mass between adjacent parcels of water traveling down a pipe.

The rate of reaction accounts for disinfectant consumption both in the bulk phase and at the pipe wall. The rate of bulk reaction can generally be described as a power function of concentration

$$r = k_b C^n \tag{3.2}$$

where k_b = a bulk reaction constant (1/time) and n = the reaction order. As mentioned in Chapter (2), the decay of disinfectants such as chlorine can be modelled adequately as a simple-first order reaction ($k_b < 0$, $n = 1$).

The overall rate of the wall reaction depends on the amount of wall area available for reaction and the rate of mass transfer between the bulk fluid and the wall. The surface area per unit of volume, which for a pipe equals 2 divided by the radius, determines the former factor; the latter factor is represented by a mass transfer coefficient whose value depends on the molecular diffusivity of the disinfectant and on the Reynolds number of the flow (Rossman et al., 1994). For first-order kinetics, it can be expressed as

$$r = \frac{2\, k_w k_f\, C}{R(k_w + k_f)} \tag{3.3}$$

where k_w = wall reaction rate constant (length/time), k_f = mass transfer coefficient (length/time); and R = pipe hydraulic mean radius (length).

Mass transfer coefficients are usually expressed in terms of a dimensionless Sherwood number (*Sh*):

$$k_f = Sh\frac{D}{d} \tag{3.4}$$

in which *D* = the molecular diffusivity of the disinfectant being transported (length^2/time) and *d* = pipe diameter. In fully developed laminar flow, the average Sherwood number along the length of a pipe can be expressed as

$$Sh = 3.65 + \frac{0.0668(d/L)\ Re\ Sc}{1+0.04\ [(d/L)\ Re\ Sc]^{2/3}} \tag{3.5}$$

in which *Re* = Reynolds number and *Sc* = Schmidt number (kinematic viscosity of water divided by the diffusivity of the disinfectant). For turbulent flow the empirical correlation of Notter and Sleicher (1971) can be used:

$$Sh = 0.0149Re^{0.88}Sc^{1/3} \tag{3.6}$$

Mixing at pipe junctions follows the mass balance equations, assuming the mixing of fluid is complete and instantaneous. At junctions receiving inflow from two or more pipes, complete mixing happens in a short time and is quickly distributed to the next nodes. Thus, the concentration of disinfectant in water leaving the node is simply the flow-weighted sum of the concentrations from the inflowing pipes. The disinfectant concentration at the specific node can be described by

$$C_{out} = \frac{\sum Q_{in}C_{in}}{\sum Q_{out}} \tag{3.7}$$

The most common water quality application of EPANET is the prediction of chlorine residuals.

3.3. EPANET-MSX

While the water quality module of EPANET is limited to tracking the transport and fate of just a single chemical specie, its extension EPANET-MSX allows to model any system of multiple, interacting chemical species.

Reactions can be divided into two classes based on reaction rates. Some reactions are reversible and fast enough in comparison with the system's other processes so that a local equilibrium can be assumed; others are not sufficiently fast and/or irreversible and it is inappropriate to use an equilibrium formulation to represent them. Theoretically, very large backward and forward rate constants (with their ratio equaling the equilibrium constant) can be used to model fast/equilibrium reactions and therefore both fast/equilibrium and slow/kinetic reaction dynamics can be written as a single set of ordinary differential equations (ODEs) that can be integrated to simulate changes in species concentrations.

In EPANET-MSX, algebraic equations are used to represent the fast/equilibrium reactions and mass conservation. Thus, it is assumed that all reaction dynamics can be described by a set of differential-algebraic equations (DAEs) that is in semi-explicit format. The system of DAEs that defines the interactions between bulk species, surface species, and parameter values can be written in general terms as

$$\frac{dx_b}{dt} = f(x_b, x_s, z_b, z_s, p) \tag{3.8}$$

$$\frac{dx_s}{dt} = g(x_b, x_s, z_b, z_s, p) \tag{3.9}$$

$$0 = h(x_b, x_s, z_b, z_s, p) \tag{3.10}$$

where x_b = vector of time-varying differential variables associated with the bulk water; x_s = vector of time-varying differential variables associated with the pipe surface; z_b = time-varying algebraic variable associated with the bulk water; z_s =

time-varying algebraic variable associated with the pipe surface; and p = time invariant model parameters.

This multispecies reaction system is linked with the water quality transport module of the EPANET model. Hence, EPANET-MSX ignores axial dispersion and tracks the movement and reaction of chemicals in discrete water volumes or segments which are transported through network pipes by the bulk velocity, and completely mix at nodes.

As introduced in Chapter 2, the chloramine decay in distribution systems involves multiple interacting chemical species, which can be only analyzed by using a multi-species approach.

3.4. WUDESIM

The WUDESIM (Washington University Dead End Simulator) model simulates the disinfectant transport in dead-end branches of the network using a dynamic 1-D Advection-Dispersion-Reaction (ADR) equation

$$\frac{\partial C_i}{\partial t} = -u_i \frac{\partial C_i}{\partial x} + E \frac{\partial^2 C_i}{\partial x^2} + r(C_i) \tag{3.11}$$

where, C_i = concentration of disinfectant (mass/volume) in pipe i as a function of distance x and time t, u_i = average flow velocity in pipe i (length/time), E = longitudinal dispersion coefficient (area/time) and r = rate of reaction (mass/volume/time) as a function of concentration that accounts for disinfectant consumption both in the bulk phase and at the pipe wall, calculated by (3.2) and (3.3).

Removing the dispersion term in the previous ADR equation reduces the model to the 1-D AR equation incorporated in the water quality module of EPANET.

The longitudinal dispersion is characterized by a non-physical coefficient E, whose value is determined by the flow and pipe properties. Many analytical models to determine the values of E under laminar (Re < 2,300) and transitional/turbulent (Re > 2,300) flow regimes are available in literature.

These models solve the governing differential equation using different velocity profiles, defined using empirical expressions. Therefore, their predictions are highly sensitive to the accuracy of the velocity profiles used (Sattar 2013).

The formula supported by the WUDESIM model for calculating the values of the longitudinal dispersion coefficient for different flow regimes are listed in Table 3.1. Under laminar flow (Re < 2,300), WUDESIM supports the classical formula developed by Taylor (1953) as well as the more recent formula developed by Li et al. (2006). The Taylor's formula only provides the ultimate value that the dispersion coefficient approaches after a certain initialization period has elapsed given by $t > 0.5\frac{R^2}{D}$. Given that extended stagnation periods are typically encountered in dead ends leading to a partial loss in the dispersion memory between demand pulses, the longitudinal dispersion in pulsating laminar flow will always be within the initialization period (Abokifa et al. 2016). Thus, Li et al. (2006) developed a formula to account for the dynamic rates of dispersion for pulsating laminar flow. The use of a highly dynamic time evolving dispersion coefficient is essential to simulate the complex nature of flow demands in dead ends.

To account for dispersive transport for the rare cases when dead-end pipes operate under turbulent regimes (Re > 4,000), WUDESIM supports the classical formula of Taylor (1954) in addition to the empirical formula developed by Sattar (2013), using gene expression programming. The Taylor's formula is only valid under highly turbulent regimes (Re > 20,000) which is unlikely to take place in dead ends where flow regimes are largely laminar with only occasional transitional to early turbulent flows, while the Sattar's formula shows reasonable description of the experimental data for Reynolds numbers in the range (2,300 < Re < 10,000) (Abokifa et al 2016). For transitional regimes (2,300 < Re < 4,000), WUDESIM calculates the value of E using linear interpolation between the two corresponding values calculated at Re = 2,300 and Re = 4,000 using the selected formula for laminar and turbulent dispersion, respectively.

For mixing at a pipe junction, WUDESIM makes the same assumptions as EPANET, i.e., the mixing obeys mass balance equations and is complete and instantaneous, following equation (3.7).

Table 3.1. Supported formula in WUDESIM for calculating the dispersion coefficient.

Flow regime	Dispersion coefficient formula	Reference
Laminar	$E = \frac{R^2u^2}{48D}$	Taylor (1953)
	$E = \frac{R^2u^2}{48D}\left\{1 - \left(\frac{\tau_0}{\tau}\right)\left[1 - exp\left(-\frac{\tau}{\tau_0}\right)\right]\right\}$	Li et al. (2006)
Turbulent	$E = 10.1Ru\sqrt{f/8}$	Taylor (1954)
	$E = \frac{1.65d^{-1.82d}}{Ref}$	Sattar (2013)

Notation: R is the pipe radius (m); u is the average flow velocity in the pipe (m/s); D is the molecular diffusivity of solute in water (m^2/s); τ_0 is a Lagrangian time scale (s) calculated as: $\tau_0 = a^2/16D$; τ is the residence time (s) calculated as: $\tau = L/u$; L is the pipe length (m); f is the pipe friction factor (-); d is the pipe diameter (m) and Re is the number of Reynolds (-).

Chapter 4

The Modulation of Nodal Outflows to Guarantee Sufficient Chlorine Residuals in a Medium Size WDN

In this chapter the modulation of nodal outflows is proposed to solve the problem of low chlorine residuals at critical dead-end nodes of WDNs. The slight increase in nodal outflows can be obtained through the opening of a blowoff at the hydrant site. The methodology was presented to the scientific community in the paper "Modulating Nodal Outflows to Guarantee Sufficient Disinfectant Residuals in Water Distribution Networks", authored by S. Avvedimento, S. Todeschini, C. Giudicianni, A. Di Nardo, T. Walski, and E. Creaco, and published in the Journal of Water Resources Planning and Management, 146(8) - August 2020. It is based on the combined use of optimization and flow routing/water quality modelling of WDNs. The effectiveness of the methodology is proven on a real medium size WDN, yielding insight into the economic feasibility of the solution.

4.1. Overview

Water quality has been gaining attention in the context of water safety plans (WSPs) (WHO 2009) to ensure the safety of drinking water using a comprehensive approach for risk assessment and management. The WSP approach encompasses all steps in water supply from catchment to consumer. Focusing on the last segment of the water technology cycle, disinfection within WDN is necessary to prevent drinking water from posing a microbial risk. As disinfectant travels through the pipes in a distribution system it can react with a variety of materials both within the bulk water and from the pipe wall (Rossman et al. 1994). However, in some terminal nodes of the WDN, disinfectant concentrations may become lower than the minimum values necessary to guarantee users' protection from contaminations (as prescribed by technical guidelines). Typical values of maximum disinfectant dose adopted at WDN sources are 5 mg/L (WHO 1996) or 4 mg/L (Ontario MOE 2006). A recommended minimal target must also be guaranteed for free disinfectant residual concentration in a WDN, as is pointed out in the Safe Water System project developed by Centeres for Disease Control and Prevention (CDC 2001) and Pan American Health Organization (PAHO), Ontario MOE (2006), and WHO (2011). 40 CFR 141.72 (CFR 2000) requires that the disinfectant "cannot be undetectable in more than 5 percent of the samples each month." However, some states in the US have much stricter limits. For example, Pennsylvania requires 0.2 mg/L (Pennsylvania DEP 2019).

The problem of low disinfectant concentrations occurs in areas such as dead-end nodes, in which low flow conditions lead to long residence times and to excessive decay of the disinfectant upstream from users. Walski (2019) provided an overview of the range of options available to maintain a disinfectant residual.

A solution to this problem may be to increase the concentration of disinfectant fed at the source node. Although this is done in some cases, it may create excessive disinfectant residuals near the feeding point, resulting in taste and odour problems, as well as in the formation of carcinogenic DBPs (Morris et al. 1992). Another possible solution to the problem is the use of additional disinfectant booster stations (e.g., Boccelli et al. 1998; Tryby et al. 2002; Propato and Uber 2004; Carrico and Singer 2009). Kang et al. (2010) proved that booster disinfection combined with the optimal operation of real-time valves can improve water quality while requiring lower chlorine doses. However, these solutions increase the installation and operational costs for the water utility. Furthermore, when critical nodes are scattered over the WDN, it may be infeasible to serve all critical dead-end nodes with a reasonable number of booster stations.

An alternative option for maintaining disinfectant residuals is based on flushing distribution pipes (Friedman et al. 2002). Although hydrants are typically used for firefighting purposes and for the calibration of WDN numerical models (Walski et al. 2001), more research endeavours proved the effectiveness of hydrants to maintain water quality (Friedman et al. 2002; Kirmeyer et al. 2002). There are two different but related mechanisms by which flowing hydrants can improve water quality: (1) providing sufficient shear stress velocity ($\geq$ 1.5 m/s) to remove deposits and debris attached to pipe wall, and (2) removing poor quality water from the WDN and replacing it with water with lower water age and improved disinfectant residual concentration. This can be accomplished by flushing.

In the scientific literature, various works were dedicated to the use of flushing strategies as a tool for the removal of deposits and contamination from water supply pipes (Carrière et al. 2005; Deuerlein et al. 2014) or as a first hydraulic response to contamination (Baranowski and Walski 2009; Poulin et al. 2010).

The use of flushing to maintain disinfectant residuals was explored for the first time by Walski and Draus (1996), who made use of numerical modelling to determine if this strategy could maintain an adequate chlorine residual in a water system under low demand conditions.

Xie et al. (2014) proposed a tool for scheduling and optimization of conventional flushing, to increase disinfectant residuals in a WDN. In their optimization framework, the objective was to minimize the flushing volume while meeting the minimum requirement of disinfectant residual. The problem was formulated as a single-objective discrete optimization, with automatic flushing device (AFD) operation patterns as decision variables. Although this approach represents an important contribution, it considered neither the effect of seasonality changes on water demand and chlorine decay conditions through the system, nor the economic feasibility of the solution, which is an important issue for water utilities. Moreover, Xie et al. pointed out potential pressure deficiency in the WDN as a result of automatic flushing.

Quintiliani et al. (2017) examined whether the use of flushing, combined with the operation of valves and pump stations, could reduce the formation of THMs in WDNs. However, they pointed out that large amounts of clean water need to be flushed out of the WDN to ensure a relevant reduction in THMs, thus making this practice infeasible as an ordinary operational intervention.

Because of the above considerations for the use of flushing to guarantee water quality, with focus on disinfectant residuals, the present research is carried out with the following novel issues:

- implementation of continuous low flow instead of hydrant flushing;
- analysis of the effect of seasonal changes on the effectiveness of the proposed procedure; and
- economic analysis of the solutions obtained.

For the first issue, the procedure proposed in this work is based on the slight increase in nodal outflows all day long, through the opening of a blowoff at a critical dead-end node, to obtain a low continuous flow.

This is expected to cause fewer undesired effects in terms of service pressure decrease compared with the typical intense flushing. Nodal blowoffs can take place at hydrant sites by slightly opening small taps installed close to these devices at critical dead-ends, or at user connections.

Figure 4.1 shows a schematic representation of a blowoff. It can be placed in proximity to a dead-end node and connected to the closed pipe. The blowoff is manually opened by turning a valve operating nut in the proper direction with a valve wrench. When the valve is open, the blowoff is turned on and the stagnant water coming from the connected dead-end pipe is discharged by the blowoff into the environment (blowoff outflow). The outflow rate, which will be lower than the one obtained from typical fire hydrants, can be gradually increased by regulating the degree of opening of the valve operating nut.

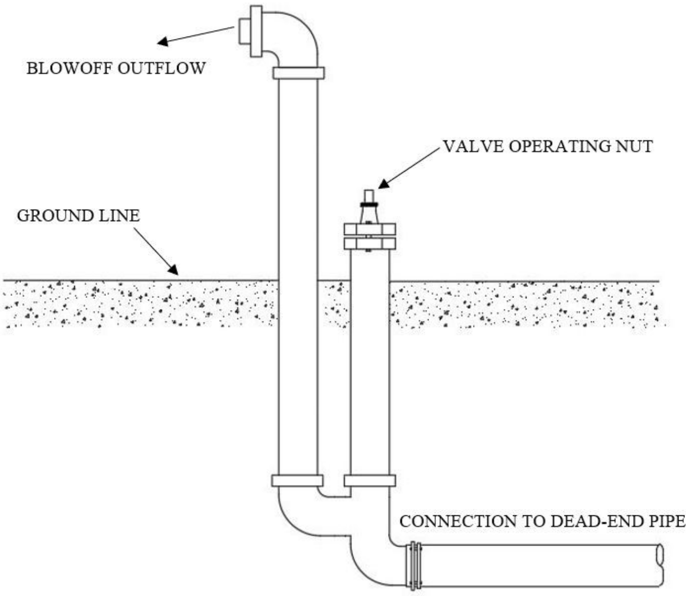


Figure 4.1. Schematic representation of a blowoff (not to scale).

In this context, incentives can be proposed to users to encourage them to use more water, for example, for irrigation purposes. The opening of these blowoffs must be modulated in such a way as to obtain the smallest increase in pipe flow that enables meeting the minimum concentrations of disinfectant at downstream critical nodes of 0.2 mg/L. Obviously, nodal pressure deficits also must be prevented when outflows are increased.

4.2. Methodology

The plan of nodal blowoffs to prevent low disinfectant concentrations in WDN can be developed by making use of models that simulate WDN behaviour in terms of both flow routing and water quality (disinfectant decay), such as EPANET version 2.0 or WaterGEMS version 10.02.00.43.

These software applications simulate the flow through an open hydrant as an emitter, with pressure-driven demand q given by

$$q = e\,P^n \tag{4.1}$$

where q = flow through hydrant (m^3/s); e = emitter coefficient (m^{3-n}/s); P = pressure head upstream of the hydrant (m); and n = emitter exponent (typically set to 0.5). Using the emitters, the flushing outflow rate in the considered node depends on the value of the pressure head in each time step.

The plan of nodal blowoffs can be tackled as an optimization problem. The multi-objective genetic algorithm NSGAII (Deb et al. 2002) is used in this work to search for solutions between two objective functions (f_1 and f_2) to be simultaneously minimized.

The first objective function, f_1, is the total volume Vol (m^3) of water input into the WDN per day

$$f_1 = Vol = \sum_{j=1}^{N_{\Delta t}} \sum_{i=1}^{n_s} Q_{i,j}\,\Delta t \tag{4.2}$$

where $Q_{i,j}$ (m^3/s) = flow rate supplied by ith of n_s source nodes at jth of $N_{\Delta t}$ time steps Δt (s) considered for the simulation. As a result of mass conservation, the variable Vol equals all nodal outflows (leakage from WDN pipes plus standard outflow plus additional outflow considered for fixing disinfectant residuals).

The second objective function, f_2, is the total mass W (kg) of disinfectant fed into the network per day

$$f_2 = W = \sum_{j=1}^{N_{\Delta t}} \sum_{i=1}^{n_s} C_{d,i}\,Q_{i,j}\,\Delta t \tag{4.3}$$

where $C_{d,i}$ = concentration of disinfectant imposed on ith supply (kg/m^3).

The decision variables are $C_{d,i}$ (i = 1, …, n_s) and the values of the emitter coefficients e at the n_n critical nodes of the network. These emitters can be considered as elements of a vector $\mathbf{E} = (e_1, e_2, \ldots, e_{n-})$.

A first set of constraints of the optimization concerns the minimum and maximum limits on the feasible disinfectant concentration at the source

$$C_{d,min,s} \leq C_{d,i} \leq C_{d,max,s} \tag{4.4}$$

where $C_{d,min,s}$ and $C_{d,max,s}$ = minimum and maximum concentrations of disinfectant allowed in the WDN sources, respectively. When fixing $C_{d,min,s}$ and $C_{d,max,s}$ it must be kept in mind that excessively high values of disinfectant concentrations may cause taste and odour problems in water close to the source nodes. However, a minimum concentration of disinfectant (e.g., chlorine) must be guaranteed to reduce microbial growth and thus to decrease the disinfectant consumption in pipes at least near the source.

A second set of constraints concerns the minimum desired value $c_{d,min}$ of the disinfectant residual (kg/m^3) to be guaranteed at WDN nodes.

A third set of constraints of the optimization concerns the minimum desired pressure heads P_{min} for full demand satisfaction at WDN nodes

$$P_{min} \leq P_{i,j} \tag{4.5}$$

where $P_{i,j}$ = nodal pressure heads calculated by the hydraulic solver at ith node and at jth time step.

The objective functions f_1 and f_2 compete against each other. The lowest value of f_1 is obtained by imposing no additional outflow through blowoffs and faucets, including water supply to users and leakage along pipes. However, under these conditions, the compliance with minimum required residual concentrations at all nodes, including the critical terminal ones, may require high disinfectant doses to be input at WDN source(s) (high values of f_2). Conversely, if low continuous flows are allowed at critical nodes, f_1 grows to some extent. However, lower disinfectant doses, leading to lower values of f_2, can be enough at the source(s) to respect residual concentrations at all WDN nodes, due to the increase in flow at dead-end nodes. The output of the optimization consists of a set of trade-off solutions (Pareto front). These low continuous flows are intended to take place near WDN nodes by means of already-existing faucets and blowoffs at hydrant sites. If these devices are not present, their purchase and installation cost should be considered.

A heuristic procedure is set-up in this work to obtain the Pareto front of optimal solutions under the simplifying assumption of a single value of disinfectant concentration C_d being used for all WDN sources in the generic solution. As a result, the solutions obtained through this procedure are rigorous only for single-source WDNs. In the case of multisource WDNs, imposing the same value of C_d on all sources may lead to suboptimality. In all cases, these solutions can be used as good solutions of the first attempt obtainable with a small computational burden, to be potentially refined in the context of more-complete multi-objective methodologies, which may require larger computational burden.

For given value for the disinfectant concentration C_d at the WDN source(s), the sequence of steps shown in Figure 4.2 is carried out.

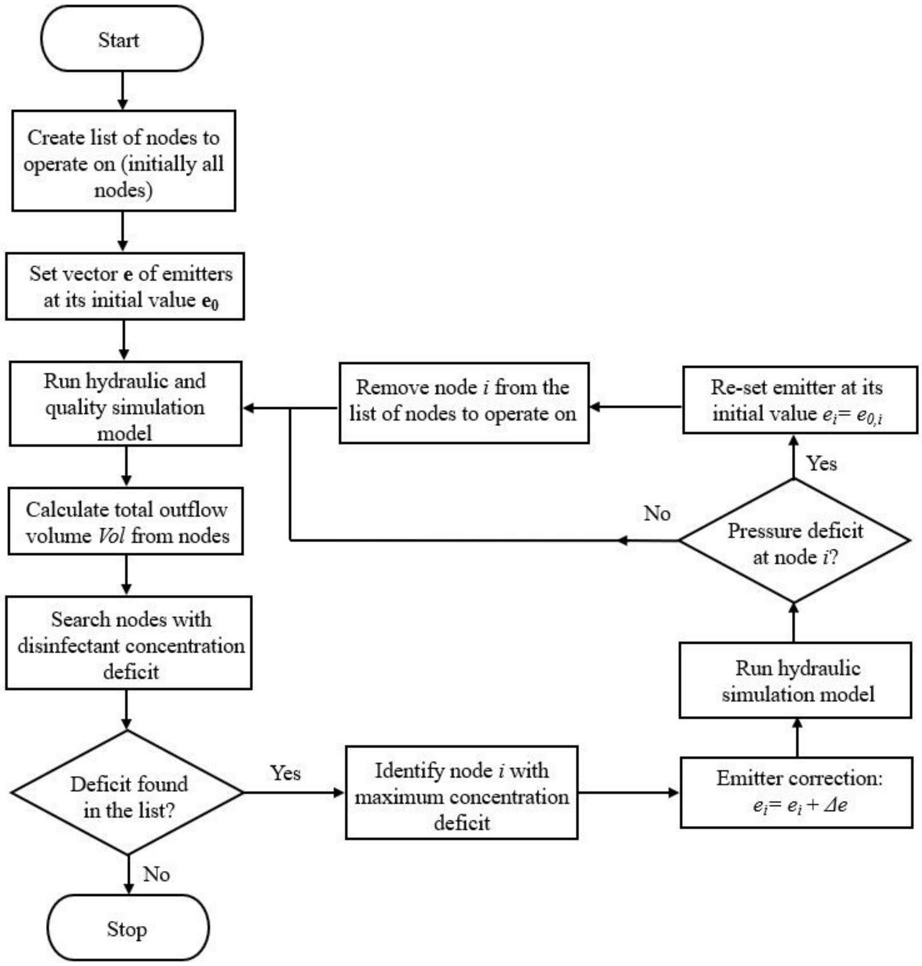


Figure 4.2. Flowchart of the steps to carry out for each value of disinfectant concentration at the source.

At the beginning, a list of nodes on which to potentially operate is created, which initially includes all WDN nodes (Step 1). Then, the vector **E** of nodal emitters is set at the initial value $\mathbf{e_0}$ (no additional outflow at any nodes, and nodal emitters equal to their initial values that account for only leakage along pipes) (Step 2). A hydraulic and water quality simulation is run to obtain the WDN behavior in a sufficiently long sequence of days, to enable the water quality of the WDN to reach cyclic conditions (Step 3). The total outflow (users' consumption plus leakage plus additional initially null outflow) from nodes is calculated in the last day of the simulation Step 4). The nodes with a deficit in the disinfectant concentration are searched for in the list of nodes on which to operate (Step 5). If no deficit is found, the procedure ends. Otherwise, the node i with the maximum deficit is identified (Step 6) and its emitter is increased to simulate faucet or blowoff opening as $e_i = e_i + \Delta e$, where Δe is a small variation considered in the analysis, e.g., 0.01 (Step 7). Then the hydraulic model is run again to calculate the pressure at the node (Step 8). Hence, the procedure restarts from Step 3 if the increase in nodal blowoff causes no service pressure deficit at the node or elsewhere in the WDN. Otherwise, the nodal emitter is reset to its initial value (Step 9) and node i is removed from the list of nodes on which to operate. Then the remaining node with the maximum concentration deficit is addressed until all nodes achieve the target disinfectant concentration. At the end of the iterations of the heuristic procedure, no more nodes with disinfectant concentration deficits will exist in the WDN, excepts the nodes removed from the list for which the increase of nodal emitter cause pressure deficits. However, for the WDN considered in this work, cases of pressure deficits were infrequent. This was due to the small outflow values typically required for the correction of disinfectant residuals.

Furthermore, by referring to the last day of WDN simulation, objective functions f_1 and f_2 can be calculated and associated with the disinfectant concentration C_d considered at the WDN source(s). By applying the heuristic procedure to various values of disinfectant concentration at the WDN source(s), an approximated Pareto front of optimal solutions can be obtained. Obviously, each solution is associated with a specific value of C_d.

The optimization methodology is programmed using Matlab version R2018b.

4.3. Application

Case Study

The case study considered in this work is a WDN serving a city in northern Italy of about 30,000 inhabitants (Creaco and Franchini 2013). It is made up of 623 demand nodes, 678 pipes and 1 source node (Figure 4.3), with total head ranging from 38 to 42 m [Figure 4.4(a)] due to tank water level fluctuations. For this WDN, the EPANET model is available, in which all the nodes are assumed to have an elevation of 0.00 m above sea level (ASL). All the pipes are assumed to feature a Manning roughness coefficient of 0.01 s/m$^{1/3}$ (a typical value for PVC); the diameter ranged from 63 to 300 mm, and the length ranged from 7 to 2,000 m. A pattern is used for the hourly demand multiplier to represent the typical daily variation in the users' demand in the system, with multiplier values ranging from 0.500 to 1.335 [Figure 4.4(b)]. The total average demand of the WDN is 68.54 L/s. WDN emitters corresponding to leakage are tuned in order to obtain a percentage of leakage around 20%, consistent with the real WDN.

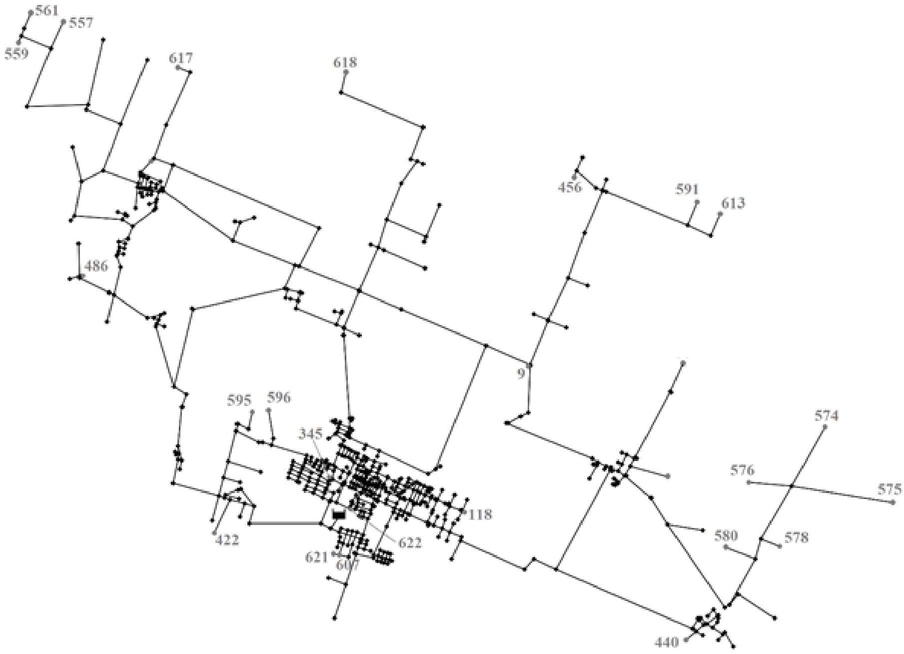


Figure 4.3. Case study layout. Grey numbers and nodes indicate dead-end nodes where blowoffs may be opened for chlorine concentrations at the source C_{cl} equal to 1 mg/L or higher.

For water quality simulations, chlorine is chosen as the disinfectant. In the chlorine decay simulation, bulk and wall reactions both are first order. The bulk decay constant (k_b) is assumed to be 1.0 d^{-1} from literature for all the links and the wall decay constant (k_w) is set to 0 because network pipes are made of plastic material (smooth surface of pipes' internal wall) (Rossman et al. 1994; Powell et al. 2000; Boccelli et al. 2003; Monteiro et al. 2013; Nejjari et al. 2013).

The simulations are run for 10 days of WDN operation to make sure that chlorine injected close to the reservoir has enough time to reach the final nodes of the network, and to reach well-established cyclical operating conditions in the last day of simulation. The hydraulic and water quality time steps are 1 hour and 5 min, respectively. The initial chlorine concentration is set to 0 at all WDN nodes.

In an initial exploratory simulation with EPANET, chlorine is injected into the source node with a constant concentration of C_{cl} = 1.0 mg/L, a typical low value used at WDN sources. EPANET is used to identify the critical nodes, with a residual chlorine concentration below the minimum constraint of $c_{cl,min}$ = 0.2 mg/L in the last day of WDN operation.

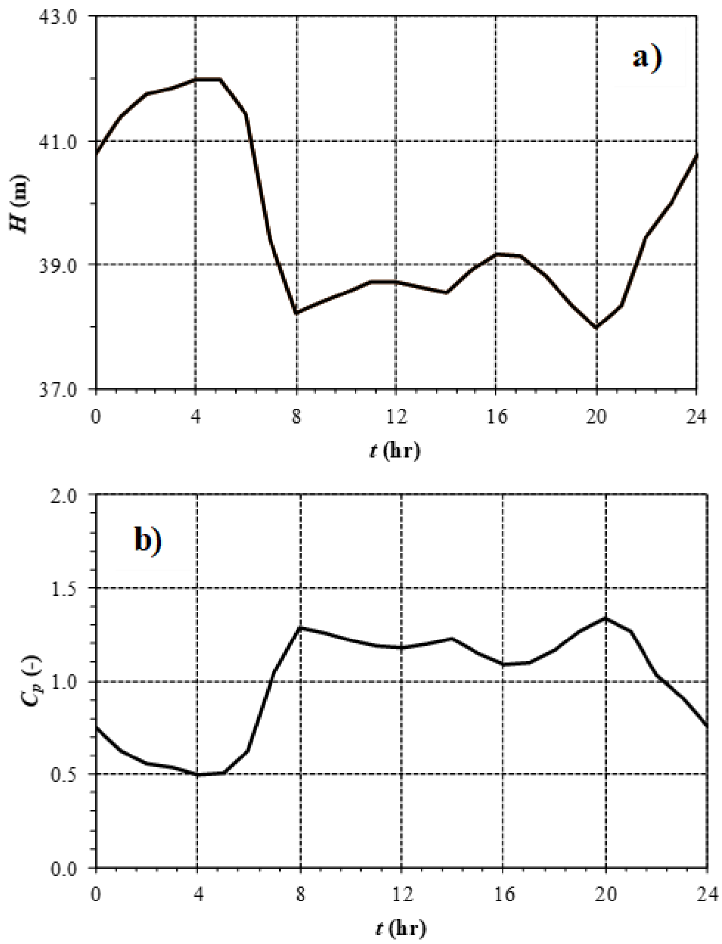


Figure 4.4. Temporal pattern of (a) head H at the source; and (b) hourly demand multiplier C_p.

Specifically, 24 nodes are identified (Figure 4.2) with minimum chlorine concentration below 0.2 mg/L. These critical nodes are scattered in the WDN and located at various dead ends. This happens due to the occurrence of low flow velocities, which cause an excessive disinfectant decay. Such a problem cannot be tackled easily by installing chlorine boosters. Numerous booster installations would be required to reach all the critical nodes, thus incurring in an exceedingly high capital investment. Furthermore, the problem cannot be solved by increasing chlorine dose at the source. In this context, other EPANET simulations show that, even when C_{cl} increase significantly, it is infeasible to eliminate all violations (Figure 4.5). Even in the case of C_{cl} = 12.5 mg/L (an infeasible and dangerous concentration in WDNs), 2 of the 24 initial violations persist.

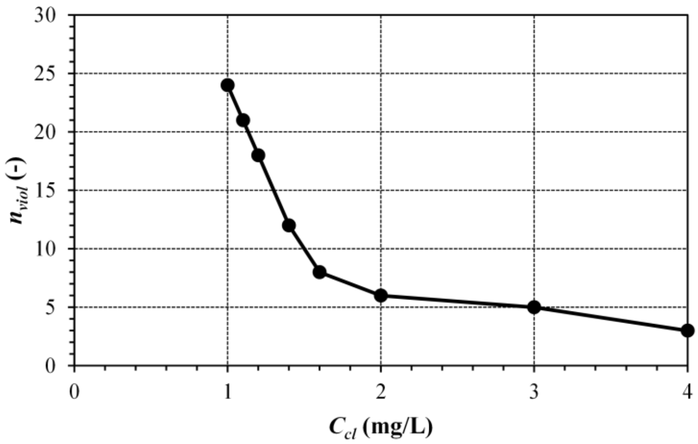


Figure 4.5. Number of violating nodes nviol as a function of chlorine dose C_{cl} at the source. The infeasible value $C_{cl,max}$ = 4 mg/L is considered here only for explicatory purposes.

A viable option to correct violations is increasing flows constantly at dead-end nodes by opening blowoffs at hydrant sites. To this end, the heuristic procedure presented in the previous section is applied to the WDN for the following eight values of C_{cl}: 1, 1.1, 1.2, 1.4, 1.6, 2, 3, 4 mg/L. Therefore, the chlorine concentration at the source is assumed to range from $C_{cl,min}$ = 1 mg/L to $C_{cl,max}$= 4 mg/L. Furthermore, as prescribed by Italian regulation (D. Lgs. 31/01), the minimum residual chlorine concentration is set to $c_{cl,min}$ = 0.2 mg/L with a minimal desired pressure head $P_{min} \geq 12 + 10 = 22$ m (12 m is the height of the average building in the network, and 10 m is the surplus of head pressure as prescribed by the Italian guidelines).

4.4. Results

In the applications, blowoffs opening never caused pressure deficits at WDN nodes. Figure 4.6 shows the Pareto front of optimal trade-off solutions in the optimization. The total mass *W* of chlorine injected at the source is plotted against the total volume *Vol* of water delivered in the network (including supply, leakage and additional outflow considered for fixing chlorine residuals).

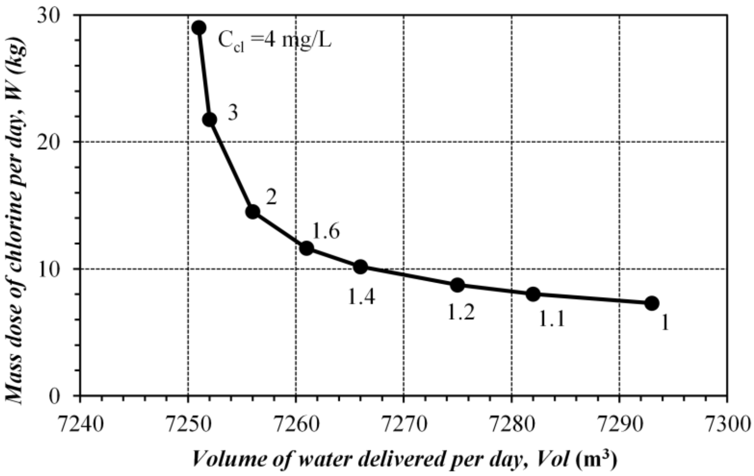


Figure 4.6. Pareto front of optimal solutions in the trade-off between water volume Vol and mass chlorine W for each value of chlorine concentration at C_{cl} at the source.

As expected, the higher values of *W,* the disinfectant feed, were associated with lower values of water volume and vice versa. This was because when there is no additional outflow (minimum value of *Vol*), it is necessary to impose high concentration of chlorine at the source with high *W* value, to meet chlorine residual requirements even at dead-end nodes. On the other hand, by imposing some additional outflows (high values of *Vol*) near suitably identified critical nodes, it is possible to satisfy the minimum desired concentrations of chlorine with low values of *W*. Each solution of Figure 4.6, featuring certain values of *Vol* and W associated with a single value of C_{cl}, was postprocessed for a cost analysis, (Table 4.1). As a function of W, the daily cost of chlorine is calculated as

$$Chlorine\ cost = uc_{cl} * W \tag{4.7}$$

where uc_{cl} = unit cost of chlorine (here, €4.89/kg); and W = total mass of chlorine injected (kg).

The daily variable operating cost for various values of water unit cost exclusive of disinfection is calculated by

$$Variable\ operating\ cost = uc_w * Vol \tag{4.8}$$

where uc_w = unit cost of water (€/m^3); and Vol = total volume of water delivered (m^3).

The total daily cost C_{tot} is

$$C_{tot} = chlorine\ cost + variable\ operating\ cost \tag{4.9}$$

The calculations in Table 4.1 indicate that, for the various values uc_w considered, the lowest values of C_{tot} were obtained for low values of chlorine concentration C_{cl} at the source (≤ 1.4 mg/L). This corresponded to solutions with lower values of W and slightly larger values of Vol.

Table 4.1. Economic analysis of solutions in Pareto front in Figure 4.5

C_{cl} (mg/L)	Vol (m^3)	Leakage + add. out. (%)	W (kg)	Chlorine cost (€)	C_{tot} (€)						
					uc_w=0.005 €/m^3	uc_w=0.0275 €/m^3	uc_w=0.05 €/m^3	uc_w=0.275 €/m^3	uc_w=0.5 €/m^3	uc_w=0.75 €/m^3	uc_w=1 €/m^3
1	7293	20.51	7.29	36	72	236	400	2041	3682	5505	7329
1.1	7282	20.39	8.01	39	76	239	403	2042	3680	5501	7321
1.2	7275	20.32	8.73	43	79	243	406	2043	3680	5499	7318
1.4	7266	20.22	10.17	50	86	250	413	2048	3683	5499	7316
1.6	7261	20.16	11.62	57	93	256	420	2054	3687	5503	7318
2	7256	20.11	14.51	71	107	270	434	2066	3699	5513	7327
3	7252	20.06	21.76	106	143	306	469	2101	3732	5545	7359
4	7251	20.05	29.00	142	178	341	504	2136	3767	5580	7392

Note: C_{tot} = total cost; and uc_w = water unit cost.

The additional nodal blowoffs can be considered as a kind of non-revenue water, in addition to leakage along WDN pipes. The percentage of leakage plus additional nodal blowoffs, calculated with reference to the total outflow from the source, only slightly increased compared to the no-blowoffs Scenario (20 %) (Table 4.1). The maximum value was 20.51% (Table 4.1, Leakage). This means that the slight opening of faucets and/or blowoffs for improving water quality at dead-end nodes only slightly worsened water losses in the WDN. However, the small values of nodal outflows may make the results of this methodology difficult to apply rigorously in the field. It is likely that the opening of blowoffs or faucets in the field may cause larger outflows than those required by the methodology.

4.5. Discussion

Additional simulations were carried out to perform a sensitivity analysis of different parameters. A single value $C_{cl} = 1$ mg/L was considered for the chlorine concentration at the source because the effects of this parameter were considered previously. A range of variation [0.75, 1.25] was considered for both the demand multiplier and the bulk decay constant to analyze the effects of demand seasonality and chlorine decay conditions, which may lead to results different from those obtained previously. In these additional simulations, the increase in nodal outflows was obtained by opening hydrant blowoffs. The software WaterGEMS was used because it enables a more tailored modeling of hydrants. In WaterGEMS, hydrants are modelled as other emitters separate from those used in the model to represent leakage. Therefore, the modeling of hydrant blowoffs at critical dead-end nodes does not result in any interference with leakage modeling in the WDN. At each node, the outflow due to pipe leakage and the outflow due to hydrant blowoffs can be calculated separately by multiplying the respective emitter coefficient by the nodal pressure head raised to the emitter exponent. The tailored blowoff emitters were tuned (Table 4.2) to obtain the lowest blowoff outflows [average daily values *q* (Table 4.2)] that corrected chlorine deficits in five different Scenarios.

The results in Table 4.2 correspond to the demands and decay rates loaded into the model to generate those outflows. In real systems, the demands and reaction rates are not constant, but vary according to weather and other factors. That is why several cases for results are provided. Cases A and E were the most conservative (highest outflows) whereas Cases C and D were the least conservative.

Globally, Table 4.2 confirms the validity of the methodology under a range of conditions of demand and chlorine decay. As expected, the decrease in demand and the increase of the bulk decay constant caused an increase in the number of blowoffs to open and in the blowoff outflows.

Table 4.2. Blowoff emitters e and average daily outflows q close to critical nodes in WaterGEMS model in five operational Scenarios, and total additional outflow from the network.

Node	*e* (L/s/m$^{1/2}$) Case A	*q* (L/d) Case A	*e* (L/s/m$^{1/2}$) Case B	*q* (L/d) Case B	*e* (L/s/m$^{1/2}$) Case C	*q* (L/d) Case C	*e* (L/s/m$^{1/2}$) Case D	*q* (L/d) Case D	*e* (L/s/m$^{1/2}$) Case E	*q* (L/d) Case E
9	0.0001	595	0.00002	116	0.000012	67	-	-	0.00007	396
36	0.00032	1912	0.00014	811	-	-	-	-	0.00061	3472
43	-	-	-	-	-	-	-	-	0.000013	74
54	0.000027	167	-	-	-	-	-	-	0.000029	173
57	-	-	-	-	-	-	-	-	0.00003	179
118	0.000032	206	0.000025	159	0.00002	124	0.000021	133	0.000052	330
345	0.00016	1047	0.00015	974	0.00014	901	0.000091	591	0.00022	1428
388	-	-	-	-	-	-	-	-	0.000014	81
422	0.00051	3352	0.00025	1638	-	-	-	-	0.00082	5368
440	0.00012	743	0.00005	301	-	-	-	-	0.00026	1554
451	0.000078	451	-	-	-	-	-	-	0.0024	12610
455	0.00016	924	-	-	-	-	-	-	0.002	10466
456	0.000089	514	0.000035	196	0.000012	64	-	-	0.0025	13085
486	0.000025	154	0.00003	182	0.000015	89	-	-	0.000064	385
488	-	-	-	-	-	-	-	-	0.00016	962
508	0.000013	77	-	-	-	-	-	-	0.000027	153
524	0.000013	77	-	-	-	-	-	-	0.000026	148
546	0.000018	106	-	-	-	-	-	-	0.000016	90
550	-	-	-	-	-	-	-	-	0.0002	1045
557	0.00084	4559	0.00055	2896	0.0004	2029	-	-	0.0027	13234
559	0.00035	1897	0.0002	1052	0.00012	608	-	-	0.0013	6351
561	0.00069	3739	0.00063	3313	0.00046	2331	-	-	0.0016	7812
574	0.0018	10057	0.0012	6498	0.00095	4946	0.00017	944	0.0031	16026
575	0.002	11171	0.00175	9472	0.0015	7808	0.00029	1610	0.0037	19115
576	0.0014	7824	0.0013	7040	0.0011	5728	0.00044	2442	0.0022	11378
578	0.00025	1403	0.0002	1088	0.00014	732	-	-	0.00043	2240
580	0.00047	2643	0.00047	2560	0.00043	2252	0.00012	668	0.00084	4388
582	0.00011	657	0.00009	521	0.000023	128	-	-	0.00038	2160
584	0.000035	204	-	-	-	-	-	-	0.00023	1261
591	0.00052	2978	0.0003	1666	0.00025	1333	-	-	0.0021	10830
595	0.00074	4839	0.0007	4547	0.00055	3543	0.00031	2014	0.0011	7139
596	0.0014	9148	0.0002	1297	0.00011	707	-	-	0.00072	4665
607	0.00019	1242	0.0002	1296	0.00017	1091	0.000076	493	0.00028	1814
613	0.0013	7440	0.00095	5273	0.00081	4315	0.000027	152	0.0025	12884
614	-	-	-	-	-	-	-	-	0.00005	265
617	0.00115	6776	0.00085	4862	0.00061	3368	0.000041	236	0.0025	14015
618	0.00027	1590	0.00017	971	0.00011	606	-	-	0.00069	3879
621	0.00087	5681	0.00075	4853	0.0006	3840	0.00032	2071	0.0012	7761
622	0.0012	7906	0.0011	7221	0.0011	7188	0.00068	4464	0.0015	9843
Total additional outflow (L/d)	-	102,081	-	70,800	-	53,798	-	15,819	-	209,058

Note: Case A: bulk decay constant = 1 d^{-1}, demand multiplier = 0.75; Case B: bulk decay constant = 1 d^{-1}, demand multiplier = 1; Case C: bulk decay constant = 1 d^{-1}, demand multiplier = 1.25; Case D: bulk decay constant = 0.75 d^{-1}, demand multiplier = 1; Case E: bulk decay constant 1.25 d^{-1}, demand multiplier = 1.

Finally, other simulations were performed to investigate if blowoffs with larger outflow (e.g., 5-10 L/s) can be flowed for a short period instead of continuous low flow to fix chlorine residuals. However, these simulations proved that blowoffs with larger outflow are ineffective at solving the problem of low disinfectant concentrations at some sites, in addition to causing higher installation and operational costs, due to the device's automatization as well as larger head-losses in the WDN. Therefore, the use of low continuous flows at blowoffs should be preferred.

An example is the blowoff close to the Node 557 in the WaterGEMS simulation with demand multiplier equal to 1, chlorine concentration at source equal to 1 mg/L and bulk decay constant equal to 1 d^{-1}. Table 4.2, Case B shows that an additional nodal outflow of 2,896 L/d can fix the minimum constraint of 0.2 mg/L for chlorine residual. However, a larger outflow of 5 L/s for 1 h from 2 to 3 a.m., leading to a total outflow of 18,000 L/d, is not able to fix this constraint (Figure 4.7, 2-3 a.m. in the last two days of simulation).

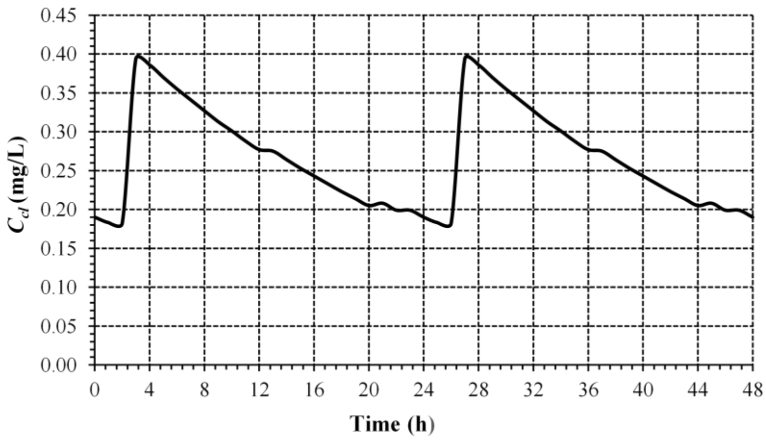


Figure 4.7. Chlorine concentration pattern close to Node 557 in the last three days of simulation in Case B with a blowoff outflow of 5 L/s from 2 to 3 a.m.

In addition to water savings and better effectiveness in terms of chlorine residuals, blowoffs have other benefits compared with hydrant flushing. Due to the lower water discharges and flow velocities, they prevent the sudden remobilization of the biofilm layer present on pipe walls, which potentially may result in undesired water discoloration.

Because hydrants are designed to emit only high flows, the solution proposed in this work could be implemented in real WDNs by installing a small tap for water immediately upstream from the hydrant. If the outflow proposed by the optimizer at the generic node is too small to obtain with the tap, this device should be adjusted to the smallest feasible setting. Of course, this would cause the results in the field to be slightly different from those obtained by the modeling. The blowoff flow can be discharged into the sewer system or into the environment (surface water), or it can be used for other purposes, such as irrigation.

A last comment concerns the practical setting of nodal blowoffs, which should be regulated to cope with seasonal variations in demand and in temperature-related bulk decay. The changes in required flow can be as much as an order of magnitude (Table 4.2). The settings may need to be adjusted as frequently as monthly. The following practical procedure could be adopted: (1) select the blowoff such that it will drain without causing any flooding damages; (2) on a monthly basis, rerun the model with the forecast demand and rate constant (temperature for that month to determine the required flow); (3) if they change significantly, have a crew adjust the opening blowoff; and (4) verify the new flow with a bucket and stopwatch test.

However, it is unlikely, although potentially feasible, that the system operators will adjust the outflows daily, for example, from a dry day with significant lawn and garden irrigation to a rainy period. It is more likely that the operators will adjust these values monthly or seasonally. How conservative the outflow values should be is up to the operators' judgement. A high outflow wastes water, whereas a low value may fail to meet the target on any given day.

Some proposed technology may have a disinfectant residual sensor built into the flushing device. However, this technology will be quite expensive and will require substantial maintenance, e.g., for the instrument configuration and data acquisition, for the replacement of the sensor parts etc. The technique in this work should be more economical.

Chapter 5

Comparison of Techniques for Maintaining Adequate Disinfectant Residuals in a Large Size WDN

The present Chapter provides a numerical comparison between different techniques - booster station and/or nodal blowoff - that can be adopted to increase the residuals in a real WDN. The study considers the injection at WDN sources of 1) chlorine and 2) chloramine. The methodology is based on the use of flow routing/water quality modelling of WDN. The study was presented to the scientific community in the paper "Comparison of Techniques for Maintaining Adequate Disinfectant Residuals in a Full-Scale Water Distribution Network", authored by S. Avvedimento, S. Todeschini, S. Manenti and E. Creaco and published in Water, 14 (7) - March 2022.

5.1. Overview

Water utilities worldwide are required to comply with national water quality regulations [e.g., in Italy (D. Lgs 31/2001) as implementation of the European Council Directive 98/83/EC, in Australia (National Water Quality Management Strategy 2011), in Canada (Health Canada 2010), in China (GB 5749-2006), and in the U.S. (Safe Drinking Water Act)] to provide safe drinking water at the consumers' taps. Maintaining a disinfectant residual within a WDN is an important task to guarantee users' protection from microbial contamination. When disinfectant is provided at WDN sources, it will be hardly maintained throughout the system. In fact, disinfectant interacts with the organic material in the bulk water and/or with the biofilm on the surface of the pipes, resulting in a fast decay (Vasconcelos et al. 1997; Boccelli et al. 2003; Zhu et al., 2020).

Particularly, terminal sections of WDN, also called dead-end sections, are well known to be problematic zones in terms of water quality degradation (Abokifa et al. 2016). In these sections, low flow conditions and high residence times lead to excessive decay of the disinfectant upstream from users. Consequently, in some terminal nodes, also called dead ends, disinfectant residuals decrease to values lower than the minimum as prescribed by the technical guidelines (D. Lgs 31/2001; National Water Quality Management Strategy 2011; Health Canada 2010; GB 5749-2006; Safe Drinking Water Act; WHO 2017).

To tackle the problem of low disinfectant concentrations, there are several operational measures that can be adopted to meet the residual target, each with its own pros and cons. A possible solution may be to increase the disinfectant dose at source(s). However, this may lead to excessive disinfectant concentrations near the feeding points, resulting in taste and odour problems, as well as the formation of DBPs (e.g., THMs, HAAs) considered harmful for public health (Villanueva et al. 2015; Li and Mitch 2018).

Another solution may be to install booster disinfectant stations throughout the network. Booster disinfection reapplies disinfectant at strategic locations within the distribution system to compensate for the losses that occur as it decays over time (Tryby et al. 2002). However, this solution causes an increase in installation and operational costs for the water utility.

In the reviewed scientific literature, many works were dedicated to the use of booster chlorine stations (Boccelli et al. 1998; Tryby et al. 2002; Prasad et al. 2004; Lansey et al. 2007; Meng et al. 2013; Goyal and Patel 2018) and their combination with other measures, i.e., with the real-time optimal valve operation in Kang and Lansey (2010). Most of the mentioned studies formulated the problem of booster stations as an optimization using different optimization techniques and searching for solutions in terms of optimal injection scheduling, operation, and locations of booster stations. Ohar and Ostfeld (2014) extended the problem of optimal design (overall placement and construction costs) and operation (chlorine dose) of booster stations to the reduction in the formation of THMs concentrations while delivering water with acceptable residual chlorine. A comprehensive literature review on the optimization of booster chlorination stations can be found in the works of Islam et al. (2017) and Mala-Jetmarova et al. (2017).

Although the placement of booster stations provides a more uniform distribution of disinfectant residuals within the system, it does not address the issue in dead-end sections of WDNs, for which other interventions, such as additional outflows or flushing, should be considered. Water flushing involves moving water at high velocity through the distribution system and discharging it through flushing devices, hydrants, or blowoff ports. The increase in flow velocity in proximity to dead-ends reduces the time available for the above-mentioned interactions thus leading to a lower disinfectant decay. Many U.S. utilities regularly schedule flushing programs [e.g., in the spring and the fall (Hubbs 2020)], while others flush on an as-needed basis (Friedman et al. 2003). Flushing is especially common during warmer months since chlorine-based disinfectants are consumed in water more rapidly at higher temperatures (Hubbs 2020). The use of flushing strategies has been proposed by many researchers as a good management practice for improving water quality in WDNs (Walski and Draus 1996; Antoun et al. 1997; Friedman et al. 2002; Friedman et al. 2003; Barbeau et al. 2005; Carrière et al. 2005; Baranowski and Walski 2009; Poulin et al. 2010; Deuerlein et al. 2014; Xie et al. 2014).

Flushing activities can be operated in two ways, i.e., continuously through a manual flusher or at intermittent times by an automated one. Instead of intensive intermittent flushes, low outflows can be used from blowoffs. A blowoff is a flushing device that allows obtaining a continuous flow at low rate at a dead-end node causing fewer undesired effects in terms of service pressure decrease, compared with typical intense flushing. It can be opened at a hydrant site close to the generic critical dead-end node, with the objective to eliminate water with low residuals. Nodal outflows through blowoffs are designed to cause the smallest increase in pipe outflows for guaranteeing sufficient disinfectant residuals. This measure involves minimal capital and labour investment but can increase the amount of revenue water (Walski 2019). The use of continuous blowoffs to correct violations in dead-end nodes of WDN was investigated in the Chapter 4. With the same water volume flushed, the effects of intermittent outflows should be investigated. In the case of intermittent outflows, water is provided for only limited durations with larger outflows. However, due to the larger outflow values, this solution may cause service pressure deficits in the WDN and undesired sediment mobilization if not done properly. Additionally, higher installation costs due to the automatization should be considered.

Regarding the disinfectant typically used in treatment plants, chlorine products have been used since the 20th century to disinfect drinking water (Zhang et al. 2017). Unfortunately, this widely used free-chlorine treatment has disadvantages, including the high reactivity of chlorine with NOM and the production of DBPs, some of which are likely human carcinogens (Duirk and Valentine 2007; Zhang et al. 2017). Hence, an option lies in permanently switching disinfectants to maintain a residual, typically from free chlorine to chloramine due to its slower decay. A study conducted by Kadwa et al. (2018) showed that for initial concentrations of chlorine (as Cl_2) and of chloramine (as NH_2Cl), both set to 2.4 mg/L, chlorine falls below detectable limits (0.05 mg/L) much earlier (7 days) compared with chloramine (11 days).

The disinfection efficiency is dependent on several factors e.g., the disinfectant concentration, contact time, pH, and temperature (Agudelo-Vera et al., 2020). Disinfectant concentration and contact time are integral to disinfection kinetics and the practical application of the CT value (LeChevallier and Au, 2004). The

CT value, equal to the disinfectant concentration (in mg/L or ppm) multiplied by the contact time (min), is commonly used to gauge the effectiveness of the disinfectant residual against different pathogens (CDC, 2022). The lower the CT value, the more effective the disinfection agent. Of the two disinfectants, chloramines are the weaker, requiring significantly higher CTs to achieve levels of inactivation of pathogens comparable with free chlorine (USEPA, 2007). A review of CT values and corresponding inactivation rates for specific pathogens in the presence of free chlorine and chloramines is provided in (USEPA, 2007). Overall, it was demonstrated that only free chlorine was able to provide 99.99 percent (4-log) inactivation of viruses. To provide 99 percent (2-log) inactivation of most species, free chlorine and chloramine required a CT of <150 and 10,000 min mg/L, respectively (USEPA, 2007). References on the effectiveness of chlorine-based disinfectants at pathogen inactivation can be found in (LeChevallier and Au, 2004; CDC, 2022; USEPA, 2007).

Starting from the early 2000s, several water systems throughout North America have converted to chloramine for improving stability, taste, odor, and DBPs control (Kirmeyer et al. 1993; USEPA 2012). Although chloramine is less reactive than free chlorine in producing regulated DBPs in combination with NOM, it still forms some DBPs, such as N-nitrosodimethylamine (NDMA) (Kirmeyer et al. 2004). Chloramines, such as monochloramine, dichloramine and trichloramine, are generated by the reaction of chlorine with ammonia (WHO 2017). The relative amounts of each of these species are dependent on pH, temperature, contact time, and the ratio of Cl_2 to NH_3-N (Huang 2008). In WDNs, monochloramine is the dominant and favourite species adopted because of its biocidal properties, relative stability, and relatively low taste and odour properties (Kirmeyer et al. 2004).

While prediction of chlorine performance can be made with widely available state-of-the-art chlorine decay modelling, the same cannot be said for chloramine performance. Starting from the first chloramine decay models proposed by (Vikesland et al. 2001) and (Duirk et al. 2005), few authors (Fisher et al. 2009; Alexander and Boccelli, 2010; Ricca et al. 2019) provided the evaluation of a multi-species chloramine model through the use of field-scale measurements and distribution system network modeling. None of the authors have explored the effect of operational techniques (i.e., booster stations or flushing) on the increase in chloramine residuals in WDNs. Of course, chloramines have their own issues such as the production of ammonia, which has the potential to promote nitrification reactions within the system. Nitrification can have adverse impacts, such as the reduction in chloramine residuals, alkalinity, the promotion of bacterial regrowth (Wilczak et al. 1996) and the corrosion of infrastructural elements (Zhang et al. 2008). Intensive monitoring of the major water quality parameters (chlorine/chloramine residuals, DBP concentrations etc.) within WDNs at a certain frequency and at specified locations is important to ensure that the water supplied is in compliance with the required guidelines and standards.

The WHO publishes and regularly updates the GDWQ, which has become an authoritative basis for the setting of national regulations and standards for water safety in support of public health (WHO 2017). With regard to disinfectant residuals, the maximum values for free chlorine and monochloramine are set to 5 mg/L and 3 mg/L, respectively (WHO 2017). These levels are greatly in excess of the residuals of chlorine and monochloramine found in drinking water supplies, which typically range from 1-2 mg/L (WHO 2017; Health Canada 2018). At the point of delivery, a minimum residual concentration of 0.2 mg/L of free chlorine should be maintained throughout the distribution system, while it is normal practice to supply water with a chloramine residual of 0.1-0.15 mg/L to act as a preservative during distribution (WHO 2017). In the absence of field measurements or as a complement, simulation models such as EPANET support water utilities operators in understanding the problem and taking operational decisions for low residuals based on local conditions.

As mentioned above, there have been various works dedicated to modeling of disinfectants chlorine/chloramine and use of booster stations, flushing hydrants/blowoffs in WDNs; however, to the best of our knowledge, little attention has been dedicated to the analysis and comparison of the effects of the various solutions, implemented alone or in a combined way, on a large-scale WDN. For example, Propato and Uber (2004) developed a simulation framework to assess the vulnerability of a WDN to microbiological contamination. Their study showed that the risk of consumer exposure is affected by the residual maintenance strategy employed. A chloramine residual, instead of free chlorine, may weaken the final barrier against pathogen intrusions. On the other hand, the addition of a chlorine booster station may improve consumer protection without requiring excessive disinfectant doses.

In the light of the above considerations, the work presented in this chapter addresses the existing research gap in analyzing and comparing different operational measures aimed at ensuring safeguard requirements for water quality in a real large-scale WDN.

5.2. Methodology

In this work four solutions are compared to increase disinfectant residuals in a full-scale WDN. As showed in the following Table 5.1, five scenarios of the network model are analysed.

Table 5.1. Number of violating nodes for each scenario analysed.

Scenario	Disinfectant	N. of Booster station	Flushing Blowoffs	N. of violating nodes
0	Chlorine	0	0	41
1	Chlorine	3	18-continuous	0
2	Chlorine	3	18-intermittent	0
3	Chloramine	0	0	18
4	Chloramine	0	12-continuous	0

Note: violating nodes = nodes with a disinfectant residual below the minimum requirement (e.g. in this study c_{min} = 0.2 mg/L).

Scenario 0 represents the network's behavior to the injection of chlorine as a disinfectant at the sources. To correct the disinfectant violations encountered in scenario 0, scenario 1 considers the installation of booster stations and continuously dripping blowoffs. Scenario 2 differs from scenario 1 due to the adoption of intermittent outflows in lieu of continuous outflow. In scenario 3, chloramine is adopted as a disinfectant and the network's response is modeled. Finally, scenario 4 considers the placement of continuous dripping blowoffs as a measure to meet the residual target in the chloraminated network. In this last case, no booster stations were placed in bulk areas since violations in disinfectant residuals occurred only at critical dead-end nodes of the network.

These potential solutions can be developed by making use of models that simulate WDN behaviour in terms of both flow routing and water quality (disinfectant decay). The software EPANET ver. 2.2 and its multi-species extension EPANET-MSX is used to simulate the chlorine and chloramine decay, respectively. It was linked to the Matlab R2021a environment to extract and analyse the water quality results at all WDN nodes. The procedure for each scenario analysed is described in the following subsections.

5.2.1. Scenario 0 - Chlorine

In this scenario, the network's response to the injection of chlorine at the WDN sources is modelled. In the model adopted, the chlorine decay simulation is first order [Eq. (2.1)]. For all the links, the bulk decay constant (k_b) is assumed to be 0.5 d^{-1} from the scientific literature (Rossman et al. 1994; Powell et al. 2000; Propato and Uber 2004; Nejjari et al. 2013) and the wall decay constant (k_{w}) is set to 0 because it is supposed that network pipes are made of plastic material (smooth surface of pipes' internal wall). This hypothesis is justified by the practical evidence that plastic is the most popular material for water pipes and the values of decay constants are consistent with those adopted in the mentioned studies.

First, a constant value of chlorine concentration C_{cl}, as an example, a concentration C_{cl} = 2.0 mg/L, is injected into the WDN sources. Then, the flow routing/water quality simulation of the WDN is run to model the hydraulic and water quality behaviour of the WDN and to create a list of nodes with a deficit of disinfectant residuals. The resulting nodes are sorted in descending order of maximum deficit. Modelling chlorine, this residual can be evaluated taking the minimum value $c_{cl,\mathrm{min}}$= 0.2 mg/L as a benchmark. It must be highlighted that the zero-demand nodes are not considered in the list of violating nodes because they are meaningless for this kind of analysis. In fact, disinfectant deficits are dangerous only in the case of water consumption at the generic node.

5.2.2. Scenario 1 - Chlorine, Booster Stations, and Continuous Blowoffs

The water quality outputs (chlorine residuals) obtained in scenario 0 are used for the implementation of booster stations and nodal blowoffs as operational measures. Simulations are used to evaluate the placement first of booster stations and then of dripping blowoffs.

Booster stations reapply disinfectant at intermediate locations of the WDN to obtain a more uniform distribution of disinfectant while keeping residuals within specific limits. It is important that a booster station delivers disinfected water to as many nodes as possible. Theoretically, a booster station can be located at any node of a WDN. Therefore, the potential number of booster stations is equal to the number of nodes in WDN. However, in large networks, the exploration of each node of the WDN as a potential location for a booster station would make the computation difficult and very demanding from the computational viewpoint. In this study, suitable locations for the installation of a booster station are chosen by simulation attempts, relying on network hydraulics, and selecting bulk areas experiencing low or intermittent residual coverage. Booster stations are modeled as set point injections, delivering a constant mass dosage rate of chlorine C_{cl} = 2.0 mg/L. A criterion used for selecting a booster location is the reachability (number of nodes that can receive disinfected water from the booster node). The reachability of a node is determined by simulating a constant chlorine concentration C_{cl} = 2 mg/L for the generic node i and by determining the resulting number of nodes with chlorine residuals greater or equal to the minimum value $c_{cl,min}$. In this study, the minimum value of reachability is set to 5, meaning that the placement of a booster station is required to increase the chlorine residuals of 5 nodes above the target $c_{cl,min}$ in the water quality monitoring window considered.

Beyond the bulk areas affected by low disinfectant residuals, there can still be critical nodes scattered in the WDN and located at various dead ends. This happens due to the occurrence of low flow velocities and high residence times that cause an excessive disinfectant decay upstream from users. Unless as many boosters are installed as the number of critical nodes, the problem cannot be solved by sticking to the booster stations solution alone. Therefore, for these nodes the implementation of continuous outflows through nodal additional outflows may be effective. This consists of a slight increase in the nodal outflow at the generic critical node all day long, through the opening of a dripping blowoff at the hydrant site. In fact, this results in an almost constant outflow at the critical dead-end node to be added to the outflow to users and to leakage outflow.

The continuous low flow scenario was proposed in the previous Chapter by making use of emitters in the software EPANET [see Eq. (4.1)].

With similar effects, the additional outflow can be obtained by introducing a new demand category at the generic node of the WDN. For the generic node i in the list of critical nodes, a new demand category with multiplicative daily temporal pattern constantly equal to 1 is added in the software to represent the nodal outflow through blowoff. Then, the lowest blowoff demand that fixes the nodal disinfectant residual is searched for by trials, corresponding to a certain daily blowoff volume V_i (L). Since this blowoff demand is applied for 24 h in a day, it is indicated as $q_{i,24.}$ (L s^{-1}). Its relationship with V_i is expressed by

$$V_i = 24 \cdot 3{,}600\ q_{i,24} \tag{5.1}$$

5.2.3. Scenario 2 - Chlorine, Booster Stations, and Intermittent Blowoffs

Keeping the operational measures identified in the previous scenario, a comparison between continuous and intermittent blowoffs is carried out. The water volume being the same, water is provided through blowoffs for all day long or for only limited durations, respectively.

At the generic node *i*, intermittent outflow sub scenarios can be created for the same daily value of V_i in Eq. (5.1). As an example, let us assume a sub scenario with *k* h of a blowoff operation and (24-*k*) h of no blowoff operation in the day. The water discharge of the blowoff can be calculated through the following formula

$$q_{i,k} = \frac{24}{k}\ q_{i,24} \tag{5.2}$$

As a particular case of an intermittent flow sub scenario, Eq. (5.2) returns the continuous flow scenario for $k = 24$, i.e., for the blowoff duration of 24 h in the day. The demand coefficient pattern for a sub scenario with *k* hours of blowoff operation is made up of *k* values equal to 1 and (24-*k*) values equal to 0. In this context, we assume that outflows are regularly spaced in time in the day. Therefore, if *j* is the hour in the day when the first outflow takes place, the following hours of blowoff will be $j + (24/k)$, $j + 2\ (24/k)$, $j + 3\ (24/k)$, and so forth, up to the end of the day.

Let us assume that we want to split a water volume $V_i = 86{,}400$ L into $k = 3$ hours of blowoff. Starting from Eqs. (5.1) and (5.2), we obtain $q_{i,24} = 1$ L/s and $q_{i,3} = 8$ L/s, respectively. If we assume that the first outflow takes place at hour $j = 1$, the following ones will be at hours 9 and 17.

Therefore, for the generic intermittent flow sub scenario with *k* hours of outflow, the hour *j* of the first blowoff becomes a decisional variable of the problem, which can take on all integer values between 1 h and $(24/k)$ h. As an example, in the case of the sub scenario with $k = 3$ hours of outflow, it ranges from 1 h to 8 h because 8 h is the last hour that enables having 3 one-hour-long blowoffs regularly spaced in the day, i.e., at times 8 h, 16 h, and 24 h. In the calculations, j is optimized in such a way as to maximize the effectiveness of the intermittent outflow for fixing the disinfectant residual deficit at the node. This is accomplished by minimizing the total duration $v_{k,j}$ (min) of residual deficit violations at the node, given by

$$f_j = \min\,(v_{k,j}) \tag{5.3}$$

The methodology is applied by considering 8 different sub scenarios of outflows' operation time: a sub scenario of continuous flow and 7 sub scenarios of intermittent flows, with k values equal to 1, 2, 3, 4, 6, 8, and 12, respectively.

5.2.4. Scenario 3 - Chloramine

This scenario investigates the effects of switching the disinfectant from chlorine to chloramine. Hence, the network's response to the chloramine injection is modeled. The chloramine reaction model used in this work was developed previously by (Vikesland et al. 2001) and (Duirk et al. 2005) and takes account of the chloramine decay due to auto decomposition alone and due to the chloramine decay because of auto decomposition in the presence of NOM. The reaction model converted into an EPANET-MSX file consists in 14 bulk species and no surface species (Table 2.4 in Chapter 2). In the absence of field measurements and in order to make a comparison with the chlorine model described previously, the initial condition for the monochloramine dose is set to 2 mg/L at each source. The mean values of parameters $CaCO_3$ (alkalinity) and pH for all nodes are set respectively to 200 mg/L and 7.75 (Shang et al., 2008). The sources are assigned a TOC concentration of 0.5

mg/L [in drinking water, values for TOC are typically < 1 mg/L, (APAT 2004)] consisting of 1% slow reacting sites and 42% fast reacting sites in the NOM structure. The values adopted are consistent with typical values founded in WDN sources and with experiments carried out in the scientific literature. Using the chloramine decay model descripted, the network is run in EPANET-MSX, and nodes with a minimum chloramine concentration $c_{ch,\min} = 0.2$ mg/L are searched for.

5.2.5. Scenario 4 - Chloramine and Continuous Blowoffs

Based on the results concerning the chloramine residuals, additional techniques (boosters or dripping blowoffs) are implemented in this last scenario. The procedure carried out in scenario 2 is repeated for the chloraminated network.

5.2.6. Estimation of Total Volume of Water and Total Mass of Disinfectant

For each scenario considered, the average total volume of water delivered Vol (m^3) and the average total mass dose of disinfectant supplied W (Kg) in the simulation analysis are estimated by using Eqs. (4.2) and (4.3).

5.3. Application

Case Study

The case study considered in this work is the network model from the Battle of the Water Sensor Networks 2006 (BWSN Network 2) (Ostfeld et al. 2008).

This large network is made up of 12,523 nodes, 2 reservoirs, a source (well), 2 tanks, 14,822 pipes, 4 pumps, and 5 valves (layout in Figure 5.1). Figure 5.1 is intended to be a schematic representation of the network model used as a benchmark in this study. This figure aims to show the network size and where the sources are located to clearly define what the disinfectant path is before reaching the final users of the network. All the pipes are assumed to feature a Hazen-William roughness coefficient of 140, a diameter ranging up to 1219 mm, and a length from to 1 to 4019 m. Nodes are assumed to have an elevation between 0.00 and 40.67 m above sea level (ASL), and a base demand ranging from 0 to 15.55 L/s. Among the network nodes, 1971 nodes are zero-demand nodes and are then excluded from the analysis of violations. Tanks use a completely mixed modelling technique. There are simple control statements that affect the operations of pumps and valves surrounding each tank. The network is subject to five variable demand patterns. WDN emitters corresponding to leakage are tuned in such a way as to obtain a percentage of leakage around 15%, which is a reasonable value for modern and well-maintained WDNs.

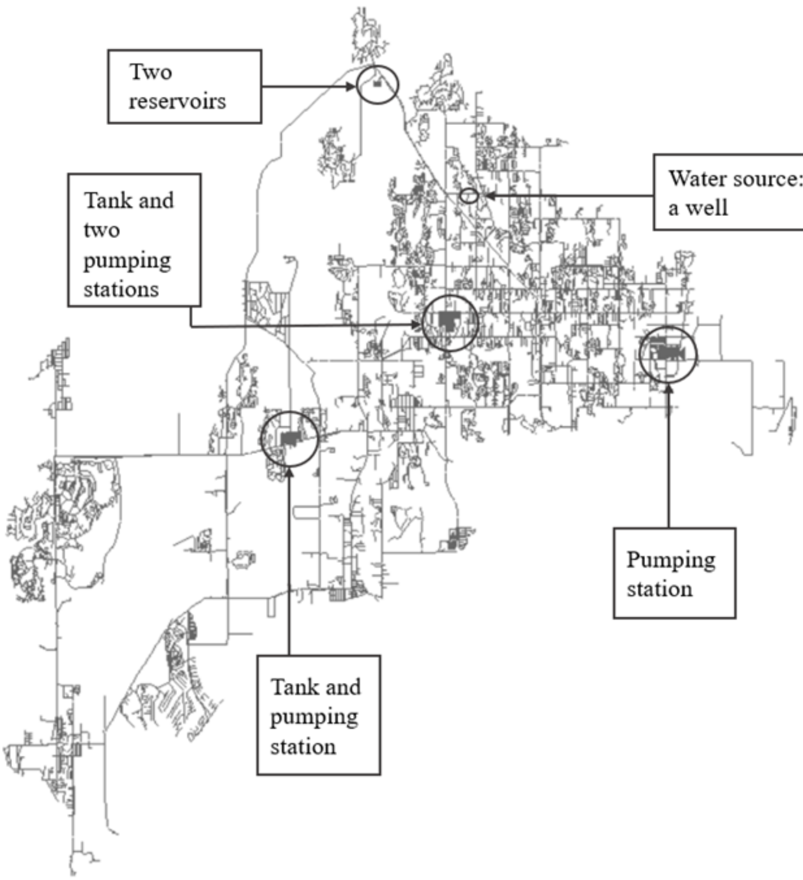


Figure 5.1. Case study layout (not to scale) based on Ostfeld et al. (2008).

For water quality simulations, chlorine and chloramine are chosen as disinfectant for scenarios 0, 1, 2 and for scenarios 3 and 4, respectively. Both disinfectants are supplied at the sources and booster stations with a constant concentration of 2 mg/L. The initial chlorine/chloramine concentration is set to 0 at all WDN nodes. A simulation duration of 240 h (10 days) is used for the analysis to make sure that the disinfectant injected into the sources has enough time to reach the terminal nodes of the network and to reach well-established cyclical operating conditions in the last day of simulation. The hydraulic and water quality time steps used in calculations are 1 h and 5 min, respectively. The constraints used in all scenario models require that residual concentrations of both chlorine and chloramine be maintained between a minimum c_{min} = 0.2 mg/L and a maximum C_{max} = 2.0 mg/L over the last 48 h monitoring time window. Therefore, this time window is considered to evaluate disinfectant violations.

5.4. Results

Generally, water quality simulations indicated that, for both disinfectants injected into the three sources, residual requirements were not satisfied in all WDN nodes without the implementation of boosters and additional outflows at critical nodes. Specifically, 41 violating nodes were identified in the network in scenario 0, with a chlorine concentration below $c_{cl,min}$ = 0.2 mg/L. (Table 5.1). Water quality simulations showed that violations occurred in both bulk areas and in terminal sections of the network affecting many dead-end nodes.

The results obtained for scenario 1 show that the placement of three booster stations and the opening of 18 nodal additional continuous outflows can increase overall chlorine concentrations in WDN (Table 5.1, Figure 5.2).

Table 5.2. Blowoff emitters *e* and average outflows *q* of flushing blowoffs in scenarios 1 and 4.

Node ID	e (L/s/m$^{1/2}$) Scenario 1	q (L/s) Scenario 1	e (L/s/m$^{1/2}$) Scenario 4	q (L/s) Scenario 4
941	0.0052	0.033	0.003	0.019
1800	0.026	0.192	0.023	0.170
2330	0.0081	0.056	0.0025	0.018
2340	0.013	0.117	0.0046	0.054
3220	0.0024	0.071	-	-
3491	0.016	0.154	0.0064	0.078
3510	0.015	0.127	0.0069	0.062
3844	0.016	0.124	0.014	0.109
3618	0.0068	0.081	-	-
3857	0.0033	0.027	-	-
4181	0.005	0.046	0.0025	0.028
4910	0.0038	0.032	-	
5056	0.038	0.238	0.033	0.208
8057	0.028	0.228	0.013	0.121
8480	0.042	0.352	0.018	0.175
8476	0.0032	0.149	-	-
8954	0.0028	0.029	-	-
10046	0.041	0.381	0.014	0.169

Note: Scenario 1: disinfectant chlorine – 3 booster stations and 18 flushing blowoffs. Scenario 4: disinfectant chloramine – no booster stations – 12 flushing blowoffs.

The analysis of the nodes in terms of low chlorine residuals and high reachability pointed out that nodes 1853, 3854, and 12,346 may be suitable locations for a booster station. Since these booster stations serve three bulk areas of the network, they helped in increasing the chlorine concentrations of the neighbouring nodes. The reachability (number of nodes that receive disinfected water from the booster node) is equal to six for both booster 3854 and 12,346 and five for booster 1853. The problem of low residuals can be solved by placing booster stations at nodes 1853 and 12,346 without any additional operational measure in the surrounding areas. Conversely, though increasing chlorine residuals in the area served, the booster at node 3854 required placement of additional nodal outflows to be used at three critical dead-end nodes not reachable by the booster. Beyond the three bulk areas, there were still critical dead-end nodes scattered in the WDN. For these nodes, 18 continuous blowoffs (including the three ones placed in the bulk area served by booster 3854) were opened all day long to fix chlorine residuals.

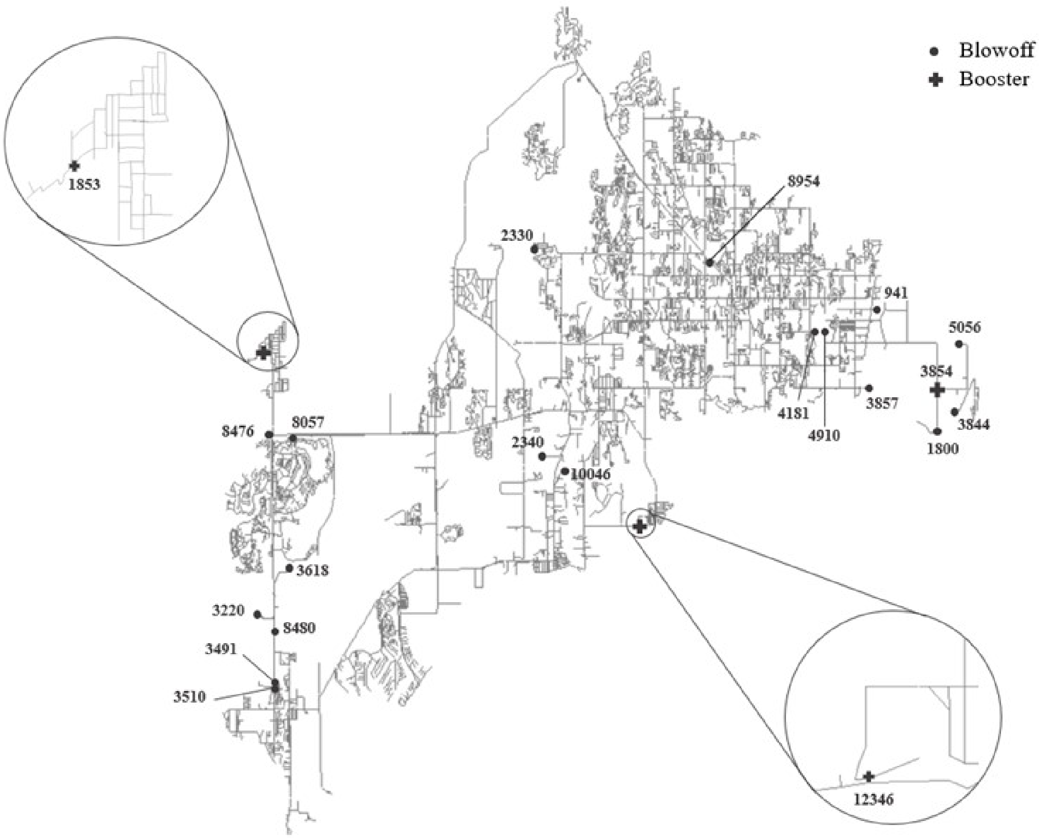
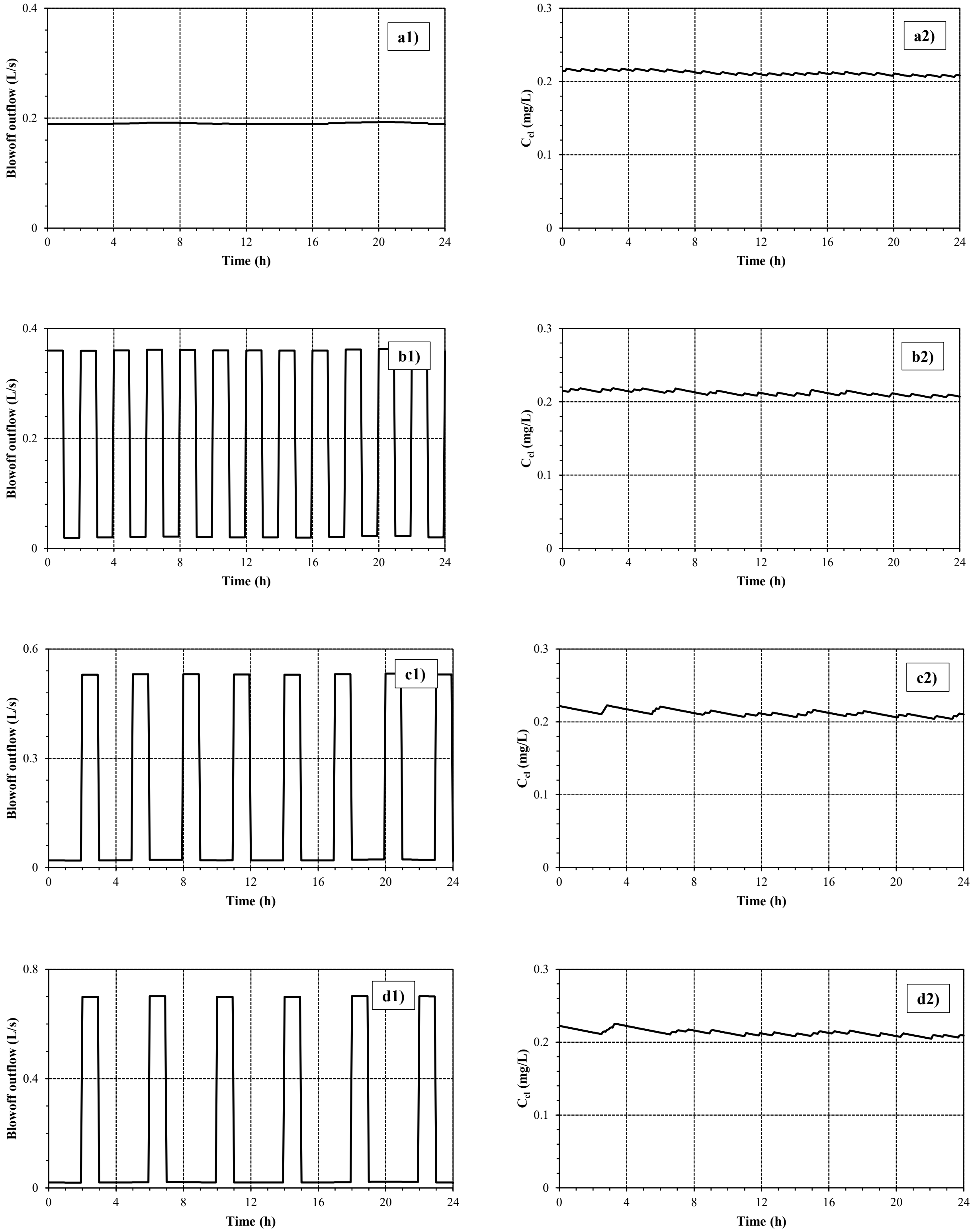


Figure 5.2. Booster stations and flushing blowoff placement in the network for scenarios 1 and 2.

The blowoff emitter coefficients were tuned in such a way to obtain the lowest blowoff outflows that correct chlorine deficits. The emitter coefficients and the nodal average outflows (including outflows to fix chlorine residuals and leakage outflows) for critical dead-end nodes in WDN for scenario 1 are reported in Table 5.2.

The comparison between continuous flow and intermittent flow was carried out for all nodal blowoffs placed in the WDN. Intermittent blowoffs were considered only in the chlorinated network due to the higher computation times required for the chloraminated network. It must be remarked that intermittent flows, like continuous flows, never cause service pressure deficits in the WDN, except in case the intermittent blowoff at node 8480 is opened 1 h per day. In fact, the average outflow of 1 h flushing for node 8480 is almost 7 L/s, an excessive value compared with the outflows obtained for the other intermittent blowoffs. Hence, it was deemed that, in all the sub scenarios of 1 h flushing considered, all intermittent blowoffs were opened at 1 h per day while considering continuous flow (24 h) only for node 8480. As a representative situation, the comparison between continuous and intermittent outflows is reported just for nodes 1800 and 3510, both located in peripheral areas of WDN. For each intermittent supply sub scenario, the one minimizing f_j was chosen.

As for node 1800, the minimum continuous outflow that fixed chlorine residual above 0.2 mg/L was that with $q_{1800,24}$ = 0.17 L/s, corresponding to a daily volume V_{1800} = 612 L. For this value of blowoff volume V_{1800}, intermittent outflow sub scenarios were generated using the procedure described in Section 5.2.3 for scenario 2. The graphs in Figure 5.3 show the patterns of flow rate supplied by blowoff and chlorine concentration at node 1800 for both continuous and intermittent supply sub scenarios for the j that minimized the total duration of residual chlorine deficit violations at the node. These patterns refer to the last day of the 10-day long flow routing/water quality simulation.



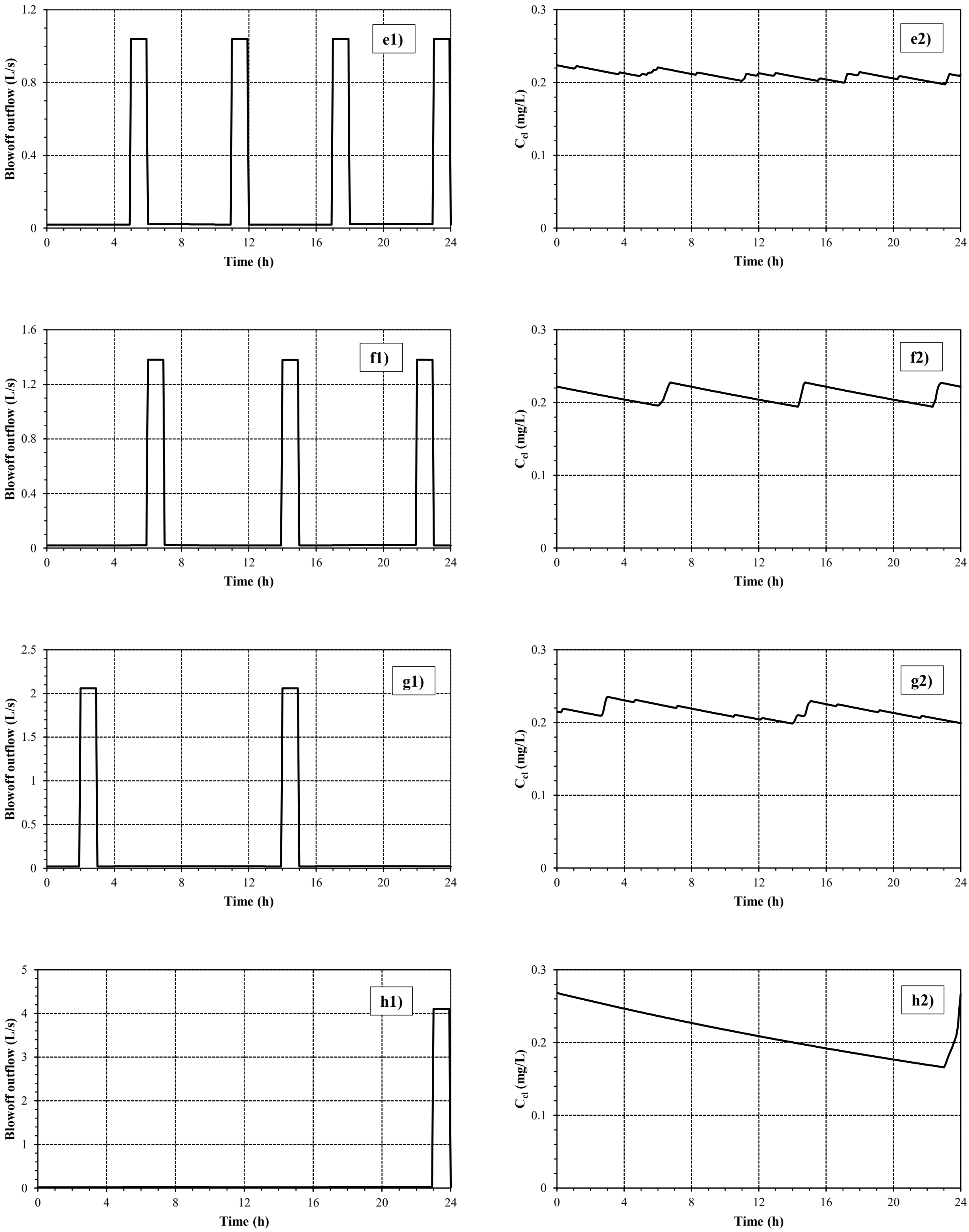


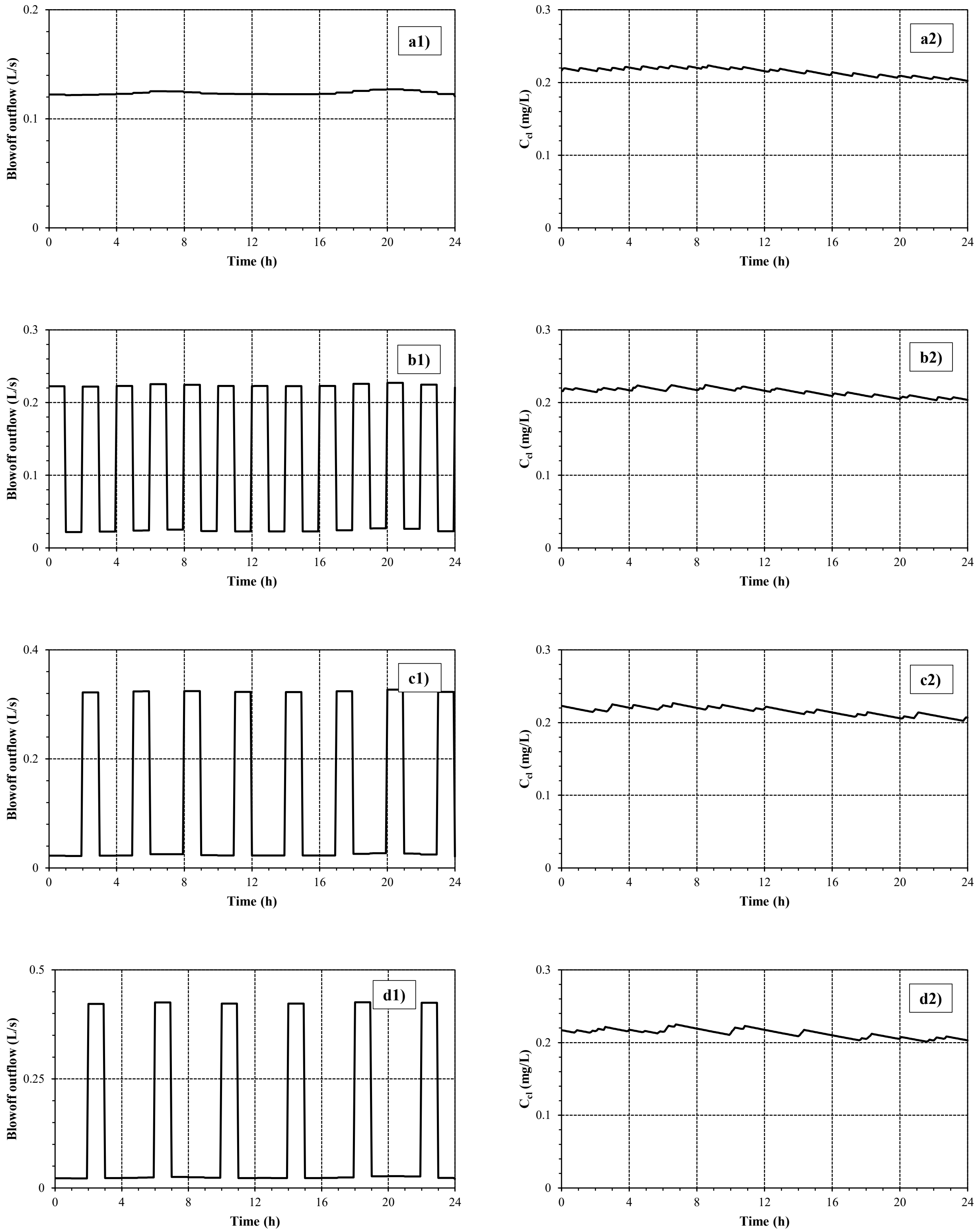
Figure 5.3. Blowoff outflow (1) and chlorine concentration (2) patterns at node 1800 in the last day of simulation in outflow sub scenarios 2 with k = 24 h (**a1,a2**), 12 h (**b1,b2**), 8 h (**c1,c2**), 6 h (**d1,d2**), 4 h (**e1,e2**), 3 h (**f1,f2**), 2 h (**g1,g2**) and 1 h (**h1,h2**) per day.

Table 5.3 reports the main features of the blowoff sub scenarios in terms of number k of hours of flow, blowoff flow $q_{1800,k}$, first hour j of outflow in the day, and duration $v_{k,j}$ of chlorine residual violations in the day.

Table 5.3. Summary of outflow scenarios for the node 1800.

Sub Scenario	k Hours of Blowoff in the Day	Blowoff Flow $q_{1800,k}$ (L/s)	First Hour j of Outflow in the Day (h)	Duration $v_{k,j}$ of Violations (min)
2a	24	0.17	1 (from 0 h to 1 h)	0
2b	12	0.34	1 (from 0 h to 1 h)	0
2c	8	0.51	3 (from 2 h to 3 h)	0
2d	6	0.68	3 (from 2 h to 3 h)	0
2e	4	1.02	6 (from 5 h to 6 h)	0
2f	3	1.36	7 (from 6 h to 7 h)	150
2g	2	2.04	3 (from 2 h to 3 h)	0
2h	1	4.08	24 (from 23 h to 24 h)	515

As for node 3510, the minimum continuous outflow that fixed chlorine residual above 0.2 mg/L was that with $q_{3510,24}$ = 0.1 L/s, corresponding to a daily volume V_{3510} = 360 L. For this value of blowoff volume V_{3510}, intermittent outflow sub scenarios were generated using the procedure described in section 5.2.3 for scenario 2. The graphs in Figure 5.4 show the patterns of flow rate supplied by blowoff and chlorine concentration at node 3510 for both continuous and intermittent supply sub scenarios for the j that minimized the total duration of residual chlorine deficit violations at the node. These patterns refer to the last day of the 10-day long flow routing/water quality simulation.


a1)
Blowoff outflow (L/s)
Time (h)
a2)
Ccl (mg/L)
Time (h)
b1)
Blowoff outflow (L/s)
Time (h)
b2)
Ccl (mg/L)
Time (h)
c1)
Blowoff outflow (L/s)
Time (h)
c2)
Ccl (mg/L)
Time (h)
d1)
Blowoff outflow (L/s)
Time (h)
d2)
Ccl (mg/L)
Time (h)

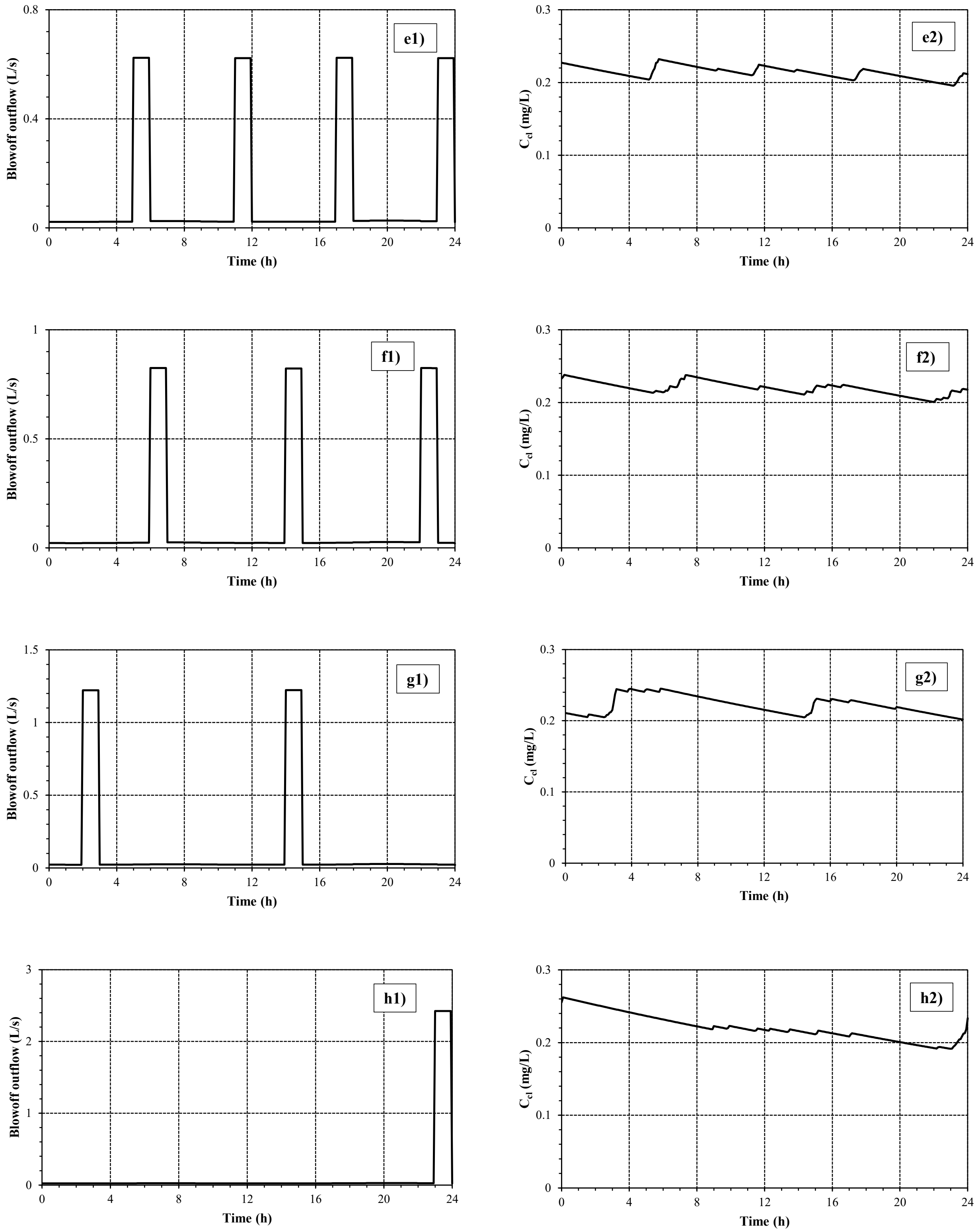


Figure 5.4. Blowoff outflow (1) and chlorine concentration (2) patterns at node 3510 in the last day of simulation in outflow sub scenarios 2 with k = 24 h (**a1,a2**), 12 h (**b1,b2**), 8 h **(c1,c2**), 6 h (**d1,d2**), 4 h (**e1,e2**), 3 h (**f1,f2**), 2 h (**g1,g2**) and 1 h (**h1,h2**) per day.

Table 5.4 reports the main features of the blowoff sub scenarios in terms of number k of hours of flow, blowoff flow $q_{3510,k}$, first hour j of outflow in the day, and duration $v_{k,j}$ of chlorine residual violations in the day.

Table 5.4. Summary of outflow scenarios for the node 3510.

Sub Scenario	k Hours of Blowoff in the Day	Blowoff Flow $q_{3510,k}$ (L/s)	First Hour j of Outflow in the Day (h)	Duration $v_{k,j}$ of Violations (min)
2a	24	0.1	1 (from 0 h to 1 h)	0
2b	12	0.2	1 (from 0 h to 1 h)	0
2c	8	0.3	3 (from 2 h to 3 h)	0
2d	6	0.4	3 (from 2 h to 3 h)	0
2e	4	0.6	6 (from 5 h to 6 h)	40
2f	3	0.8	7 (from 6 h to 7 h)	0
2g	2	1.2	3 (from 2 h to 3 h)	0
2h	1	2.4	24 (from 23 h to 24 h)	155

The results presented in Figure 5.3 a) and Figure 5.4 a) confirm the validity of continuous outflow scenario. Referring to intermittent flow sub scenarios of the node 1800, nodal blowoff can fix the minimum constraint of 0.2 mg/L in all the cases except for cases (f) and (h). In case (f), the chlorine residual become slightly lower than the target close to 5th and 6th h ($c_{cl,min}$ = 0.196 mg/L), 13th and 14th h ($c_{cl,min}$ = 0.194 mg/L), and from 21st to 22nd h ($c_{cl,min}$ = 0.194 mg/L). The worst case is the last one, case (h), in which there is a progressive decrease in chlorine concentration, starting from 14th h to 23rd h ($c_{cl,min}$ = 0.166 mg/L). Instead, the use of intermittent blowoff close to node 3510 seems to have benefits on the chlorine residual in all the cases except for cases (f) and (h). Besides the times when the minimum constraint is slightly violated as in the case (f) close to the 22nd and 23rd h ($c_{cl,min}$ = 0.195 mg/L), there is a violation in the case (h) from the 21st to 23rd h ($c_{cl,min}$ = 0.191 mg/L).

Globally, results proved that intermittent outflows are effective at solving the problem of low disinfectant concentrations for all WDN blowoffs, if a percentage of violation of 10-15% for a few hours per day is considered acceptable.

Finally, the choice of using chloramine as an alternative to chlorine was investigated. As expected, chloramine tended to have a slower decay than chlorine. Specifically, 18 violating nodes were identified in the network in scenario 3, with a chloramine concentration below $c_{ch,min}$ = 0.2 mg/L (Table 5.1). These violations occurred only at critical dead-end nodes of the network. Therefore, in scenario 4 no booster stations were placed in bulk areas and fewer flushing blowoffs were installed than those of scenario 1 (Table 5.1, Figure 5.5).

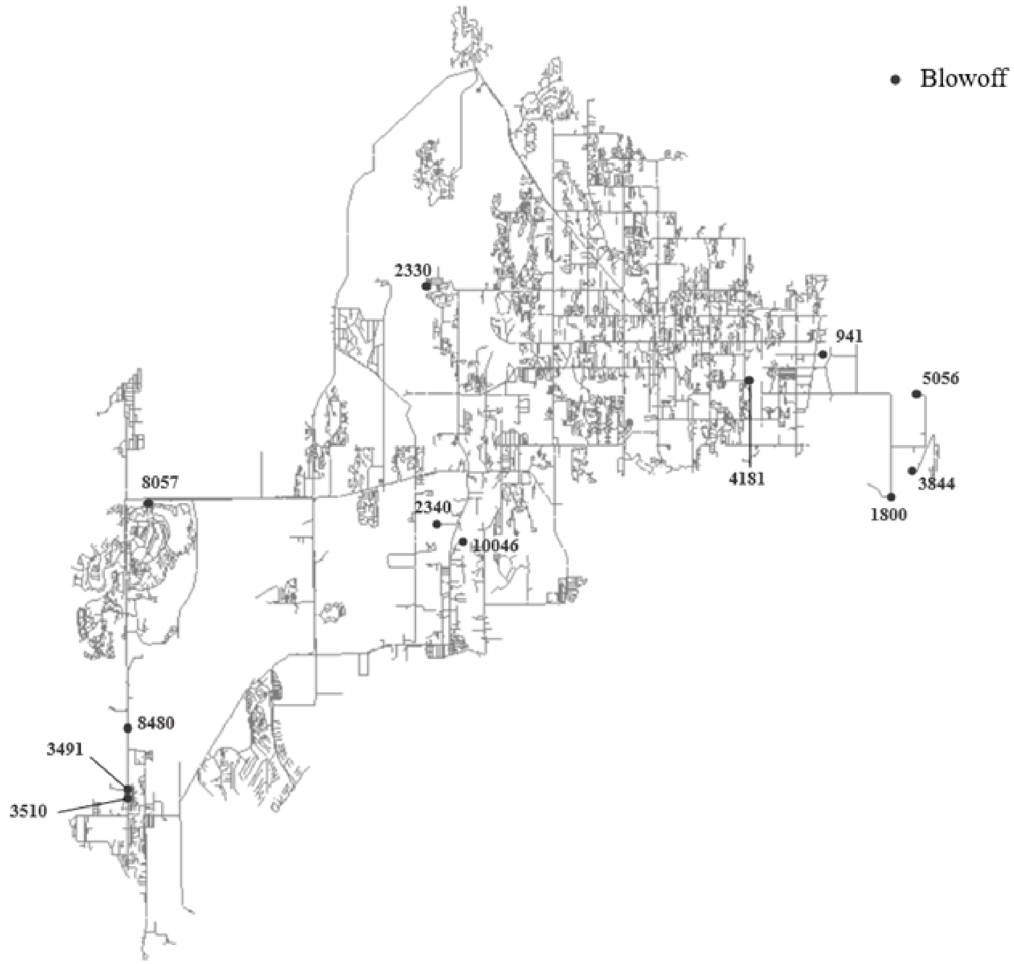


Figure 5.5. Flushing blowoff placement in the network for scenario 4.

The blowoff emitter coefficients were tuned to obtain the lowest blowoff outflows that correct chloramine deficits. Therefore, the injection of chloramine at WDN sources led to a decrease in the number of blowoffs to open and in the blowoff outflows. The emitter coefficients and the average nodal outflows (including outflows to fix chloramine residuals and leakage outflows) for critical dead-end nodes in WDN for scenario 4 are reported in Table 5.2. For all scenarios considered, the average total volume of water delivered *Vol* and average total mass dose of disinfectant *W* supplied in the 10 days of simulation analysis for each scenario were estimated and are reported in Table 5.5.

Table 5 Daily average total volume of water delivered *Vol* and daily mass dose of disinfectant *W* supplied in the 10 days of simulation analysis for each scenario.

Scenario	W (Kg)	Vol (m^3)
0	3,171	1,585,416
1	3,174	1,586,871
4	3,172	1,586,177

Note: Scenario 0: disinfectant chlorine – no booster stations and no flushing blowoffs. Scenario 1: disinfectant chlorine – 3 booster stations – 18 flushing blowoffs. Scenario 4: disinfectant chloramine – no booster stations – 12 flushing blowoffs.

As it is shown, the average total volume *Vol* (including supply, leakage, and additional outflow by blowoffs considered for fixing disinfectant residuals) only slightly increases in scenarios 1 and 4 compared to the no-blowoffs scenario (scenario 0). The fewer flushing blowoffs needed in the chloramine model (scenario 4) led to a decreased volume of water supplied compared to the chlorine model (scenario 1).

Generally, the slight opening of nodal blowoffs for improving water quality at dead-end nodes worsened water losses only slightly in the WDN. Referring to the average total mass dose *W* (including disinfectant mass dose injected in sources and booster stations), the maximum value was obtained for scenario 1, in which three booster stations were placed to meet the residual target in bulk areas. However, this value was only slightly larger than the values obtained for scenarios 0 and 4, for which no booster station was necessary.

5.5. Discussion

The problem of low residuals cannot be solved by simply increasing disinfectant dose at the sources. In this context, other EPANET and EPANET-MSX simulations showed that, even when disinfectant concentration C_d at sources grows, it is infeasible to eliminate all violating nodes (Figure 5.6).

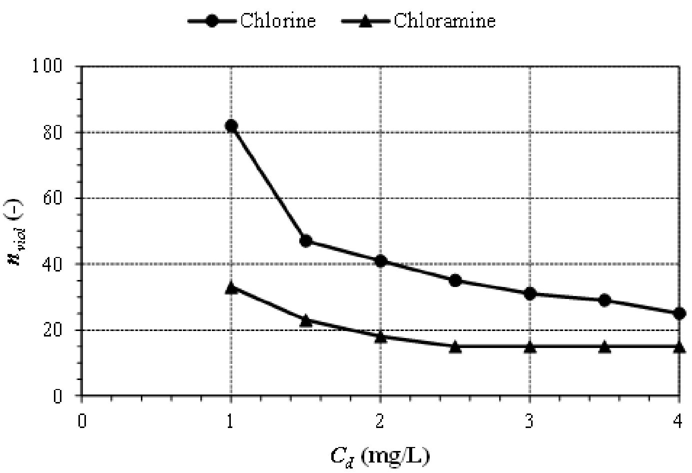


Figure 5.6. Number of violating nodes for each disinfectant dose Cd at sources in the chlorine and chloramine model.

In fact, even in the case of C_d = 4 mg/L (a high value compared with typical disinfectant concentrations found in WDNs), 25 and 15 violating nodes persist for chlorine and chloramine, respectively. Results confirm the slower decay rate of chloramine than chlorine for each dose at sources. The chloramine curve tends to stabilize at C_d = 2.5 mg/L after which, while increasing the dose at sources, 15 violating nodes are always detected. The choice of using a disinfectant concentration of C_d = 2 mg/L at sources may be a good compromise between keeping the disinfectant residuals in the target and avoiding the production of harmful DBPs that are known to be caused by excessive doses of disinfectant within WDN.

In light of the results reported in this work, the implementation of continuous or intermittent additional outflows at critical dead ends can contribute to the solution of this problem, in combination with the installation of disinfectant booster stations. Though flushing is a practice that can increase nonrevenue water, incentives can be proposed to users to encourage them to use more water, for instance, for irrigation purposes. The solution of nodal blowoffs can be implemented in real WDNs by installing a small tap for water immediately upstream from the hydrant. If the outflow proposed by simulations at the generic node is too small to be obtained by the tap, this device should be adjusted at the smallest feasible setting. Some automated flushing technology may have a disinfectant residual sensor built into the flushing device. However, this technology will be quite expensive and will require substantial maintenance.

It is clear there is no best solution for improving disinfectant residuals in WDN. Each of the alternatives shown has its own pros and cons. Booster stations are effective to obtain a more uniform distribution of disinfectant and can be placed in suffering bulk areas, though requiring a significant capital investment for the water utility. Continuous nodal blowoffs, instead, seem to be a necessary solution for the numerous and scattered suffering dead-end nodes in WDNs. They can be obtained by manually regulating blowoffs at critical nodes, e.g., close to the hydrant site; therefore, causing no costs for the automatization. Furthermore, they cause no service pressure deficits in the WDN. Cons include that the very low water discharges associated with their operation can be hardly obtained in the field. Therefore, larger water discharges than those predicted through the modeling, and larger water losses as a result, should be obtained in the field. In the case of intermittent blowoffs, the total outflow volume being the same, larger water discharge values, more easily obtainable in the field, were obtained. However, as cons, this solution may have higher installation costs due to the automatization and could cause local service pressure deficits in WDN due to the larger outflows. With higher velocities, this solution may also stir up sediment if not carried out properly. The use of chloramine as a possible alternative to chlorine led to an overall increase in residuals within the WDN and consequently to a decrease in the number of operational measures to be implemented.

The choice of a solution depends on multiple factors such as local conditions (i.e., decay rate), the water utility's choice for incurring additional capital, labor, etc. Hydraulic/water quality models, such as EPANET, can be very effective in comparing operational alternatives to solve the problem of low residuals in the system under study. However, the use of software modeling should be combined with field sampling in order to obtain a more complete picture of the system and to reflect local conditions (i.e., decay rate), as well as to consider factors neglected in the EPANET modeling, such as the chemical diffusion/dispersion effects. Indeed, these effects may play a role in alleviating the problem of disinfectant residual violations that may arise due to low flow conditions in proximity to dead ends.

The proposed approaches can be extended to other real WDNs. However, the characteristics (e.g., the number and location) of the operational measures implemented in this work are strictly related to the network model considered, which is a meaningful example of a real large-size WDN. This study demonstrates that chlorine booster stations and continuous or intermittent nodal blowoffs are valid solutions to improve the residuals when chlorine or chloramine is used as a disinfectant at the sources. However, the results obtained are influenced by the assumptions underlying the study. Other factors should be investigated, such as the variability in decay rates and in flow demands. In real systems, disinfectant decay rates are not constant but vary due to seasonal variations in network conditions. For example, an increase in the water temperature or in the organic content in the treated water causes the growth of disinfectant consumption. Similarly, flow demands are not constant but vary due to users' habits as well as seasonal patterns. The variability in these parameters can affect the water quality results and hence the solutions to be implemented. It should be expected that a decrease in flow demands in the WDN and a growth of the disinfectant decay cause an increase in the number of booster stations and blowoffs to be located, and vice versa.

The last comment concerns the potential nitrification problem that may occur in a WDN when chloramine is used as a disinfectant. Nitrification reactions, due to the production of ammonia, can have the adverse impact of reducing chloramine residuals, promoting bacterial regrowth. Furthermore, it can be accompanied by a decrease in pH, thus promoting the corrosion of the infrastructure. The chloramine decay model implemented in EPANET-MSX takes account the formation of ammonia (NH_3). A health-based guideline has not been derived, as reported in the GDWQ (WHO 2017) since ammonia is not of direct importance for health in the concentrations to be expected in drinking water. However, referring to the European (Council Directive 98/83/EC) and Italian (D. Lgs. 31/2001) regulations, concentrations lower than a guideline value of 0.5 mg/L should be guaranteed. Results obtained show that 39 out of 10,551 nodes have ammonia concentrations above the target 0.5 mg/L in the monitoring time window (last 2 days of simulation) with values up to 0.59 mg/L. However, this percentage of violation (18%) occurs only for very few nodes in the WDN.

Chapter 6

Investigating the Effects of Dispersion and Demand Pulses on Chlorine Residuals in a Medium Size WDN

In this Chapter the effects of considering i) the dispersion transport and ii) the pulsed nature of demand on water quality and chlorine residuals at dead-end nodes are investigated. These aspects are usually neglected in the flow routing and water quality modelling of WDNs. The methodology uses a flow routing/water quality model and a stochastic demand generator to simulate demand pulses at a fine time scale in a medium size WDN.

6.1. Overview

As was seen in Chapter 3, the prediction of disinfectant residuals in WDNs can be made by using hydraulic/water quality models.

Prevailing water quality models, such as EPANET apply 1-D AR transport model (Rossman et al. 1994) for quality simulations, assuming that steady plug flow exists at all times in every pipe. Furthermore, the effects of dispersion are neglected. However, while this assumption does not generally compromise the accuracy of simulations for main transmission lines where the advection component plays a major role, previous studies (Tzatchkov et al. 2002; Abokifa et al. 2016) showed that 1-D AR transport model fails to accurately predict field observed chlorine concentrations in the dead-end branches where the role of dispersion is not negligible.

A dead-end branch is topologically defined as a pipe or a set of pipes connected in series, linked to the WDN from only one inlet node (Abokifa et al. 2020).

Dead-end branches are located at the peripheral zones of the distribution system and serve a significant fraction of the residential population (Buchberger and Lee 1999). In these areas, the flow regime is usually dominated by intermittent laminar conditions with frequent stagnations during no-consumption periods (Buchberger et al. 2003; Abokifa et al. 2020). In the presence of laminar flow, hydrodynamic dispersion can be a predominant transport mechanism in the solute spreading. By neglecting the dispersion term in the governing transport equation, as is the case with EPANET, inaccurate results may be obtained in these peripheral zones of WDN as was shown by Lee (2004), Basha and Malaeb (2007), Tzatchkov et al. (2009), Abokifa et al. (2016). Therefore, dispersion is important and necessary to be considered in water quality models in order to predict the actual disinfectant concentrations in the dead-ends of the WDN.

Abokifa et al. (2016) released an ADR model, denominated WUDESIM (Washington University Dead End Simulator), for simulating single-species reactions in the dead-end branches of WDNs. WUDESIM uses the hydraulic engine incorporated in EPANET for the flow-routing. As for water quality module, it includes species transport by longitudinal dispersion. The model predicted in a more accurate way the field measured concentrations of fluoride tracer and chlorine compared to those stimulated by EPANET (Abokifa et al. 2016).

In addition to the fundamental shortcoming of neglecting dispersion transport, temporal averaging of the water demand is typically employed in the hydraulic, and subsequently water quality, simulations conducted with EPANET (Farina et al. 2014; Menapace et al. 2018). Temporal averaging of the water demands involves using demand patterns at an hourly resolution. This averaging procedure masks the actual flow patterns in WDN happening at the sub-hourly level. Buchberger and Wu (1995) and Blokker et al. (2008) showed that sub-hourly temporal demand distribution can affect water quality results.

Several models were developed for generating residential water demand at high temporal resolutions through pulses featuring arrival time, duration and intensity. Among these, there are models that use stochastic processes to reproduce the overall water demand of the household as in the Poisson Rectangular Pulse (PRP) process (Buchberger and Wells 1996; Buchberger et al. 2003; Garcia et al. 2004) and in a generalization of the PRP, the cor-PRP model (Creaco et al. 2015; Creaco et al. 2016). A comprehensive overview and comparison among models for generating residential water demand pulses can be found in Creaco et al. 2017. Both PRP and cor-PRP are based on three main steps: (1) generation of the pulse frequency of household water use through a non-homogenous Poisson process; (2) generation of pulse duration (in s) and intensity flow (in L/s) through probability distributions and (3) sum of the intensities of all active pulses to obtain the total demand (L/s) at any time. The main difference between the PRP and cor-PRP lies in the step (2) in which the pulse durations and intensities are generated in a correlated way in the cor-PRP model as confirmed by experimental evidence (Creaco et al. 2015).

In light of the above considerations, the present work is carried out, which represents a follow-up of the study presented in Chapter 4. The scope of this research is to evaluate the effect of (1) dispersion and (2) dispersion coupled with demand pulses at a fine time scale on chlorine concentrations in dead-end nodes of the WDN.

6.2. Methodology

In this work three water quality models are used for the prediction of the chlorine residuals at dead-end nodes of the WDN, as showed in Table 6.1. The models differ from each other for the type of analysis conducted on the dispersion and on the temporal demand distribution.

Table 6.1. Type of analysis conducted for each model used.

Model(s)	Type of Analysis	
	Dispersion	**Temporal demand distribution**
Mod. A: EPANET	No	Averaged (hourly basis)
Mod. B: WUDESIM	Yes	Averaged (hourly basis)
Mod. C: WUDESIM + cor-PRP	Yes	Stochastic demand pulses

The EPANET model (hereinafter referred to as Model A) is the most used one for the prediction of disinfectant residuals in WDNs. It considers the solute transport only controlled by advection and reaction, neglecting the dispersion, and typically considers flow demands averaged at an hourly resolution. The WUDESIM model (hereinafter referred to as Model B) includes the effect of the dispersion with the same assumption on flow demands as in Model A, while the coupled WUDESIM + cor-PRP model (hereinafter referred to as Model C) considers the effect of the dispersion when flow demands are generated as stochastic pulses at a sub-hourly level.

As for the WUDESIM model, it calculates the solute concentrations of the dead-end branches of the network. First, the dead-end branches of the network are searched for by checking the upstream nodes, one-by-one, connected to only one pipe. The dead-end branch is linked to the WDN from only one inlet node, which is a node connected to two or more pipes of the network. Then, WUDESIM calculates the time-series profiles of the solute concentration for each dead-end branch node and the time-series of the Reynolds number and residence times for each dead-end branch pipe.

As seen in Chapter 3, WUDESIM model supports different formula to calculate the longitudinal dispersion coefficient under different flow regimes.

In this study the classical formula developed by Taylor (1953) is used in Model B for the estimation of the dispersion coefficient under laminar flow regime (Re < 2,300), while the dynamic dispersion rate formula developed by Li et al. (2006) for pulsating laminar flows is implemented in Model C.

Besides the low flow conditions that characterize dead-ends, larger flow rates can also take place during peak demand hours for which occasional transitional/turbulent conditions may occur leading to advection-dominated transport. Hence, the longitudinal dispersion coefficient is also considered under turbulent flow regime (Re > 4,000) and calculated using the formula developed by Sattar (2013) in both scenarios. The choice of using the Sattar (2013) formula instead of Taylor (1954) formula is due to the basic assumption of the latter one, which is only valid under highly turbulent regimes (Re > 20,000) (Ekambara and Joshi, 2003). A similar condition is unlikely to occur in dead-end branches of WDN. For the transitional regime (2,300 < Re < 4,000) the dispersion coefficient is calculated by linear interpolation between the two values calculated at Re = 2,300 and Re = 4,000.

In Model C, WUDESIM is applied after pulsed demands are generated through the cor-PRP model. This model is based on a non-homogeneous Poisson process to generate the pulse arrivals (frequency), and on a bivariate probability distribution to generate the pulse durations D (in s) and intensities I (flows in L/s). By summing the intensities I of all active pulses, the total nodal demand Q (L/s) at any time is obtained. The total demands Q are then averaged over a specified period which is assumed to be 5 min in this study to sufficiently represent the effects of stochastic demands on model hydraulics and transport. Indeed, the analysis at smaller time steps would have required unsteady flow models to be used for the flow routing, which is out of scope of this work. The underlying equations used in the cor-PRP model are described in Creaco et al. (2015). The statistical parameters (average and standard deviations) used for both demand intensities and durations are taken from Creaco et al. (2017) and summarized in Table 6.2. At each node and temporal step, the pulse arrival frequency is estimated in such a way as to reproduce the average value of demand.

Table 6.2. Statistical parameters of variables *D* and *I* from the pulses generated by the cor-PRP model (Creaco et al. 2017).

	Cor-PRP model (Creaco et al. 2015)				
Scenario	**μ(*D*) (s)**	**σ(*D*) (s)**	**μ(*I*) (L/s)**	**σ(*I*) (L/s)**	**ρ(D,I)**
2	48.91	103.38	0.097	0.066	0.33

Note: μ(*D*) and σ(*D*) mean and standard deviation associated with the pulse durations *D*; μ(*I*) and σ(*I*) mean and standard deviation associated with the pulse intensities *I*; *ρ(D,I)* duration-intensity correlation.

6.3. Application

Case Study

The methodology described above is applied to the WDN (Figure 6.1) previously used in Chapter 4. The description of the network model is provided there. The network model uses a demand multiplier pattern at 1 hour level to represent the typical daily variation in the users' demand in the system.

For the water quality simulations, chlorine concentration at the source, bulk and wall decay coefficients are set to C_{cl} = 1.0 mg/L, k_b = 1.0 d^{-1} and k_w = 0 respectively, to match the values used in the previous study (Rossman et al. 1994; Powell et al. 2000; Boccelli et al. 2003; Monteiro et al. 2013; Nejjari et al. 2013). Simulations are run for 10 days of WDN operation in order to establish cyclic conditions in the hydraulic and water quality conditions.

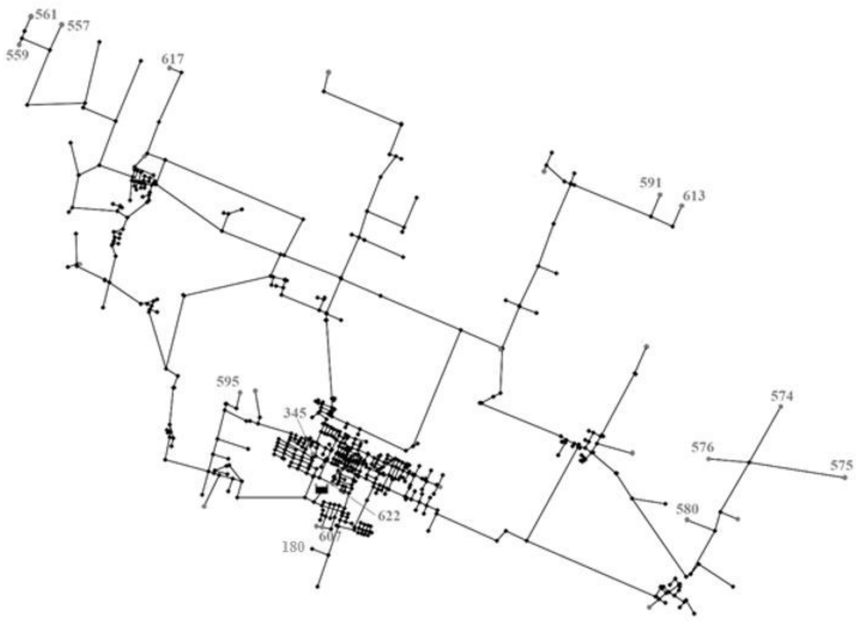


Figure 6.1. Case study layout. Grey numbers and nodes indicate dead-end nodes with a chlorine residual below to $C_{cl,min}$ = 0.2 mg/L in the last day of simulation for Model B.

The chlorine residuals in the dead-end nodes of the WDN, as simulated in the two Models B and C described above, are compared with those obtained by the Model A, the results of which are already reported in Chapter 3.

6.4. Results

The network has 244 dead-end branches, comprising a total of 264 pipes and nodes connected. By evaluating the average Reynolds number for the entire period of simulation (10 days), the majority of dead-end branch pipes (226) operate under laminar flow regime, 24 under transitional flow regime and only 14 under turbulent flow regime.

Water quality results showed a general increase in chlorine residuals in all WDN dead-end nodes when Model B is used compared to Model A. For Model A, as was seen in Chapter 4, 24 critical dead-end nodes with a chlorine residual below the minimum of $c_{cl,min}$ = 0.2 mg/L were found in the WDN in the last day of simulation. By switching to a more accurate model, i.e. the ADR model (Model B), the number of critical nodes decreases to 14 (displayed in Figure 6.1). However, considering the effect of demand pulses in combination with the dispersion (Model C), the situation is again similar to Model A (number of violating nodes = 22). This different behaviour between the models is also evident by plotting the chlorine residuals profiles for two nodes of the WDN featuring a different flow regime.

The first node, node 180, is the terminal junction of pipe 175, which operates under turbulent flow regime conditions during almost the entire period of simulation, with an average flow velocity of u = 0.15 m/s and an average Reynolds number of Re ~ 10,300. Under these flow conditions, the advection is expected to play a significant role in the transport of the disinfectant.

Figure 6.2 shows the 10-days of chlorine concentrations profile for the node 180 in the three models (a), of nodal outflow (inclusive of leakage) at 1 hour resolution in Models A and B (b) and of nodal outflow (inclusive of leakage) at 5 min resolution in Model C (c).

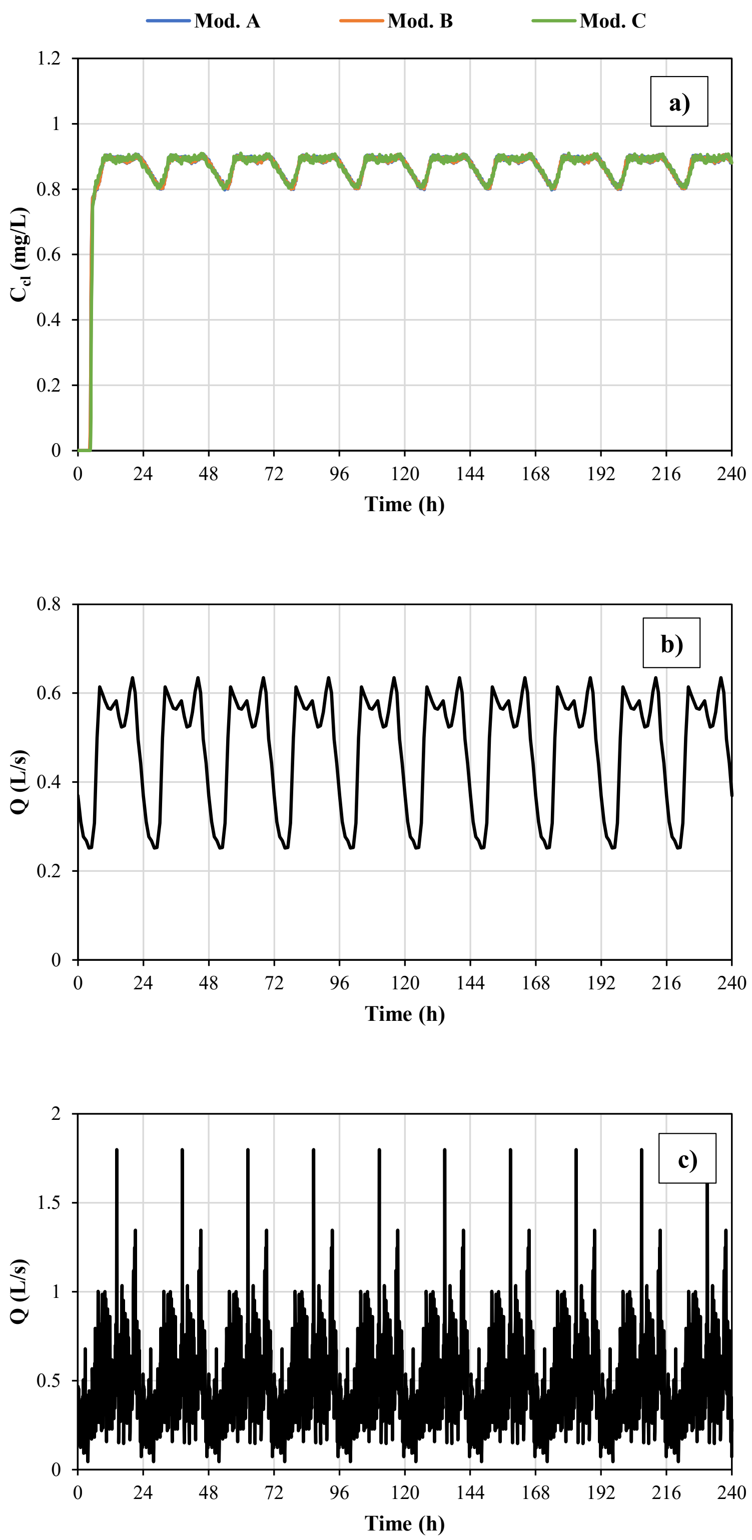


Figure 6.2. 10-days of chlorine concentrations profile for the node 180 in the three models (a), of nodal outflow (inclusive of leakage) at 1 hour resolution in Models A and B (b) and of nodal outflow (inclusive of leakage) at 5 min resolution in Model C (c).

Obviously, the nodal outflow with an interval of 5 min [Figure 6.2 (c)] has much higher variability than that with 1 hour resolution [Figure 6.2 (b)], due to the presence of demand pulses.

As it can be seen from Figure 6.2 (a), there is no significative difference in the prediction of chlorine residuals as simulated by the three models. Here, the advection transport dominates over dispersion leading the models to give practically the same results. Neither dispersion nor the temporal demand variation has relevant effect on the chlorine residuals for turbulent flow zones.

The second node, node 580, is the terminal junction of pipe 586 which operates under laminar flow regime conditions during almost the entire simulation period, with an average flow velocity of u = 0.003 m/s and an average Reynolds number of $Re \sim 370$. Under this low-flow conditions, the longitudinal dispersion is expected to play a significant role in the transport of the disinfectant.

Figure 6.3 shows the 10-days of chlorine concentrations profile for the node 580 in the three models (a), of nodal outflow (inclusive of leakage) at 1 hour resolution in Models A and B (b) and of nodal outflow (inclusive of leakage) at 5 min resolution in Model C (c).

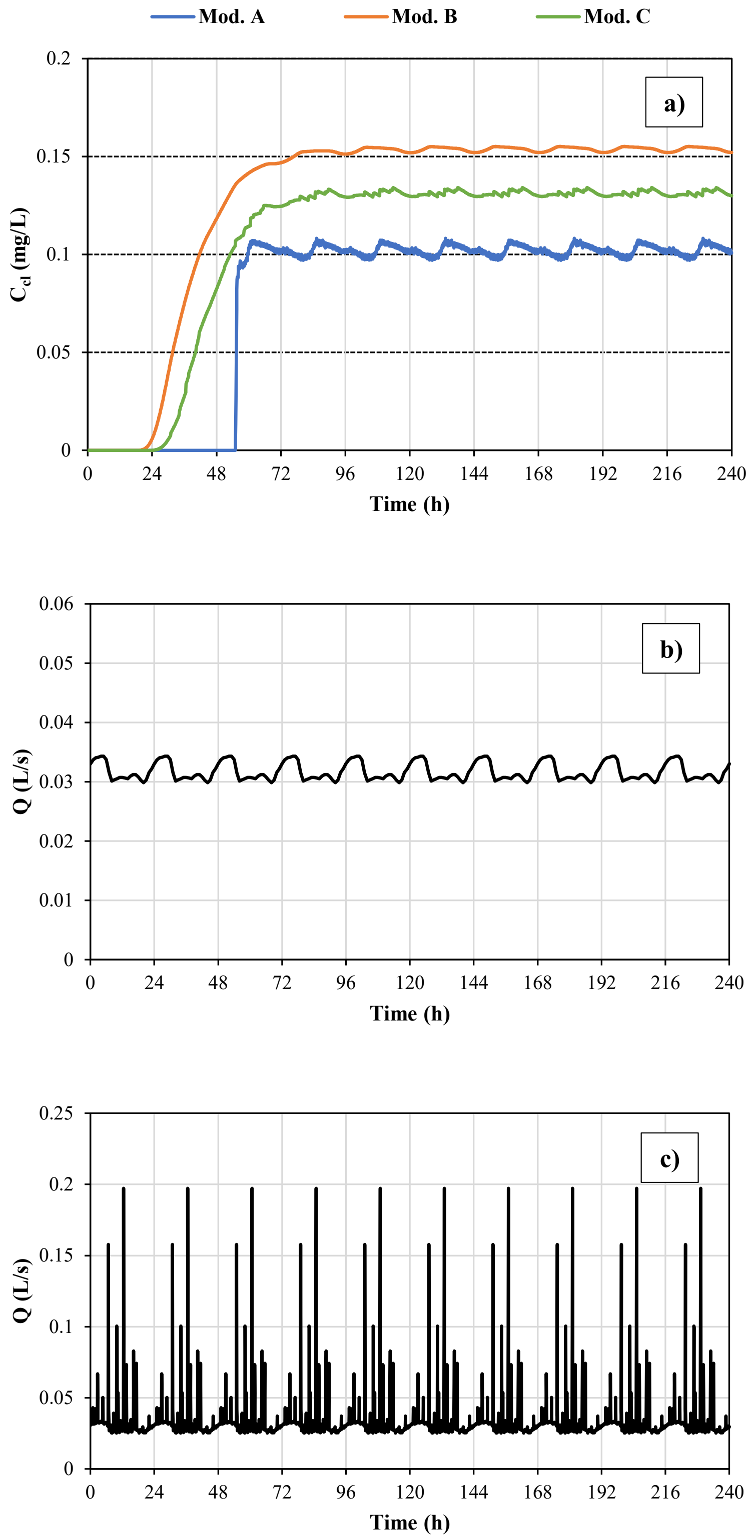


Figure 6.3. 10-days of chlorine concentrations profile for the node 580 in the three models (a), of nodal outflow (inclusive of leakage) at 1 hour resolution in Models A and B (b) and of nodal outflow (inclusive of leakage) at 5 min resolution in Model C (c).

The nodal outflow with a interval of 5 min [Figure 6.3 (c)] has much higher variability and range than that with 1 hour resolution [Figure 6.3 (b)].

For this node, there is a significative difference in the chlorine residuals simulated in the three models. Comparing the results yielded by Model A (blue profile) and Model B (orange profile), it can be seen how the dispersion transport prevails over advection (Abokifa et al. 2016) in this zone leading to higher chlorine residuals compared to the ones obtained in its absence. Furthermore, the Model B profile appears to be smoother than the Model A one. This is because EPANET moves the concentration pattern across the network almost by pure advection producing large fluctuations of concentration. The high dispersion in this low-velocity pipe dampens these fluctuations yielding to a more realistic pattern of the concentration evolution.

When demand pulses are added, the pattern of concentration tends to decrease, approaching the pattern obtained in Model A. This happens as a result of flow stagnation occurring between the arrival time of two subsequent demand pulses.

Chapter 7

Conclusions

This work has been related to the water quality aspects in WDNs, with the aim of guaranteeing high standards of drinking water quality in WDNs.

A range of issues related to water temperature in WDNs and its potential impact on water quality in these networks was addressed in Chapter 2. The methodology adopted is based on a literature review and a stakeholder survey in order to understand legislation and local practices adopted to monitor water temperature in WDN.

Technical solutions for ensuring the satisfaction of disinfectant residuals requirements throughout the WDNs were implemented in Chapter 4 and Chapter 5, for a medium-size WDN and for a large-scale WDN respectively.

Specifically, Chapter 4 proposed a novel solution to the problem of low chlorine residuals at critical dead-end nodes of WDN. This solution is based on the slight increase in nodal outflows all day long, through the opening of a blowoff at the hydrant site. The methodology used a multi-objective optimization algorithm coupled with the EPANET software in a heuristic procedure. In the optimization problem, the concentration of chlorine on supply and the values of emitter coefficients at critical nodes (associated with blowoff openings) were used as decisional variables to be optimized. Two objectives were minimized, namely the total volume of delivered water and the total mass of disinfectant fed into the network, while meeting the minimum value of disinfectant at all nodes of 0.2

mg/L. The effectiveness of the methodology was proven on a real medium-size WDN suffering from low chlorine concentrations at various dead-end nodes, yielding an insight into the economic feasibility of the solution. Furthermore, a sensitivity analysis was carried out to analyze the effects of demand seasonality and chlorine decay conditions on the solution proposed.

In the case study analysed in Chapter 5, the problem of low concentrations occurs not only at dead-end nodes but also in some large internal areas of the WDN. To address this combination of problems, the modulation of nodal outflows was combined with other operating measures. First, while considering chlorine as disinfectant, the implementation of booster stations in bulk areas and continuous outflows at dead-end nodes was carried out in the network. Afterwards the comparison between continuous and intermittent outflows was performed. The water volume being the same, water was provided through blowoffs for 24-hrs or for limited durations, respectively. The work ended with the investigation of switching chlorine with chloramine in combination with the continuous nodal outflows solution to meet the residual target. The methodology used the EPANET software, and its multi-species extension EPANET-MSX to simulate the chlorine and chloramine decay, respectively.

Finally, Chapter 6 investigated the effects of considering i) the dispersion transport and ii) the pulsed nature of demand on water quality and chlorine residuals at dead-end nodes of WDN. These aspects are usually neglected in the flow routing and water quality modelling of WDNs, such as EPANET. Two models for simulating chlorine residuals at dead-ends of WDN were considered. The WUDESIM model investigated the effect of the dispersion when flow demands are typically averaged at an hourly level (same assumption of EPANET), while the coupled WUDESIM + Cor-PRP model considered the effect of the dispersion when flow demands are generated as pulses at a sub-hourly level (5 min in the study). The chlorine concentration outputs of the two models were compared with the ones simulated by the EPANET model. For explicatory purposes, the comparison between the three models was made for two representative nodes of the network which operated under different flow regimes, laminar and turbulent respectively.

The main findings of the research activities conducted can be summarized below. Based on the information obtained from both literature review and stakeholder survey, the following observations on drinking water temperature were made:

- water temperatures are monitored, but this is not done systematically, and data collected varies substantially across different countries. In most cases, water temperature is most frequently monitored at sources and treatment plants. There is limited and sporadic monitoring in the WDN. This monitoring should be done more systematically for a number of reasons, including improved compliance testing and underpinning future research in this area. In many countries, temperature is already measured, such as part of when measuring for chlorine residuals on site but is not recorded. Therefore, such data could readily be gained with minimal additional effort.
- it is widely acknowledged in the literature and engineering practice of different countries that a link exists between drinking water temperature and quality, with lower temperatures linked to improved quality. However,

this link is currently not well understood for a range of potential water quality issues. This includes the significance of the 25 °C threshold, which water utilities in some countries are already asked to comply with.

- water temperature varies as it travels from the water treatment works to a tap, primarily due to exchange with the surrounding ground and ground water. Whilst plausible models could be proposed to simulate the processes involved, these remain unverified at present. There is a need for research in this area, including the interaction between the soil and land cover, the presence of anthropogenic heat sources and pipe hydraulics.
- a number of future changes in the surrounding environment are likely to impact the water temperature in the WDN. These include climate change, urbanisation, more integrated urban planning, rainwater use, greywater reuse and wider application of water saving and other technologies. The impact of these changes on the WDN temperature and consequential water quality is currently not well understood and hence requires future research.

As for the study on the modulation of nodal outflows to solve the problem of low residuals at dead-ends of WDN:

- the post-processing of the solutions in the WDN considered proved the economic profitability of increasing nodal outflows for solving problems of low disinfectant concentrations at dead-end nodes. The lowest operational costs for the WDN are obtained using lower chlorine doses at the source and larger nodal outflows. Nevertheless, the nodal outflows from blowoffs are always tiny compared with those that would be required for firefighting.

- the percentage of water losses in the WDN only slightly increases with the implementation of blowoffs in the value of 20.51% compared to 20% in the no-blowoff scenario.
- the decrease in water demand and the growth of the bulk decay constant causes the increase in the number of blowoff to open and in the blowoff outflows.
- other simulations proved that for the WDN analyzed, hydrants with larger outflow (i.e., 5-10 L/s) running for a short period (i.e., 1 h per day) are ineffective at solving the problem of low disinfectant concentrations at some sites of the WDN. For this case study, a continuous outflow at low rate obtained by the opening of a blowoff prove a valid solution to improve water quality.

On the comparison of techniques that can be implemented in a large-scale WDN, results showed that all the techniques analysed, each with their own pros and cons, are effective to tackle the problem of low disinfectant residuals in WDN. Particularly:

- booster stations are effective to obtain a more uniform distribution of disinfectant and can be placed in suffering bulk areas, though requiring a significant capital investment for water utility.
- continuous nodal blowoffs seem to be a necessary solution for the numerous and scattered suffering dead end nodes in WDNs. They can be obtained by manually regulating blowoffs at critical nodes, e.g., close to the hydrant site, therefore causing no costs for the automatization. Furthermore, they cause no service pressure deficits in the WDN. Cons

include that their very low water discharges, which can be hardly obtained in the field. Therefore, larger water discharges than those predicted through the modelling, and larger water losses as a result, should be obtained in the field.

- in the case of intermittent blowoffs, the total outflow volume being the same, larger water discharge values, more easily obtainable in the field, are obtained. Intermittent blowoffs have a similar performance to the continuous blowoffs if a percentage of violation of 10-15% for few hours per day is considered acceptable. However, as cons, this solution may have higher installation costs due to the automatization and could cause local service pressure deficits in WDN due to the larger outflows. With higher velocities, this solution may also stir up sediment if not done properly.
- the use of chloramine as a possible alternative to chlorine led in general to an overall increase in residuals throughout the WDN and consequently to a decrease in the number of blowoffs to open and in blowoff outflows. Of course, chloramines have their own issues such as the production of ammonia, which has the potential to promote nitrification reactions within the system. Nitrification can have the adverse impacts of reducing chloramine residuals, promoting bacterial regrowth. Furthermore, it can be accompanied by a decrease in pH, thus promoting the corrosion of the infrastructure.
- other factors should be investigated, such as the variability in decay rates and in flow demands. In real systems, disinfectant decay rates are not constant but vary due to seasonal variations in network conditions. For

example, an increase in the water temperature or in the organic content in the treated water causes the growth of the disinfectant consumption. Similarly, flow demands are not constant but vary due to users' habits as well as seasonal patterns. The variability in these parameters can affect the water quality results and hence the solutions implemented. Similarly to results obtained for the case of continuous nodal blowoffs, it should be expected that a decrease in flow demands in the WDN and a growth of the disinfectant decay cause an increase in the number of booster stations and blowoffs to be located, and vice versa.

- it is clear that there is no best solution for improving disinfectant residuals in WDNs. Each of the alternatives has its own pros and cons. The choice of a solution depends on multiple factors such as local conditions (i.e. decay rate), the water utility's choice for incurring additional capital, labour etc. Hydraulic/water quality models, such as EPANET, can be very effective in comparing operational alternatives to solve the problem of low residuals in the system under study. However, the use of software modelling should be combined to field sampling in order to obtain a more complete picture of the system and to reflect local conditions.

Regarding the influence of dispersion and demand pulses on chlorine residuals, it was proved that:

- neither dispersion nor the temporal demand variation has relevant effect on the chlorine residuals at dead-ends operated under turbulent flow regime. In these zones the advection transport is predominant over dispersion, leading the three models (WUDESIM, WUDESIM + Cor-PRP and EPANET) to give similar results.

- both dispersion and demand pulses have effect on the chlorine residuals at dead-ends operated under laminar flow regime. The chlorine transport in these zones is mainly controlled by dispersion leading the dispersion model (WUDESIM) and the dispersion + demand pulses model (WUDESIM + Cor-PRP) to predict higher chlorine residuals compared to ones obtained by EPANET.
- with the same dispersion effect, the kind of temporal demand pattern also affects the predicted concentration profiles under laminar regime. Higher chlorine residuals are obtained when a low flow at the time scale of 1 h is considered at the generic node instead of higher intermittent pulses at the time scale of 5 min.
- regardless of the different approach in considering the temporal demand distribution, the inclusion of dispersion into the water quality model led to an overall increase in chlorine residuals compared to those obtained in its absence. Furthermore, the number of violating dead-end nodes in WDN (nodes with a chlorine residual below a minimum required, e.g. 0.2 mg/L) decreases when the dispersion effect is considered. Therefore, by implementing the solution of nodal blowoffs at dead-end nodes, a decrease in the number of blowoffs and of the blowoffs' outflows would be expected.

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