PRESSURE RELATED STANDARDS AND PERFORMANCE OF WATER DISTRIBUTION SYSTEMS

By

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Abstract

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The standard design approach of water distribution systems requires that pressure at any point in the system is maintained within a range whereby the maximum pressure is not exceeded so that the likelihood of a pipe burst is reduced and the minimum pressure is always maintained or exceeded to ensure adequate flows for satisfying expected demands. High pressure systems tend to cause more frequent pipe breaks and an increase in energy use and leakage. Low pressure systems cause consumer complaints, make the system more susceptible to negative pressures, and possibly to the ingress of contaminants during transient events. The overall goal of establishing pressure standards is to balance these opposing tendencies to achieve a safe, reliable, and economic operation of the system. Yet, there are no universally acceptable or established rules or guidelines for establishing a pressure standard for water distribution system design, and few studies have considered whether the traditional standards are still applicable in modern systems. This study has made a critical appraisal on what pressure standards mean, where they are violated, and where they need revision to achieve a comprehensive picture about what the pressure

standards really mean. The research also highlights the inter-related issues associated with pressure criteria. Assessment of the relationships governing water pressure, leakage, energy use and economics is realized via the analytical investigation of single pipes and the simulation of representative networks using the steady state analysis software EPANET 2. The role of minimum pressure standards, storage, pumping strategy, and resource prices on the energy and water loss of systems is analysed and assessed. In anticipation that pressure contributes to pipe break rates, a probabilistic approach considering uncertain water demand and pipe's roughness modeled with a Monte Carlo simulation (MCS) algorithm is presented. This study also explores how the minimum pressure standards affect transient pressures and reviews how destructive transient pressures may be controlled to limit reduced pressure surges within acceptable limits even when the minimum steady state pressure is relatively low. In order to place the research in practical context, this study develops a surge limit control algorithm for the design of a portable device for limiting the down-surge pressures by creating a pressure control boundary in a pipe system during hydrant operations. This boundary is established using the portable control device to safely operate a hydrant in a water distribution system. This study also highlights the notion that high level of pressure standards may lead to a troublesome squandering of water and energy and may disrupt the performance of water distribution systems. Given the too often degraded nature of water supply infrastructures, the on-going challenges of urban growth, and the increased stress on natural resources, the significant benefits of better controlling water pressure are not only welcome but urgently needed.

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Notation

Symbol	**Meaning**
A	cross-sectional area of the pipe
A_1	areas of piston
a	wave speed
a_0	leakage fraction
B	burst frequencies before pressure reduction
$Br(h)$	break rate function
C	discharge coefficient
C_h	Hazen Williams's coefficient
C_{v}	coefficient of variation
C_v	the valve coefficient
c_t	unit conversion factor
D	pipe diameter
E	supplied energy at the source
E_i	initial energy use
E_{save}	energy saving
$E(q)$	mean of the leakage rate
$E[Br]$	expected pipe break rates
e_0	efficiency at the best efficiency point
e_p	pump efficiency
F	forces to piston 1 and 2
f	Darcy-Weisbach friction factor
$f(h)$	probability density function of pressure

g	acceleration due to gravity
H_{cr}	MPC
H_f	head loss
H_{LS}	low surge limit
H_{m0}	initial MPC
H_{min}	minimum transient pressure
H_s	the supplied pressure head
H_T	total head
H_t	partial derivatives of piezometric head with respect to t
H_x	partial derivatives of piezometric head with respect to x
H_{US}	up surge limit
$H(t)$	pressure head at time t
h	pressure
h_f	head loss in the pipe
h_{loss}	head loss
I_V	intensity of violation of MPC
K	pipe resistance coefficient
k	spring constant
K_c	controller parameters
K_r	controller parameters
L	pipe's length
m	an exponent
N	emitter exponent
n	exponent of the pressure-demand relationship
P_0	operating pressure before pressure reduction

P_1	operating pressure after pressure reduction
P_a	pressure below which there are no breaks
P_s	up/or low surge limits
Q	flow rate
Q_0	flow at the best efficiency point
Q_d	required flow
Q_{de}	desired demand
Q_r	required demand
Q_s	supplied demand
q	leakage rate
R_f	head loss ratio
RN	nodal reliability
R_P	relative delivery head
R_Q	relative flow
t	time
T_c	negative pressure index
T_m	duration of violation of MPC
T_p	period of violation of MPC
t_{comp}	total computational time
V_t	partial derivatives of the fluid velocity with respect to t
V_x	partial derivatives of the fluid velocity with respect to x
$Var\ (q)$	variance of leakage rate
X	displacement of disks 1 and 2
x	distance along the centerline of pipe
x_s	displacement of spring

ΔH	amount of reduction in MPC
ΔH_0	head difference across the valve
ΔH_s	pressure difference of the low and up surge limits
η	efficiency factors of the original and lower speed pumps
μ_c	mean discharge coefficient
μ_H	mean pressure
$\sigma^2{}_c$	variances of discharge coefficient
$\sigma^2{}_H$	variances of pressure
τ	dimensionless valve opening

Chapter 1

Introduction

1.1 Introduction

Water distribution systems (WDSs) are designed to provide safe drinking water for residential, commercial, and industrial users and to provide an adequate supply of water, at an acceptable pressure, to deal with routine and emergency conditions (e.g., pump failures, pipe breaks, and fire flow requirements). In standard design, the pressure at any point in the system should be maintained within a range whereby the maximum pressure is not too high so that the likelihood of pipe burst is greatly reduced and the minimum pressure is maintained in order to provide adequate flow for expected demands. Although pressure is a necessary parameter for operating WDSs, it must be carefully managed because its excess or deficit can cause a hazard or an inconvenience. High system pressures may cause an increase in water demands (e.g., faucets, showers, and lawn watering), energy use, leakage, and the frequency of pipe breaks (Lambert 2000) as well as problems with valve operation and device erosion. While, low pressures may make systems more susceptible to low pressure failures, either hydraulic (e.g., an inability to supply the required flow, problems in reservoir operation such as inability to completely fill tanks, and problems with secondary pump operation including poor suction conditions) or safety related (e.g., risks from a transient event such as pipe damage and collapse as well as intrusion (or ingress) of contaminant flow). The overall goal of establishing pressure criteria is to provide safe, reliable and economic operation of WDSs. Yet, there are no universally acceptable or established rules or guidelines to specify the minimum and maximum pressure criteria for WDSs design.

A review of guidelines and regulations indicated that there is no specified value, in most regions, for the maximum acceptable pressure in WDS design and operation. But, quite reasonably, all guidelines strongly recommend that maximum pressures including maximum operating pressures plus transient pressures must be controlled below the maximum permissible pressure of pipelines. Standards for minimum pressure also vary widely around the world, in both the value of standard and in the condition that it should be set. For example, in most provinces in Canada, the minimum pressure criterion (MPC) is 14 m but in Australia, it is 20 m and in the UK, it is only 10 m. Different criteria of water pressure for WDSs design imply that the pressure delivered to a customer might be judged high enough to meet standards in some countries, while water delivered under the same pressure in other countries is considered unacceptable. Moreover, in optimization models of WDSs, the pressure criterion is considered as a constraint to ensure the system pressure is above the criterion (Walski et al. 2007). But depending which value of pressure standard is considered distribution systems may either be overdesigned with the associated excessive costs or have very little capacity to handle extreme situations.

While minimum pressure is enforced in WDSs design during times of high water usage, problems can arise during off-peak hours, e.g., late night and early morning, and at low elevation areas such that the pressures become too high. Figure 1.1 depicts the pressure profiles at four fire hydrants (in different pressure zones) in a mid-west region in Wisconsin, USA. As is clear from the figure, the pressure is much higher than requirement (i.e., 20 psi according to AWWA (2008) recommendations) in high pressure zones and even in low pressure zones, the pressure is more than required during hydrant operations. This raises such key questions: why, in essence, do we need to supply such a high pressure, much more than requirements? How often does low pressure occur? What metrics can best evaluate the severity of violations of MPC in WDSs design?

Traditionally, pressure management is an effective way to control leaks and pipe bursts in a system during off-peak hours (Gomes et al. 2011). To perform pressure management, however, the optimization of the number of pressure reducing valves, their locations, and the determination of the optimal adjustment for valve openings are challenging tasks. Moreover, in pressure management strategies, the energy use of the

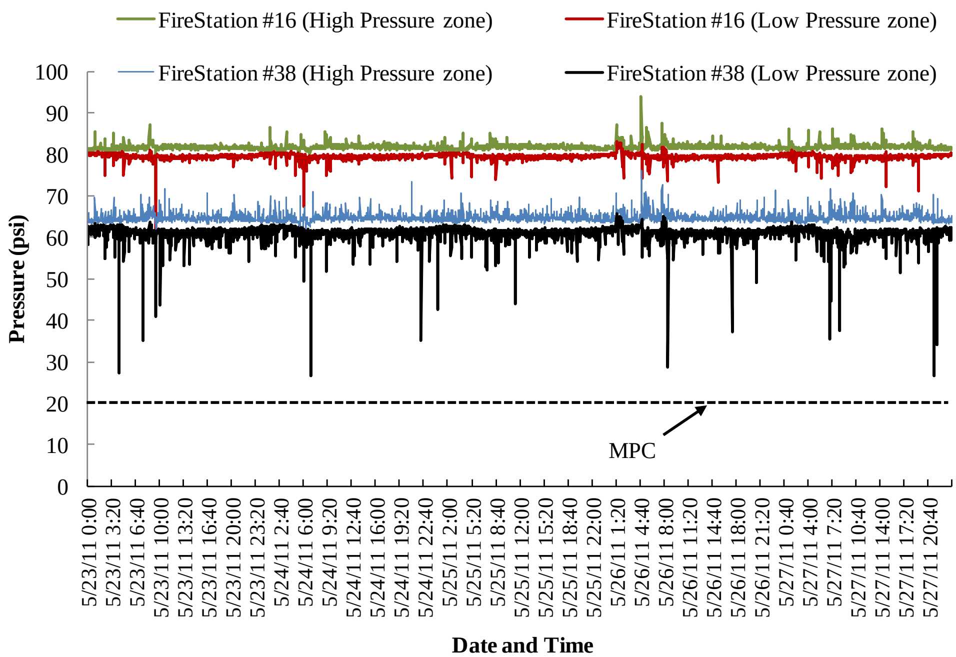


Figure 1.1. Pressure monitoring data of hydrants located in different pressure zones in 2-minute intervals in a mid-west region system in Wisconsin USA (LeChevallier et al. 2014a)

system may change little and it might be still suboptimal. Although it has been demonstrated that reduction in water pressure can influence the energy requirement and operational costs, there is an absence of literature regarding the consequences of reduction in pressure standards that is achieved by reducing the energy supplied, one of the main focus of this thesis, which is beyond pressure management.

The connection between pressure standards and factors influencing WDS operation is shown in Figure 1.2. All the issues caused by changes in pressure standards affect the system operating costs. Thus, there is a link between pressure standards and a constellation of "infrastructure report card" issues. Pressure should be limited not only because of the usual benefits of leakage reduction and possible decrease in pipe bursts, but for the key reason that energy has, both financial and environmental costs. But pressure management might be the ideal strategy in one system, and in another it might

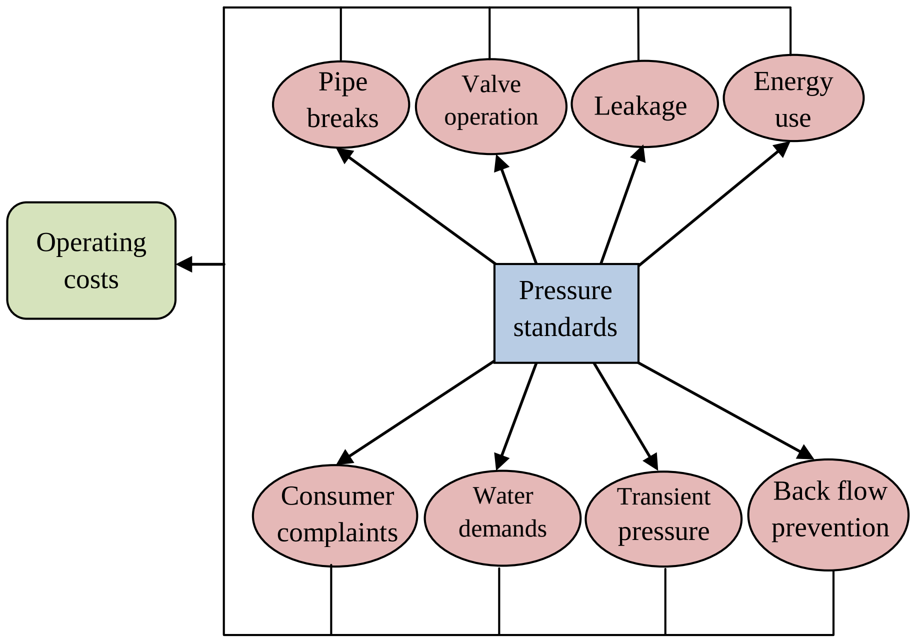


Figure 1.2. The connection between pressure standards and factors influencing WDS operation

well be to replace the pumping system. Certainly, if systems didn't leak as much, if pipes didn't burst as often, if backflow prevention was better managed and ensured, and if transient events were better controlled, designers could probably have even less stringent low pressure standards, and still be better off.

This thesis explores the notion to what extent pressure standards are universally accepted and what revisions in pressure standards are desirable for WDSs design. This thesis seeks to explain that if the criteria themselves, and the means to evaluate them, are too vague, so will be the outcomes used to correct any deficiency. In other words, this thesis does not aim to resolve all issues but rather attempts to highlight the problematic issues associated with pressure, indicating the intrinsic relationships between pressure and factors influencing the performance of WDSs. This significantly helps giving more comprehensive picture of high and low pressure problems in WDSs. Figure 1.3 shows a linked diagram to indicate the experts of different branches that may be interested in

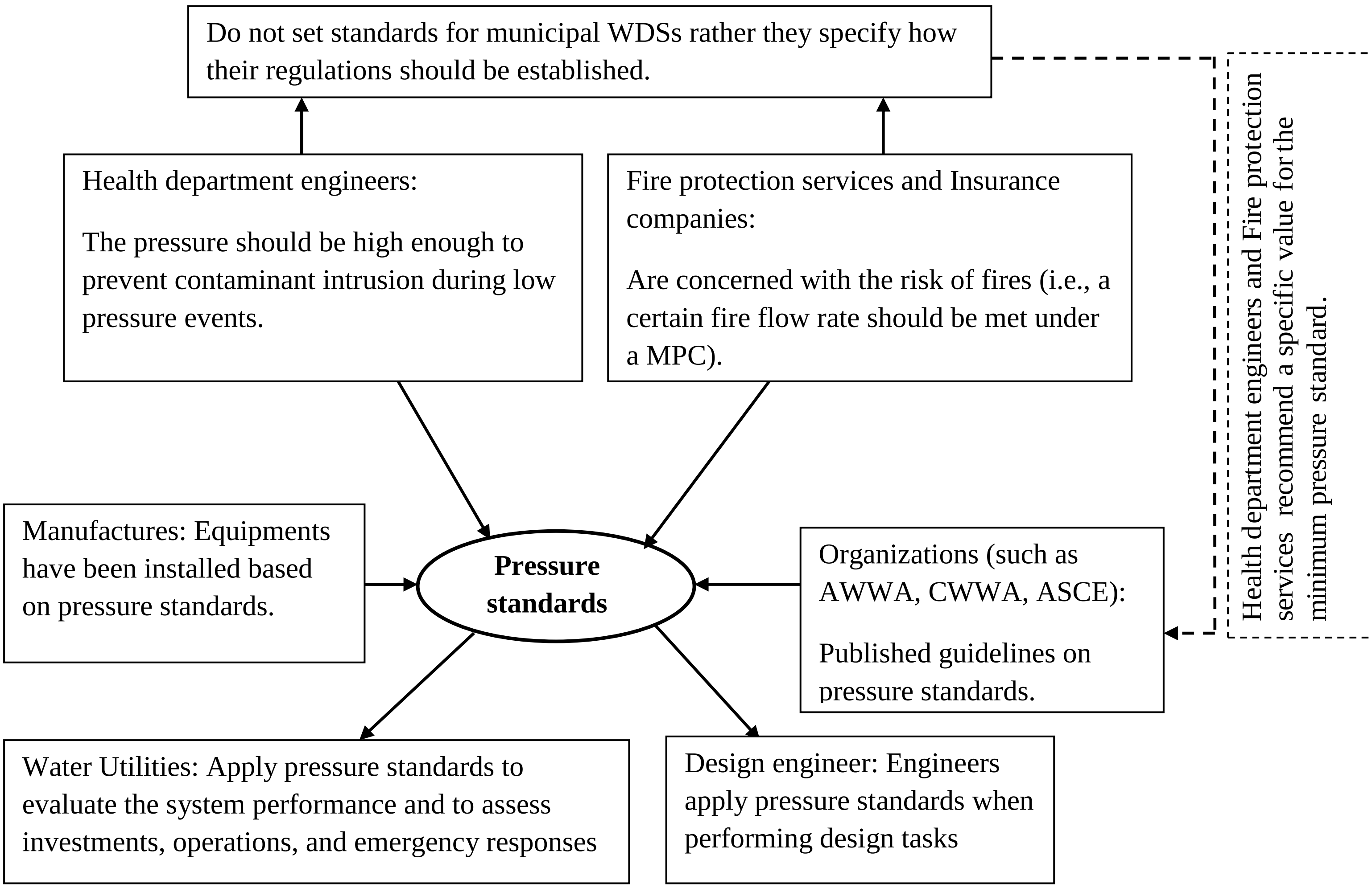


Figure 1.3. Experts of areas associated with WDSs design and operation that deal with pressure standards (the outward arrow shows that the pressure standards are applied in these circumstances and the inward arrow shows the experts who recommend a specific value for the standard).

thesis results. As can be seen, experts from several technical areas of WDSs deal with pressure standards.

1.2 Thesis Objectives

The main objectives of this thesis are to first highlight the importance and complexity of defining suitable pressure standards in WDSs design and operation, and second, to develop a novel idea in order to control transient pressures during hydrant operations. The detailed objectives of this thesis are as follows:

1- The main purposes of pressure standards are to achieve an economic design of WDSs and to assess WDSs performance. Both high and low pressures can put WDSs at risk of disruption. In design, the attempt has been made to maintain the system pressure between minimum and maximum standards for safe, reliable and economic operation. Chapter 3 establishes a critical discussion to answer to these important and interesting questions: What do MPCs mean? Where are they violated? And where do they need revision? Specifically, what kinds of pressure transgressions are most crucial to system performance and economics and what are merely inconvenience?

2- Criteria which specify the minimum pressure at which water is to be delivered to customers from a WDS differ around the world. Thus, interestingly, water pressure delivered to customers might be deemed high enough in some countries while the same delivered water pressure in other countries is considered unacceptable. A critical appraisal of the consequences of changes in the MPC is undertaken in chapter 4 with the aim of addressing an important question: how would consumers and WDSs be affected by changes in the MPC?

3- The standard design approach requires that WDSs be appropriately designed so that customers receive a satisfactory delivery pressure. Chapter 5 aims at a preliminarily assessment of the implication of changes to the delivery pressure on system energy use and cost, leakage, excess pressure, and environmental impact. This chapter sets out to assess the role of delivery pressures, on the energy and water loss of systems, trade-off key influential factors that exist in many systems.

4- A measure named the expected pipe break rate that can be used to determine how high system pressures contribute to pipe break rates is developed in chapter

6. The previous pipe break studies have relied on field data and few studies, which are only based on field data, have considered the relationship between pressure and pipe breaks. This chapter presents a probabilistic approach considering uncertain water demand and pipe roughness modeled with a Monte Carlo simulation (MCS) algorithm to show the link between system operating pressure and pipe break rates. The assessment of network performance is also performed with the probabilistic measures of reliability across the network.

5- Low pressure standards can put systems at risk during transient events: a risk to the pipeline, to its associated hydraulic devices and to those in their vicinity, and a risk of water contamination and thus to human life. Thus, although it may not have been part of the original intent, there is a direct connection between MPC and transients that should not be ignored. Chapter 7 looks specifically at the role of MPC and how it affects the system's transient response to raise the awareness about issues that can arise from changes in MPC.

6- Following the last step, to make the system safe during hydrant operations, a challenging task in WDSs operation, a surge limit control algorithm is developed in a manner that the down-surge is controlled in a predetermined level during hydrant operations. Chapter 8 explains first the use of a portable device for limiting the down surge pressures by creating a down surge control boundary in a pipe system during hydrant operations. And second, it explores how this surge limit control algorithm is created. Since determining a specified opening time for every hydrant is a challenging task, because of existing many hydrants scattered at different locations of WDSs, developing a surge limit control algorithm and new surge control device to control the down-surge pressures during hydrant operations would seem to be a worthwhile task.

1.3 Thesis Organization

The structure of this thesis was largely shaped by preparation and presentation of conference papers and the submission of journal papers. Figure 1.4 shows the framework and the connection between chapters of this thesis. Chapters 3 to 6 narrow the focus to the concept that pressure standards are established and disruption of WDSs performance resulting from high delivery pressure which might not to be necessary for the system operation. This is where the bulk of analysis is to be found and where contributions of the research are. Chapters 7 and 8 focus on the linkage between pressure standards and transient pressures and how destructive transient pressures may be controlled to limit down-surge pressures to an acceptable limit even when the delivery pressure is relatively low. In the light of this, a novel and significant idea is developed (in chapter 8) in order to control pressure change during hydrant operations with the use of a portable device for limiting the down-surge pressures in a pipe system. It is hoped that the findings of this research motivate utility companies to rethink about pressure standards in WDSs.

The thesis comprises 10 chapters (including the present chapter). Chapter 2 offers a literature review covering generally the major themes of contemporary and historical issues of WDSs. The issues of high and low pressure consequences in WDSs are reviewed. More specifically the focus is on the connection between pressure and other factors affecting WDSs performance. Also, the role of water pressure in WDSs analysis (steady state and transient analysis) and uncertainty and reliability analysis in WDSs are reviewed.

Chapters 3 to 8 represent the core contribution of this research. In chapter 3, the critical discussion has been made on what pressure standards mean, where they are violated, and where they need revision to achieve a comprehensive picture about what the pressure standards really mean. This chapter introduces this notion that there may be still room for revising pressure standards. This chapter begins with the discussion of why the pressure standards need. Then, a critical appraisal has been made to what it means to comply with a pressure standard. Some useful metrics are derived to determine the

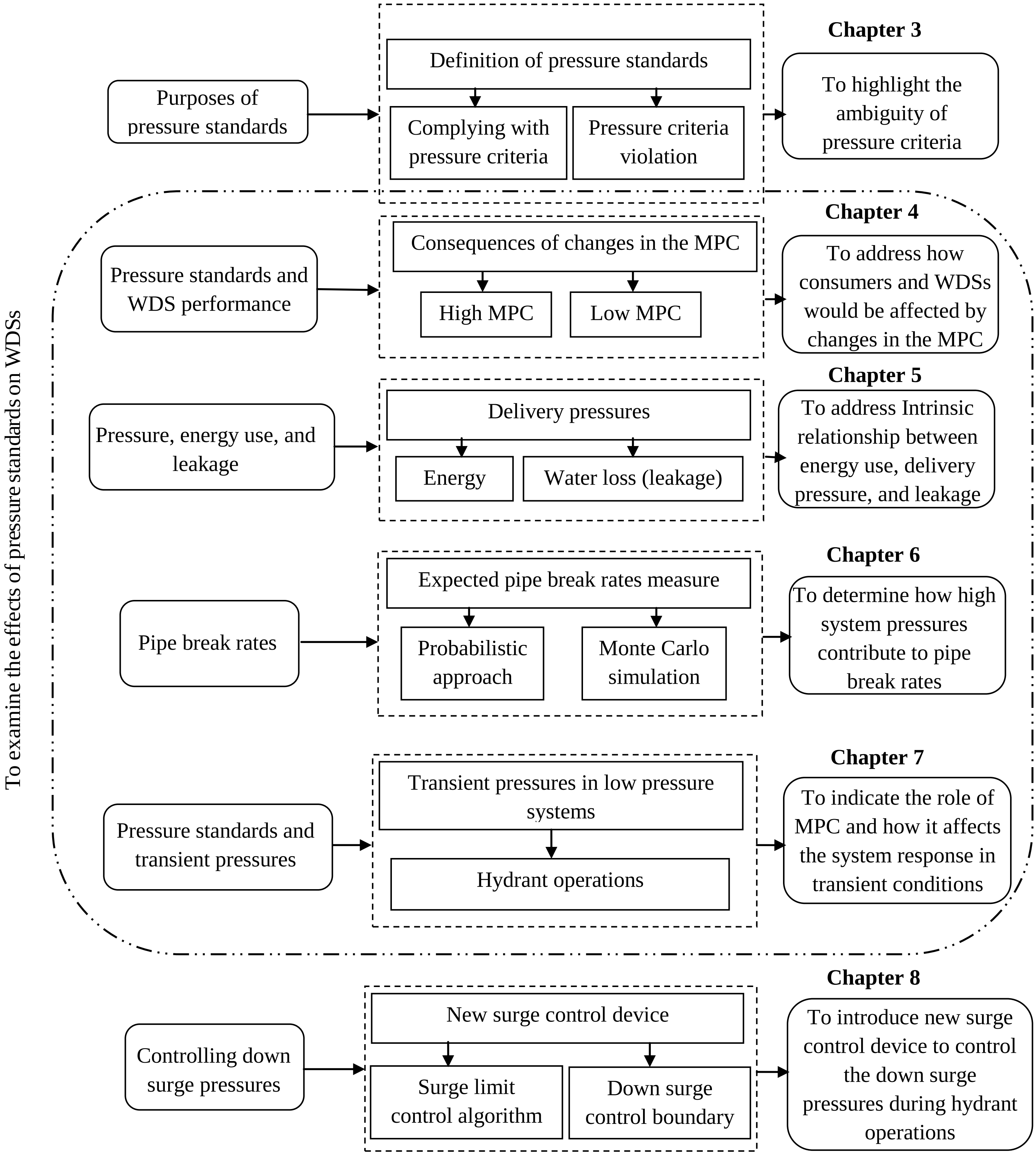


Figure 1.4. The connection between chapters

intensity of transgression of MPC, during transient events, in WDSs.

In chapter 4, the pros and cons of reduction in the MPC are explained. The tension that low value of pressure standard creates, e.g., an inability to supply the required flow and increasing the risk of an intrusion event associated with hydraulic transients, and the benefits of reducing this standard, e.g., a decrease in pressure-based demands and also the improvement in system performance through reduced energy use, leakage, and the frequency of pipe breaks, are discussed. The purpose of chapter 4 is to set the stage for the following chapters.

Chapter 5 focuses on the system energy use and leakage associated with changes in delivery pressure. In this chapter, the effectiveness of changes in the pressure on system energy use, leakage, and environmental impact is examined. An analytical expression is first developed to characterize the relationship between energy use, leakage and pressure for a single pipe segment. Then, two case studies, Anytown network and the unrehabilatated Anytown without storage tank, are considered to obtain a preliminary understanding of how a system gains the benefits of reduction in pressure supplied.

In chapter 6, a probabilistic approach is developed to introduce pipe break rates associated with system operating pressures. The chapter is divided into two parts. In the first part, an expected pipe break rates measure is developed to model pipe break rates which occur in the network experiencing high pressures. This is achieved by means of an expected value function that mathematically connects to pressure conditions at the network. Then, a MCS algorithm is developed to simulate uncertain municipal demands and pipes roughness coefficients. This algorithm also simulates the hydraulic response of a network, i.e., pressure heads across the network, to the sequence of loads. In the second part, the MCS algorithm and probabilistic approach are applied to the part of Hamilton network to address the expected pipe break rates due to distributed high pressures in WDSs.

Chapter 7 represents the fundamental approach to set the stage for chapter 8. This chapter presents the vital necessity of transient analysis. This chapter looks specifically at the role of MPCs and how they affect system response in transient conditions in order to raise the awareness about issues that can arise from changes in pressure standards enforced in WDSs design. Case studies are developed to show the potential the risk of transient pressures to the pipeline and how such a destructive transient pressure may be controlled to limit down surge pressures to an acceptable limits even when the MPC is relatively low.

Chapter 8 creates a new strategy of transient pressure control using down-surge control boundary in a pipe system during hydrant operations. In this chapter, the down-surge control boundary is first established using the portable control device to safely operate a hydrant in WDSs. Then, the extended Method of Characteristic (MoC) for transient analysis and the mathematical model of the control valve are described. Based on the numerical model of the control valve, a new application of down-surge control boundary is developed to limit the down-surge. In essence, the idea is to sense the pressure change at a hydrant location and then, according to these pressure changes, adjust the opening of the control valve so as to limit the down-surge, and thus the residual pressure, to a predefined level. Case studies are then developed to explore, at such a boundary, how the valve is able to adjust its opening automatically in response to the pressure changes to control the transient pressures at desired levels.

Chapter 9 contains additional background information for supporting the discussion which has been made in chapter 4 and for the methodology described in chapter 5. These are materials that could not be included in the associated papers of chapters 4 and 5 because of length constrains but can be justified for inclusion here. This chapter mostly determines the effectiveness of operating policy on system energy use and cost and the excess delivery pressure. The implication of changes in pressure standards on system energy use and cost, the system operating pressure, and leakage is also examined.

And finally, chapter 10, the last chapter of the thesis, summarizes the thesis, lists the major conclusions, and discusses possible future works.

1.4 Publications Related to the Thesis Research

As explained in the previous section, the contributions of this thesis have been disseminated in published format. The following list comprises journal and conference papers that are related to this thesis.

Ghorbanian, V., Karney, B., and Guo, Y. (2015). "Minimum Pressure Criterion in Water Distribution Systems: Challenges and Consequences." *EWRI 2015: Floods, Droughts, and Ecosystems: Managing Our Resources Despite Growing Demands and Diminishing Funds*, EWRI, 17-21 May, Austin, Texas, USA. (Source paper for chapter 4)

Ghorbanian, V., Karney, B.W., and Guo, Y. (2015). "The Link between Transient Surges and Minimum Pressure Criterion in Water Distribution Systems." *In*: *Pipelines Conference: Recent Advances in Underground Pipeline Engineering & Construction*, ASCE, August 23-26, Baltimore, Maryland. USA. (Source paper for chapter 7)

Ghorbanian, V., Karney, B., and Yiping, G. (2015). "Pressure Standards in Water Distribution Systems: A Reflection on Current Practice with Consideration of Some Unresolved Issues." *J. Water Resour. Plan. Manage.,* ACSE, Accepted for publication. (Source paper for chapter 3)

Ghorbanian, V., Karney, B., and, Guo, Y., (2015). "Intrinsic Relationship between Energy Consumption, Pressure, and Leakage in Water Distribution Systems." *Urban Water Journal*, Submitted for publication. (Source paper for chapter 5)

Ghorbanian, V., Guo, Y., and Karney, B. (2015). "Field Data Based Methodology for Estimating the Expected Pipe Break Rates of Water Distribution Systems." *J. Water*

Resour. Plan. Manage., ACSE, Accepted for publication. (Source paper for chapter 6)

Ghorbanian, V., Karney, B., and Yiping, G. "Development of a Control Valve Algorithm to Limit Pressure Down-surges During Hydrant Operations." Will be submitted to the *Journal of Hydraulic Engineering, ASCE.* (Source paper for chapter 8)

Chapters 3 to 8 are based on conference and journal papers either in print, under review, or will be submitted for publication. As primary author, I wrote the papers listed above and performed all of the detailed research and analysis that are presented in them. The co-authors of this paper, Bryan Karney and Yiping Guo, are also my PhD thesis supervisors. Professors Karney and Guo proofread and edited the manuscripts before submission and offered excellent suggestions and ideas to improve the work. I have received the permission and endorsement from the aforementioned co-authors to include in this thesis document all materials found in the publications listed above.

Chapter 2

Literature Review

2.1 Introduction

This chapter reviews the literature pertaining to issues associated with undesirable pressures in WDSs. Pressure is an important factor in WDS design and operation, and is also the main focus of this thesis. Most of the work related to pressure has historically concentrated on the consequences of high and low pressures in WDSs. Because this thesis examines the interconnections amongst several broad research areas, the review is selective as opposed to comprehensive, and only representative papers are chosen from each area. This chapter begins with the issues influenced by pressure to indicate the connection between pressure and other factors affecting WDSs performance. The role of water pressure in WDSs analysis (steady state and transient analysis) is described. The focus then turns to uncertainty and reliability analysis. Here, research papers that present methods to quantify uncertainty and to determine reliability in WDSs are reviewed in detail.

2.2 Pressure, Leakage, and Energy Use

WDSs are historically designed to deliver safe and reliable drinking water to end users under sufficient pressures. The pressure in WDSs should be controlled within a range whereby the allowed maximum pressure reduces the likelihood of pipe bursts and the specified minimum pressure ensures adequate flow for expected demands. A review of many guidelines and regulations suggests that MPC ranges from 10 m to 28 m under different flow conditions, i.e., during fire flow, during maximum hourly demand, and during normal conditions, and there is no set values for the maximum pressure standards in most regions and just for most provinces of Canada, the maximum pressure is specified as 70 m.

Pressure, as a form of energy delivered to every customer, is one of the most fundamental metrics to evaluate the safe and efficient distribution of water to customers. If pressure reduces, water consumption (e.g., consumptions from faucet, showers, and lawn watering) may decrease. A study in Denver, Colorado illustrated that reduction in water consumption was about 6 percent in a year for homes that received water service at lower pressures compared that at higher pressures (http://water.epa.gov/polwaste/nps/chap3.cfm).

Another important aspect of the pressure is its influence on leakage. Leak detection and control has been a growing concern of municipalities since leaky distribution systems cause significant water and energy losses. A 2004 audit at Toronto, Ontario, Canada estimated leakage to be 103 MLD (Lalonde et al. 2008). Flow through leaks depends upon the water pressure in a pipe at the leaky location (Colombo and Karney 2002; Giustolisi 2008; Wu et al. 2010; Gao et al. 2010). Colombo and Karney (2002 and 2005) examined the impact of leaks on the energy consumption in water supply systems. They concluded that leaks increase operating costs in all systems and energy costs increase more than proportionately with leakage. They also found that leaky systems with storage may not essentially have lower operating costs and energy use as compared to direct pumping systems. Colombo and Karney (2009) studied the effects of delivery head reduction and demand curtailment associated with leaks in a single pipe and found that pressure management with the aim of leak reduction is more influential in low resistance pipes.

The relationship between leakage and pressure has been known to be nonlinear

$$q_l = C_l H_l^N \tag{2.1}$$

where q_l is the leakage rate, C_l is the discharge coefficient, H_l is the pressure head at the leaky location, and N is an exponent. Traditionally, N is assigned a value of 0.5 assuming the normal hydraulic relationship of flow from a fixed orifice. But several studies have shown that the value of N could range from 0.5 to 1.5 (Hiki 1981; Lambert 2000). C_l can change depending on the flow regime through the leak and/or internal pipe pressure. Lambert (2000) indicated a relationship between C_l and Reynolds Number, R_e, for flow in a 15 mm diameter cooper pipe with a 1 mm diameter hole drilled into it in order to simulate a leak. For laminar flow through the leak (Re < 3000), C_l was found to increase sharply from about 0.3 to 0.7. For transitional flow (3000< Re <8000), C_l varied between 0.7 and 0.85. In the turbulent flow regime with Re > 8000, C_l levelled off to about 0.75. Clearly, from Eq. (2.1) reduction in pressure causes leakage to decrease. Thus, pressure reduction and careful pressure management can be important strategies of leak reduction and system management and are yet the areas that warrant further investigation.

In the United States, electricity represents about 75% of the cost of municipal water processing and distribution (USDOE 2006). Total electricity consumption for pumping and water distribution in the city of Toronto in 1998 was approximately 386 GWh/year (Filion 2008). American Water, the largest investor-owned water and wastewater company in the United States, reported that 97% of its electricity consumption and 90% of its greenhouse gas emissions are the products of the water delivery process (Young 2010). The US Environmental Protection Agency estimates that drinking water and wastewater services emit approximately 45 million tons of greenhouse gases into the atmosphere each year (Wallis et al 2008). In WDSs, reduction in pressure causes a decrease in both flow and pressure, and consequently the system energy use and leakage reduce. Pressure management is considered jointly with energy management because both problems are interlinked, hence pressure management reduces leakage and subsequently reduces energy consumption by reducing the pumped volume of water and therefore reduces unnecessary energy costs (Colombo and Karney 2002; 2005). These energy costs depend on the energy usage and the energy rate. Energy-saving measures in WDSs can be realized in different ways, from field-testing and proper maintenance of equipment to the use of optimal control. Energy usage can be reduced by decreasing the volume of water pumps, lowering the head against which it is pumped, and increasing the efficiency of pumps. Ormsbee and Lansey (1994) reviewed the existing optimal control methodologies for water-supply pumping systems, and classified methodologies on the basis of the type of system to which the methodology can be applied (single source-single tank or multiple source-multiple tank), the type of hydraulic model used (mass balance, regression, or hydraulic simulation), the type of demand model used (distributed or proportional), the type of optimization method used (linear programming, dynamic programming, or nonlinear programming), and the nature of the resulting control policy (implicit or explicit).

Jowitt and Germanopoulos (1992) presented a method based on linear programming for determining an optimal schedule of pumping. Both peak and peak-off electricity charges were considered, as well as the relative efficiencies of the available pumps, the structure of the electricity tariff, the consumer demand pattern, and the hydraulic characteristics and operational constraints of the network were taken into account. Walenda et al. (2006) proposed a novel idea of a feedback control of a WDS considering the time dependent electrical tariff. The approach was based on a decision surface concept constructed using a bundle of optimal trajectories which are obtained by solving the open loop scheduling problem for different initial reservoir levels. The decision surface was approximated locally during real time control by a convex polytope. These methods can reduce the energy use of WDSs and consequently GHG can be decreased.

The energy consumption can be used to evaluate the GHG impacts, which depend on the sources of water and energy. Physical infrastructure such as water, wastewater, solid-waste disposal, and transportation systems have been highlighted by the growing awareness of environmental issues. Life-Cycle analysis (LCA) has been widely applied to improve the environmental performance of products and services. LCA studies in the area of water cycle management have mainly addressed specific aspects of wastewater systems, e.g., the emission of greenhouse gases of municipal wastewater (Monteith et al. 2005), urban water systems, e.g., impacts of water distribution on environmental indicators (Sahely and Kennedy 2007), examining the potential environmental impacts of water systems (Undie et al. 2004), and quantifying energy expenditures in WDSs (Filion et al. 2004).

Racoviceanu et al. (2007) quantified the total energy use and greenhouse gas (GHG) emissions for the city of Toronto municipal water treatment system. They developed Life-Cycle inventories to assess impacts of chemical manufacturing and transportation and operational environmental effects. They found that on-site pumping accounting for the most operational burdens was dominant in contribution to the total energy use and GHG emissions, whereas the environmental impacts from the transportation of chemicals were appraised insignificant. They also reported that the average GHG emission for Ontario's electricity mix was 224 g CO_2 eq/kWh. A methodology was presented by Filion et al. (2004) to conduct a life-cycle energy analysis (LCEA) of a WDS. They concluded that energy expenditures for pipe fabrication and pipe repair are, in some cases, an order of magnitude larger than energy expenditures for pipe recycling and disposal. Examination of the impact of WDSs on energy use and the environment and identification of the opportunities for reducing energy consumption may reveal ways to reducing the environmental impacts associated with this infrastructure.

2.3 Pipe Breaks

Pipe breaks are an important issue of concern to municipalities. A study conducted in the USA and Canada indicated that the overall failure rate of water mains, during a 12 month period (in 2011), for all pipe materials is 11 failures/100 miles/year (Folkman 2012). Many factors contribute to the deterioration of water pipes that result in pipe breaks. These factors can be either dynamic (e.g., pipe age, water pressure, temperature, soil corrosivity, water contents of surrounding soil, and previous pipe breaks) or static (e.g., pipe diameter or pipe material) (Wang et al. 2009). Morris (1967) presented a few possible causes of water main structural failures that include soil aggressiveness or corrosivity, soil stability, weather conditions, bedding conditions, construction quality, and land development. Some other factors influencing in pipe failures include internal corrosion, pressure surges, and faulty anchorages at branches, bends and dead ends (Kleiner and Rajani 2001). Shamir and Howard (1979) extended the list to include manufacturing flaws and traffic loading. Marks et al. (1987) studied the importance of joints leading to structural failure.

The weight of the contribution of each factor to pipe breaks is still not universally agreed in the literature but it has been established that some factors are more high-risk than others. The most commonly used pipe materials in a water distribution system are cast iron (CI), ductile iron (DI), polyvinyl chloride (PVC), high density polyethylene (HDPE), asbestos cement (AC), steel, and concrete. The pipe material is accounted for a factor influencing pipe breaks since the physical properties of a pipe changes after it has been installed and consequently the resistance and life span of the pipe change (Kleiner and Rajani 2001). In North America, CI, DI, and PVC are the most dominant material of pipes used in WDSs (Folkman 2012). However, it has to be noted that the distribution of pipe materials is likely to change in the future due to the current extensive use of plastic pipes (http://www.truthaboutpipes.com/a-progressive-solution-to-materials-used-for-water). It was identified that the majority of breaks occur in CI pipes that are the oldest pipes, often installed more that 50 years ago (Pelletier et al. 2003; Singh and Adachi 2013; Kimutai et al. 2015).

Pipe diameter is identified as one of the factors affecting pipe failure rates (Clark et al. 1982; Berardi et. al 2008). It was reported consistent in the literature that small diameter pipes have greater number of failures that of larger diameter pipes (Berardi et. al 2008; Wang et al. 2009; Kimutai et al. 2015). The majority of statistical models that are used for predicting water main breaks using available historical data considered pipe age as the important factor describing the time dependence of pipe breakage (Berardi et. al 2008; Wang et al. 2009). It was reported by many researchers that pipe failure varies with pipe age in a bathtub curve (Andreou et al. 1987; Kleiner and Rajani 2001; Singh and Adachi 2013). The bathtub curve consists of three periods (Kleiner and Rajani 2001). The first is known as burn-in period, in which breaks occur mainly as a result of faulty installation or improper pipes sizes, with a decreasing failure rate. The second period is a normal life period, in which the pipe operates relatively trouble free but failures may occur due to heavy loads, third party interference, etc., with a relatively low constant failure rate. The third period is called a wear-out period that exhibits an increasing failure rate due to pipe deterioration and ageing.

Environmental factors such as precipitation, soil conditions, frost and traffic loading, and quality of external underground water have been identified to contribute to the failure rate of pipes in water networks (O'Day 1989; Rajani and Zhan 1996; Kleiner and Rajani 2001; Kimutai et al. 2015). The relationship between pipe failure rate, soil crossevity, temperature, and rainfall was studied by Pratt et al. (2011). They found that the pipe break rates (for CI pipes) positively correlated to soil corrosivity (ferro and cement corrosivity), and rainfall but the failure rate is negatively correlated to the temperature. Generally, low temperature and rainfall have been identified to increase pipe break rates in WDSs (O'Day 1982; Brander 2001; Kimutai et al. 2015).

High operating pressure of WDSs is an important factor that causes pipe breaks. A survey conducted by Pearson et al. (2005) showed that reducing pressure by approximately 50 m (during high period of pressures) by installing control valves in a real system, part of a large network in the UK, causes the burst frequency drops from 3 per month to one every six months on average. They also concluded that a nonlinear relationship between relative burst frequency and operating pressure can be established

$$\frac{B_1}{B_0} = \left(\frac{P_1 - P_a}{P_0 - P_a} \right)^{N_2} \tag{2.2}$$

where B_0 and B_1 are respectively the burst frequencies before and after pressure reduction, P_0 and P_1 are operating pressure before and after pressure reduction, respectively, P_a is a positive pressure at which the burst frequency would be zero, and N_2 is an exponent. They pointed out that the burst frequency would be zero for pressures below 20 m. The value of N_2 was reported to vary from 0.2 to 12 for 50 water networks studied and the mean and standard deviation of N_2 were 2.36 and 3.29, respectively. Eq. (2-2) is valid for the case studies conducted in the UK and may not be generalized in every system.

Thornton and Lambert (2005) analysed data collected from 21 utilities of 11 countries on breaks (or repairs) before and after pressure management and concluded that management of surges and excess pressures can reduce numbers of breaks from 28% to 80% per year. They also concluded that the percentage reduction in pipe bursts often exceeds the percentage reduction in average maximum operating pressure; e.g., in the USA, 36% reduction in average maximum operating pressure caused 50% reduction in pipe breaks. Creaco et al. (2016) provided a methodology in order to assess how variations in district service pressure affect leakage, electricity costs for the operation of the pumps, and pipe break rates. They reported that the average service pressure reduction from 48.23 to 30 m (i.e., 38% reduction in delivery pressure), in the Abbiategrasso district, Italy, leads to reduction in leakage, pipe break rate, and energy savings by 27%, 5.3%, and 53%, respectively. More details on the relationship between pressure and pipe breaks have been discussed in Chapter 6 of this thesis.

Many studies have established the link between pipe aging, structural deterioration, the onset of leakage, and the increase in pipe breaks (Kettler and Goulter 1985; Kleiner and Rajani 2001; Wang et al. 2009). Shamir and Howard (1979) developed a model with the use of regression analysis to predict the number of pipe breaks per year per length of pipe:

$$N(t) = N(t_0)e^{A(t-t_0)} \tag{2.3}$$

where $N(t)$ and $N(t_0)$ are number of breaks per 1000-ft of pipe in years t and t_0, t is current year; t_0 is the base year for the analysis (the year that the pipe was installed or the year for which data is available), and A is coefficient of breakage rate growth (year^{-1}). Note that $N(t_0) \neq 0$, which means that on average a pipe is assumed to always have a breakage frequency, albeit very small in the beginning of its life. The scarcity of data on factors contributing to pipe deterioration and pipe breaks (e.g., aging of material, corrosivity of soil, frost heaving and settlement, poor installation, service pressures, surface loading) and the difficulty in developing models that account for these factors have led researchers to resort to statistical methods to simulate histories of pipe breaks in systems (Andreou et al. 1987; Li and Haims 1992; Kleiner and Rajani 1999). A general statistical failure prediction model proposed by Cox (1972)

$$h(t, Z) = h_0(t)e^{\beta Z} \qquad (2.4)$$

where t is time, $h(t, Z)$ is the hazard function which is the instantaneous rate of failure (probability of failure at time $t+Dt$ given survival to time t), $h_0(t)$ is an arbitrary baseline hazard function, Z is the vector of covariates acting multiplicatively on the hazard function, β is the vector of coefficients to be estimated by regression from available data. The baseline hazard function, $h_0(t)$, can be interpreted as a time-dependent aging component, while the covariates represent environmental and operational stress factors that act on the water main to increase or reduce its failure hazard.

All models presented to determine pipe break rates are only estimation tools because of lack of required data. This gives support to the argument for collecting data in different networks in order to determine the exact rate of pipe breaks. Moreover, few researches have been conducted to obtain a relationship between pipe break rates and operating pressure, the key step in the development of economic models to determine the financial benefit of reducing pressure.

2.4 Pressure Management

Traditionally, pressure management is an effective way of reducing the excess pressures and the amount of water lost in a WDS during off-peak hours (Gomes et al. 2011). The most common methods of pressure reduction include pump and pressure control, fixed outlet control valves, flow modulated control valves, and remote node control (Thornton 2002; Thornton et al. 2008). Pressure management strategies can be implemented without affecting service levels when activated during low demand periods such as late night and early morning. In the pressure management process, minimum night flow analysis is usually conducted in order to calculate the factors related to losses since most of the users are not active during the night and pressures are high throughout the systems (Walski et al. 2006a; Gomes et al. 2011; Campisano et al. 2012).

Many studies have been conducted to determine the location and the required calibration of a pressure reducing valve (PRV) in the pipe networks with the optimization of the number of valves and their locations (Sterling and Bargiela 1984; Vairavamoorthy and Lumbers 1998; Liberatore and Sechi 2009). In the proposed optimization techniques, linear objective functions and linear constraints are obtained to minimize deviations from referenced pressures at the nodes. The practical challenge to operate a PRV is to determine the optimal adjustment for opening the valve. The location and the opening setting of the valves were assessed by minimizing the mean square error between the actual and the target pressure (Fontana et al. 2012) calculated as

$$Z_j = \sum_{i=1}^{n} \gamma \left[\left(P_{i,j} - P_{\min} \right) \right]^2 \tag{2.5}$$

where Zj is the objective function, j is the time step, i is a node of the network, $P_{i;j}$ is the pressure calculated in the node i at time j, P_{min} is the target pressure, and γ is a penalty coefficient. At all demand nodes, the pressure should be larger or equal to P_{min} to ensure adequate service.

In recent years, the opportunity of replacing PRVs with turbines or pumps as turbines (PATs) was highlighted. Micro hydro (the water turbine with a capacity less than 100 kW) and mini hydro (the water turbine with a capacity from 100 kW to 1 MW) generators can be installed in WDSs to ensure both pressure control for leakage reduction and energy production (Ramos and Borga 1999; Mhylab and ESHA 2010). The hydropower plant installation in a water supply system may be either a reservoir-service tank hydropower plant or a distribution line hydropower plant (Afshar et al. 1990; Mhylab and ESHA 2010). In a reservoir-service tank installation type system, the hydropower plants are installed at the different sections of the transmission pipeline connecting supply reservoirs such as dams to service tanks which are placed to provide stable water pressure in a given district. This type of installation is quite similar to the conventional type of hydro plant installations. In a distribution line hydropower plant, a power plant is installed in the community's service main just before it connects to the distribution system. This type of installation is subject to daily and seasonal fluctuations and is only possible if there is high water level in the service tank. (Afshar et al. 1990; Vieira and Ramos 2009). Another idea that seems to be worthwhile for reducing excess pressure in WDSs is strategies that decrease the energy supplied which can be achieved by reducing the pressure standards. This novel idea has not been researched yet.

2.5 Pressure dependent demand analysis

To analyse the performance of water networks, governing equations based mass and energy conservation should be solved for nodes and loops. In distribution systems, water consumption is assigned at nodes and defined as nodal demands which are treated as known values and then nodal hydraulic head is determined. This approach is often called demand driven analysis assuming that consumer demands are always satisfied regardless of the pressures throughout the system. This approach is valid in normal operating conditions; however, in abnormal cases, e.g. pipe outages, power failures at pump stations and fire flow conditions, nodal pressure may not supply the desired demand. In these cases, accurate analysis cannot be achieved without considering the impact of the pressure change on the supplied flow (Gupta and Bhave 1996). To accurately predict the performance of the system under deficient-conditions, pressure dependent demand (PDD) analysis is necessary.

In PDD analysis, nodal demand is assumed to vary with the nodal pressure and when the nodal pressure rises to a certain level, i.e., the desired pressure, 100% of the demand is supplied. The PDD function is

$$Q = \begin{cases} 0 & \text{if } P \leq 0 \\ Q_{de}\left(\dfrac{P}{P_{de}}\right)^{n} & \text{if } 0 < P < P_{de} \\ Q_{de} & \text{if } P \geq P_{de} \end{cases} \tag{2.6}$$

where P represents the pressure head at a node, Q_{de} denotes the desired demand at the node, Q is the actual flow rate at the node, P_{de} is the desired pressure which is often considered to be equal to MPC, and n is the exponent of the pressure-demand relationship. Other equations have been also used for prediction of water distribution systems behaviour under pressure deficient conditions. The following are functions proposed by Wagner et al. (1988) and Chandapillai (1991).

$$q_j^{avl} = q_j^{req} \qquad \text{if } H_j^{avl} \geq H_j^{des} \tag{2.7}$$

$$q_j^{avl} = q_j^{req}\left(\frac{H_j^{avl} - H_j^{\min}}{H_j^{des} - H_j^{\min}}\right)^{\frac{1}{n}} \qquad \text{if } H_j^{min} < H_j^{avl} < H_j^{des} \tag{2.8}$$

$$q_j^{avl} = 0 \qquad \text{if } H_j^{avl} \leq H_j^{min} \tag{2.9}$$

$$H_j^{des} = H_j^{\min} + R\left(q_j^{req}\right)^{n} \tag{2.10}$$

where q_j^{avl} is the available flow; q_j^{req} is the required design demand; H_j^{avl} is the available nodal head; H_j^{des} is the desired head; H_j^{min} is the minimum required head; R is a resistance constant; and n is an exponent. In the original paper, R and n were taken as 0.1 and 2, respectively.

Several methods have been proposed to analyse WDSs under insufficient conditions. Germanopoulos (1985) introduced empirical functions to include pressure dependent demand and leakage terms in simulation models for WDSs. Chandapillai (1991) suggested a parabolic relationship between demand and available pressure to analyse low pressure conditions in WDSs. Gupta and Bhave (1996) compared several available methods for prediction of WDSs' behaviour under pressure deficient conditions and concluded that the node flow analysis proposed by Wagner et al. (1988) and Chandapillai (1991) are the best. Assela (2010) modified the EPANET2 computational engine using an emitter function to determine the hydraulic performance of WDSs under insufficient pressures but the proposed model has a drawback for modeling the combined volume-based and pressure-dependent demand scenarios.

2.6 Transient Events

Transients can introduce large pressure forces and rapid fluid accelerations into a pipeline system. These disturbances may result in pump and device failures, pipe and equipment ruptures, and the backflow/intrusion of contaminated water. Many transient events can lead to column separation which can result in catastrophic pipeline failures due to severe pressure rises following the collapse of vapor cavities. Thus, transient events increase leakage and health risks and decrease system reliability. Transient flow simulation has become an essential requirement for assuring safety and the safe operation of drinking WDSs. Transient pressures are most significant when the rate of flow is changed rapidly, resulting from rapid valve closures and openings or pump stoppages. Flow disturbances, whether caused by design or accident, during transient events may create traveling pressure and velocity waves of excessive magnitude. These transient pressures are superimposed on the steady state (static) conditions in the pipeline at the time the transient occurs. The total force acting within a pipe can be obtained by summing the steady state and transient pressures in the pipeline system. Thus, the severity of transient pressures must be accurately determined so that the pipes can be properly designed to withstand these additional forces.

Transient flow is a response of the fluid to some change in the hydraulic facilities that control and convey the fluid. Starting up or switching off water pumps, opening and closing valves, and fire hydrants operation result in rapid flow changes that produce pressure waves, which have both positive and negative phases. All transient flows are transitions, of long or short duration, from one steady flow state to another. Transient pressures can be significant sometimes creating pressure changes that are large enough to burst pipes and small enough to cause contaminant intrusion, column separation, and consumer complaints. Two equations, the momentum and continuity equations are generally used to model the transient flow in pipeline systems (Wylie, and Streeter 1983, Chaudhry 1987). The governing equations can be written as:

Continuity equation:
$$H_t + \frac{a^2}{g} V_x = 0 \tag{2.11}$$

Momentum equation:
$$V_t + gH_x + \frac{f}{2D} V|V| = 0 \tag{2.12}$$

where x is the distance along the centerline of pipe, t is time, H_x and H_t are the partial derivatives of piezometric head with respect to x and t, respectively, V_x and V_t is the partial derivatives of the fluid velocity with respect to x and t, respectively, D is pipe diameter, f is Darcy-Weisbach friction factor, a is wave speed, and g is acceleration due to gravity. The general assumptions, to derive Eqs. (2.11) and (2.12), are the flow is one dimensional, the convective term and pipes slope are small, pipes properties, e.g., diameter, wave speed, and temperature, are constant, and the friction factor is approximated by the Darcy–Weisbach formula for steady state flow.

A general solution to Eqs. (2.11) and (2.12) is not available because of the nonlinearity of the momentum equation and the complexity of water networks. However, the method of characteristics (MoC) is the most popular approach by which the partial differential equations can be transformed into ordinary differential equations (Wylie and Streeter 1983). The MoC combines the momentum and continuity expressions to form two compatibility equations in terms of head, H, and discharge, Q, which are valid along the so-called positive and negative characteristic equations (denoted as C^+ and C^- in Figure 2.1):

$$C^+: H_p = C_p - BQ_p,\ C_p = H_A + BQ_A - RQ_A|Q_A| \tag{2.13}$$

$$C^-: H_p = C_M + BQ_p,\ C_M = H_B + BQ_B + RQ_B|Q_B| \tag{2.14}$$

In the above two equations, $B=a/gA$, A is cross-sectional area of the pipe; $R=f\Delta x/(2gDA^2)$. To achieve the stable numerical solution, the x-t grid is usually chosen to ensure $(a\Delta t/\Delta x)=1$. H and Q having the letter A and B are always available either as given initial condition or as the results of a previous stage calculation. This numerical solution is performed in chapter 8 of the thesis to develop the surge control algorithm.

Pressure transients in WDSs are inevitable and will normally be most severe at pump stations and control valves, in high-elevation areas, in locations with low static pressures, and in remote locations that are distant from reservoirs (Friedman et al 2004). WDSs operating under high pressures may experience severe transient pressures. Conversely, WDSs operating under low pressures may be susceptible to negative

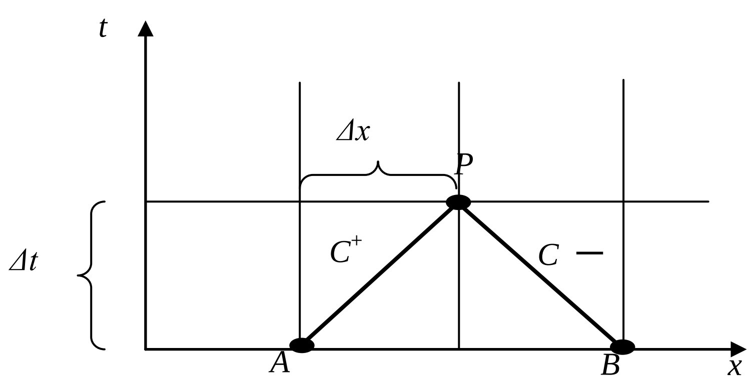


Figure 2.1. Method of Characteristics *x-t* grid (Karney and McInnis 1992)

pressures. In general, among scenarios considered for WDS designs, there are two of them under which water systems may experience low pressures: maximum hour demand or maximum day demand plus required fire flow. Pressures in WDSs should be maintained at an acceptable level during transient conditions in order to avoid pipe bursts resulting from high pressures and to sustain the minimum pressure so that adequate flows are supplied to consumers. Intrusion, the flow of contaminated water into drinking water systems through leaks, cracks, and submerged air valves, also occurs due to transient low or negative pressures (CPWSDS 2006).

Protection of WDSs against transient pressures is necessary. To minimize a system's susceptibility to surge pressures, surge control strategies for mitigating pressure transients must be performed. Surge control strategies have been divided into three categories: engineering strategies, maintenance strategies, and operational strategies. Devices such as surge anticipation valves, pressure relief valves, air release/vacuum valves, surge tanks, and air vessels are generally used to control surge pressures in the system. Surge anticipation valves are generally installed in pumping systems to open immediately after pump failures (Lescovitch 1967); however, these valves are not efficient to prevent column separation although they can minimize the impact of cavity occurrence (Larock et al. 2000). Surge tanks are used to absorb sudden rises of pressure in pipelines and must be sized to admit the maximum possible upsurge and not to allow air to be drawn into pipelines (Larock et al. 2000). In some researches the optimal location, type and size of surge protection devices have been determined by employing a transient model and an optimization model (Laine and Karney 1997; Lingireddy et al. 2000; Jung and Karney 2003, 2006).

Maintenance activities can help to minimize the likelihood of intrusion during down surge occurring in WDSs. The possibility of employing hydraulic transient models for quantifying levels of deterioration and identifying the pipes requiring repair have been addressed in the literature (Arbon et al. 2006; Misiunas et al. 2007; Gong et al. 2013). For this purpose, the magnitude of transient wave reflections from the segment of pipeline is used to assess the condition of the pipe segment (Misiunas et al. 2007). At locations along the pipelines with poor conditions, the wave reflections have a larger magnitude than less damaged sections (Misiunas et al. 2007).

Proper valve motions and fire hydrant operations, as well as suitable rates of switching pumps can all be used to maintain transient pressures within acceptable levels. Using variable-speed drives for pumps may lead to the avoidance of high surges during the start-up of pumps, but their effect is negligible in case of a power failure (Huo 2011). Valve stroking procedure, explained by Wylie and Streeter (1983), is a technique in which transient pressures are controlled within acceptable pressure limits. The time of valve closure is explicitly restricted to a duration of $4L/a$, where L is the pipe length and a is the speed of sound in the pipe (Wylie and Streeter 1983). Goldberg (1987) subsequently developed a technique, called quick stroking (QS), to control surge pressures in the minimum time of valve closure of $2L/a$. Zhang et al. (2011) developed a nonreflective boundary condition in which the opening of an active control valve is adjusted using dynamic proportional-integral-derivative (PID) controllers to eliminate or attenuate system transient pressures. Careful operation of systems are required to perform in accordance to appropriate operational strategies; many of them need the combination of a remote sensor and a control system to implement in WDSs which are costly for operating water networks.

The transient conditions occurred during the operation of WDSs can be controlled by the aforementioned surge control strategies. However, in the case of hydrants operation, it is impossible to employ a surge control device at each hydrant to control transient pressures since there are many hydrants scattered at different locations of a WDS. Therefore, transient pressures induced by hydrants should be controlled through slow opening of the hydrant. How slow the hydrant opening motion should be is still a question.

2.7 Uncertainty

The traditional design strategy of WDSs is to size WDS features, i.e. pumps, tanks, and proper pipe diameters, to provide existing and future water demands above the MPC with a possible minimum cost. Design demands comprise a diurnal demand pattern and peak scenarios, i.e. maximum hourly demand and maximum day demand plus fire flow requirements. This few number of design demands results in system response against a small fraction of the vast combination of existing loading conditions that can be explored. This design strategy is often called the deterministic approach because the design parameters are not treated as random variables. Overall, the main limitation of the deterministic approach of WDS design is that the performance of WDSs is predicted using a small number of design parameters.

Since the number and types of future consumers can not be accurately determined, the future required demands for WDSs design are uncertain. The estimation methods to determine needed fire flow (NFF) are also subject to uncertainty since the estimated flow may be sufficiently high to control the fires occurring in the network. Another uncertain parameter in the design of WDSs is the roughness coefficients of pipes duo to pipes' corrosion and deposition of debris during the period of operation. Consequently, the computed pressures which are the real concern of WDSs reliability are not certain either. The reliability of WDSs is inversely associated with hydraulic failure considering inadequate delivery of flow and pressure head at demand points. To incorporate uncertainty in WDSs design, the demand including fire flow and pipe roughness must be considered as random variables.

Consumer consumption varies across the world and is assessed by determining the amount of water that actually is used by consumers. The water consumption includes a customer’s faucet (i.e., municipal, commercial, and industrial use), a leaky main, and an open fire hydrant. All aforementioned three types of water consumption are defined as water demand in WDSs. The primary step to analyse WDSs is to determine the baseline demands including both customer demands and unaccounted-for water (leak). The baseline demand usually equals the average day demand calculated from monthly or quarterly meter readings and billing records. In WDSs design, demand patterns obtained by multiplying the demand multiplication factors are used to simulate the behavior of the quasi-dynamic system over a period of time in which hydraulic demands and boundary conditions change with respect to time. Multiplication factors from average day to maximum day range from 1.2 to 3, and factors from average day to peak hour are typically between 3 and 6 (Walski et al. 2007). The most common simulation duration is typically a multiple of 24 hours, because the most recognizable pattern for demands and operations is on a daily basis. A diurnal demand pattern is the temporal variation in water usage for municipal water systems (Figure 2.2).

Since current and future water demands are not accurately predictable because of lack of available field data, it is common to assign identical diurnal demand curves to all

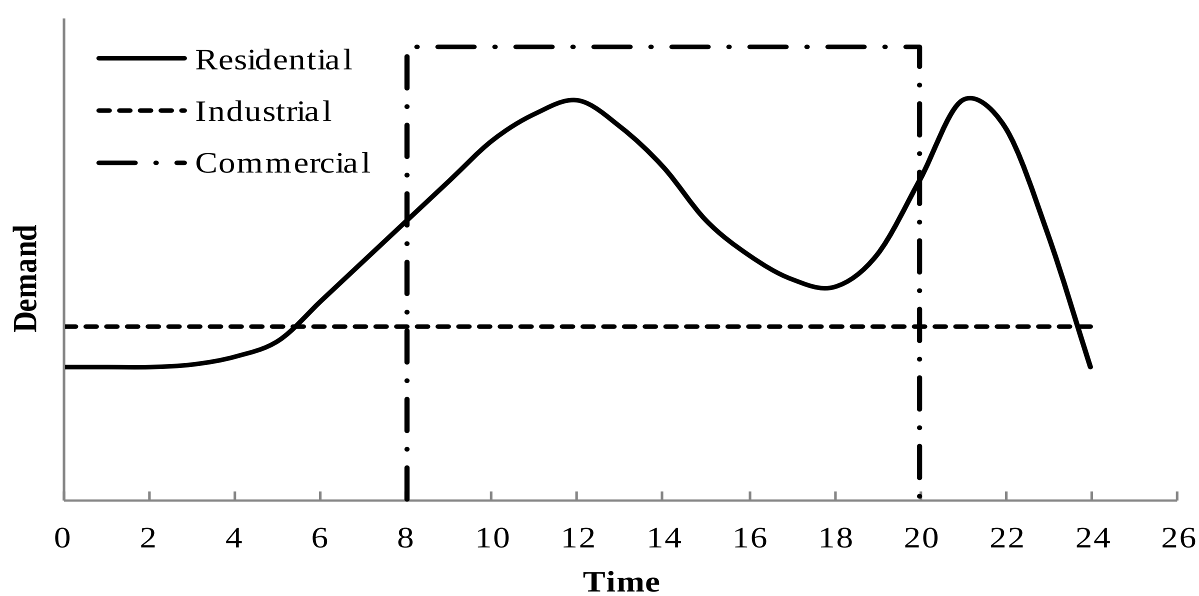


Figure 2.2. Demand patterns for different users (Filion et al. 2007a)

users of the same type in WDSs design. Applying a diurnal demand pattern to all users implies that demand pairs are perfectly correlated in water networks, implying that all users react simultaneously and in exactly the same way to normal and peak demand conditions. However, in a real system, users consume water based on social habits and financial constraints. Filion et al. (2007a) argued that the assumption of correlating demand pairs may not be precise in real WDSs and this presumption can lead to more expensive designs. They concluded that collecting data on demand may help utilities to better determine the relationship between demand pairs. Blokker et al. (2008) however pointed out that the assumption of high cross correlation is valid at 1-hr time scales and demand nodes representing more than 10 connections. Therefore, the nodal demands may assume to be perfectly correlated or uncorrelated with each other in WDSs. In developed models for WDSs design under uncertainty, the nodal demands were considered to be correlated (Gomes and Karney 2005, Kapelan et al. 2005, Filion et al. 2007a) or independent of each other (Lansey et al. 1989, Bao, and Mays 1990, Kang et al. 2009). In both assumptions of dependency and independency of nodal demands, the demand is assumed to follow normal or log normal distributions with the known mean and standard deviation.

Fire flows are considered as the additional demands at the specified nodes in WDSs under maximum day demand. The fire flow is required to control fire estimated with some uncertainty according to guidelines. The required fire flows are described in fire codes published by cities, counties, or other political jurisdictions. These codes describe the water requirements associated with firefighting in terms of quantity, duration of service, pressure, and hydrant placement (AWWA 2002). There are no fire flow standards that apply to all countries and it is the responsibility of each country to develop or adopt its own fire flow requirements (AWWA 2002). The needed fire flow (NFF) is the rate of flow considered necessary for suppressing a major fire in a specific building. The required fire flow duration is 2 hours if NFF is equal or less than 2500 gpm (158 L/s) and it is 3 hours for NFF equaling 3000 to 3500 gpm (189-221 L/s) (AWWA 2008). The fire flow requirement has been studied as the stochastic phenomenon in WDSs analysis.

Gomes and Karney (2005) presented a methodology to evaluate the reliability of WDSs under a fire condition. Filion et al. (2007b) proposed a stochastic design approach including the expected annual damages due to low and high pressure hydraulic failures in a water network. They assumed that fires occur independently both in time and in space. They also used the Poisson distribution to simulate the eruption of fires at each node and a normal distribution for fire flow rate. Conditional probability of fire flow failure, developed from the numerical or analytical probabilistic model, was incorporated into the multi-objective optimization model of water networks (Filion and Jung 2010; Jung et al. 2013). The uncertainty in NFF in water networks can be represented with a probability distribution function (PDF) with the known mean and standard deviation.

The friction coefficient of a pipe depends on the properties of the fluid that is passing through the pipe, the speed at which it is moving, the internal roughness of the pipe, and the length and diameter of the pipe (Walski et al. 2007). Among factors influencing the friction, the roughness of the pipe cannot be determined directly. Pipe roughness values may be estimated in two ways: using values from literature or directly from field measurements (Walski et al. 1988). The frequently used head loss expression, particularly in North America, is the Hazen-Williams formula (Williams and Hazen 1920):

$$h_L = \frac{aL}{C_h^{1.852} D^{4.87}} Q^{1.852} \tag{2.15}$$

where h_L is the head loss due to friction (m), L and D are respectively the pipe length and diameter (m), C_h is the Hazen-Williams coefficient, Q is the flow rate (m^3/s), and a is the unit conversion factor (10.7 for SI unit). It was acknowledged that the friction factor of a pipe changes, i.e., decreases in Hazen-Williams coefficient, over the life of a pipe (Sharp and Walski 1988; Boxall et al. 2004). The friction factor of a pipe is directly influenced by the internal roughness of the pipe which changes duo to the corrosion of the pipe and deposition of residuals inside the pipe during the period of operation (Sharp and Walski 1988). Lamont (1981) found that the decrease in C_h factor, associated with the pipe's age, depended heavily on the corrosiveness of the water being carried. Sharp and Walski (1988) proposed an equation to predict the C_h factor for future times which shows how the Hazen-Williams coefficient decreases with time:

$$C_h = 18 - 37.2\log\left(\frac{e_0 + at}{D}\right) \tag{2.16}$$

where e_0 and a are the pipe's initial roughness and the roughness growth rate, respectively, and t is the time since the installation of the pipe.

The high cost of tests to determine the friction coefficient in time prohibits the acquisition of exact data for various water pipes in a network. Therefore, the way to deal with the uncertainty in the friction factor of pipes is to use a probabilistic approach. A pipe's roughness may be assumed to follow a normal distribution defined by the known mean and standard deviation for uncertainty analysis of WDSs (Kang et al. 2009; Lansey et al. 1989). Bao and Mays (1990) studied different types of distributions, i.e. normal, log-normal, Gumbel (minimum value and maximum value), uniform, triangular, Pearson type III, log-Pearson type III, Weibull, and trapezoidal distribution, for examining the sensitivity of the hydraulic reliability of a water system to distributions of pipe roughness. They concluded that the system reliability is somewhat insensitive to the types of probability distributions of pipe's roughness when the coefficient of variation is less than 0.4. Furthermore, if the coefficient of variation is greater than 0.4, different types of probability distributions resulted in greater variations in system reliability. In the work of Kapelan et al. (2005), pipe roughness was assumed to follow a uniform PDF with known lower and upper bounds and was incorporated into a multi-objective optimization design of WDSs.

Many researches have been conducted to incorporate uncertain design parameters in WDSs analysis. Lansey et al. (1989) introduced a chance constrained model to include the uncertainties in required demands, required pressure heads, and pipe roughness coefficients for the minimum cost design of water distribution networks. The random variables, i.e., the demands, required pressure heads, and roughness coefficients, were assumed to be normally distributed with known means and standard deviations. They concluded that uncertainties in the aforementioned variables have significantly affected the optimum network design and cost. They also concluded that there is an inverse relationship between the network cost and reliability. Thus, increasing the reliability level will result in a greater increase in the system cost. Kang et al. (2009) examined the ability of approximate methods, the first-order second-moment (FOSM) method, a quasi-MCS method, and Latin hypercube sampling (LHS), for uncertainty analysis of WDSs. Results indicated that FOSM provides good estimation for pressure uncertainty while LHS performed well for both chlorine concentration and pressure head at nodes compared with MCS. Basupi and Kapelan (2014) presented a flexible methodology that combines sampling techniques (Monte Carlo or Latin Hypercube simulations), decision tree analysis, and genetic algorithm optimization. They proposed the methodology incorporating the future uncertain water demand to determine flexible and optimal decisions for WDSs design. Risk-based optimization has been researched to incorporate uncertainty to solve WDSs' design problems (Tung 1986; Xu and Goulter 1999; Kapelan et al. 2006; Yannopoulos and Spiliotis 2013). In the proposed methods to determine optimal WDS designs, uncertainty was usually incorporated into the problem formulation as a constraint to either maximise the overall WDS robustness, the probability of satisfying minimum pressure head constraints at all nodes in the network, or to minimise the total WDS risk or the probability of pressure failure at nodes.

2.8 Reliability

To provide numerical estimations of the stochastic features of the system response, MCS is used. In MCS, the system response of interest is repeatedly calculated under various system parameter sets generated from the known or assumed probability distributions. MCS has been widely used to determine the hydraulic reliability and optimal design of WDSs considering uncertainty in input variables, i.e, demands, pipe's roughness, and fire flow events (Bao and Mays 1990; Lansey et al. 2001; Wagner et al. 1988; Filion et al.

2007b). Wagner et al. (1988) presented two reliability indices, reachability and connectivity, to assess the reliability of a WDS where reachability is defined as the overall probability that a given demand node is connected to at least one source and connectivity denotes the probability that every demand node is connected to at least one source. They concluded that these two indices can be used to identify basic sources of unreliability in a system, e.g. lack of network interconnections or extremely unreliable links. Bao and Mays (1990) presented a methodology based on a MCS to estimate the nodal and system hydraulic reliabilities of WDSs. In that study, the nodal reliability was defined as

$$R_n = P(H_S \geq H_r \mid Q_S = Q_r) \tag{2.13}$$

where H_S and H_r are respectively supplied and minimum required pressure head and Q_s and Q_r are supplied and required water demand, respectively. They also proposed that the system reliability be defined as the arithmetic mean, i.e. $R_{sa} = \sum R_n/I$ where R_n is the nodal reliability and I is the number of demand nodes, or the weighted average, i.e., $R_{sw} = \sum R_n Q_{sa}/\sum Q_{sa}$ where Q_{sa} is the mean value of water supply at the node. They concluded that the difference in system reliability between the arithmetic mean and the weighted mean of nodal reliabilities is insignificant; however, they recommended the use of weighted mean to combine nodal reliabilities to assure the overall system reliability.

Li et al. (1993) studied the capacity reliability of water networks including single demand node when there is uncertainty in internal roughness coefficients of pipes. They defined the capacity reliability as the probability that the carrying capacity of a network meets the demand as follows

$$R_h = \sum P(Q_N \geq Q_D \mid A_i) P(A_i) \tag{2.14}$$

where Q_N is the capacity of the WDS, Q_D is the water demand at the demand node, and $P(A_i)$ is the probability of pipe break for configuration A_i. Ostfeld et al. (2002) presented a probabilistic simulation model for analyzing the reliability of WDSs. In that method, demand, source concentration, and failure components (pipes, pumps, or sources) were treated as random variables considering a presumptive probability distribution. They defined three reliability measures: the fraction of delivered volume (FDV), the fraction of delivered demand (FDD), and the fraction of delivered quality (FDQ). The *FDV* is defined as $FDV = \sum V_j/V_T$ where V_i is the volume supplied to consumers at each node and V_T is the total required volume of demand. The *FDD* is defined as $FDD = \sum t_j/T$ where t_j is the total duration at the node j when the demand supplied is above the demand factor which is a percentage of the required demand at a node predefined by the user, and T is the duration of simulation. And, The *FDQ* is defined as $FDQ = \sum tq_j/T$ where tq_j is the total duration at node j for which the concentration is below the threshold concentration factor. They concluded that FDD tends to be greater or equal to FDV, and FDQ depends on the water quality distribution at the sources, and the consumers' water quality demands. Gomes and Karney (2005) presented a methodology to evaluate the WDS reliability under a fire condition. They denoted the reliability index as

$$R_i = \sum_{j=1}^{n=node} \left[1 - \Pr(P_i \le P\lim_i \,\middle|\, fire\ node_j) \right] \times \Pr_N(fire\ node_j) \tag{2.15}$$

where P_i is pressure at node i; $P\lim_i$ is pressure limit for node i; $Pr_N(fire\ node_j)$ is the relative frequency of a fire condition at node j. The results illustrated that the relative frequencies of fire events and the nodal position in the network are two key factors in the reliability of WDSs.

Reliability analysis has been addressed for the design of WDSs. In the reliability based optimal design of WDSs, the reliability index is generally incorporated into the optimization approach as a constraint to maximize the system reliability (Babayan et al. 2005; Kapelan et al. 2005; Atkinson et al. 2014). Many useful researches have been conducted on least-cost design of networks considering reliability to compare network costs against different levels of reliability (Xu and Goulter 1999; Tolson et al. 2004;

Babayan et al. 2005; Farmani and Butler 2013). The general probabilistic algorithm cited in the literature for optimization model is as follows

Minimize:

$$f(D_1, D_2, \ldots D_n) = \sum_{i=1}^{n} C(D_i, L_i), \qquad (2.16)$$

Subject to:

$$Q_{in} - Q_{out} = Q_D \qquad \text{for all nodes} \qquad (2.17)$$

$$\sum h_f - \sum E = 0 \qquad \text{for all loops} \qquad (2.18)$$

$$P(h \geq h_{\min}) \geq R_l \qquad \text{for all nodes} \qquad (2.19)$$

where $C(D_i, L_i)$ is the objective cost function which is generally a nonlinear function of diameter D_i and length L_i of the pipes in the network. N is the number of pipes, Q_{in}, Q_{out}, and Q_D are respectively pipe flow into the node, pipe flow exiting the node, and existing and future design demands, h_f is the head loss across a pipe as modeled by Hazen-Williams or Darcy-Weisbach equation, E is the energy added to the water by pumps, h and h_{min} are the pressure and MPC, respectively, and R_l is a predefined reliability level. The probabilistic constraint in Eq.(2.19) states that the proportion of time pressure head h at each node, is at or above the MPC must exceed some reliability level R_l.

No research discusses how the reliability index and network costs are linked to uncertain parameters. In other words, no research has been conducted to show how the system performance and the network reliability can be improved in terms of changes in water tank volume, pumping capacity, or changes in pipe diameters by considering uncertain design parameters of WDSs. Which one of the improvements is more feasible is something that has not been investigated either.

2.9 Summary

Despite the key role that a WDS plays in providing service to the public, few and different standards exist for design and evaluation of WDSs performance. The pressure standard identified is a fairly common requirement for minimum water pressure under fire flow conditions and there is no universally accepted standards for monitoring, measuring, and assessing the performance of WDSs. Undesirable pressures have a link to the problematic issues of WDSs, e.g., leakage, pipe breaks, high energy use, and contaminant intrusions. Thus, most researches have focused on quantifying the impacts of low and high pressures in WDSs and few studies have been conducted on controlling the pressure in the system.

Although it is widely accepted that there is a link between pressure, leakage, and the system energy use, exact understanding of these connections in WDSs still remains elusive. Nonetheless, progress has been made on this front. Pressure management is traditionally performed as an effective way of reducing the excess pressures and leakages in a system. To control the high pressure during low demand conditions, pressure reducing valves are generally installed in the pipe networks but the optimization of the number of valves, their locations, and the determination of the optimal opening adjustment are all challenging tasks. Also, in a pressure management process, the energy supplied does not change and it may still be high enough to waste both energy and money. Some studies have conducted to determine the leakage-pressure relationship for accurately modelling the behavior of leaks in order to quantify the effect of pressure modifications. Several researchers have tackled the issue of energy use by considering the optimisation of pump selection, pumping schedules, and tank operations. Mathematical programming and simulation are often employed to solve these multiconstraint problems. Despite such an implicit awareness to improve the performance of WDSs, the link to the pressure has usually received only passing mention or cursory treatment (if addressed at all).

Pipe breaks are an inevitable aspect of WDSs management and cause significant water losses. Several models for predicting breakage rates of water mains have been developed to show break behavior and break patterns. These previous studies relied on field data and have not focused on the relationship between pressure and pipe breaks. Only very few studies have been conducted on the relationship between pressure and pipe breaks from limited field data (due to historical lack of awareness at water utilities of the importance of collecting such data). Moreover, the pressure, which is an important factor of WDS design and planning, is not certain either. To address uncertainty in WDS design and planning, several models have been developed to improve the performance of WDSs at minimum costs and to achieve a high reliability level. However, these previous studies have focused only on the hydraulic performance of systems, e.g., to increase the probability of supplying water demand above a predefined MPC at minimum network costs. Few efforts have been made to explore how pipe break rate might change, if the system pressure changes in a WDS considering uncertain design parameters, e.g., water demands and pipe's friction factor.

To assure safely design and operation of WDSs, pressures across the network is considered to be above an MPC. But this criterion is almost always temporarily violated under transient conditions. All studies in the area of transient analysis have been conducted to show how to control transient pressures using surge control strategies in order to maintain pressures within a range whereby the maximum pressure does not exceed the maximum permissible pressure of pipes and the minimum pressure is not negative. Thus, a transient-MPC is different from that of steady state conditions. Considerable thoughts need to be given to what standards mean, how to design for their satisfaction, and what constitutes violations. The primary objective of this thesis is to fill this gap in the literature and draw attention to this important aspect of pressure standards.

Since the inter-relationship between pressure and problematic issues of WDSs appears not to have been developed or scrutinized in the literature, this thesis aims to identify and evaluate the benefits that might be gained and the tensions that are created by changes in pressure standards. In particular, chapter 3 points out the ambiguity in applied pressure standards in WDSs design and highlights some questions that are not completely covered by these criteria. In chapter 4, all the consequences of pressure change in WDSs are examined. In chapter 5, the special relationship between pressure, leakage, and energy use is analytically examined for single leaky pipes and numerically simulated for hypothetical networks to indicate how reduction in the supplied energy affects the performance of WDSs. An approximate probabilistic method, no known equivalent of which has been found in the literature, is developed in chapter 6 to determine the expected pipe break rates in order to quantify the relationship between pressures and break rates in WDSs. The role of the delivery pressures and their effectiveness on the system response under transient conditions is examined in chapter 7 in order to highlight that MPC is not really properly considered as what it really means in the literature, particularly when taking into account transient conditions. To safely operate WDSs during fire flow conditions, a surge limit control algorithm is developed in chapter 8 in order to control the down-surge pressures, a novel idea that has never been paid any attention so far.

Chapter 3

Pressure Standards in Water Distribution Systems: A Reflection on Current Practice with Consideration of Some Unresolved Issues

This chapter is based on the paper entitled “Pressure Standards in Water Distribution Systems: A Reflection on Current Practice with Consideration of Some Unresolved Issues.” by Vali Ghorbanian, Bryan Karney, and Yiping Guo; which is accepted for publication by the *Journal of Water Resources Planning and Management*.

Pressure standards assist in the design of water distribution systems and the assessment of their performance. Although exact thresholds are sometimes rather vague, unusually high and low pressures are widely understood to increase costs and put systems at risk from events like pipe bursts at the pressure high end to the risk of contaminant intrusion or poor fire-fighting conditions at the low pressure end. Interestingly, since the definition of what conditions constitute acceptable pressures differ around the world, a delivery pressure might be considered acceptable in some regions and unacceptable in others. But, if a wider range of system conditions is considered, including transient events, an interesting question arises, as to what exactly the standards might mean and how violations should be evaluated. Specifically, what kinds of pressure transgressions are most crucial to system performance and economics and what are merely inconvenient? Certainly the issue of evaluating consequences is relevant, but also complex, since some consequences are not easily attributed to specific system conditions. As the chapter considers, the issue of the frequency, duration and intensity of the pressure violation are all relevant, but so is the vulnerability of the system to those violations. Few of these issues have yet received adequate attention, but it is to raising such a discussion that this chapter is aimed. There is no doubt that pressure standards can help to assess system performance and to trigger system evolution (i.e., operational and capital investments). But if the criteria themselves, and the means to evaluate them, are too vague, so will be the corrective outcomes.

3.1 Introduction

The aim of a water distribution system (WDS) is to safely deliver adequate quantities of drinking water to end users under sufficient pressures to permit or facilitate a wide range of human endeavors. In design, the system pressure is generally expected to be maintained between minimum and maximum standards for safe, reliable and economic operation. These standards were generally set to be both reasonable and economic measures, with the goal of specifying a moderate range that would generally prevent intrusion or ingress of contaminants, limit consumer complaints, avoid damage due to inadequate fire protection, avoid problems in reservoir operation, and reduce problems with secondary pump operation (e.g., poor suction conditions). Pressure standards also aim to reduce excess demands, e.g., through reduced flow rates from faucets, showers, and lawn watering, the frequency of pipe breaks, leakage rates, and excess energy use during high pressures.

However, the variation in standards among countries shows there is room for debate between the tensions that too high and low a pressure create and the benefits achieved through the pressure standards themselves. But revising (or clarifying) standards is never an easy task. Residential and industrial equipments have been installed and infrastructure has been constructed based on existing standards and their interpretation. Thus, modifications to the service level, including pressure and pricing, are a multi-constraint decision which must consider wide stakeholder opinions and sufficient time for adaptation.

Though often neglected, transient pressures also influence both the performance of WDSs and the interpretation of what pressure standards might actually mean. Such pressure surges occur whenever flow conditions are altered in the network. Transient events are generally most severe when rapid and coordinated flow changes occur, such as those associated with the power failure of a pump or rapid valve and hydrant operations. Transient events are generally characterized by fluctuating pressures and velocities and can be high enough to break or damage pipes or equipment, while transient low pressures can disrupt delivery conditions. Certainly an effective practice to reduce risk from intrusion of contaminants is to always maintain distribution system pressures higher than external pressures including transient events, but many utilities do not collect or submit pressure monitoring data or records of low pressure events for regulatory compliance (LeChevallier et al. 2011; Kirmeyer et al. 2001). LeChevallier et al. (2011) reported that no significant relationship was observed between pressure and the monitored water quality parameters of free chlorine residual, conductivity, pH, and temperature at all monitoring locations of a system serving a hilly terrain (elevations range from 169 to 521 m); however the system had several areas of frequent pipe breaks. Other cross connection control and backflow prevention programs might efficiently reduce the risk of infection rather than increasing pressure in the system (AWWA 2004).

In design, WDSs are also required to deliver large fire flows at adequate pressures. From transient perspective, the designer must provide a system that can establish fire flows as quickly as practical. Yet a rapid hydrant opening can easily generate a transient low pressure event in the system particularly if fire crews receive little specific instruction on opening a fire hydrant. Hence, one critical but too-often-forgotten issue that is raised here is how to determine whether transient pressures violate pressure standards.

This chapter highlights the ambiguity of pressure standards relating to WDSs design and raises some key questions that are only currently incompletely addressed by the published criteria. A section discusses how to better interpret the available pressure standards, where they succeed the best, and where a revision might be helpful. Some useful metrics are used to evaluate the violation of minimum pressure criterion (MPC) under transient events. Finally, the consequences of a reduction in a system's minimum operational pressure are briefly explained. This chapter addresses some issues associated with pressure criteria most applicable to developed countries in which continuity of water supply is generally taken for granted. Certainly, different design approaches must clearly be adopted for use with intermittent supply systems.

3.2 Why are pressure standards required?

At the first glance, several obvious but rather crucial questions might be raised: Why do we need pressure standards? In what ways do they help to deliver continuous and safe water to customers? How do pressure standards influence the methods for the evaluation of WDSs? One point is immediately obvious: WDSs design process is a challenging task. There are countless decisions to make, i.e., pipes and their sizes, materials, pressure classes, the pumps and their various capacities, the size and location of reservoirs, the types and location of valves, the whole range of monitoring equipment, and the pressure district boundaries. Thus, by setting pressure standards one rapidly establishes a benchmark for a reasonably cost effective and efficient design. Walski (1985) pointed out that existing standards are performance standards (i.e., to evaluate the performance of a system) rather than design standards which states that how a system should be built. High and low pressures are both problematic and undesired in WDSs. High pressure systems may cause more frequent pipe breaks and an increase in energy use and leakage (Lambert 2000). Low pressure systems lead to consumer complaints and they make systems more susceptible to negative pressures and possibly contaminant intrusions during transient events (Friedman et al. 2004). The overall goal of establishing a pressure criterion is to balance these opposing tendencies to ensure that reasonably safe, reliable and economic operation of WDSs is almost always achieved.

Pressure standards are intended to help to monitor and to assess system performance. By a thoughtful and well executed monitoring program, utility managers can determine how WDSs performance compares to established standards and how the system is evolving over time. The overall adequacy of WDSs is clearly to be measured in terms of how well the customers are served. Hydraulic performance measures relate to the delivery of an adequate supply of water and are usually measured in terms of pressure and flow parameters. The desired pressure generally is a “medium” pressure to be between the high and low limits set for the system. Industrial equipment and residential appliances are designed for specific pressure ranges. A minimum pressure is also clearly required to supply adequate water from faucets and shower heads for customer satisfaction. The failure to meet these operational standards can cause customer dissatisfaction and complaints. Yet, what equipment should be used to monitor the pressure and how is the outcome of that monitoring to be interpreted? We return to these surprisingly vexing questions later. At the moment, it is perhaps enough to say that it is clearly not sufficient to a glance at a some convenient pressure gauge or even to directly use the output from a Supervisory Control and Data Acquisition (SCADA) system. It is not uncommon that pressures drop below MPC in WDSs and this violation can only occasionally be captured through existing monitoring programs in most utilities (LeChevallier 2014b).

A key performance requirement as part of the pressure standard is the maintenance of a minimum residual pressure during fire flows. A minimum pressure is required to overcome friction losses at the hydrant and in the suction and delivery hoses so that adequate pressure is provided for supplying the required fire flow. In most US states and Canadian provinces, governments are responsible for building codes and fire prevention regulations and these regulations are generally enforced by the local fire marshals or fire chiefs with the added weight of insurance provisions. But, in most European countries, water companies and fire authorities are jointly responsible to provide water for firefighting. It is historically accepted that a fire can be extinguished by spreading water on it which can be achieved with the use of pressurised water. To provide required fire flow, the pipes of distribution systems should be sized to deliver the required flow rates at the desired pressure. But in the absence of a fire, flow rates and velocities are often much smaller, leading to larger pipes that can substantially increase the residence time of water in the distribution system leading to degradation of water quality

(Snyder et al. 2002). Fire flow requirements can induce powerful and not always well understood stresses on system design and performance.

3.3 Current pressure standards

There are no universally acceptable or established rules or guidelines for the specification of pressure standards for WDSs design. Table 3.1 shows some of the examples for pressure standards applied in WDSs design around the world. This table clearly shows that there are some inconsistencies and variations in acceptable standards in terms of the required pressure and the conditions that the minimum standard is recommended to enforce in design. The minimum pressure criterion (MPC) is technically defined as the required pressure above which there is no deficiency in system performance. In some guidelines, the minimum pressure standard has been specified for supplying a minimum required flow, e.g., in the UK and Wales, a flow of 9 litres/min at a minimum pressure of 10 m (14 psi) is required at each point in WDSs as the design standard (Hayuti et al. 2006). The lack of globally accepted regulations has led water utilities to develop their own criteria for design and operation of distribution systems [e.g., the primary MPC in use in most US states is 14 m (20 psi) during fire flow or emergency conditions (Ten State Standards 2007)]. However, the tentative guidelines developed by local utilities tend to focus on specific system elements, e.g., enforcing a pressure criterion to supply fire flow requirement, rather than on overall distribution system performance such as water quality, pipe breaks, leaks, and system operating pressures.

According to Table 3.1, the minimum pressure of 14 m (20 psi) is an acceptable MPC in several regions. The principal reason of enforcing a 14 m MPC may be in order to provide a minimum flow and to overcome friction losses in the customer's service branch, meter, and house piping at the second story level of a house (Walski 1985). But the standards do not specify whether this pressure criterion should be met at the elevation of the pipe, at the elevation of the ground, or at the first floor of the customer. The utilities appear unanimous in their belief that an evaluation of distribution system performance must reflect the level of service received by the customers. But a modern

Table 3.1. Pressure standards based on review of guidelines and regulations (m (psi))

Region		Minimum pressure				Maximum pressure
		Condition				
		During fire flow	During maximum hourly demand	During normal conditions	During all conditions	
Canada	British Columbia	14 (20)	28 (40)	-	-	70 (100)
	Alberta	15 (22)	35 (50)	-	-	56 (80)
	Saskatchewan	14 (20)	35 (50)	-	-	70 (100)
	Halifax	15 (22)	28 (40)	-	-	63 (90)
	Manitoba	14 (20)	21 (30)	-	-	Not specified
	Other provinces	14 (20)	-	28 (40)	-	70 (100)
USA	Louisiana	-	-	-	10.5 (15)	Not specified
	Connecticut, Oklahoma, & Delaware	-	-	-	17.5 (25)	
	Michigan	14 (20)	-	24.5 (35)	-	
	Other states	-	-	-	14 (20)	
UK and Wales		-	-	-	10 (14)	Not specified
Brazil		-	-	-	15 (22)	
Australia		20 (29)	-	-	-	
New Zealand		10 (14)	25 (36)	-	-	
South Africa		-	-	-	24 (34)	
Netherlands		-	-	-	20 (29)	
Hong Kong		-	-	-	20 (29)	

question of some import is whether the fire standard must be met continuously even when no fire is being fought in the system? Is response to a fire in the time of emergency sufficient and equivalently reliable?

As is clear from Table 3.1, there is no universally agreed on value that specifies the maximum acceptable pressure in WDS design and operation. In practice, such values are usually constrained by pipe considerations, such as working pressure and pipe rating concerns. But the design standard requires that water mains be designed to withstand total forces (i.e., static and transient pressures) acting on pipelines. The maximum allowable transient pressure cited in different national and international codes and standards is up to 1.5 times the design pressure (Pothof and Karney 2012). Design pressure is normally defined as the pressure of the system during normal operation. Moreover, wide ranges of acceptable MPC imply that water delivered under the same pressure might be acceptable in some countries while unacceptable in others. Hence, water distribution costs (both capital and operating) to meet the same flow requirements inevitably vary from region to region even for the same or similar system topology and conditions.

Beyond regulatory requirements, Friedman et al. (2010) recommended five pressure performance goals (i.e., above 0 m during emergencies such as main breaks and power outages, above 14 m under maximum day demand and fire flow conditions, above 25 m under normal conditions, less than 70 m under normal conditions, within ±7 m of average pressure in greater than 95% of the time) in order to optimize WDSs in terms of reducing unnecessary water losses, main breaks, and/or energy usage. The pressure criteria are invoked by designer and operators to help size distribution mains and services that are used for the final stage of delivering water to the end customers; yet these criteria are not always applied to the transmission mains that convey larger amounts of water over greater distances, typically between major facilities within the system (Walski et al. 2007). In distribution systems, it is not uncommon for the pressure to be relatively low (less than 14 m or 20 psi) at locations close to ground tanks, whereas the discharge headers from pump stations often experience high pressure. Yet such departures are usually tolerated since there are seldom service connections close to these two critical points.

For adequacy, pressure related performance could be measured as how often operating pressures are above the MPC. In a risk-based optimization model to determine optimal design of WDSs, the overall WDS robustness is defined as the probability of satisfying minimum pressure head constraints at all nodes in the network (Kapelan et al. 2006; Yannopoulos and Spiliotis 2013). Future required demands, roughness coefficients of pipes, human behavior to operate WDSs, the estimation methods to determine needed fire flow, and demand patterns for residential, commercial and industrial sectors are all subject to uncertainty, thus the computed/measured minimum pressure which are the real concern of insurance companies and WDSs' reliability, is not certain either. Nor can any monitoring program of current conditions assure that the required performance will be achieved when needed. Many utilities, even those with online monitoring, only measure pressures once every few minutes. Therefore, there are a lot of uncertainty about the specifics of pressure monitoring and management (LeChevallier et al. 2014b). Pressures may well be below the MPC in some circumstances due to the upset of uncertain parameters in design and operation of WDSs. Therefore, the enforced/ensured pressure criteria cannot continuously guarantee the adequacy and availability of the required water and the required pressure to all consumers.

The required fire flows are described in fire codes published by insurance companies or other oversight jurisdictions. These typically specify the so-called needed fire flow (NFF) which is the rate of flow considered necessary for suppressing a major fire within a specific building. Based specifically on AWWA's M31 Manual (AWWA 2008), the required fire flow duration is 2 hours if NFF is equal to or less than 158 L/s (2500 gpm) and it is 3 hours if NFF is equal to 189-221 L/s (3000-3500 gpm). Specific properties with a NFF in excess of 221 l /s (3500 gpm) are evaluated separately and assigned an individual classification (ISO 2012). In North America, the minimum pressure standard is often recommended by insurance companies that are concerned not so much with human comfort but with the risk of fires. Insurance companies require that a certain fire flow rate (e.g., NFF) be met under a specific MPC, which is often 14 m (20 psi), measured as the residual pressure at the discharge point. Insurance companies do not provide engineering advice on water supply improvements, they just provide guidelines on water demand for any new community development and the required pressure during fire flows for evaluating water supply systems in order to rate them (ISO 2012). If the pressure provided by water supply is too low, customers have to pay more for home insurance premium (ISO 2012). Even in the absence of a fire, the conventional design approach requires that the minimum standard be met assuming the possible occurrence of fires. This obligation to provide fire protection substantially affects WDSs design and operation (Snyder et al. 2002).

Regardless of the efforts that have been made to provide secured fire flow under the established MPC, this criterion is almost certainly temporally violated during the transient event associated with the initial opening of a hydrant and in power outage conditions (Ghorbanian et al. 2015a; LeChevallier et al. 2011). Moreover, the fire flow requirement may be supplied under a pressure less than the MPC even in steady state conditions since the hydrant outflow is also controlled by the hydrant's outlet nozzle diameter. That is, there is an important pressure-flow relationship that is established partly for convenience, partly by convention, and partly because of necessity. The pressure-flow relationships also show some variation over the world. Therefore, the specific reasoning for establishing a particular is somehow ambiguous around the world. Different residual pressure with the same outlet size of hydrants is assumed to provide the same flow rate, this is not acceptable in terms of hydraulic calculation.

Fire prevention is a philosophy of the selection of equipments, materials, and processes that will eliminate or lower the risk of a fire. However, regulations do vary in the requirements for methods of operating or installation in Europe and North America. In Europe, a different set of regulations for fire code exists which may determine that quick opening of a hydrant is not required. This contrasts with the regulations in North America. For instance, in the U.K. many data centers use either a gaseous or mist system for fire protection in data halls whereas in North America it is more common to use pipe water systems (Elliott 2006). North American codes tend to provide prescriptive solutions that favor active fire suppression while European codes tend to provide performance requirements that favor passive fire protection. In North America, it is common to use automatic sprinkler systems to control fire in non-residential buildings and much less water is used to extinguish fire (Hickey 2008; AWWA 2008). For a building protected by automatic sprinklers, the NFF is that needed for the sprinkler system, converted to 14 m (20 psi) residual pressure, with a minimum of 32 l /s (500 gpm) (AWWA 2008). But the installation of automatic sprinkler systems transfers a significant cost – from the installation of automatic sprinkler systems, to their maintenance, and periodic testing – to the private sector (Hickey 2008). There are economic incentives in the form of insurance premium reductions for commercial property owners with installed and properly maintained sprinkler systems (Hickey 2008). Even here, of course, an alternative exists. For example, with the use of foam to help extinguish fires, less water is indeed, reducing the water demand associated with firefighting (Cote and Linville 1986). Hence, collecting and sharing data on processes of extinguishing fire seem to be extremely useful.

From the above discussion, it is clear that pressure criteria and standards can be evaluated from a variety of overlapping but sometimes distinct points of view, representing the perspectives of regulatory agencies, health and environmental agencies, water utilities, fire departments, and customers. This number of participants is perhaps a reflection of how universal water uses are and how many people have an interest in the water supply system. Such complexity is further intensified by the reality that systems, standards and operation are constantly evolving in time.

3.4 Complying with a pressure standard

In conventional design approaches, all components should be sized to comply with the regulations. While an MPC is enforced in the design of WDSs to ensure supplying adequate demands during periods of peak consumptions, e.g., the greater of the maximum hour demand and the maximum day demand plus fire flow, many systems experience higher pressures than necessary during off-peak demand periods. This is so much so that, in certain instances, customers might need to install pressure reducing valves in their houses. Excessive pressure can also be controlled by regional pressure management that is now recognised as one of the most efficient and cost effective strategies to reduce pressure, burst and leakage rates (Ulanicki et al. 2008; Gomes et al. 2011). Additional benefits might be gained by including specific strategies that decrease the energy supplied. Current regulations and guidelines indicate that pressure as a measure of performance should be based at least on MPC; however, consideration of both maximum and variations of pressure is also necessary, though seldom stipulated, to reduce system costs and the risk of failure.

The unquestioned supposition is usually that if certain design standards are adopted, then the network will provide pressures at or above the required minimum during peak demands, and therefore, the probability of hydraulic failure will be highly unlikely. Conceptually this is simple but what does this mean in practice? If one installs a pressure gauge in the system, pressures will be seen to vary; if the pressure gauge is more sensitive and read more often, pressures typically vary much more. It may be easy to dismiss certainly momentarily transgressions of the pressure standard, but a judgement call is already needed to estimate the consequences/significance of the violation. This is seldom easy. Is small violation allowed every hour, every day or every week? How much of violation is considered to be small? Do the duration as well as the frequency and magnitude of the variation matter? Are all parts of the system equally vulnerable to the same degree of transgression? Should this be prioritized based on the importance of the system component, or its material (e.g., flexible vs. rigid pipe wall), its age or perhaps its failure history? One might turn pressure data into a kind of a pressure-duration-frequency curve with the goal of assessing to how often, how intense and how long low or high pressures actually occur. But what would one do with such a curve?

The continuous monitoring of water quality, hydraulics, and system pressure is typically undertaken with up-to-date SCADA systems. Data are centrally archived and used for infrastructure management and system evaluation. However, typical SCADA systems seldom have sufficient temporal resolution to resolve transient pressures, and the full analysis of the dynamic data is often inaccessible (LeChevallier et al. 2014b). Currently, many utilities collect data on pressures at key locations in a network (e.g., pumping stations and boundary of pressure zones) with low-resolution SCADA data. Yet, in the light of all these fluctuations, utilities may wish to reassess how the data collected by SCADA systems already in place is used in the future. A variety of metrics might be considered in the light of such a task.

If the pressure delivered to an area changes, whether as a reflection of a new standard or a new operational approach, new hydraulic grade lines are established. Therefore, before existing pressure zones are realigned by changing pumps or adjusting pump settings, thoughtful public notification and consultation is essential. Moreover, the feasibility to implement pressure changes is system specific and requires a detailed engineering study (LeChevallier et al. 2014b). To create new pressure zones, topography and customer acceptance of new pressure are often the limiting factors (Walski et al. 2007).

3.5 How to evaluate MPC violation in transient events

Rapid flow changes during transient events (e.g., valve closure or pump switching) cause pressure fluctuations in a WDS. Pressure fluctuations have many implications in the light of pressure standards; in particular, some transient events would certainly cause the MPC to be violated. The undesired transient pressures (i.e., too high pressures and negative pressures) are controlled by surge control strategies (Boulos et al. 2005). But to plan and deploy the transient mitigation, the aim is to avoid negative pressures rather than pressures below the MPC (LeChevallier 2011). Technically, air release/vacuum breaker valves are placed at locally high elevations where the system is more susceptible to negative pressures under transient events. These valves admit air into the system in order to maintain local pressure near the atmospheric (0 m) pressure. However, valve vaults can be flooded and contaminant intrusion is possible through valves during low pressure transients if they are not well maintained (Ebacher et al. 2013). Indeed, the associated transient data obtained from actual system data often supports that reality that pressure standards are not continually met. AWWA recommends installation of air valves at intervals along ascending, descending, and horizontal lines (AWWA 2001). But this may be a conservative approach, and it is seldom specified how critical each location along a pipe profile is nor the consequences of poor sizing (Ramezani et al. 2015). Proper sizing and positioning of air release/vacuum breaker valves can help to reduce or limit contaminant intrusion.

Figures 3.1 and 3.2 clearly indicate the MPC violation during a transient event. For simplicity, the centerline of the pipe is set at 0 m. The reservoir water level, H_0, is set at 58 m. Transient condition can be introduced into this case by a sudden valve closure (in 1 s) at the most downstream end of the pipe (Figure.3.1). Clearly from Figure 3.2,

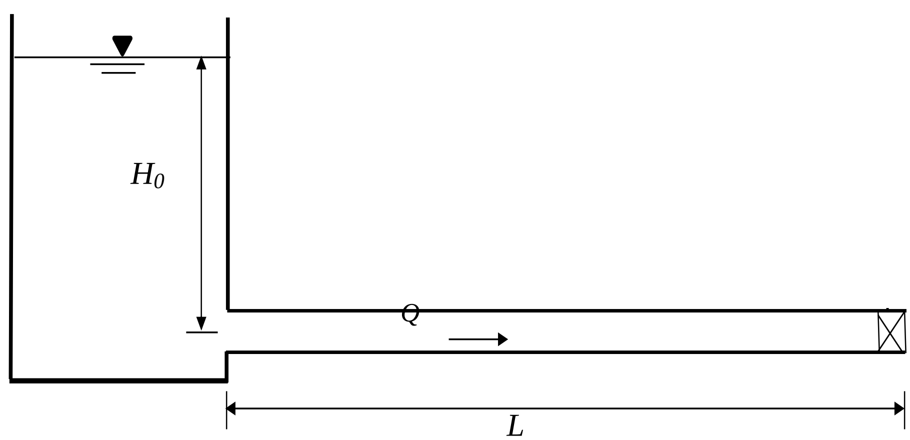


Figure 3.1. Simple system configuration (flow rate Q = 0.5 m^3/s, length L= 4000 m, pipe diameter D = 1 m, Darcy-Weisbach friction factor f = 0.015, and wave speed a = 1000 m/s)

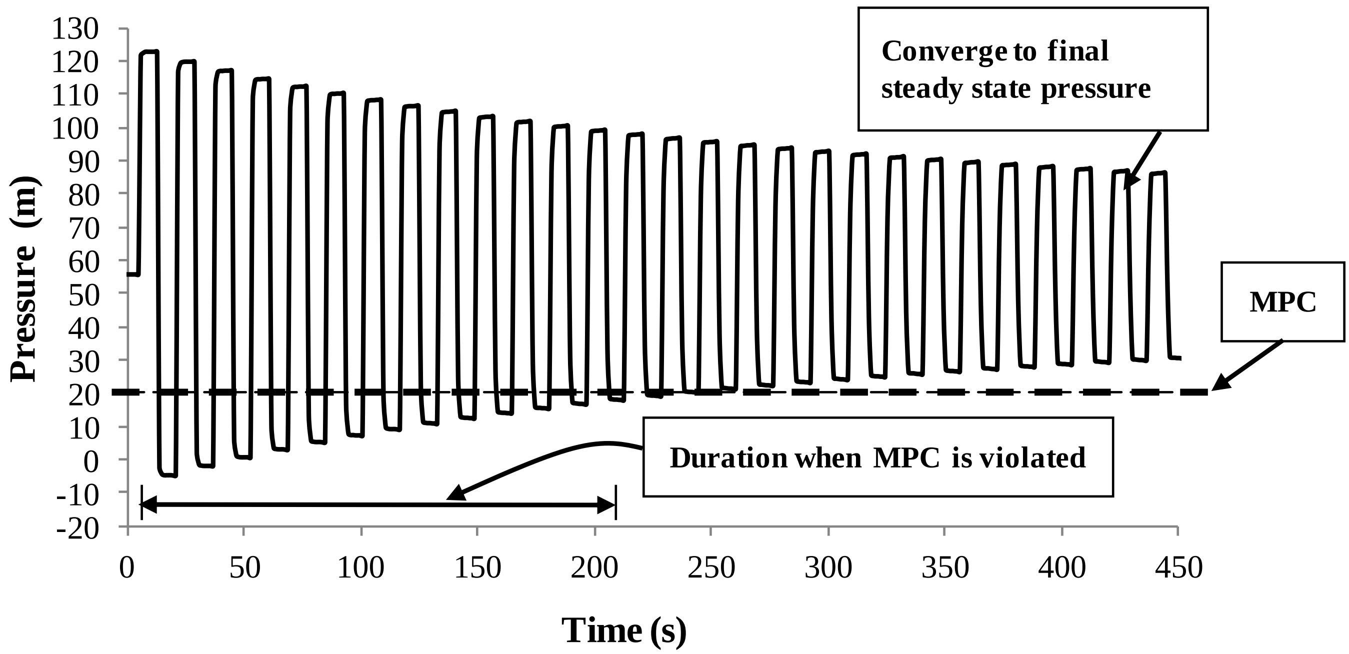


Figure 3.2. Minimum and maximum transient pressure waves

pressure fluctuations (a sequence of transient waves) during transient events often violate the regulation of minimum standard for water pressure (the MPC is considered to be 20 m in the system). Results in Figure 3.2 raise in more concrete terms the previously unanswered questions: what does it mean to achieve (or violate) the standards? How often is the standard transgressed, by how much, and for how long? Is a 1 s or 30 s violation serious? Do frequency and severity of violation matter? Should the standard be set for transient events? What does a pressure standard mean if transients are considered? In the background, are other perhaps even more subtle questions. The kind of response shown here is typical of a numerical model using so called quasi-steady friction approximations; in other words, it neglects unsteady friction effects which would typically cause the transient train to decay more rapidly. How good does a model need to be to assess system performance and by what measure? Field data would appear to be better but gathering such data still has challenges including the frequency of data collection and a host of measurement errors that can complicate the interpretation, not to mention the danger of experimenting on real systems with potentially severe events.

One immediate but vexing question any analyst faces is the choice of suitable metrics to evaluate the severity of transient events? Little thought or reflection has historically been given to this important question. Of course, such metrics should evaluate the desired performance of WDSs. Table 3.2 indicates some of the metrics used to quantify the severity of transient events. Clearly from Table 3.2, all metrics are associated with maximum and minimum transient pressures occurring in the system and very few indices have been defined to quantify the severity of negative pressures and almost none of them consider the transgression of MPC. These considerations highlight the ambiguity in using pressure standards and the key question of whether a certain pressure sequence is acceptable or unacceptable. For example, can pressures fall below the MPC for a mere second? Clearly many things are at stake including intensity and frequency. For example, an extreme negative pressure of zero absolute pressure for 1 s is much more dangerous than the pressure of zero gauge pressure for 10 s. For full negative pressures (-10 m), column separation is almost assured which may give rise to sudden pressure spikes when the cavities collapse. To date, few efforts have been made to evaluate pressure criterion in transient events.

In Figure 3.3 depicting several time steps of transient response of the simple system shown in Figure 3.1, several additional and useful metrics are shown. These metrics can be derived in order to determine the severity of violation of MPC in a WDS (Eqs. 3.1-3.4). The negative pressure index T_c can be determined as

$$T_C = \sum_{i=0}^{T} t_{ci} \tag{3.1}$$

where t_{ci} is the time when pressure is negative, and T is transient event duration. The duration of violation of MPC T_m is defined as

Table 3.2. Metrics used to quantify the severity of transient pressures

Author	Index	Definitions of variables	Comments
Friedman et al. (2004)	Intrusion Potential	The total number of nodes experiencing negative pressures and the total time when those nodes experience negative pressures.	To determine severity of surge and the intrusion potential during transient events.
Jung and Karney (2006)	$H_{\max} - H_{\min}$	H_{max} and H_{min} are respectively the maximum and minimum pressures.	The goal of minimising the difference between the maximum head and minimum head during transient events.
Jung and Karney (2011)	$SPDF = \int_{i \in N_{node}} H_i dt$	H_i is the pressure at each node that is either greater than H_{max} (the maximum allowable pressure) or smaller than H_{min} (the minimum allowable pressure).	Surge damage potential factor (SPDF) to determine the likelihood of a damaging transient event.
Martin (1983)	$S = \frac{T_{SC} a}{2L}$	S is the severity of cavity index and T_{SC} is the duration when cavity occurs.	To determine the severity of cavitation during transient events.
Radulj (2009)	$TRI^{+} = \int_{0}^{T^{+}} P_{\max}$ $TRI^{-} = \int_{0}^{T^{-}} P_{\min}$	TRI^{+} and TRI^{-} are respectively positive and negative transient risk index, T^{+} and T^{-} are the maximum return period from the data set associated maximum and minimum pressures, respectively (in day).	To quantify the risk assessment associated with hydraulic transient.
Shinozuka, and Dong (2005)	$D = -\frac{H_2 - H_1}{t_2 - t_1}$	D is the damage index, and H_2 and H_1 are pressure heads at a node at time t_1 and t_2, respectively.	To locate damaged pipe or malfunctioned equipment when the water system exhibit acute transient behavior.

3Figure

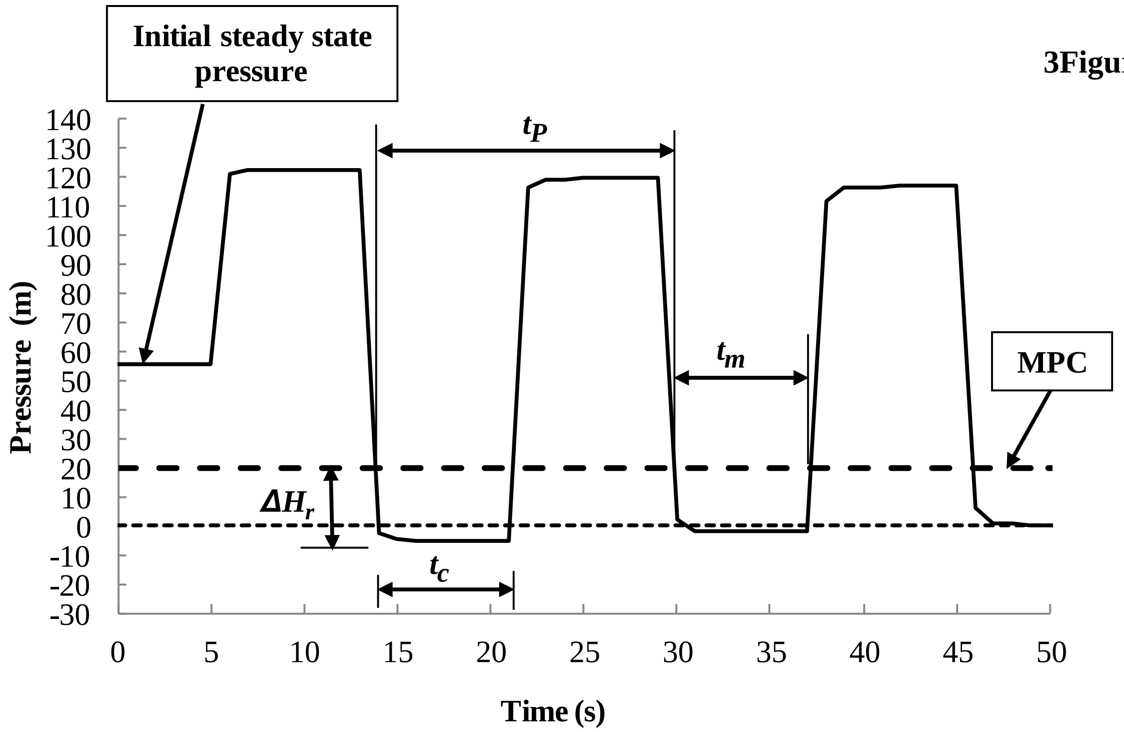


Figure 3.3. Metrics to quantify the violation of MPC

$$T_m = \sum_{i=0}^{T} t_{mi} \tag{3.2}$$

where t_{mi} is the time when pressure is below MPC. The period of violation of MPC T_p and intensity of violation of MPC I_V can be determined as

$$T_P = \frac{4L}{a} \tag{3.3}$$

$$I_V = \frac{\Delta H_r}{H_{cr}} \tag{3.4}$$

where L is the pipe's length, a is the wave speed in the pipe, H_{cr} is the MPC, $\Delta H_r = H_{cr} - H_{min}$, and H_{min} is the minimum transient pressure. Clearly from Figure 3.3, the MPC is violated every 16 s ($4l/a$) in this case study. In case of water networks, L would be the characteristic length of the network which is the sum of the pipe lengths from the source of the surge to the upstream reservoir or the energy source of the system. The duration and the number of times of violation of an MPC, during a transient event, are greater when MPC is considered to be higher. Figure 3.4 confirms this presumption. As is clear from Figure 3.4, both the number of times and the duration when the system experiences pressures less than a certain value are greater for higher MPCs. For instance, the number of times and duration at which the pressure is less than 10 m during the transient event are respectively 53 and 0.77 min while for the pressure less than 20 m, they are respectively 106 and 1.5 min.

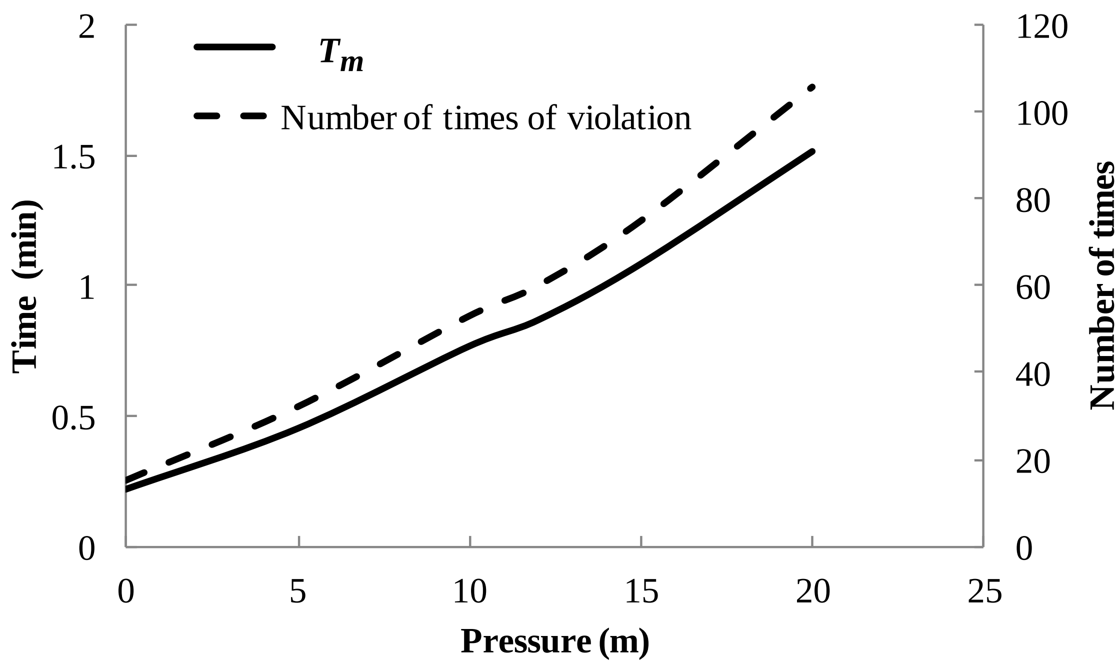


Figure 3.4. Duration and the number of times of violation of the MPC

Surge control, particularly control of high-pressure events, has typically been thought of in terms of preventing pipe bursts and efforts have been made particularly to reduce maximum pressures. Concerns regarding negative transient pressures and their public health implications have received less attention (CPWSDS 2006). Minimum transient pressure standards should be set to prevent intrusion and prevent structural problems. The consequences of low pressure failure in transient events including vacuum conditions, cavitation, and risk of contamination should be identified and it deserves particular attention. Thus, evaluation of MPC should be part of the surge analysis. The minimum allowable pressure is rarely explicitly addressed in transient conditions. The commonly accepted minimum incidental pressure in WDSs is atmospheric pressure or the maximum groundwater pressure necessary to avoid intrusion at small leaks. But, how comprehensively this transient-related MPC is achieved and scrutinized is not yet specified. Certainly other actions might sometimes be taken. For example, negative pressures can sometimes be reduced by using plastic pipes (e.g., PVC and polyethylene) in WDSs where the viscoelasticity of the pipe material significantly influences the pressure wave dissipation as well as the time-propagation (Ramos and Covas 2006).

3.6 The response of WDSs to changes in pressure standards

WDSs performance is inevitably influenced by changes in pressure standards with leakage being a case in point. Average leakage losses in water systems are reported to be around 16% but of this up to 75% is likely be recoverable (Thornton et al. 2003). Water loss control strategies (e.g., pressure management programs) have been explained in Thornton et al. (2003). Leakage rate has long been known to be related to the internal pressure of the pipe at leaky locations. Thus, lowering the pressure throughout the pipeline systems causes leakage to reduce (Lambert 2012). The impact of leaks on the energy consumption in water supply systems was examined by Colombo and Karney (2002 and 2005). They concluded that for systems with equivalent performance leaks increase operating costs in all systems and energy costs increase more than proportionately with leakage. Several relationships between leakage rate and pressure have been developed indicating leakage varies nonlinearly with pressure and can be reduced with a decrease in system pressure (Hiki 1981; Lambert 2000; and Thornton 2003). Reduction in pressure may not only decrease leakage rate but also may reduce the rate at which new leaks occur (Lambert 2000). LeChevallier et al. (2014b) reported that 24% reduction of an average pressure (by 20 m) using flow modulated pressure reduction in a case study caused reduction of about 83% in background leakage.

In water supply systems, most of the energy is consumed by pumping to provide the necessary heads and flows. A pump must supply energy to lift water from a source to the point that satisfies a MPC and to overcome the frictional head loss along the pipe to ensure that the adequate demand reaches the downstream point. If a lower value of MPC is to be considered, less power is required. Overall, the change from the higher pressure to the lower one results in a decrease in Break Horse Power (BP). The net rate of pumping energy savings is simply equal to the difference in the power requirements between the two scenarios of the MPCs. LeChevallier et al. (2014b) reported a reduced net energy input via service pressure could be achieved through adjusting pumping and decreases in dissipated energy. A case study in the US indicated that significant energy savings and improvement of distribution system energy efficiency were achieved via reducing excessive pressure at customer taps (LeChevallier et al. 2014b).

WDSs operating under high pressure are susceptible to more frequent pipe breaks. Lambert (2012) from the collected data on pressures associated with 7 Zones in an Australian Utility reported that high pressures in WDSs may cause high pipe break rates. Traditionally, pipe breaks can be prevented through active rehabilitation and replacement programs which are the most common practices of utilities. The contribution of internal pressure to pipe breaks occurring simultaneously with one or more other sources of loads (e.g., thermal loads, soil cover loads, and traffic loads) have been addressed by many authors (e.g., Kiefner and Vieth 1989; Rajani et al. 1996; Rajani and Makar 2000). Reduction in MPC influences the frequency of high pressures; thereby the probability of pipe breaks can reduce. A case study in the US showed that if the existing break frequency is high, small reductions in pressure can cause significant reductions in new break frequencies (LeChevallier et al. 2014b). To better incorporate optimization of pressure management, relationships between pressure and other distribution system performance indicators such as leakage, breaks, and energy usage should be identified (LeChevallier et al. 2014b).

A reduction in operational pressures may cause systems to become more susceptible to negative pressures and contaminant intrusions during transient events. Turning pumps on or off, opening and closing valves, and fire hydrant operations are all associated with routine actions but cause sometimes important transient conditions associated with the flow changes. To limit these pressures within an acceptable level, surge control strategies including engineering, maintenance, and operational strategies must be performed. Even WDSs that are operated under low pressures have risk of high pressure transients, but both high and low transient pressures can be efficiently controlled using surge control strategies (Ghorbanian et al. 2015a).

End users are the primary stakeholders who are influenced by low/excess water pressures. If water pressure is high at a building, it can cause both dangerous conditions (e.g., bursting heaters, boilers, piping, lime-clogged relief valves) and costly building flooding. To reduce excessive pressure, pressure reducing valves are often installed at building connections even if individual appliances are equipped with safety devices. Low pressure condition can cause customer dissatisfaction (e.g., unpleasant showering, malfunctioning dishwashers, clothes washers, and boilers). System energy is clearly wasted if water pressures are greater than the required (Ghorbanian et al. 2015b).Yet clearly a certain minimum pressure for appliance operations should be supplied.

3.7 Conclusions

Pressure standards are a foundation for safe and reliable operation of WDSs and the evaluation of WDSs performance. Design guidelines require that an MPC is maintained across the network in order to supply required fire flow during emergency condition. But, the established pressure standards are different around the world implying that water distribution costs (both capital and operating) to meet the same demand vary from region to region even for the same or similar system topology and conditions. While WDSs are often designed to maintain a minimum pressure standard in the system during peak demand periods, the system may frequently experience high pressures (i.e., in a typical day during off-peak periods) that cause the system performance to be suboptimal. Although an MPC is enforced in WDSs design, this criterion may be temporally violated during transient conditions. So, though intuitively appealing, it is not in practice a simple matter to determine if, and by how much, pressure standards are violated. Several metrics exist to evaluating the severity of transient pressures but almost none of them consider a precise definition of transient transgressions, nor the significance of such violations. In this chapter, several new metrics are introduced to quantify violations of MPC during transient events; they are appealing but it is not year clear how accurately any of them map into real system consequences for the range of conditions actually found in water delivery systems. Significantly, even the so-called fire flow requirement, often the main concern of insurance companies, may have somewhat fuzzy boundaries when the range of real conditions found in the field is considered.

Changing a pressure standard, whether by relaxation or tightening, is bound to have consequences to the design, operation and performance assessment of WDSs. Reducing the pressure may improve the WDS performance through reduced water demands and leakage, and possibly significantly decrease energy use. The probability of pipe breaks can also be reduced by lowering the pressure. Reduction in the MPC generally would cause the system to be at lower pressures, and therefore, would make the system more vulnerable to low pressures. But, the risk of low pressure can be curtailed by implementing surge control strategies and/or by implementing effective ways for backflow prevention. The required pressure for appliance operations may place a practical limit on pressure standards. Higher pressures are inevitably associated with greater energy needs.

Pressure standards are one of many utility managers concerns that also involves energy use, leakage, water quality through contaminant intrusion in transient events, pipe breaks, economic and insurance considerations, fire-fighting capabilities, and both public health agencies and individual concerns about contamination. Yet all of these considerations are tied in one way or another to issues related to the adopted pressure standard, and there is almost certainly room for much more thought and debate on these critical and fascinating interactions.

Chapter 4

Minimum Pressure Criterion in Water Distribution Systems: Challenges and Consequences

This chapter is based on the paper entitled "Minimum Pressure Criterion in Water Distribution Systems: Challenges and Consequences." by Vali Ghorbanian, Bryan Karney, and Yiping Guo presented in *EWRI 2015 Conf. Floods, Droughts, and Ecosystems: Managing Our Resources Despite Growing Demands and Diminishing Funds,* EWRI, ASCE, Austin, Texas, USA.

Criteria which stipulate the minimum pressure at which water is to be delivered to customers from a WDS differ around the world. Thus, interestingly, the pressure delivered to a customer might be judged high enough to meet standards in some countries, while water delivered under the same pressure in other countries is considered unacceptable. This chapter provides a description of consequences and implications of changes in the MPC in WDS design and operation. Reducing the MPC may cause a decrease in pressure-based demands such as faucets, showers, and lawn watering and also improve system performance through reduced energy use, leakage, and the frequency of pipe breaks. However, lowering this criterion may make the system more susceptible to low pressure failures, either hydraulic (e.g., an inability to supply the required flow) or safety related (e.g., increasing the risk of an intrusion event associated with hydraulic transients). Therefore, there should be a clear understanding of the consequences and challenges prior to changing pressure standards. Moreover, policies to control and avoid low pressure events are seldom fully linked to the value of the minimum pressure standard and the issue how the MPC is enforced/ensured in WDSs. The inter-related issues associated with MPC are raised here as important but neglected issues.

4.1 Introduction

The aim of WDSs is to safely deliver adequate quantities of drinking water to end users under sufficient pressures. In design, the system pressure is generally to be maintained between minimum and maximum acceptable levels for safe, reliable and economic operation. High pressure systems tend to cause more frequent pipe breaks and an increase in energy use and leakage (Lambert 2000). The maximum permissible pressure is determined according to pipe's strength which is related to its material, wall thickness and general condition. Low pressure systems cause consumer complaints, make the system more susceptible to negative pressures and possibly contaminant intrusions during transient events. The overall goal of establishing a MPC is to balance these opposing tendencies to ensure that safe, reliable and economic operation of WDSs is achieved. Yet, there are no universally accepted or established rules and guidelines for the establishment of MPC for WDS design.

Indeed, the available criteria for minimum pressure are quite different around the world (Table 3.1). Wide ranges of acceptable MPCs imply that water delivered under the same pressure might be acceptable in some countries while unacceptable in others. Hence, water distribution costs (both capital and operating) to meet the same flow requirements inevitably vary from region to region. The required pressure is usually specified as a MPC above which it is considered there is no deficiency in system performance. Most insurance companies are concerned not so much with human comfort but with the risk of fires. Insurance companies often require that a certain fire flow rate be met under a MPC, which is often 14 m (20 psi), measured as the residual pressure at the discharge point. However, this MPC is almost certainly temporally violated during the transient event associated with initially opening the hydrant. Moreover, the fire flow requirement may be supplied under a pressure less than the MPC even in steady state conditions since the hydrant outflow is also controlled by the hydrant’s outlet nozzle diameter. That is, there is an important pressure-flow relationship that is established partly for convenience, partly by convention and partly because of necessity. These rules also show some variation over the world.

Reduction in the MPC may cause a decrease in water demands (e.g., faucet, showers, and lawn watering), energy use, leakage, and the frequency of pipe breaks. However, lowering this criterion may make systems more susceptible to low pressure failures, either hydraulic (e.g., an inability to supply the required flow) or safety related (e.g., risks from a transient event). Designer need a clear perception of consequences and trade-offs. Certainly, if systems didn't leak as much, if pipes didn't burst as often, if backflow prevention was better managed and ensured, and if transient events were better controlled, designers could probably have even less stringent low pressure standards, and still be better off. Thus, there is a key link between pressure standards and a constellation of "infrastructure report card" issues. This chapter provides an exploration of the consequences of changes in the MPC to achieve a comprehensive picture that what the MPC really means. Some parts of this chapter – particularly the system energy use, leakage, and transient events – will need more detail that will be explained in chapters 5 and 7.

4.2 How will consumers be affected by changes in the MPC?

End users are the primary individuals who are influenced by low water pressures. To provide adequate water supply and pressure in multi-story buildings, booster pumps (often with supplemental water storage tanks) are often used to lift water for the consumers or facilities. In these cases, the target minimum water pressure provided by municipalities becomes much less relevant. However, if water pressure is reduced, pump performance can deteriorate and the local energy costs can grow. Moreover, booster pumps must then be installed at a greater number of buildings, essentially at all buildings with a height more than the minimum supplied pressure (Figure 4.1). Therefore, the owners of these buildings will be affected because they are financially responsible for the consequential costs of pressure reduction.

Nonetheless, municipalities are still responsible to supply required pressure to provide adequate demand for one-story and two-storey buildings which rarely use pumps to boost water pressure. In this case, minimum pressure should be high enough to supply water at the faucets and showers on the top floor of a house. This minimum pressure is somehow equal to the highest fixture elevation plus the head loss of interior pipes in the house as well as the required pressure at the fixture to meet water demand. The outflow rate from fixtures (e.g., faucets, showers) is often relatively small; consequently, there is rarely much head loss in interior water pipes of a house and only a small additional pressure is usually required to supply fixtures. These considerations imply that the minimum required pressure for pressure dependent appliances (such as faucets and showers) is strongly determined by the highest fixture elevation to be supplied without booster pumping.

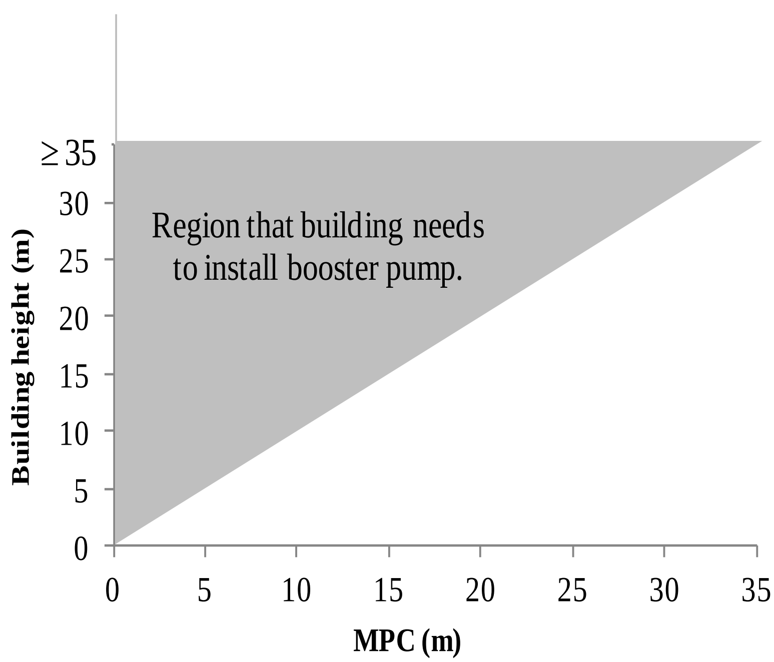


Figure. 4.1. Relationship between MPC and necessary installation of booster pumps at buildings

Volume-based demands are typical of toilets, bath tubs, clothes washers, and dishwashers. Their performance is only slightly affected by the water pressure since their functions are fulfilled when the required volume of water is delivered. Volume-based appliances are like pressure reducing valves in WDSs. These appliances reduce the water pressure to near atmospheric pressure when in use at their installed locations. Since the flow rate is related to the supplied pressure at the appliances, an increase in water pressure increases the flow rate. In pipeline systems, considering constant pipes diameter, head losses become greater as the flow rate increases. The head loss equation is expressed as

$$h_{loss} = KQ^m \tag{4.1}$$

where K is pipe resistance coefficient, Q represents pipeline flow rate, and m is an exponent. Replacing the emitter equation ($Q = CP^n$) in the Eq. (4.1) results in:

$$h_{loss} = KC^m P^{mn} \tag{4.2}$$

where P is the pressure head, n is an exponent, and C is the discharge coefficient. Figure 4.2 shows the response of relative head loss to changes in water pressure. The increase in head loss ratio is relatively linear and is approximately proportionate to the relative increase in water pressure. Clearly from the curve, as the pressure in the network becomes greater the head loss ratio increases. Since pressure head is a directly proportional to the mechanical energy that water mass possesses at the specific time and location, the energy therefore will be wasted if the water pressure is greater than the required pressure for appliance operations. Some volume-based appliances, however, require a certain minimum pressure for operation. Therefore, some end-user appliances require a minimum pressure to operate, setting another control on the minimum limit for pressure. The ranges of minimum water pressures under which appliances such as clothes washer, dishwashers, and boilers can operate are from 5 m (7 psi) to 10 m (14 psi) (Whirlpool 2000; Jacobs and Strijdom 2008; Ideal 2011).

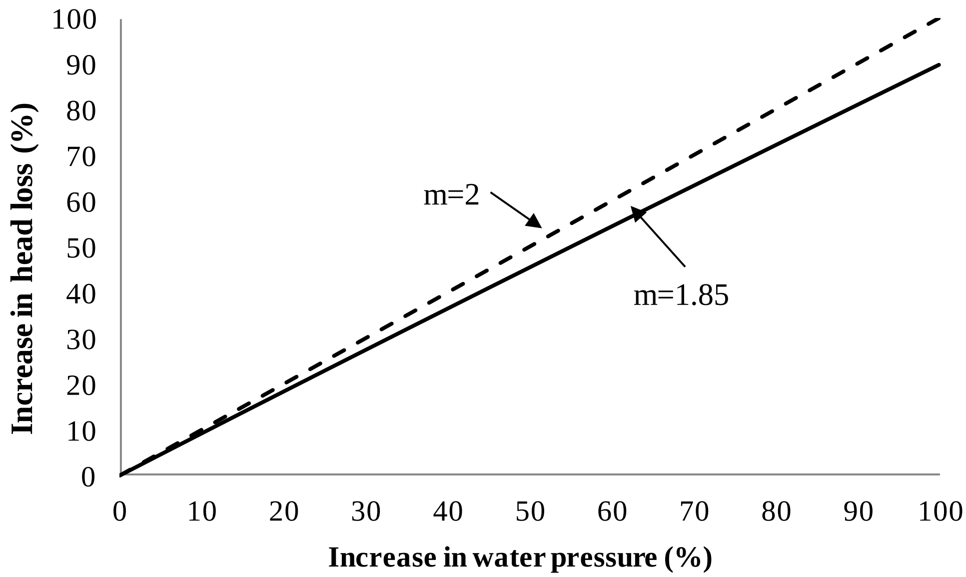


Figure 4.2. Increase in head loss ratio as a function of water pressure for discharges into the atmosphere (*n*=0.5, *m*=1.852 and *m*=2 with respectively the Hazen-Williams and the Darcy Weisbach formulas for head loss expression)

4.3 How will WDSs be affected by changes in the MPC?

A reduction in the MPC leads to a decrease in overall pressure of the system, and therefore results in improved operating conditions, a decrease in the probability of main burst and to disturbances to the public, as well as a decline in system energy consumption. The following sections discuss how a WDS is affected by changes in the MPC.

4.3.1 Energy use and MPC

To simply assess the energy effectiveness of changes in MPC, the energy use of the simple system, shown in Figure 4.3, is expressed as a dimensionless term:

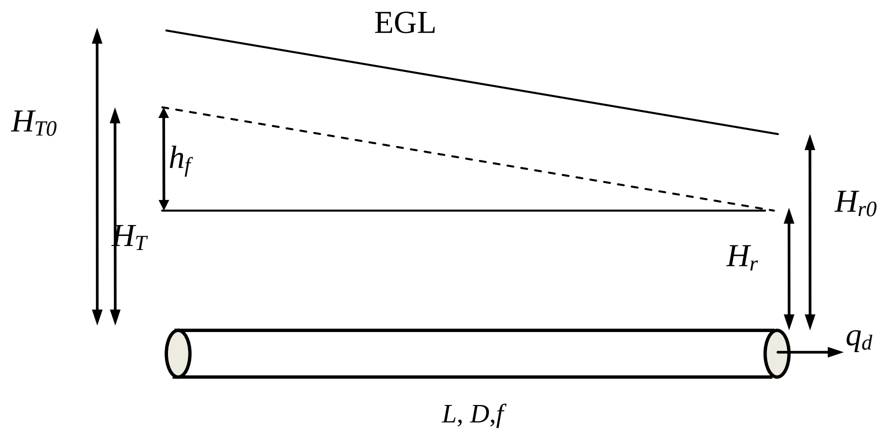


Figure 4.3. Total dynamic head in a single pipe system (L = pipe length, D = pipe diameter, f = Darcy-Weisbach friction factor, q_d = demand, and the energy is assumed to be supplied by pumps)

$$\frac{E}{E_0} = \frac{\eta_0 \gamma q_d H_T}{\eta\, \gamma q_d H_{T0}} = \frac{\eta_0 H_T}{\eta\, H_{T0}} = \frac{\eta_0}{\eta}\left(\frac{\dfrac{h_f}{H_{ro}} + \dfrac{H_r}{H_{r0}}}{\dfrac{h_f}{H_{ro}} + 1} \right) \tag{4.3}$$

where E_0 and E are the supplied energy at the source for different scenarios in which the MPCs are H_{r0} and H_r, respectively, h_f is the head loss in the pipe, η_0 and η are respectively the efficiency factors of the original and lower speed pumps, and H_{T0} and H_T is the total head supplied upstream to meet MPCs, i.e., H_{r0} and H_r, at the most downstream node, respectively. Figure 4.4 shows the response of reduction in relative energy use ($1-E/E_0$) to changes in the MPC for different values of h_f/H_{r0}. For smaller reduction in the MPC, the reduction in energy ratio changes only slightly with h_f/H_{r0}; however, as the reduction in the MPC becomes greater, the dependence upon h_f/H_{r0} is more noticeable. What is clear is that the MPC definitely influences the system energy consumption with all lines. The energy saving, as a consequence of reduction in the MPC, can be expected to be greater for low friction pipes.

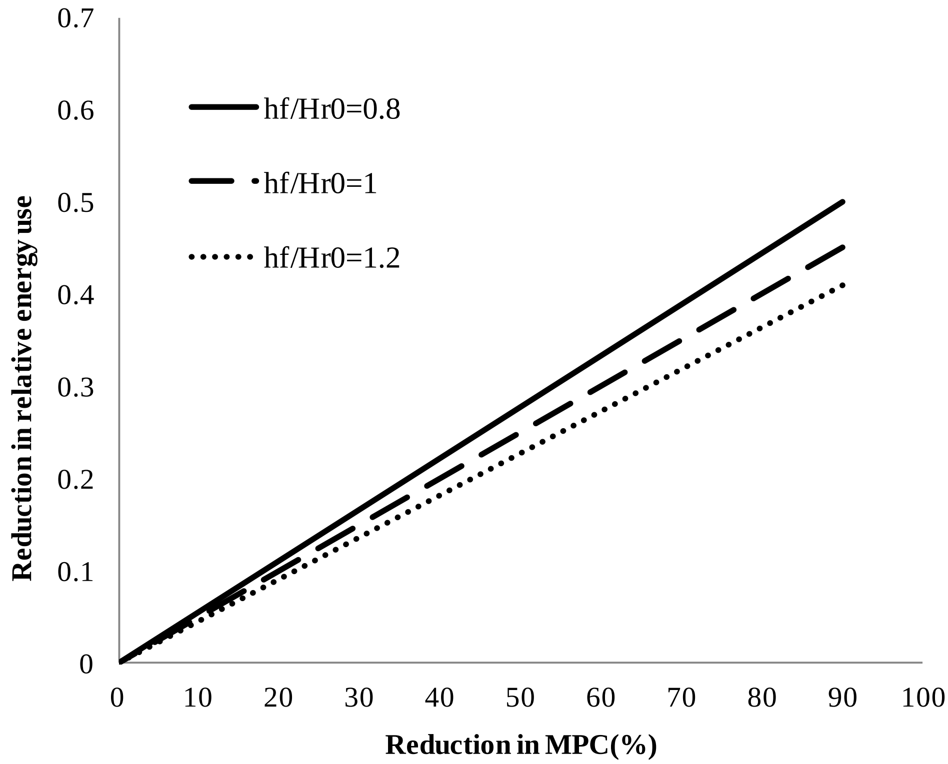


Figure 4.4. Reduction in relative energy use against the MPC (η_0/η =1)

4.3.2 Leakage and MPC

Reduction in the MPC influences leakage. Flow through leaks depends upon the water pressure in a pipe at the leaky location (Colombo and Karney 2002; Giustolisi et al. 2008; (Wu et al. 2010). Therefore, lowering the MPC causes reduction in the overall pressure throughout the system, thereby, leakage decreases. Colombo and Karney (2002 and 2005) examined the impact of leaks on the energy consumption in water supply systems. They concluded that leaks increase operating costs in all systems and energy costs increase more than proportionately with leakage. Pressure management is an effective way to control the amount of water lost in WDSs. In the pressure management process, the factors related to losses are calculated during minimum night flow since most of the users are not active during the night and pressures are high throughout the systems (Walski et al. 2006a; Gomes et al. 2011; Campisano et al. 2012). High pressure in WDSs can also be controlled by installing pressure reduction valves and the number of valves and their locations should be optimized and calibrated in the pipe networks which are a challenging task (Liberatore and Sechi 2009). Reduction of the MPC in WDSs is another idea that causes a decrease in overall system pressure and consequently causes leak reduction. The relation between the MPC and leakage is a recent issue which needs more attention.

Flow through leaks can be calculated as:

$$q = Ch^a \tag{4.4}$$

where q is the leakage rate, h is the pressure at the leak location, and a is the exponent. a is traditionally assigned a value of 0.5, however, values are also recommended from 0.36 to 1.5 by several researchers (Hiki 1981; Thornton 2003). In 50 tests conducted by Lambert (2000 and 1997), the exponent N ranged from 0.52 to 2.59. Figure 4.5 shows the relationship between relative leak and reduction in pressure for different a coefficients. All three curves descend at a decreasing rate for parameters a; however, the leak reduction is more sensitive for higher a. Pressure reduction has a non-linear effect on the relative leakage rate. According to Figure 4.5, 30% decrease in pressure causes a reduction of around 17% in leakage-related water losses.

4.3.3 Water Quality and MPC

Water quality may be influenced by changes in the MPC causing changes in water age. If the MPC is reduced, a pipe with smaller diameter might be selected to meet the required pressure at the most downstream node, and thus water age decreases for a given demand. Water age is a popular indicator of the general water quality (USEPA 2002). A key factor affecting water age is the flow velocity that is a criterion in WDSs design. A high water age increases the vulnerability of the system for regrowth (biological processes) because of decreased disinfectant residual, the reduction in transportation of sediments, and the increase of water temperature in the summer (van der Kooij 2003). Long water age provides an environment conducive to the growth and formation of poor taste and odor,

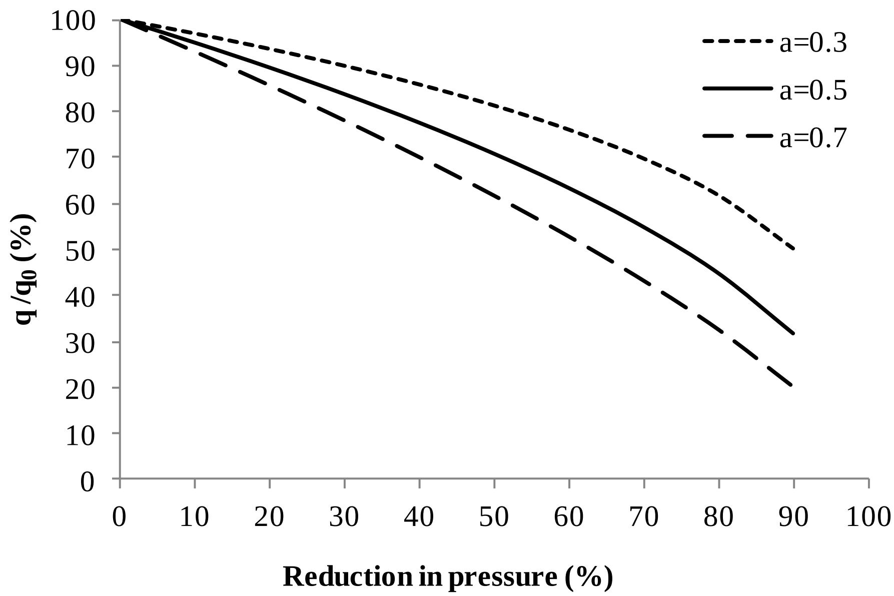


Figure 4.5. Relative leakage as a function of reduction in pressure

thereby causing customer complaints (USEPA 2002). Water age provides an environment conducive to the growth and formation of poor taste and odor, thereby causing customer complaints (USEPA 2002).

The effect of MPC on water age is easily displayed in Figure 4.6. The parameters considered here are changes in pipe diameter and total head. The goal is to identify how reduction in the MPC affects water quality and to identify the most influential factors. A reduction in water pressure can be achieved by selecting smaller pipe diameter and/or decreasing total supplied energy. Obviously, in existing WDSs, the change in the MPC does not affect water quality without rehabilitation/changes in the pipes or changes in storage volumes. In Figure 4.6, steeper energy grade line (*EGL*) shows more friction losses, due to reduction in pipe diameters, in order to achieve the desired MPC (H_r). The residence time, t, for water with the velocity of v travelling in the pipe with a length of L is $t = L/v$. The dimensionless residence time related to the pipe diameters, D and D_0 corresponding to MPCs of H_{r0} and H_r, respectively, is:

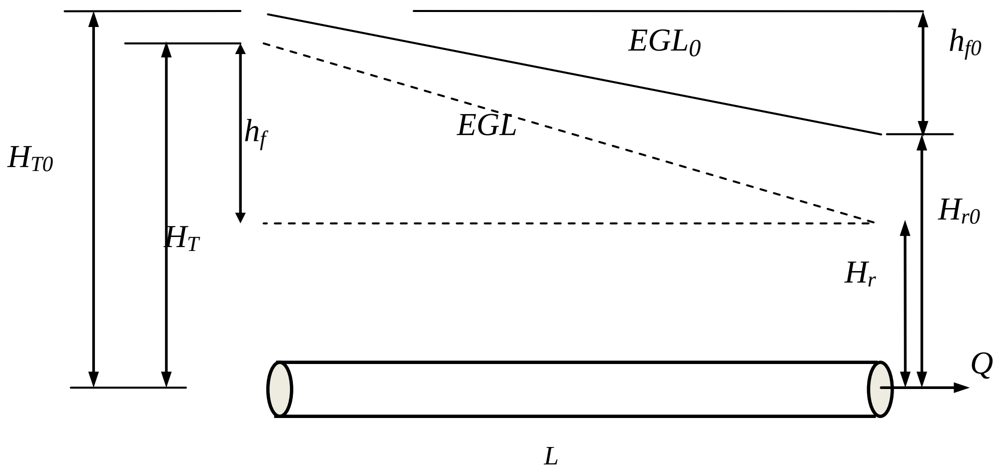


Figure 4.6. Energy grade line affecting water quality in the single pipe system

$$\frac{t}{t_0} = \left(\frac{D}{D_0}\right)^2 \tag{4.5}$$

where t_0 and t are residence times corresponding to MPCs ofH_{r0} and H_r, respectively. The friction loss according to the Hazen-Williams equation for the two scenarios (*EGL* and *EGL0*) shown in Figure 4.6 are:

$$h_{f0} = H_{T0} - H_{r0} = \frac{a}{(D_0)^{4.87}} \tag{4.6}$$

$$h_f = H_T - H_r = \frac{a}{(D)^{4.87}} \tag{4.7}$$

where h_{f0} and h_f are friction head losses associated with MPCs of H_{r0} and H_r, respectively. $a = c_t L(Q/C_h)^{1.85}$, c_t is the unit conversion factor, C_h is the Hazen Williams's coefficient, and Q is the flow rate. Dividing Eq. (4.6) by Eq. (4.7) and substituting the result into Eq. (4.5), the relative water age is given as:

$$\frac{t}{t_0} = \left(\frac{\frac{H_{T0}}{H_{r0}} - 1}{\frac{H_T}{H_{r0}} - \frac{H_r}{H_{r0}}} \right)^{0.41} \tag{4.8}$$

Figure 4.7 shows how relative water age varies with the dimensionless parameters of supplied heads and the MPC. The ratios H_{T0}/H_{r0} and H_T/H_{r0} indicate how the amount of energy supplied at the source compares when the MPC is adjusted. For any combination of values for H_{T0}/H_{r0} and H_T/H_{r0}, reduction in the MPC results in the decrease of the water age ratio. The reduction of relative water age is noticeable for a lower percentage reduction in the MPC when H_T/H_{r0} is smaller. For a certain percentage of reduction in the MPC, the relative water age increases as H_T/H_{r0} reduces (comparing the curves for H_T/H_{r0} = 5 and H_T/H_{r0} = 3). If, in general, the supplied head is unchanged

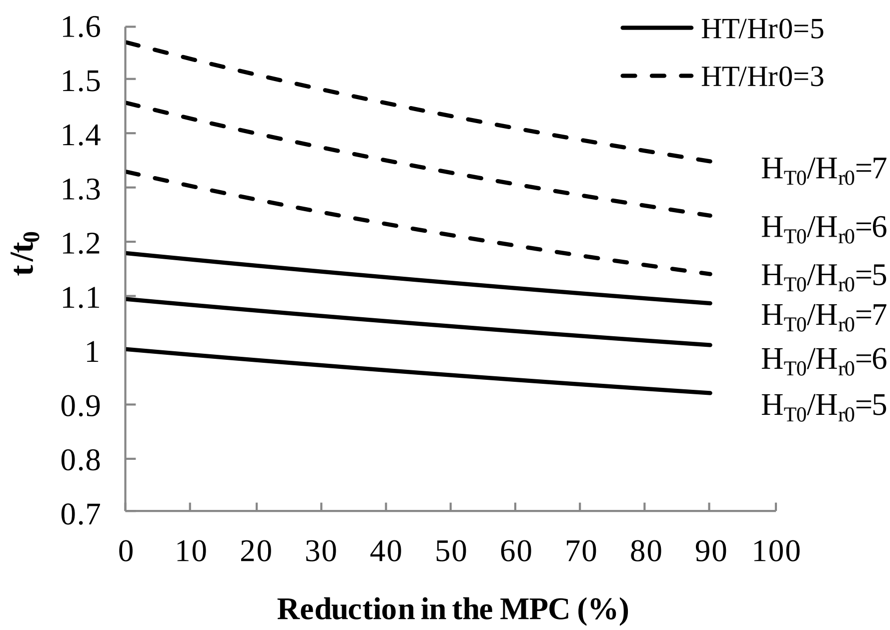


Figure 4.7. Relative water age as a function of relative supplied head and reduction in the MPC

to meet a desired reduction in the MPC, a smaller pipe diameter may be selected; thus, the relative water age decreases, which implies an improvement in water quality.

4.3.4 Transient events and MPC

It is acknowledged that low pressure systems are more vulnerable to low or negative transient pressures than high pressure systems. Therefore, reduction in the MPC may cause WDSs susceptible to negative pressures and contaminant intrusions. Starting up or switching off water pumps, opening and closing valves, and fire hydrant operations result in rapid flow changes. These disturbances generate pressure waves, which have both positive and negative phases as shown in Figure 4.8, that propagate throughout a distribution system. Pressure fluctuations during transient events imply that although a MPC is enforced in WDSs design, some transient events would certainly cause the MPC to be violated.

Low/negative transient pressures inside a pipe might allow the entry of contaminants to the pipe if pathways and contaminant sources exist. Leakage points in water mains, submerged air valves, cross-connections with non-potable water pipes, and faulty seals or joints can all be entry paths for external contaminants. Friedman et al. (2004) monitored the frequency and magnitude of negative pressures at seven WDSs in the USA. They reported that the observed negative pressures lasted approximately 40-50 seconds, and went as low as -10 psi. Although there is a link between low/negative pressure and health risk, there are some effective ways to reduce the risk of contaminant intrusion rather than enforcing high MPC in WSs design and operation. The best practices to prevent contaminant intrusion during low pressure events include transient pressures control, cross-connection control programs, repairing leaky pipes, and preventing air valves to be flooded (USEPA 2003; Friedman et al. 2004).

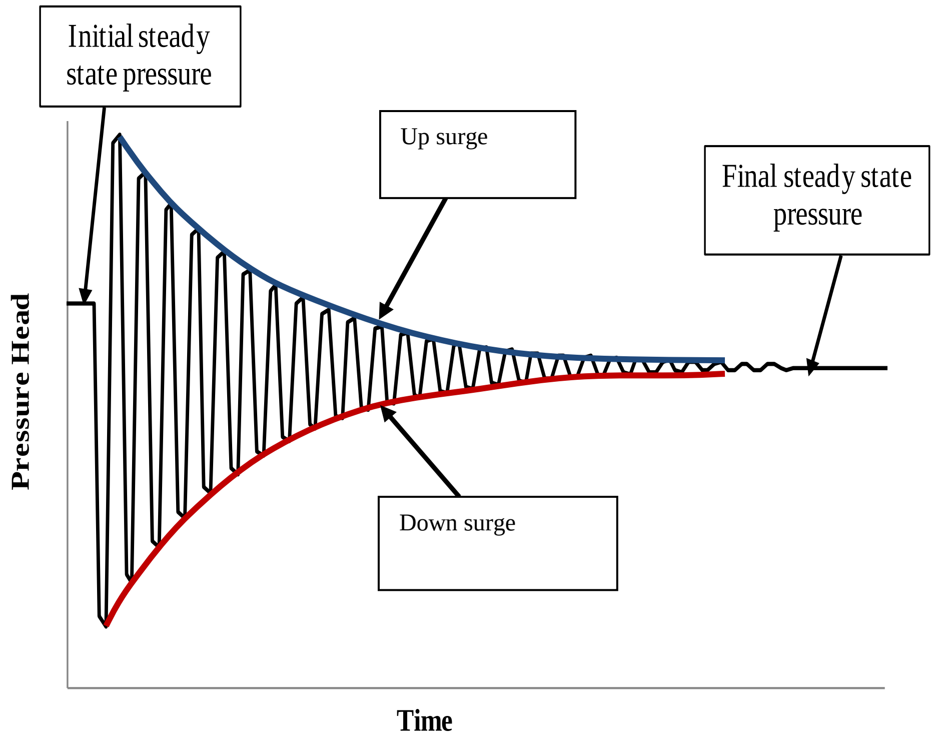


Figure 4.8. Evolution of a transient pressure waves (a typical transient pressures profiles)

To minimize a system's susceptibility to surge pressures and to control down surges to a minimum acceptable level, surge control strategies must be performed. Surge control strategies have been divided into three categories: engineering strategies, maintenance strategies, and operational strategies. Devices such as surge anticipation valves, pressure relief valves, air release/vacuum valves, surge tanks, and air vessels are generally used to control surge pressures in the system (Laine and Karney 1997; Larock et al. 2000; Lingireddy et al. 2000). Controlling WDSs' operations (e.g., controlled valve motions and fire hydrant operation), and adjusting the rate at which pumps are switched, can help maintain transient pressures to acceptable levels (Wylie and Streeter 1983; Huo 2011). Careful operation of systems are required in accordance with appropriate operational strategies; many of these strategies need the combination of remote sensors and a control system which can be costly and disconcerting for operators who are not used to active control systems.

The transient pressures occurring during the operation of WDSs can be controlled by the aforementioned surge control strategies. However, in the case of hydrant operations, it is impossible to equip a surge control device at each hydrant to control transient pressures since there are too many hydrants scattered at different locations of a WDS. Pressure fluctuations due to hydrant operations should be controlled to the minimum desired level by the slow opening of hydrants regardless of what the minimum steady-state pressure is. This is explained in chapter 8 in which a new strategy of transient pressure control is created using down surge control boundary in a pipe system during hydrant operations.

4.3.5 Pipe breaks and MPC

A high value of the MPC, one that requires relatively high system pressures, can result in more frequent pipe breaks. There is clearly an expense both the system and to related services associated with bursts and breaks. Moreover, water contamination is possible during pipe breaks and repairs. There were 237,600 water main breaks in the United States in 1994 (Kirmeyer et al. 1994) and an average of 850 water main breaks occur daily in North America at a total annual repair cost of over $3 billion (http://www.watermainbreakclock.com). Traditionally, pipe breaks can be prevented through active rehabilitation and replacement programs which are the most common practices of utilities. The contribution of internal pressure to pipe breaks occurring simultaneously with one or more other sources of loads (e.g., thermal loads, soil cover loads, and traffic loads) have been addressed by many authors (e.g., Kiefner and Vieth 1989; Rajani et al. 1996; Rajani and Makar 2000).

To illustrate how reduction in the MPC influences the frequency of high pressures in WDSs, the Anytown network presented in Walski et al. (1987) is considered here as a reasonably representative network with storage. The layout of the system is shown in Figure 4.9 and details can be found in Walski et al. (1987). The system topology for pipes and system configuration are set according to Gessler's optimization (Gessler 1985). The nodal demands and diurnal demand pattern for 2005, as explained in the original paper, are considered in this chapter. Three identical pumps in parallel with a fixed pump efficiency of 75% are taken into account to drive the system. The pump characteristic curves for cases where MPCs are 30 m and 20 m are defined by the curve H = 91 – $11\times10^{-5}Q^2$ and H=83 –$108\times10^{-5}Q^2$, respectively, where H is in meters and Q is in liters per second. Extended period simulations of 72 h are conducted to determine pressures throughout the system.

The percent of high pressures in the system (during low demand conditions) is depicted in Figure 4.10 where MPCs are 30 m and 20 m. As expected, reduction in the MPC causes a decrease in the system pressures during low flow conditions. The pressure at or below 60 m is accounted for 85% of nodes where the MPC is 20 m and only for 40% of nodes where the MPC is 30 m in the system. According to Figure 4.10, the average maximum pressure in the system decreases from 63 to 53 m (18% of reduction), where the MPC is reduced by 1/3. This proves that reduction in the MPC causes a decrease in the system pressure during low demand conditions, where pressures are high in the system, consequently the breakage rate may reduce.

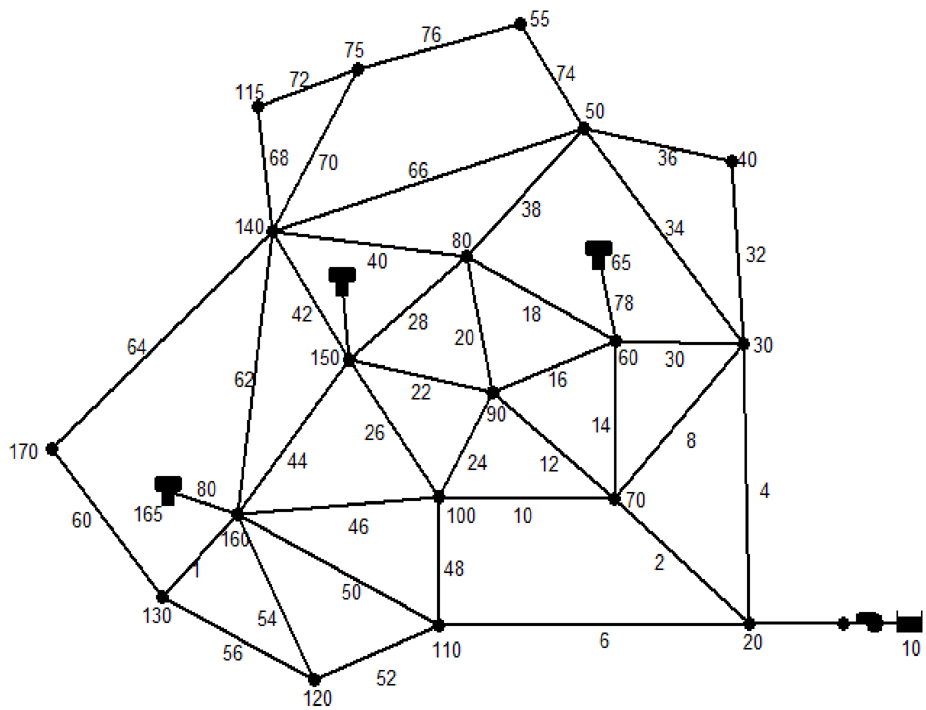


Figure 4.9. Layout of Anytown network-from Walski et al. (1987)

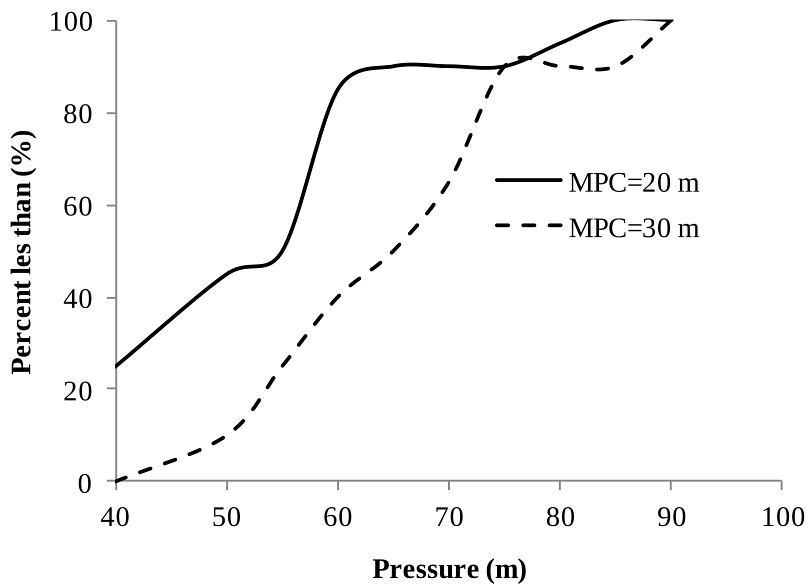


Figure 4.10. The distribution of pressure in the Anytown system

4.4 Conclusions

Reduction in the MPC has both benefits and costs. Reducing the MPC may cause decrease in water demands and leakage, and quite significantly, energy use. However, if the water pressure is reduced, booster pumps must be installed at buildings with a height greater than the minimum supplied pressure; thus some of the savings are moved to building owners because of costs of installing new booster pumps at a greater number of buildings. Lowering the MPC can also reduce the probability of pipe breaks. Water quality will be affected by changes in the MPC but in existing systems, a change in the MPC does not affect water quality if rehabilitation of pipes or changes in storage volumes do not occur. Policies to control and avoid low pressure events may not yet be fully linked to the value of the minimum pressure standard. Thus, how the MPC is enforced/ensured in WDSs is an important issue.

While an MPC is enforced in WDSs design, this criterion may be temporally violated during transient conditions. Thus, how often MPC is violated and how severe this violation could be are not a simple matter in practice. Developing some metrics to evaluating the severity of violation of MPC during transient events would seem to be helpful. Reduction in the MPC causes the system to be at lower pressures and therefore makes the system more vulnerable to low/negative transient pressures. Consequently, reduction in the MPC makes WDSs more susceptible to contaminant intrusions. Examining the consequences of changes in the MPC in terms of the system energy use, leakage, and transient events needs more details that will be provided in chapters 5 and 7 of this thesis.

Chapter 5

Intrinsic Relationship between Energy Consumption, Pressure, and Leakage in Water Distribution Systems

This chapter is based on the paper entitled “Intrinsic Relationship between Energy Consumption, Pressure, and Leakage in Water Distribution Systems.” by Vali Ghorbanian, Bryan Karney, and Yiping Guo submitted to the *Urban Water Journal*.

The basic implications of changes in delivery pressure on system energy use and cost, on leakage, excess pressure, and environmental impacts are explored. An analytical expression is first developed to characterize the primary relationships between energy use, leakage and pressure for a simple pipe segment. Then, two more realistic case studies, based on varying versions of the Anytown network, are considered. The results indicate that energy use responds more to changes in the delivery pressure in systems with higher leakage rates while reductions in pressures curtail energy use and leakage more dramatically in low resistance systems. Perhaps more surprisingly, systems with more effective water storage and thus uniform pressures tend to have to higher leakage rates, greater energy usage, and higher GHG emissions relative to systems relying on direct pumping. The generalization that results from these studies is perhaps predictable but has profound implications: the higher the delivery pressure the greater will likely be the amount of water wasted and energy dissipated.

5.1 Introduction

Water distribution systems (WDSs) are historically designed to deliver safe and reliable drinking water with sufficient pressure. Yet, there are no universally accepted guidelines to specify the appropriate pressure standards for WDS designs. For example, most

Canadian provinces and US states use a minimum pressure criterion (MPC) of 14 m, but Australia often requires 20 m and the UK only 10 m (Ghorbanian et al. 2015b). Maximum pressure standards are not established for many regions though a number like 70 m is sometimes suggested (ACWWA 2004). However, design standards do generally require that water mains be designed to be strong enough to withstand the maximum operating pressures in addition to transient pressures.

A lower value of the delivery pressure may reduce water consumption (e.g., consumptions from faucet, showers, and lawn watering) and also leads to efficient operation through reduced energy use, leakage, and frequency of pipe breaks. However, if the delivered pressure is too low, the system may be more susceptible to intrusion events resulting from hydraulic transients, and the system may also be incapable of supplying the required flows. Water utilities set a minimum pressure to ensure the delivery of adequate flows to fire hydrants and consumers at remote and high elevation areas. Most insurance companies are concerned with the risk of fires that often require a certain fire flow rate that meets a specific pressure standard. However, such a pressure standard is almost certainly temporally violated during the transient event associated with the initial opening of the hydrant (Ghorbanian et al. 2015a). Moreover, the flow of a hydrant is governed by the orifice relationship and depends on the hydrant's outlet nozzle diameter, thus the required fire flows may be supplied under a pressure less than the MPC under steady state conditions (Ghorbanian et al. 2015b).

Colombo and Karney (2002 and 2005) examined the impact of leaks on the energy consumption in water supply systems. Not surprisingly, they concluded that leaks increase both operating and energy costs, but that energy costs increase more than proportionately with leakage. They also found that leaky systems with storage may often have higher operating and energy costs as compared systems with direct pumping systems. If a system is operating at high pressures, its delivery conditions and its energy use is suboptimal. Since this point is so critical, it seems logical to further explore these key pressure relations.

Traditionally, pressure management has been considered an effective way of reducing the excess pressures and the amount of water lost in a system during off-peak hours (Gomes et al. 2011). To limit pressures, pressure reducing valves are installed with the number of valves, their location, and their set-points optimized (Liberatore and Sechi 2009). The pressure reducing valves setting can be adjusted automatically on the basis of the measurements of pressure at the control node and water discharge in the pipe if real time control is applied (Campisano et al. 2012; Creaco and Franchini 2013). In implementing a pressure management strategy, the total energy supplied does not necessarily change and this energy may be still too high even though the system operating pressure is controlled. Additional benefits might be gained by reducing the energy supplied. A case study showed that significant energy savings can be achieved through reduced energy input (to decrease delivery pressure) for the service pressure using pumping at lower head (LeChevallier et al. 2014b). However, only a few studies to date have considered the consequences of changes in the delivery pressure in terms of leakage, energy use, and environmental impact.

The pressure supplied by WDSs can be either above the requirement for service level or in a deficient condition (e.g. pipe outages, power failures at pump stations and fire flow condition). For the former, the demand driven analysis (DDA) is often performed to determine performance of WDSs under normal condition. In demand driven models, the supplied demand is assumed to be independent of pressure and this approach is valid when the pressure is above the MPC considered according to design guidelines. In deficient condition however, the pressure driven models should be used to more accurately predict the system response (Wu et al. 2009). In pressure driven analysis (PDA), nodal demand is assumed to vary with the nodal pressure and when nodal pressure rises to a certain level, i.e., the MPC, the total demand is supplied. Several methods have been proposed to analyse water distribution system under insufficient conditions (Gupta and Bhave 1996; Ang and Jowitt 2006; Siew and Tanyimboh 2011). The PDA can be also modeled by the emitter feature in the EPANET2 which needs iterations at each node for computation of accurate head and demand (Assela 2010; Jun and Guoping 2013). The MPC is a key parameter to distinguish whether the total demand can be supplied, but this value varies around the world (Ghorbanian et al. 2015b).

In this paper, the aim is to determine the potential for energy savings and benefits of leak reduction from reduced energy input for service pressure (i.e., reduction of delivery pressure). It is naturally assumed that such a reduced delivery pressure can still supply total demand (e.g., the delivery pressure is not lower than the assumed MPC). Thus, a demand driven analysis is still generally valid. Of course, in leaky system, the leakage rate depends upon the pressure at leak location and should be modeled based on PDA. Leakage is generally modelled here using the emitter feature of EPANET2 (Rossman 2000).

The context of this paper is slightly different from the study conducted by Colombo and Karney (2002 and 2005). In these previous researches, leak size and location effects on water loss and energy use were examined as were leakage levels on energy costs in systems including storage tanks. In particular, in the work of Colombo and Karney (2002 and 2005), the attempt was made to highlight the impact of leaks on the energy use and pumping costs of a system. No effort was made to determine the effectiveness of pressure reduction as a leak management strategy which is the main focus here. In particular, the current paper explores the effectiveness of changes in delivery pressure on system energy use, leakage, and environmental impact. A simple pipe is first considered to derive analytical expression to characterize the relationship between energy use, leakage and pressure. The derived analytical equations offer a concise description of how pressure influences leakage rate and energy requirements. Then, the Anytown network (Walski et al. 1987) is considered to highlight the impact of high delivery pressure on energy consumption, excess pressure, and leakage. To compare a typical network without storage with Anytown system, the unrehabilatated Anytown without storage tank in which the tanks are removed from the Anytown network is also considered.

5.2 Consequences of pressure reduction

To illustrate the essential response to a change in pressure, the simple system in Figure 5.1 is first considered. Although almost trivial, concise equations can be derived to describe of how pressure influences both leakage and energy values. Although clearly idealized, the simplified approach helps focus attention on the key variables.

5.2.1 Pressure, leakage, and headloss

In Figure 5.1, it is assumed that the required flow Q_d is supplied at prescribed downstream heads, i.e., H_{m0} and H_m denoted as delivery heads. Because of leakage, the flow in the pipe exceeds Q_d by q_0; moreover, a steeper energy grade line (EGL_0) occurs due to greater friction loss. The modified EGL reflects the effects of head loss and leakage if pressure at the demand end of the pipe decreases. The total leakage rate, q_0, can either be expressed as a proportion of demand, $q_0 = a_0Q_d$, where a_0 is the leakage fraction, or it can be modeled using the emitter function, $q_0 = CH^N$, where C is the discharge coefficient and N is an exponent. The emitter exponent N is thought to vary depending of type of leak. Lambert et al. (2013) pointed out that N could be mostly in the range 0.5 (fixed area leaks) to 1.5 (variable area leaks). In 50 tests conducted by Lambert (2000 and

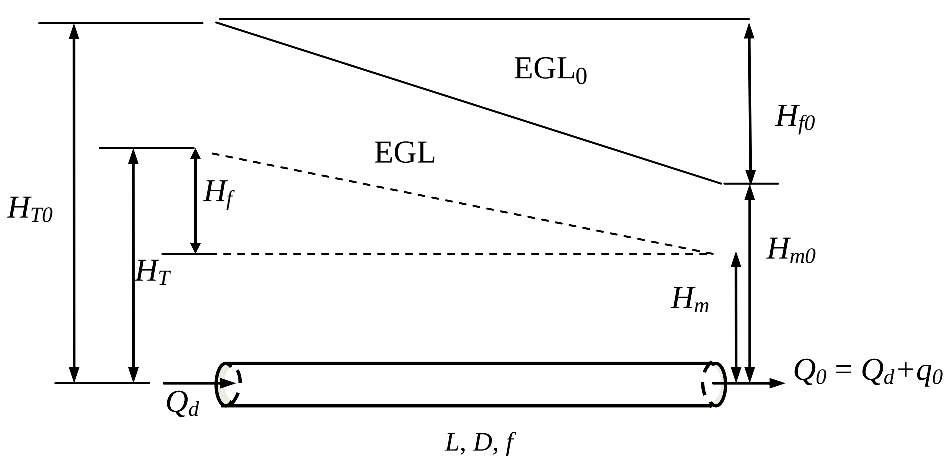


Figure 5.1. Effects of reduction in pressure in a leaky pipe segment (L = Pipe length D = Pipe diameter, f = Darcy-Weisbach friction factor, q_0 = leakage rate)

1997), the exponent N ranged from 0.52 to 2.59. From the emitter expression, the relative leakage rate can be expressed as

$$\frac{q}{q_0} = \frac{a}{a_0} = R_P{}^N \tag{5.1}$$

where a and a_0 are leakage fractions associated with delivery heads of H_m and H_{m0}, respectively, and R_P is relative delivery head, $R_P = H_m/H_{m0}$. The relative flow, R_Q, at the end of the pipe can be expressed as

$$R_Q = \frac{1+a}{1+a_0} = \frac{\frac{1}{a_0} + R_p{}^N}{\frac{1}{a_0} + 1} \tag{5.2}$$

Eq. (5.2) compactly indicates that if the delivery head is lowered, pressures at leak locations decrease and water loss is diminished. Because leak is modeled as an increment to required demand, reduction in leakage causes total flow of the pipe to reduce. A sensitivity to reduction in pressure confirmed that the flow reduction as a result of reduced pressure is more noticeable for higher leak fraction with the greater N. The head loss equation for fully developed turbulent pipe flow, $H_f = KQ^{E}$, where H_f is the head loss, K is pipe resistance coefficient, and $E = 2$ considering the Darcy-Weisbach formula for head loss expression, relates the head loss in a pipe to the flow ($Q_0 = Q_d + q_0$) it conducts. For a pipe with a single leak discharging a_0Q_d at the demand node, the resulting expression for the head loss ratio, R_f, becomes a quadratic function of a and a_0

$$R_f = \frac{H_f}{H_{f0}} = \left(\frac{1+a}{1+a_0}\right)^2 = R_Q^2 \tag{5.3}$$

Figure 5.2 shows how the head loss ratio (1- R_f) varies with leak fraction and reduction in pressure. Clearly, as pressures decrease (for all leakage fractions and values of the exponent N), the reduction in head loss ratio increases implying that lowering pressure in a pipe segment causes both pressure and leakage decrease. For higher leakage fractions and greater N, the reduction in head loss ratio is more noticeable, thus effects of reduction in pressure in systems with high leakage rate is more than that of systems with low leakage rate in terms of reduction in head loss. Although Eq. (5.3) indicates that the head loss ratio is nonlinearly related to leakage fraction, the results in Figure 5.2 shows that the reduction in head loss ratio is linearly increasing with pressure reduction ranged from 0 to 50%. Of course, the nonlinear effect of reduced pressure on reduction in head loss ratio becomes evident for higher values of pressure reduction (not shown). The reduction higher than %50 in delivery pressure may not be practical and is not considered for the single pipe system. Despite each curve following the shape of a linear function, there is no simple ‘‘rule of thumb’’ for relating decrease in head loss ratio to reduction in pressure for WDSs. Clearly, however, leaks are expected to affect the head losses and the distribution of pressure.

5.2.2 Pressure-energy relationship

If a system is leaking, reduction in pressure causes a decrease in both flow and pressure. The total head at the supply source decreases by $\Delta H_T = \Delta H_m + H_{f0} - H_f$, where $\Delta H_m = H_{m0} - H_m$, if H_{m0} is reduced by ΔH_m. Thus, the amount of reduction in total supply head is greater than that of the pressure, i.e., $\Delta H_T > \Delta H_m$. The leakage rate also reduces by $\Delta q = a_0 Q_d\ (1 - R_P^{\ N})$. The amount of reduction in the energy at the source (ΔE) of the pipe segment shown in Figure 5.1 is proportionate to $\Delta H_T \times \Delta q$. To simply assess the energy effectiveness of changes in delivery pressure in a pipe segment, the energy use of the system, shown in Figure 5.1, can be expressed as a dimensionless term

$$\frac{E}{E_0} = \frac{\gamma Q H_T}{\gamma Q_0 H_{T0}} = R_Q R_P \left(\frac{1 + \frac{R_f}{R_P} R_O}{1 + R_O} \right) \tag{5.4}$$

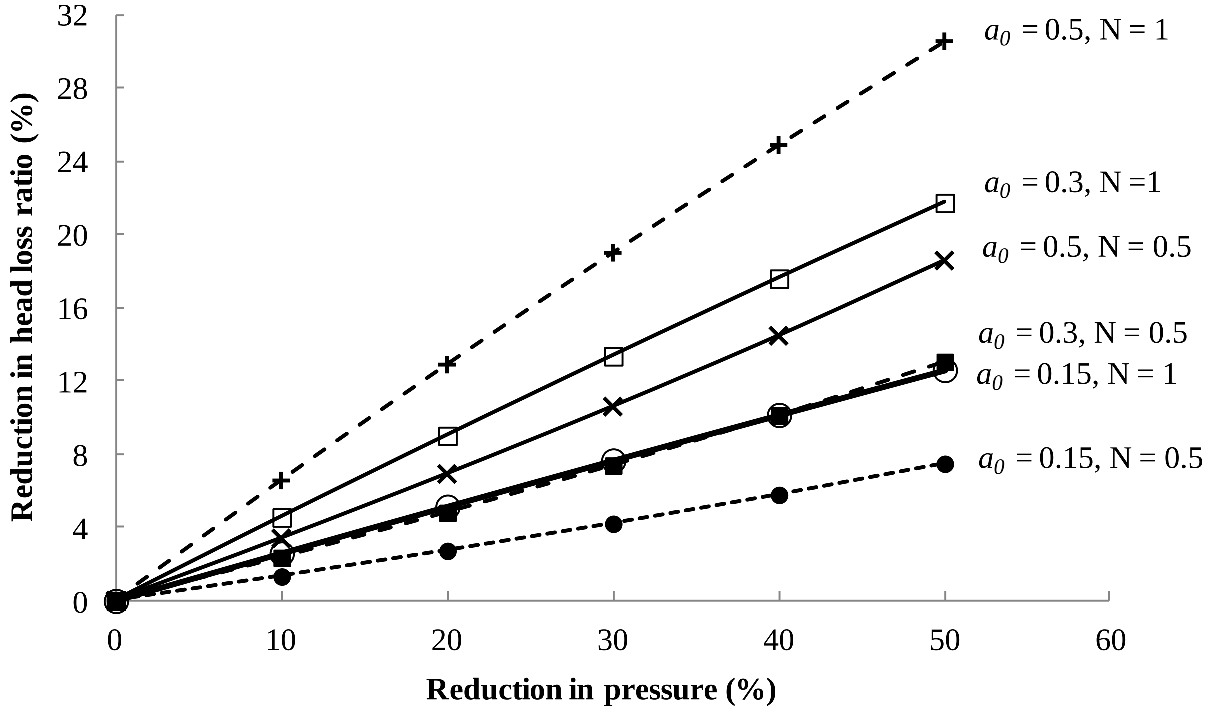


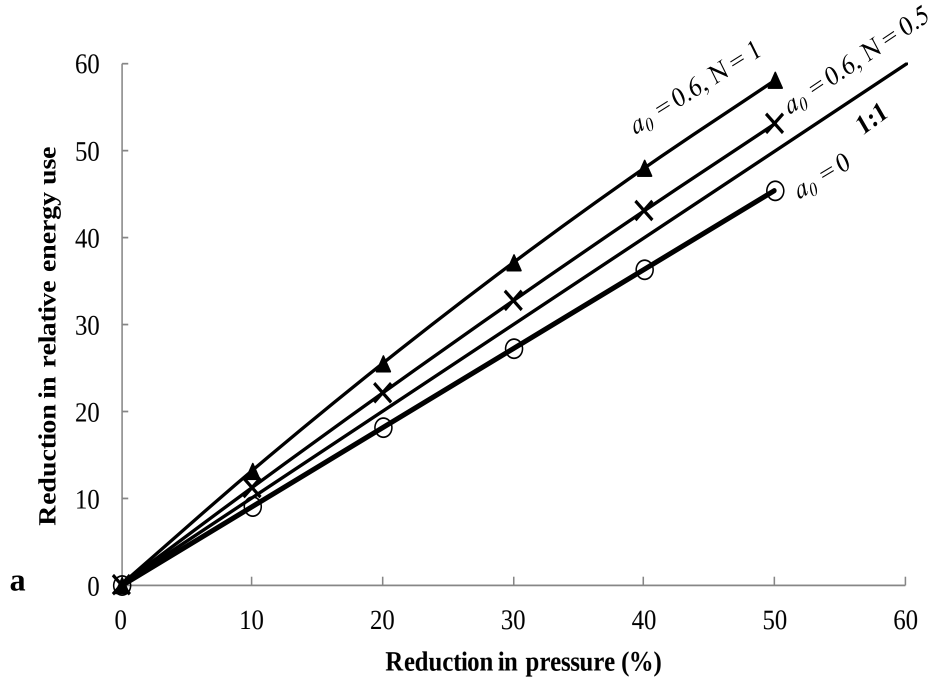
Figure 5.2. Relative head loss as a function of reduction in pressure

where E_0 and E are the energy supplied at the source for different scenarios in which delivery heads are H_{m0} and H_m, respectively, H_{T0} and H_T is the total head supplied upstream to meet pressure heads, H_{m0} and H_m, at the most downstream node, respectively, and $R_O = H_{f0} / H_{m0}$ indicating that how the amount of friction loss is compared with the delivery head. In Eq. (5.4), the supply efficiency associated with scenarios of H_{m0} and H_m is considered to be unchanged. From Eq. (5.4), reduction in relative energy use depends upon R_Q, R_P, R_O, and R_f / R_P. If in a specific system, the delivery pressure reduces, both R_P and R_Q are decreasing and R_O is not influenced. Also, in a leaky system, if the pressure decreases a becomes less than a_0, then $R_f / R_P > 1$ (this could be achieved by solving the inequality $R_f > R_p$ considering Eqs. (1) and (3) and N = 0.5). A sensitivity analysis of reduction in pressure indicated that the ratio of R_f / R_P is greater for a low leak system (e.g., $a_0 = 0.1$ and $N = 0.5$) with respect to reduction in pressure.

In a high leakage system, the response of system is more noticeable in terms of energy use. The reason is that pressure reduction in a high leak system causes both R_Q and

R_f / R_p reduce more than that of low leak counterpart. Thus, the reduction in energy use due to reduced pressures in systems with high leakage rate is more than that of low leak systems. The obvious point from Eq. (5.4) is that if the term (R_Q (1+(R_F / R_P)R_O) / (1+R_O)) < 1, the percent reduction in energy use is more than the percent of decrease in pressure. For this purpose, R_Q and R_O should be relatively small, meaning the system should comprise high leakage rate and low friction regime. This is confirmed in Figure 5.3 which depicts the response of reduction in relative energy use (1-E /E_0) to changes in delivery pressure for different cases. Figure 5.3(a) indicates the effects of leakage. This Figure represents a system with low friction regime (R_O = 0.1) and shows a reduction in pressure causes the relative energy use decreases and this reduction is more noticeable for high leakage rate. As indicated in Figure 5.3(a), all curves associated with a leaky system (a_0 > 0) lie above the 1:1 line indicating that the percent of decrease in relative energy use is more than the percent of reduction in pressure. Indeed, leakage is a key parameter influencing the response of reduction in energy use to reduced delivery pressure. Pipe friction is also another key factor influencing reduction in the system energy use due to lowering delivery head.

Figure 5.3(b) depicting reduction in relative energy use against reduction in pressure highlights this presumption. For smaller reduction in pressure, the reduction in energy ratio changes only slightly with R_O; however, as the reduction in pressure becomes greater, the dependence upon R_O is more noticeable. From Figure 5.3(b), it is clear that the energy saving, as a consequence of reduction in pressure, can be expected to be greater for low friction pipes including high leakage rate (the curves associated with R_O = 0.01 and 0.1 with N = 0.5 and 1 are above the 1:1 line). The main assumption to develop all curves as shown in Figures 3(a) and (b) is that the supply efficiency (e.g., pump efficiency) is considered to be equal for all scenarios involving changes in delivery pressure. However, if the supply efficiency changes, the relative reduction in energy use may become either more or less than what is indicated in Figures 5.3(a) and 5.3(b). Of



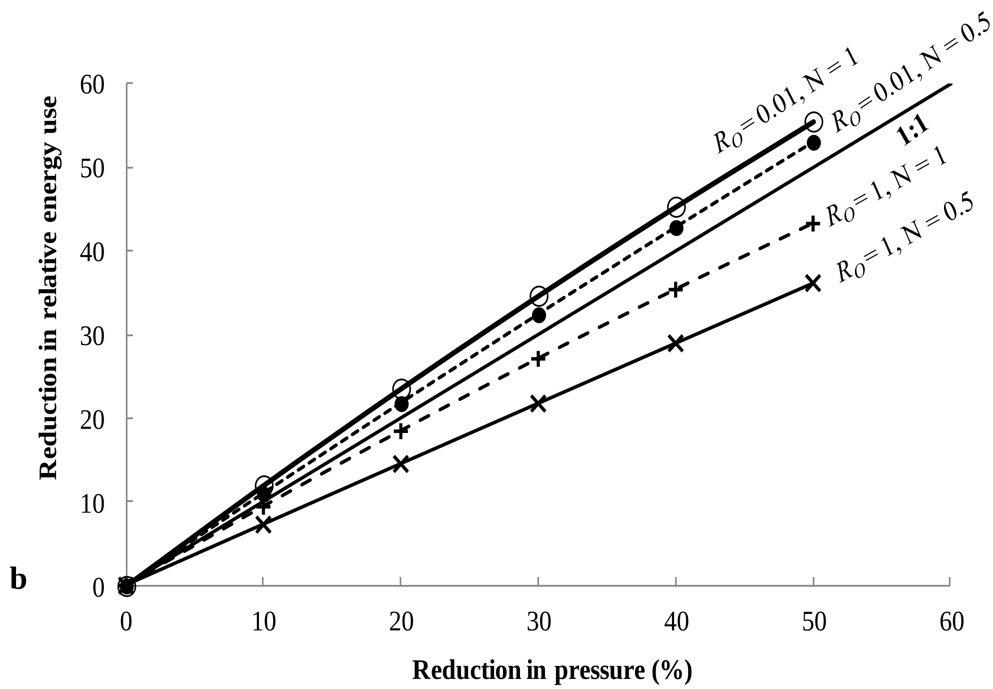


Figure 5.3. Reduction in relative energy use against changes in pressure: (a) Different leakage fraction (R_O = 0.01, a_0 = 0 denotes no leakage rate), (b) Different friction regimes (a_0 = 0.3)

course, increase in the supply efficiency strongly affects the energy saving in the system.

5.3 Case studies

Changes in energy use resulting from a reduction in pressure depend upon a wide variety of factors including system topology, pipe characteristics, pump arrangement, and operating policies. The priority here is to evaluate how changes in the delivery pressure affect the water loss, energy requirement, and maximum operating pressure in WDSs that is a key factor in pressure management strategies. To demonstrate the fundamental influence of changes in pressure, the Anytown network presented in Walski et al. (1987) is considered here as a representative network with storage. The Anytown system is considered as a realistic benchmark which has the topological complexity typical of many real-world systems. The layout of the system is depicted in Figure 4.9 and details can be found in Walski et al. (1987). The system topology for pipes and system configuration are set according to Gessler's optimization (Gessler 1985). The nodal demands and diurnal demand pattern for 2005, as explained in the original paper, are considered in this paper. Since the original data was in U.S. customary units, all units were converted to SI equivalents. Tanks T_1 and T_2 are cylindrical with a diameter of 11.7 m and a height of 12.1 m and tank T_3 is cylindrical with a diameter of 19 m and a height of 12.1 m. All tanks operate with the initial depth and maximum depth of 3 m and 10.6 m, respectively. Tanks' bottom elevations are all 92 m for the scenario in which the delivery pressure is 35 m at the highest demand node elevation. To determine energy costs, the base price of \$0.11/kW h is considered during the peak hours with price factors of 0.55, 0.85, and 1 for the hours 0 – 6 and 20 – 24, 6 – 12 and 18 –20, and 12 - 18, respectively. To evaluate the effects of reduction in pressure on energy use of the storage scenario, the Anytown system without storage, represented as a network with direct pumping strategy, is also tested.

For the Anytown, three identical pumps, defined by the curve H=160 – 5 × $10^{-4}Q^2$ (H is in meters and Q is in liters per second), in parallel are considered where the delivery pressure is set to be at least 35 m in the system. The system is considered to operate based on the tank water level, i.e., pumps are set to turn on when the tank level is at 3 m and to shut off when tanks are refilled to the level of 10.6 m. For the Anytown without storage tank, three identical pumps with the characteristic of H=165 – 8 × $10^{-4}Q^2$ (H is in meters and Q is in liters per second) are used where the delivery pressure is 35 m. The on–off pump controls are specified for pumps operation. To consider the pump efficiency, the relationship between pump discharge (Q) and pump efficiency (e_p) can be estimated by (Walski et al. 2007)

$$e_p = a_1 Q + a_2 Q^2 \tag{5.5}$$

where $a_1 = 2e_0/Q_0$, $a_2 = -e_0/Q_0^2$, e_0 is the efficiency at the best efficiency point (%), and Q_0 is the flow at the best efficiency point. The best efficiency is set to 80% (that is assumed to be wire-to-water efficiencies, i.e., both motor and pump efficiencies) for all scenarios.

The reduced delivery pressure, for the Anytown system, is achieved by changing the tank elevations and pump curves. Extended period simulations of 72 h are also conducted to model daily demand and tank level fluctuations and to ensure stationary pressure at nodes. For the Anytown without storage, pumps are taken into account with lower supplied head in order to achieve lower delivery pressure in the system. The performance of the two systems is tested for all scenarios with delivery pressures changing from 10 m to 35 m using EPANET2 simulations. Leakage rate is modeled as a percentage of demand although it has no revenue for municipalities and is not usable for customers (Colombo and Karney 2002). This is just a simplification to perform analysis considering the assumption that the demand node does not change and the leakage rate only depends on the pressure at leak locations. Leaks at nodes, in EPANET2, are modeled with the use of emitter feature in which the flow rate is considered to be a function of pressure.

The emitter coefficient generally reflects the size and shape of a leak and is often adjusted when modeling leaks of different magnitudes. In the equation proposed by

Germanopoulos (1985) and Tucciarelli et al. (1999), the emitter coefficient was considered to relate to pipe length, pipe diameter, and leaks surface per unit pipe surface of the pipe assuming that there is a constant leaking area per unit area of the pipes surface. Thus, the emitter coefficient depends on the system characteristic and can vary from one system to the other. However, the attempt here is to determine how the total leakage rate would alter if pressure in the system reduces considering other factors contributing to emitter coefficient remain unchanged. Of course, some pipes might have more leakage rate (depending on their lengths and diameters) than others but the total volume of leak for the system should remain constant for the benchmark case (i.e., total leakage rate is assumed to be 30% of the total daily demand volume). In other words, the aim here is to examine how a system, if it leaks a certain leakage orifice, to what extent that leakage would change due to changes pressure. A leak at a particular node is assumed to represent the existence of leaks in some or all of the incident pipes and indicates the equivalent leak concept (Colombo and Karney 2002). Considering this, the emitter coefficient is assumed to be the same for each leaky node, and the emitter exponent is set to be 0.5 throughout the analysis (Colombo and Karney 2002 and 2005; Jun and Guoping 2013).

Leakage rate is controlled by the value of the emitter coefficient and in the first stage of analysis it is set to be 30% meaning that total amount of water lost through the leak in a 24-hour cycle is 30% of the total daily demand volume (the emitter coefficient for the Anytown network is set to be 1.18×10^{-3} $m^{2.5}/s$, and for the Anytown system without storage, the coefficient is 1.27×10^{-3} $m^{2.5}/s$). It establishes as the reference leakage when the delivery pressure is maintained at 35 m in the system. Leaks then are computed for all pressure scenarios.

Changes in energy use and cost against reduction in the delivery pressure are shown in Figure 5.4. As expected, energy use and costs decrease for all scenarios as the delivery pressure is reduced and changes in energy use and costs are greater for the no storage configuration compared with storage counterpart as pressure changes. For a smaller percentage of reduction in pressure, the energy consumption changes only slightly for the two systems; however, as the reduction in pressure becomes greater, the dependence upon scenarios is more noticeable. The energy cost in storage configuration depends on the time of day that pumps operate and the price of energy during the hours that pumps operate. Thus, the trend of energy cost and energy use may not be consistent (as shown in Figure 5.4). According to Figure 5.4, a 30% decrease in pressure causes a reduction of about 5.5% and 13% in energy use for the Anytown network with and without storage, respectively.

Figure 5.5 depicts the leakage reduction curves for the two networks. For the no storage configurations, leak reduction response is more than its storage counterparts in terms of reduction in pressure. From the results shown in Figure 5.5, 30% decrease in pressure causes a reduction of about 12%, and 10% in leakage for respectively the unrehabilitated Anytown without storage, and the Anytown network. Comparison of results for reduction in delivery head indicates that leakage decreases against reduction in pressure for the two systems under study and the leakage percentage for direct pumping is less than the storage configuration (not shown).

Not surprisingly, reduction in delivery pressure causes a decrease in the excess pressure of the system. Figure 5.5 also depicts the decrease in maximum operating pressure in the system with respect to reduction in the delivery pressure. The average maximum pressures in the system, defined as the maximum operating pressure, is estimated by taking the arithmetic mean of computed maximum pressures, during the simulation period, at all nodes of the system. Clearly from the Figure, reduction in average excess pressure is more noticeable for the no storage configuration against lowering the delivered pressure. From Figure 5.5, a 30% decrease in delivery pressure causes a

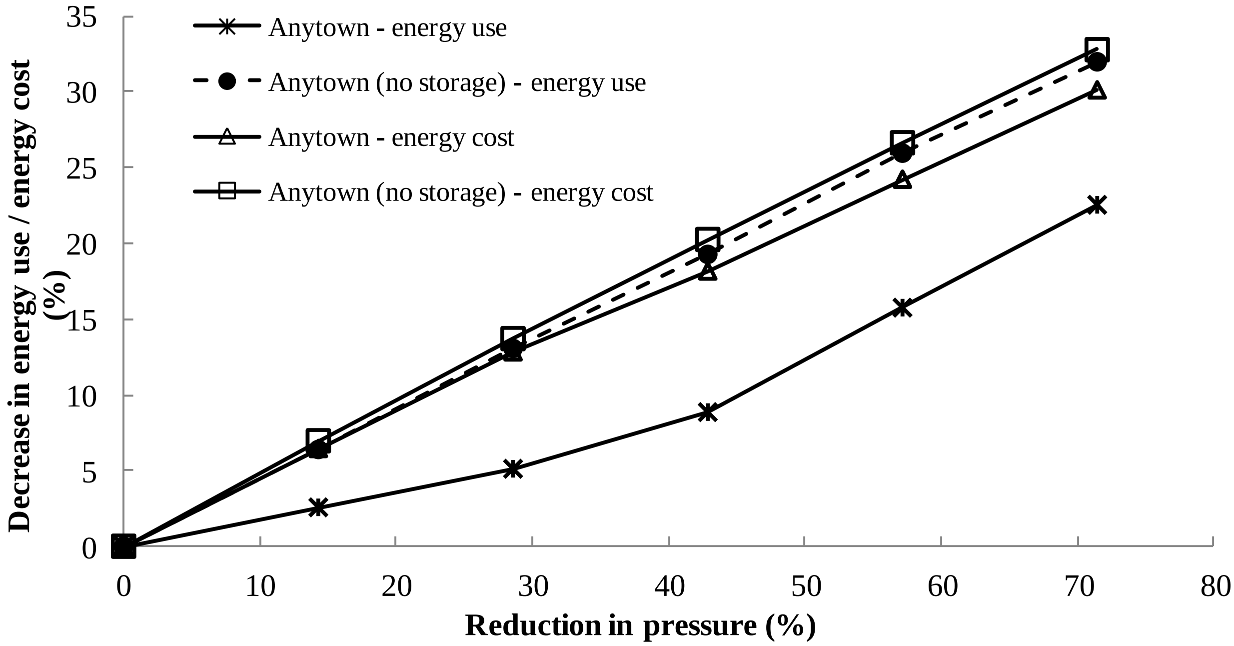


Figure 5.4. Energy use response to changes of MPC

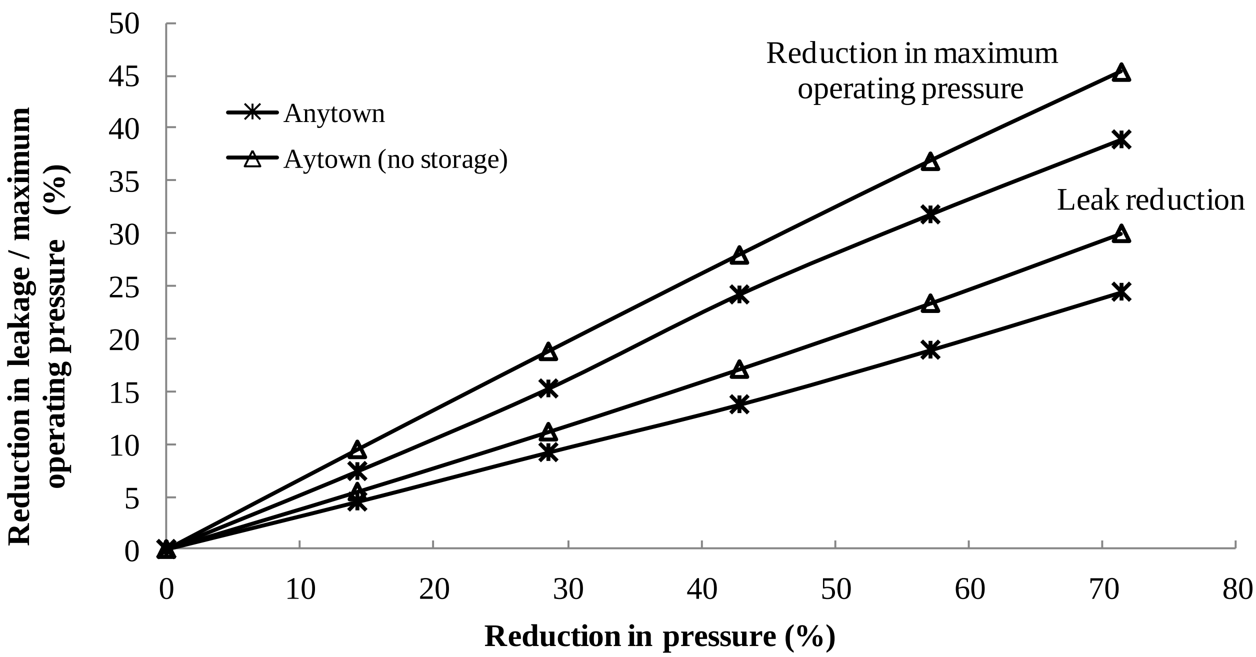


Figure 5.5. Leakage reduction and decrease in maximum operating pressure against reduction in pressure

reduction of about 16% and 19% in average excess pressure for respectively the Anytown with and without storage.

5.4 Delivery pressure and environmental impact

To consider and quantify the environmental impacts associated with a product or process, Life Cycle Analysis (LCA) has been widely used (Park and Seo 2006). Racoviceanu et al. (2007) quantified the total energy use and greenhouse gas (GHG) emissions for the City of Toronto's municipal water treatment system. They found that on-site pumping accounting for the most operational burdens was dominant for contribution of total energy use and GHG emissions, whereas the environmental impacts from chemicals transportation were appraised insignificant. Examination of the impact of WDSs on energy use and the environment and identification of opportunities for reducing energy consumption reveal the way to diminish environment impacts associated with this infrastructure. Clearly reducing pumping energy by replacing with more efficient pumps, decreasing demands through demand management programs, and possible reduction in delivery pressure in WDSs could mitigate the associated environmental impacts due to decrease in energy use.

Rather than conducting a LCA study, the aim here is to preliminarily study to quantify the GHG emissions with respect to energy use and to show how reduction in delivery pressure affects the GHG emissions in WDSs. Two environmental indicators, total energy use and GHG emissions, are generally meaningful for decision-makers to identify the environmental performance of existing water infrastructure. In WDSs, energy is required to pump water into the system. This required energy is supplied from electrical power that is generated from hydroelectric, nuclear, coal, oil, or gas plants. Each unit of generated energy emits a certain amount of GHG. For combined electricity generations, GHG emissions are estimated by summation of GHG emissions from each type generation source. Racoviceanu et al. (2007) reported that the average GHG emission for Ontario's electricity mix was 224 g CO_2 eq/kW h.

A preliminary examination to illustrate how changes in pressure affect the GHG emissions is conducted by revisiting the two Anyown networks. Taking into account the emission factor for Ontario's electricity mix and the computed energy for each scenario, total GHG emissions are depicted in Figure 5.6(a). As indicated in the Figure, the environmental impact is greater for the system with storage implying that the system operating with storage results in more energy use. The reason is that operating system with a storage tank requires operation of the network at high capacity during off-peak time or when the tank level is at the minimum set point. In order to increase the flow, the pressure needs to be increased and the head loss becomes great, consequently, the energy use increases. Figure 5.6(b) depicts the changes in GHG emissions against reduction in pressure. As indicated, no storage scenario results in a higher sensitivity to changes in pressure. According to Figure 5.6(b), a 30% decrease in pressure supplied causes a reduction of about 5.5% and 13.5% in GHG emissions respectively for the Anytown network with and without storage tank.

5.5 Conclusions

Control and reduction of delivery pressure have the benefits of reduced energy expenses, leaks, and environmental impacts. The decrease in head loss due to reduction in pressure is more noticeable in systems with high leakage rate. For systems including low friction pipes and high leakage rate, the energy saving, as a consequence of reduction in pressure, can be expected to be greater. In other words, pressure reduction aimed at leakage reduction and energy saving is more effective in newer systems with smoother pipes. The inclusion of storage tank causes the system energy use increases because of boosting pressure in the system during off-peak time or when the tank level is at the minimum set point. For two case studies considered in this paper, the difference in energy consumption is evident. A 30% reduction in delivery pressure would decrease 10% to 12% in leakage, about 5.5% to 13% of energy consumption, about 16% to 19% of average excess pressure, and about 5.5% to 13% in GHG emissions for the systems under study.

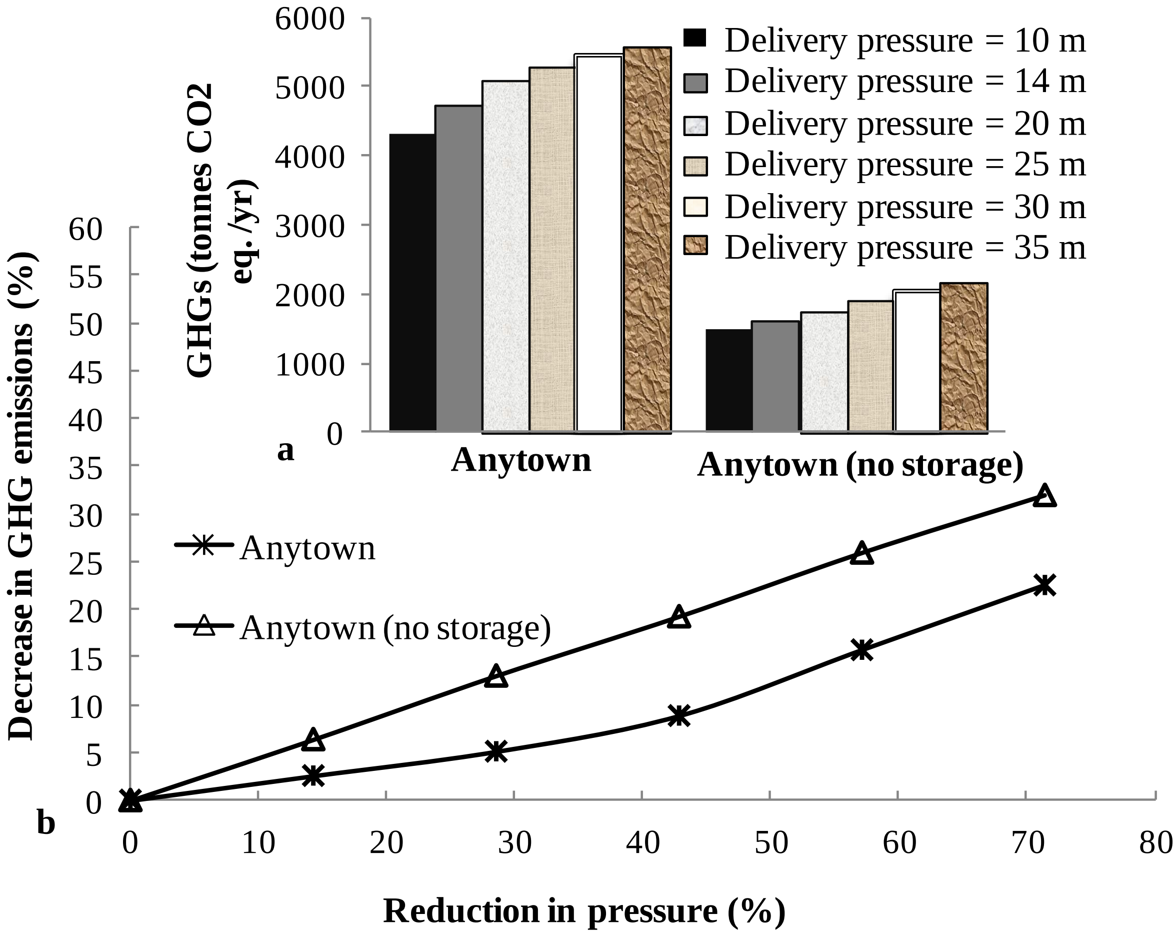


Figure 5.6. (a) GHG emissions for each scenario, (b) Decrease in GHG emissions as a function of reduction in pressure

Reduction in delivery pressure is shown that leaks decrease, however the inclusion of storage capacity decreases the leakage reduction response to the decrease in pressure.

The results of this study are patterned on an analytical expression developed for a single pipe segment and the Anytown network with and without storage tank to highlight the potential impact of high system operating pressure on energy use, leakage, and environmental impacts. The results clearly show that pressure should be limited not only for the purpose of the usual benefits of leakage reduction and possible decrease in pipe bursts, but for the key reason that energy must be paid, both financially and environmentally. Pressure management might be the ideal strategy for one system, but for another system, a better strategy might be to replace/refurbish the pumping system. The findings demonstrate how better control and management of pressures and to rethink about pressure standards setting a criterion for delivery pressure.

Chapter 6

Field Data Based Methodology for Estimating the Expected Pipe Break Rates of Water Distribution Systems

This chapter is based on the paper entitled “Field Data Based Methodology for Estimating the Expected Pipe Break Rates of Water Distribution Systems.” by Vali Ghorbanian, Yiping Guo, and Bryan Karney which is accepted for publication by the *Journal of Water Resources Planning and Management*.

Presented in this paper is a filed data based probabilistic approach to quantifying the expected pipe break rates of water distribution systems. The uncertain demands and variations of the roughnesses of pipes during their service lives are described as random variables. Sample values of these random variables are generated and inputted to a distribution system model to determine the resulting minimum and maximum pressures in Monte Carlo simulations. Based on an estimated break rate-maximum pressure relationship, the sample maximum pressures obtained from a Monte Carlo simulation are transformed to a sample of break rates and the expected pipe break rate can subsequently be determined. The sample minimum pressures are used to gain a better understanding of the distributi on network. This probabilistic approach is used for a part of the City of Hamilton network in Ontario, Canada. The results show that the frequency of occurrence of low pressure events is very small but higher minimum pressure criterion would inevitably increase the expected pipe break rates. Local field data collection is necessary in order to use the proposed methodology, savings resulting from reduced pipe break rates justify costs associated with data collection.

6.1 Introduction

The aim of a WDS is to safely deliver water to all customers of the system in sufficient quantity and quality as economically as possible. To ensure safe and reliable delivery of water across a WDS, the system pressure should generally be maintained between the minimum and maximum acceptable levels. Pressure is a key factor for operating WDSs and must be carefully managed, its excess or deficit can cause hazard or inconvenience. In the standard design of WDSs, it should be ensured that pressures throughout the system are all above a minimum pressure, known as the MPC, when the system experiences the worst-case loading which is considered to be the greater of the maximum hour demand and the maximum day demand plus fire flow (Filion et al. 2007b). The MPC is mainly established to prevent direct contamination and to provide safe drinking water from the source to all individual taps.

For the safe, reliable, and economic operation of WDSs, various local standards for pressure have been established so that sufficient pressure is always provided (but not so high as to cause a danger). While inadequate pressure and lack of pressure monitoring are of public health concerns, excessive pressure is seldom a regulatory criterion but can be problematic as well (LeChevallier et al. 2014). Creaco et al. (2016) provided a methodology for energy and leakage minimisation in which the relationship between demand and service pressure as well as the relationship between leakage and service pressure were assessed first. Then, pumping energy consumption was optimised based on the on-off setting for pumps (the pump settings were expressed as a function of the water level in the tank) associated with different service pressure values. After that, the energy and cost savings associated with the operation of the pumps were assessed. And finally, the way variations in district service pressure affecting leakage, electricity costs for the operation of the pumps, and pipe break rates was assessed. They reported that an average service pressure reduction from 48.23 to 30 m (i.e., 38% reduction in delivery pressure) in the Abbiategrasso district, Italy, leads to reductions of 27% in leakage and 5.3% in pipe break rates as well as energy savings of 53%. It is clear that excess pressure can cause higher burst rates, increased leakage and costs. There is a direct link between system pressure and pipe break rates.

Pipe breaks are inevitable and cause significant water losses. An average of 850 water main breaks occur daily in North America with a total annual repair cost of over $3 billion (http://www.watermainbreakclock.com). Frequent occurrence of water main breaks is a concern of municipal decision makers worldwide. Pipe failures are commonly classified into two main categories: leaks and bursts. Leakage losses that have a flow rate below a certain threshold value are categorised as background leakage (Lambert and Hirner 2000). Water utilities can use leakage as a metric to help evaluate the condition of the water system given that more leakage is often associated with a deteriorated physical condition of the system (Lambert et al. 2013). Thus, reducing leakage and replacing leaky pipes can help reduce pipe break rates. Thornton and Lambert (2006) and Lambert et al. (2013) developed practical prediction methods and empirical equations to estimate the beneficial influences of pressure management on leakage and burst frequency. Water utilities can assess leakage using leak detection and location techniques. These techniques enable water utilities to develop performance indicators to assess water losses and benchmark themselves with other water utilities (Fanner et al. 2007; AWWA 2009; Hughes et al. 2011).

Only a few studies have been conducted to investigate the relationship between pressure and pipe breaks using limited field data (Lambert et al. 2013; Martınez-Codina et al. 2015). Field data is limited because of the historical lack of awareness of the importance of collecting such data. Indeed, a high operating pressure can result in more frequent pipe breaks. While the design strategy of WDSs ensures that pressures in the system during peak demands are not less than a minimum, high pressures during low demand periods, e.g., late night and early morning, and at low elevation areas may cause pipe breaks (Thornton and Lambert 2006; Fanner et al. 2007); but this does not mean that most failures occur during minimum or night flow conditions. Traditionally, pressure management is regarded as an effective way to control leaks and pipe bursts during off-peak hours (Gomes et al. 2011). Several studies have been conducted for the determination of the settings of flow control valves in order to minimize leakage losses (Creaco and Pezzinga 2014). As part of the pressure management process, minimum night flow analysis is conducted to identify factors affecting losses since most of the users are not active during the night and pressures are high throughout the systems (Walski et al. 2006a; Campisano et al. 2012). To perform pressure management, however, the optimization of the number of valves, their locations, and the determination of the optimal adjustment of valve openings are all challenging tasks. The problem of the optimal location and regulation of control valves for leakage reduction and excess pressure minimization has been widely investigated (e.g., Jowitt and Xu 1990; Ali 2014; Creaco and Pezzinga 2014). Reducing pressure using pressure reducing valves (PRVs) might result in only minor energy savings (LeChevallier et al. 2014). Pressure management is frequently considered in system master planning, engineering studies, or hydraulic modeling. Practitioners can better incorporate optimization for pressure management if they have a clear understanding of the relationships between pressure and other distribution system performance indicators such as leakage, breaks, and energy usage (LeChevallier et al. 2014). No effort has been made to develop an indicator so that pipe break rates could be incorporated into the design of WDSs for long-term economic efficiency.

Distribution of pressures in water networks depends on pumping heads and water levels in tanks, pipe diameters and roughnesses, and water demands. Since the number and types of future consumers cannot be exactly determined, the future demands for WDS design are uncertain. The roughness coefficients of pipes are also uncertain due to the aging of pipes during the period of operation. Consequently, the computed pressure, which is an important factor of WDS planning and design, is not certain either. Therefore, to gain a clear picture of how system pressure affects the potential of pipe breaks, demands and the hydraulic conductivity of pipes should be treated as uncertain quantities and modeled with probability distribution functions (PDFs).

To address the uncertainty in WDS design, several models have been developed in order to improve the performance of WDSs at minimum costs (Lansey et al. 1989; Kang et al. 2009; Filion et al. 2007b). Risk-based optimization has been used to incorporate uncertainty in design of WDSs (Tung 1986; Xu and Goulter 1999; Kapelan et al. 2006; Yannopoulos and Spiliotis 2013). In many of the previously proposed methods for determining the optimal design of WDSs, uncertainty was usually incorporated into the problem formulation as a constraint to either maximise the overall WDS robustness, i.e., the probability of satisfying minimum pressure constraints at all nodes in the network; or to minimise the total WDS risk, i.e., the probability of pressure failure at any nodes. Reliability analysis has been conducted for the better design of WDSs. In the reliability-based optimal design of WDSs, reliability indices are generally incorporated into the optimization framework as a constraint in order to maximize the overall system reliability (Babayan et al. 2005; Kapelan et al. 2005; Gomes and Karney 2005; Atkinson et al. 2014). However, these previous studies have focused primarily on the hydraulic performance of systems in an attempt to meet basic delivery requirement.

In the chance-constrained optimization schemes developed for the design of WDSs, pipe breaks are modeled as a stochastic process (Shinstine et al. 2002; Filion et al. 2007b). Pipe break rates are often used as an index for system performance by practitioners, but little efforts have been made to develop methods to better quantify the mean pipe break rates. The mean pipe break rates determined by considering a wide range of uncertain demands and pipe roughnesses may be used as an indicator in design and to help redefine what is optimal.

This study explores the linkage between the maximum operating pressure of WDSs and the expected (or mean) pipe break rates; and at a more practical level, provides a more comprehensive understanding about the effect of system pressure and pressure standards on pipe break rates. The quantified mean or expected pipe break rates serve as an indicator for designers (e.g., it can be incorporated into optimization models as an additional objective function in order to minimize the expected pipe break rates) and it can help a utility to strike a balance between cost and pipe breaks. Uncertain water demands and pipe roughnesses are modeled with a Monte Carlo simulation (MCS) algorithm. The MCS algorithm is used for the computation of expected daily break rates over an extended period. This probabilistic approach is applied to a water network, part of the City of Hamilton network in Ontario, Canada, to determine the expected yearly break rates.

This paper is comprised of three parts. In the first part, prediction models and causes of pipe breaks are briefly reviewed. In the second part, a probabilistic approach for computing expected break rates is presented. In the last part, the probabilistic approach is applied to a case study to quantify the frequency of pipe breaks, the effect of MPC on system pressure and pipe break rates is also examined.

6.2 Prediction Models for Pipe Breaks

Several models for predicting the break rates of water mains have been developed to show break behavior and break patterns. These models can be classified into deterministic and probabilistic categories (Kleiner and Rajani 2001). Deterministic models often use two- or three-parameter equations to derive breakage patterns based on pipe age and diameter, as well as breakage history (Kleiner and Rajani 2001). The division of pipes into groups with similar properties (operational, environmental and pipe type) is often necessary which requires efficient grouping schemes (Makar and Kleiner 2000). Probabilistic models are used to estimate pipe life expectancy or failure probability. The efficiency of rehabilitation planning according to the projected pipe break patterns depends on the quality and quantity of available data (Kleiner and Rajani 2001).

Shamir and Howard (1979) and Walski and Pelliccia (1982) used exponential functions to predict break rates based on recorded break data. Clark et al. (1982) conducted a replacement cost analysis to determine the optimal timing of pipe replacement. Kettler and Goulter (1985) suggested a linear relationship between pipe breaks and age based on a sample of pipes installed within a 10-year period in Winnipeg, Manitoba. Jacobs and Karney (1994) proposed a model, using GIS, to estimate the probability of occurrence of a day with no pipe breaks and the probability of an independent pipe break, defined as a break that occurs more than 90 days after and/or more than 20 m from a previous break. Kleiner and Rajani (2001) reported that the breakage rate of buried pipes could be related to pipe deterioration as well as climatic conditions and soil shrinkage behavior. Achim et al. (2007) developed a neural network model in order to predict the number of breaks/km/year for water mains based on 3 years of recorded data in Melbourne, Australia. Wang et al. (2009) developed five deterioration models that predict the annual break rates of water mains considering pipe material, diameter, age, and length.

Le Gauffre et al. (2010) derived relationships between pipe break rates and climate variables including the number of hours with temperature less than 0°C, the number of hours with temperature greater than 30°C, the maximum number of consecutive days with daily precipitation less than 1 mm, and the maximum number of consecutive days with daily precipitation greater than 1 mm from available data for the duration of 1993-2010 for the Greater Lyon area in France. They concluded that rainfall and freezing duration tend to increase pipe break rates. Kimutai et al. (2015) studied three statistical models, the Weibull proportional hazard model (WPHM), the Cox proportional hazard model (Cox-PHM), and the Poisson model (PM) for predicting pipe failures for the City of Calgary's water network in Canada. The results indicated that WHPM and PM were suitable for metallic and PVC pipes, respectively. From the statistical models, they also showed that physical covariates (e.g., pipe diameter, length) compared with environmental covariates (e.g., temperature) were more critical in affecting the pipe failure rates. These previous studies relied on field data and did not focus on the relationship between pressure and pipe breaks

6.3 Causes of Pipe Breaks

Many factors contribute to the deterioration of pipes that eventually result in pipe breaks. These factors include pipe age, water pressure, temperature, soil corrosivity, water contents of surrounding soils, previous pipe breaks, pipe diameter, pipe material, and construction practices (Wang et al. 2009; Morris 1967; Kleiner and Rajani 2001). A study of the New York water supply system conducted by the US Army Corps of Engineers (1981) revealed that leakage increases the moisture content of the surrounding soils and expedites corrosion. Some studies indicated that more breaks are expected in a water network as temperature decreases (O'Day 1982; Kimutai et al. 2015). The relative weight of contribution of each factor to pipe breaks is still not universally agreed upon in the literature but it has been established that some factors have more weights than others. It was identified that the majority of breaks occur in cast iron (CI) pipes that are the oldest pipes, often installed more that 50 years ago (Pelletier et al. 2003; Singh and Adachi 2013; Kimutai et al. 2015).

Pipe diameter was identified as one of the factors affecting pipe failure rates (Clark et al. 1982; Berardi et al. 2008). It was reported consistently in the literature that small diameter pipes have a greater number of failures than larger diameter pipes (Berardi et. al 2008; Wang et al. 2009; Kimutai et al. 2015). The majority of statistical models used for predicting water main breaks considered pipe age as one of the important factors (Berardi et al. 2008; Wang et al. 2009). It was reported by many researchers that pipe failure varies with pipe age in accordance with a bathtub curve (Andreou et al. 1987; Kleiner and Rajani 2001; Singh and Adachi 2013). Environmental factors such as precipitation, soil conditions, frost and traffic loading, and the quality of external groundwater have been identified as factors contributing to the failure rate of pipes in water networks (O'Day 1989; Rajani and Zhan 1996; Kleiner and Rajani 2001; Kimutai et al. 2015). Generally, low temperature and rainfall tend to increase pipe break rates (O'Day 1982; Brander 2001; Kimutai et al. 2015).

As explained above, there are many factors influencing pipe breaks (e.g., pipe deteriorations, properties of pipes, and environmental conditions). But, without pressure water would only be nominally present in a distribution system (no leak and pipe breaks would occur) and the system would be unable to deliver water to the users. Thus, pipe breaks mostly occur due to pressure. Few studies have been conducted to demonstrate the effects of pressure management on reducing pipe break rates. Pearson et al. (2005)

illustrated that reducing pressure by approximately 50 m during high pressure events with the installation of control valves in a real system, part of a large network in the UK, causes a dramatic reduction in burst frequency: the burst frequency dropped from 3 per month to one every six months on average. They also established a relationship between relative burst frequency and pressure based on collected data from 50 WDSs (Eq. (2.2)).

Thornton and Lambert (2006) analysed data collected from 21 utilities of 11 countries on breaks (or repairs) before and after pressure management and concluded that the percentage reduction in pipe bursts often exceeds the percentage reduction in average maximum operating pressures. For example, in Halifax, Canada, 18% reduction in average maximum operating pressure caused 23% reduction in pipe breaks. Lambert et al. (2013) summarized some works done on the prediction of the benefits of pressure management in WDSs and concluded that reduction of excess pressures could have a substantial influence on reducing bursts (e.g., 1% reduction in average pressure causes 1.4% reduction in burst frequency). They also developed a relationship between burst frequency and pressure from data collected by an Australian utility:

$$BF = BF_{\text{npd}} + A \times (AZP_{\max})^{N_3} \tag{6.1}$$

In Eq. (6.1), BF is the burst frequency, BF_{npd} is pressure-independent burst frequency, $AZP_{\max}$ is the maximum pressure at the average zone point (AZP), A is a coefficient influencing the slope of the pressure-dependent part of the relationship, and N_3 is an exponent recommended to be close to 3. The value of BF_{npd} can be estimated as the lower boundary of the data points in a plot depicting burst frequency as a function of average zone night pressure (Lambert et al. 2013). Martınez-Codina et al. (2015) presented a new methodology based on a maximum pressure indicator in order to identify the range of suitable of maximum pressures that most likely reduces pipe breaks. They concluded that the maximum pressure should have an upper limit to reduce the probability of pipe breaks (e.g., the upper limits of 79, 96, and 70 m for 3 case studies in Madrid, Spain where presented in that study). But they suggested that as pipes age and deteriorate, these thresholds need to be updated and the model results may change with time.

6.4 Expected Pipe Break Rates

High pressures coupled with the physical and environmental conditions of pipe networks result in an increase in water main breaks. The increase in pipe break rates causes increase in operation and maintenance costs, increase in loss of water and social costs such as loss of service, disruption to traffic, disruption to business and industrial processes, and disruption of residential areas. Annual break rates (breaks/km/year) are often used as one of the controlling criteria in rating the conditions of water mains. Technically, to determine the effects of parameters [e.g., pipe material deterioration, external loads (frost, traffic, and temperature), and quality of pipe installation] affecting pipe breaks, field data are necessary but collection of sufficient data is a challenging task to water utilities. Sophisticated techniques must be used to sift through collected field data in order to identify meaningful information on the status of a distribution system (Speight 2008). Thus, break data is often segregated into homogenous groups of materials, diameters, ages, environmental and operational conditions, and mechanisms of failure to better identify breakage patterns (Kleiner and Rajani 2001; Martins et al. 2013).

The only parameter that can be easily measured in WDSs is water pressure. A relationship between pressure and pipe break rate can be extremely useful in implementing pressure management strategies and even in the design of WDSs. A pipe break rate function that maps a predictor variable (i.e., maximum pressure h) at a system to a unique average level of pipe break rate (Br) forms the basis for calculating the expected pipe break rates (Figure 6.1). The hypothetical brake rate function depicted in Figure 6.1(b) may be valid for pipes with homogeneous properties (e.g., the same type of pipe) and under similar environmental conditions. Of course pipe materials, the environment in which the pipes are laid, and the operating characteristics of the system influence the likelihood of pipe breaks as well (Kleiner and Rajani 2001). Ideally, pipe material, size, age, type of bedding, soil characteristics, operating pressures, water

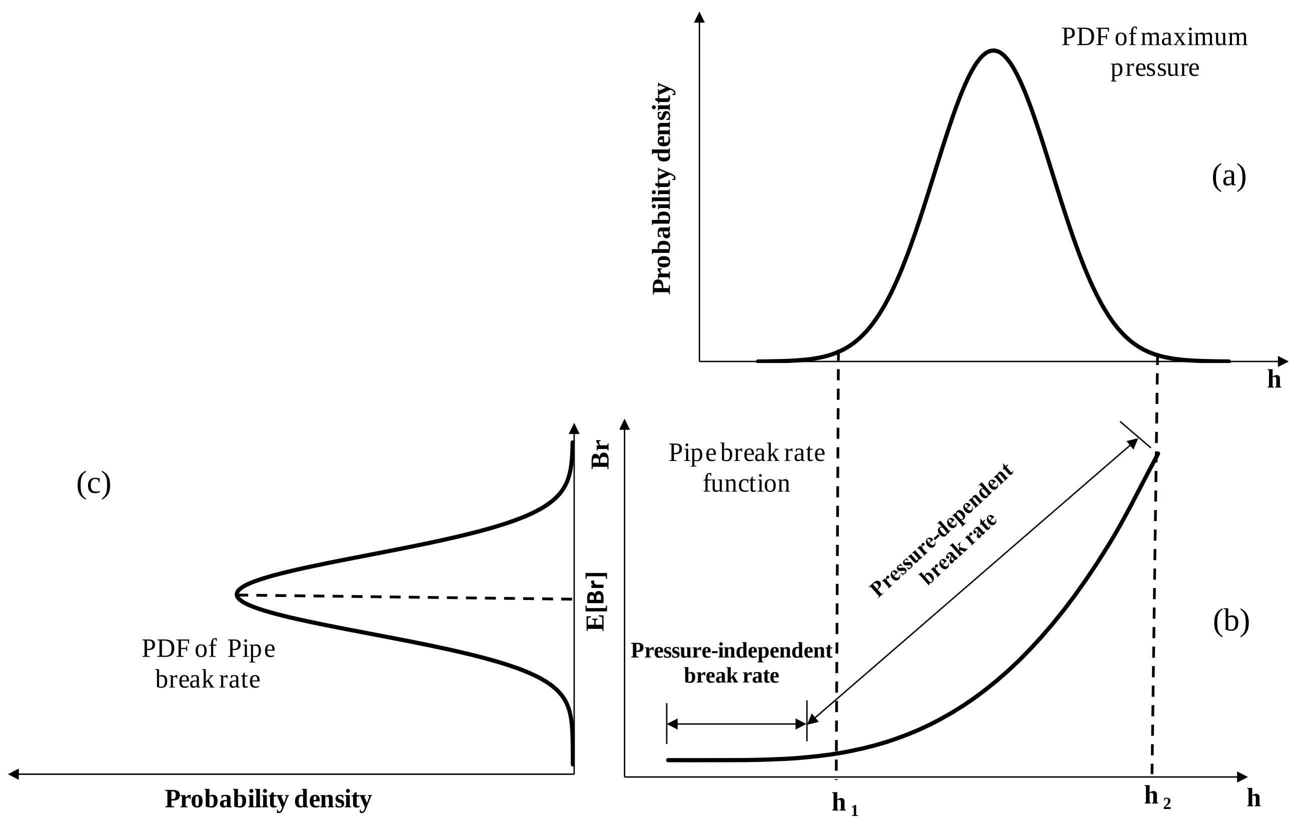


Figure 6.1. PDF of pressure, continuous break rate function, and PDF of break rate

temperatures, time, place and type of historical breaks should be available to derive breakage patterns based on all the factors influencing pipe bursts (Makar and Kleiner 2000). However, in many cases, only partial sets of data exist. The failure rate of a distribution system can be assumed to have a relatively low value until a particular pressure is exceeded and then the failure rate increases rapidly for small increases in pressures (Lambert et al. 2013). This is as depicted in Figure 6.1(b).

The brake rate curve (as depicted in Figure 6.2) can move to the left over a period of years and also seasonally due to other influential parameters contributing to pipe breaks (Lambert et al. 2013). Moreover, the zone of pressure-independent failure rate may vary in WDSs due to the condition under which pipes are laid. Other factors such as pipe diameter, soil condition, and the quality of installation may be included in Figure 6.2

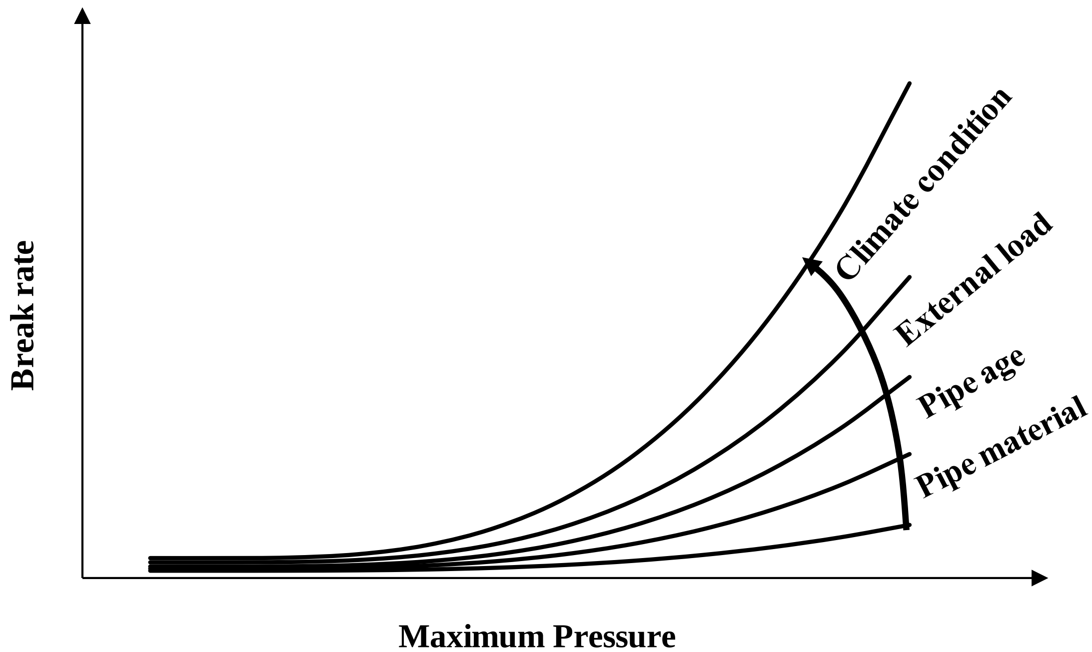


Figure 6.2. Hypothetical shapes of break rate functions influenced by factors contributing to pipe breaks

in order to describe more accurately the relationship between pressure and break rate. Thus, for each pipe material and under each of the environmental and operational conditions, the pipe break rate function can be represented by a unique curve. To develop such a pipe break rate curve, the maximum pressure indicator (the maximum pressure at the AZP of a pressure zone) is often used since the maximum pressure has been known to be a key control parameter for reducing the number of pipe breaks in pressure management (Lambert et al. 2013; Martınez-Codina et al. 2015).

Two simplifications are necessary for developing a pipe break rate function. First, no attempt is made to associate a predictor variable (i.e., maximum pressure) to a precise level of break rate observed in the field during a particular period of time. In reality, if a particular level of the predictor variable is encountered frequently (e.g., pressures above a threshold level), break rates will vary in each instance depending on the specific circumstances that exist at the time the system experiences those high pressures. The break rate function only associates an average break rate to each level of the predictor variable. The second simplification is that maximum pressure is assumed to be the only predictor variable controlling pipe break rates.

Once a pipe break rate function is established, an estimate of the expected pipe break rate is calculated by integrating the product of the continuous break rate function with an empirical PDF of the maximum pressure over the possible range of maximum pressures as

$$E[Br] = \int_{h_1}^{h_2} Br(h)\, f(h)\, dh \tag{6.2}$$

where $E[Br]$ = the expected pipe break rate (breaks/km/day) during the service life of a water distribution system; h_1, h_2 = the lower and upper limits of the maximum pressures that the system may encounter in its life time (m); $Br(h)$ = the break rate function that associates maximum pressure h to an average pipe break rate (breaks/km/day); and $f(h)$= the probability density function of maximum pressure. Figure 6.1 graphically represents the above procedure. Note that the pipe break rate here is expressed as a function of random variable h, i.e., $Br(h)$. Eq. (6.2) theoretically gives the expected value of the pipe break rate. Based on Eq. (6.2), numerical integration can be carried out once the break rate function $Br(h)$ and $f(h)$ are both known numerically in order to determine the expected pipe break rate.

The break rate function as shown in Figure 6.1(b) has a logical and reasonable shape but its exact form associated with different physical and environmental conditions of WDSs remains a topic for future research. The shape of a break rate function will vary from one system to the other, and can be established using observed data. It is beyond the scope of this paper to determine the exact shape of this pipe break rate function for a specific system. However, as long as the break rate function follows a similar logical shape, the probabilistic approach presented in this paper can be used and a reasonable estimation of the expected pipe break rates can be provided.

Eq. (6.2) provides a simple way of estimating the expected pipe break rates considering wide ranges of loadings and influencing factors (e.g., uncertain demands and pipe roughnesses). The estimated expected pipe break rates can be used as an indicator for the development of economic models to determine the financial benefits of reducing pressures. In essence, Eq. (6.2) itself does not include all the factors influencing pipe breaks rather the pipe break rate function depicted in Figures. 6.1(b) and 6.2 is used to describe the influence of a variety of factors, e.g., pipe characteristics, environmental conditions, and system pressures contributing to pipe breaks. In practice, pipe break rate functions can be developed based on historical pressure data obtained from SCADA systems. However, in the absence such data for a specific system, pipe break rate functions developed for other similar systems may be used. Limited local data may be used to verify or fine-tune the pipe break rate functions developed for other similar systems. The main objective of this study is to develop a procedure for determining an indicator that can be incorporated into the design of new networks using collected data on pipe break rates. The need for collecting adequate data on pipe breaks is also highlighted when the usefulness of the indicator is explained.

6.5 Simulation Algorithm for Expected Pipe Break Rates

Expected pipe break rates as defined previously can be computed using a MCS algorithm. Demand and pipe roughness often vary over a long time period (Bao and Mays 1990). It is assumed that a WDS, during its lifespan, experiences the entire ranges of possible demands and pipe roughnesses in order to determine the system pressures under a wide range of design loads (Bao and Mays 1990; Kapelan et al. 2005). In other words, the expected pipe break rates here is computed over the range of possible high pressures considering uncertain design parameters (i.e., demand and roughness coefficient) that a WDS may experience in a typical day (that is one of the most strong assumptions made in the design process) to determine an additional indicator that can be used for the design of WDSs. First, in each run of an MCS, one demand for each node and one roughness for each pipe are generated and entered into the network solver EPANET2 (Rossman 2000) to compute pressure heads under a typical diurnal demand pattern. As indicated in Table

6.1, an MCS is comprised of N independent runs for M nodes of a network. The extended period simulation (EPS) should be used to ensure that pressures can stay above the MPC during peak demand times (the duration of simulation in each MCS run is chosen to be 24 hours). At each run, the computed minimum pressure at each node during a 24-hr simulation is compared to the MPC to ensure that nodal pressures are always above the MPC. Of course, the supply pressure should be increased to ensure for satisfying the MPC. The maximum nodal pressure at each run of MCS for each node (e.g., $h_{2,1}$ in Table 6.1) is also recorded. These maximum pressures are used to derive the PDF of the maximum pressure. After reaching the end of the last run, the PDF of maximum pressures is derived and the expected pipe break rates $E[B]$ is numerically computed based on Eq. (6.2) incorporating the associated pipe break rate function.

The tasks performed in an MCS run are shown in Figure 6.3. First, base demands and pipe roughnesses are generated according to their corresponding PDFs. The base demand usually equals the average day demand calculated from monthly or quarterly

Table 6.1. Monte Carlo simulation runs and the calculated maximum pressure

	Node					
Run	1	2	3	4	. . .	M
1	$h_{1,1}$*	$h_{1,2}$	$h_{1,3}$	$h_{1,4}$		$h_{1,M}$
2	$h_{2,1}$	$h_{2,2}$	$h_{2,3}$	$h_{2,4}$		$h_{2,M}$
3	$h_{3,1}$	$h_{3,2}$	$h_{3,3}$	$h_{3,4}$		$h_{3,M}$
.						
.						
.						
N	$h_{N,1}$	$h_{N,2}$	$h_{N,3}$	$h_{N,4}$		$h_{N,M}$

*Note $h_{1,1}$ is the maximum pressure observed for node 1 during 24-our simulation of the first MCS run. Those maximum pressures are later on used as a sample for estimating the PDF of maximum pressure.

meter readings and billing records. In WDS design, demand patterns obtained by multiplying base demand by demand multiplication factors are used to simulate the behavior of a quasi-dynamic system over a period of time during which demands and boundary conditions change with respect to time. For the proposed MCS, the PDF of each uncertain parameter can be determined on the basis of measured data of the system. However, because of the scarcity of field data, hypothetical PDFs are used for uncertain design parameters. According to the authors' review of literature, the following limitations exist in using field data:

1- In any real system there can be hundreds or thousands of unknowns (i.e., roughness coefficient for each pipe and demand at each node) and only a relatively small number of field observations. Wu et al. (2002) have observed that when the number of unknowns greatly exceeds the number of useful observations, there is little confidence in the calibration results. This is because there are too many different parameter values that would produce results close to the observed values (Walski et al. 2006b).

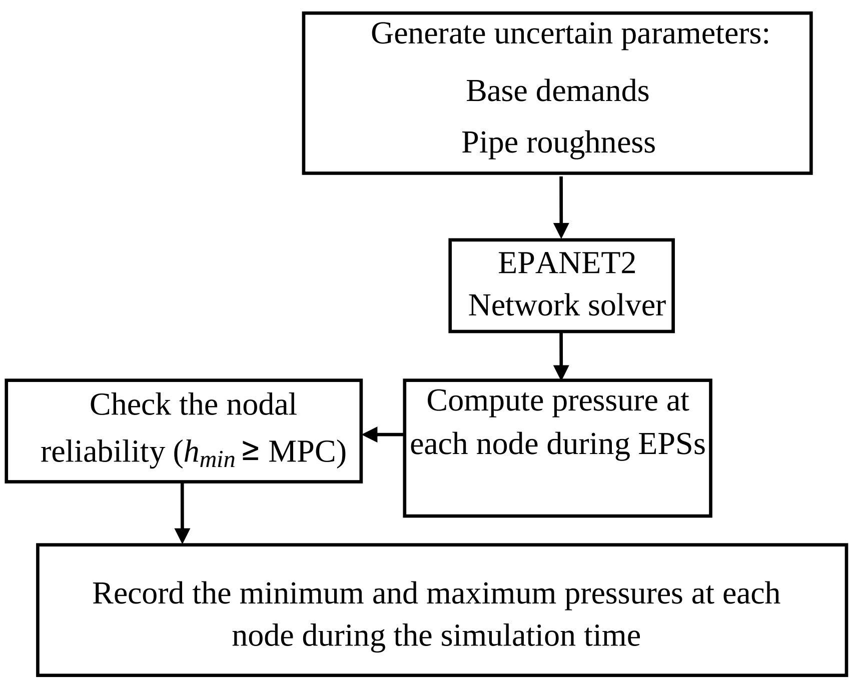


Figure 6.3. Steps to compute pressures in an MCS run

2- Consumer demands occur along pipes at many separate locations. The nodal water demand is an aggregation of the consumption of individual houses and buildings in the vicinity and is allocated to a demand node at a junction of pipes (Kang and Lansey 2009b). Even with this aggregation, due to the complexity of network systems with spatially distributed user types and lack of available field data, individual node's demand estimation is a challenging task.

3- Field measurements from supervisory control and data acquisition (SCADA) systems play a critical role in determining the nodal demands and pipe roughnesses. Given the limited number of monitoring locations, the topographical and spatial distributions of meters in networks strongly influence the quality of the estimation of demands (Kang and Lansey 2009b). If some portions of the system are insufficiently measured, the demand and roughness estimates and subsequently model predictions in those portions would be inaccurate and contain a high degree of uncertainty.

4- To reduce the number of unknowns, the roughness coefficients for a subset of pipes are assigned the same value according to the pipe's age, material, diameter and relative locations (Mallick et al. 2002). This could be achieved if the precise age of each pipe in the network is known. However, determining the year when each pipe segment was laid down can be fairly tedious or sometimes impossible particularly for older pipes in a real water distribution system.

5- Field data collection for the estimation of pipe roughness is generally performed by a fire flow test, which is intended to cause large head losses so as to make the system sensitive to the roughness coefficients (Ormsbee and Lingireddy 1997). Data collection for this purpose is conducted rarely, e.g. every 5 years, since pipe roughness changes slowly.

In many previous studies, demands and roughness coefficients are assumed to follow normal distributions with known means and standard deviations (Lansey et al. 1989; Gomes and Karney 2005; Kapelan et al. 2005; Filion et al. 2007a). In this work, water demand and pipe roughness are assumed to be normally distributed as well. These assumptions were made for simplicity reasons. In real systems, demand pairs may tend to be perfectly correlated in water networks implying that all users react simultaneously to normal and peak demand conditions. In this case, nodal demands are dependent of each other. To generate correlated random samples of demands, a procedure suggested by Iman and Conover (1982) can be used. The focus of this paper is not to determine the impact of correlation between demand pairs on pipe break rates. The methodology developed and presented here can certainly be used to handle correlated random variables.

In the second part of the MCS, the EPANET solver is used to compute nodal pressure heads. The minimum pressure head at each node in a 24-hr simulation is used to determine the nodal reliability index. Hydraulic failures resulting from inadequate delivery of flow and pressure head at demand points decrease the reliability of a WDS (Bao and Mays 1990). A hydraulic failure is defined as the occurrence of the scenario that a given demand node receives insufficient flow rate under inadequate pressure head. In most studies conducted to examine WDSs' reliabilities, the hydraulic reliability index was defined as the probability that pressure at each node is above the MPC given that adequate demand is supplied (Bao and Mays 1990; Gomes and Karney 2005; Atkinson et al. 2014). In this work, nodal reliability is defined according to Bao and Mays (1990) as

$$RN = P(H_s \geq MPC | Q_s = Q_r) \tag{6.3}$$

where RN = the nodal reliability, H_S = the supplied pressure head, Q_S = the supplied demand, and Q_r = the required demand. Because the hydraulic simulator EPANET 2 always satisfies demand but not necessarily pressure head, this approach automatically assumes that the water demand is satisfied. Therefore, hydraulic failure is considered to be due to inadequate pressures at demand points. Of course, pressures supplied by WDSs can sometimes be lower than the requirement under deficient service conditions (e.g., with pipe outage, power failure at pumping stations, and under fire flow conditions) and these conditions would affect the system reliability. Under deficient conditions, pressure driven analysis (PDA) should be performed to accurately predict the system response with pressure deficits (Wu et al. 2009; Jun and Guoping 2013). If, however, the nodal reliability (Eq. 6.3) is maintained at 100%, i.e., pressures at or above the MPC are supplied for all nodes at all times, demand driven models such as EPANET 2 can be used to simulate the performance of water networks. Pipe failure also affects system reliability but the focus of this study is not to determine the system reliability, rather the purpose of using reliability index here is to have an indicator for ensuring that minimum pressure at each node is above the MPC as is required in the general design of WDSs. The same method has been used by other researchers for conducting their studies (e.g., Babayan et al. 2005; Kapelan et al. 2005 and 2006; Bao and Mays 1990). In the last part of the simulation, after all individual MCS runs are performed, the PDF of maximum pressure is determined and the expected pipe break rates is computed based on Eq. (6.2).

6.6 Case Study

The above presented probabilistic approach is applied to a part of the City of Hamilton's distribution system in Ontario, Canada. This network consists of 240 nodes, 1 source, and 273 pipes (Figure 6.4). Three parallel pumps are connected to the source. The average total system demand is 593 l/s and 27 nodes have no demand. To model diurnal fluctuations, a diurnal demand pattern depicted in Figure 6.5 is applied to all nodes. Three identical pumps in parallel with the characteristic curve defined by $H = 66.7 - 3.8\times10^{-4}Q^2$, where H is in meters and Q is in l/s, are considered. The on–off pump controls are specified for pump operations. Figure 6.6 presents the percentages of pipe length within different categories of diameters in the Hamilton network. The vast majority of pipes have diameters less than 450 mm (86.7%). For the case study presented here, all pipes were grouped together and one pipe break rate function was considered. It may be practical to assign different break rate functions to different groups of diameters to obtain a proper estimate of expected pipe break rates if sufficient data on pipe breaks is available. Since there is no data on pipe breaks for the Hamilton network, the pipe break rate function (Figure 6.7) was adapted from Lambert et al. (2013).

Random number generation was used to generate uncertain design parameters, i.e. demands and pipe roughnesses. An MCS is comprised of 1000 (i.e., N = 1000 in Table 6.1) independent runs each with generated random parameter values and run for 24 hours.

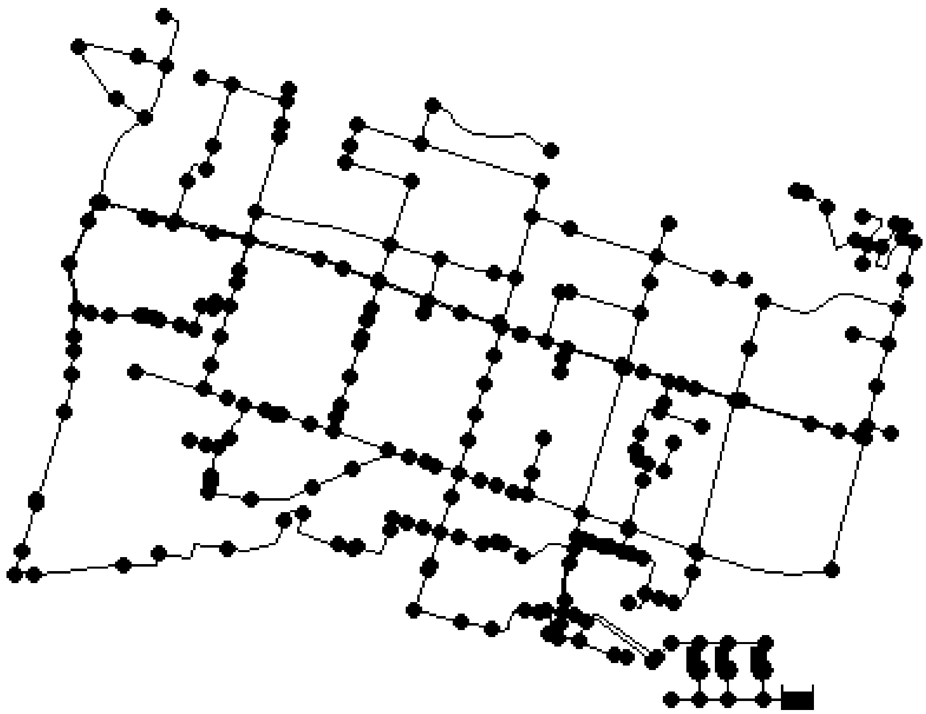


Figure 6.4. The north-east part of the Hamilton network

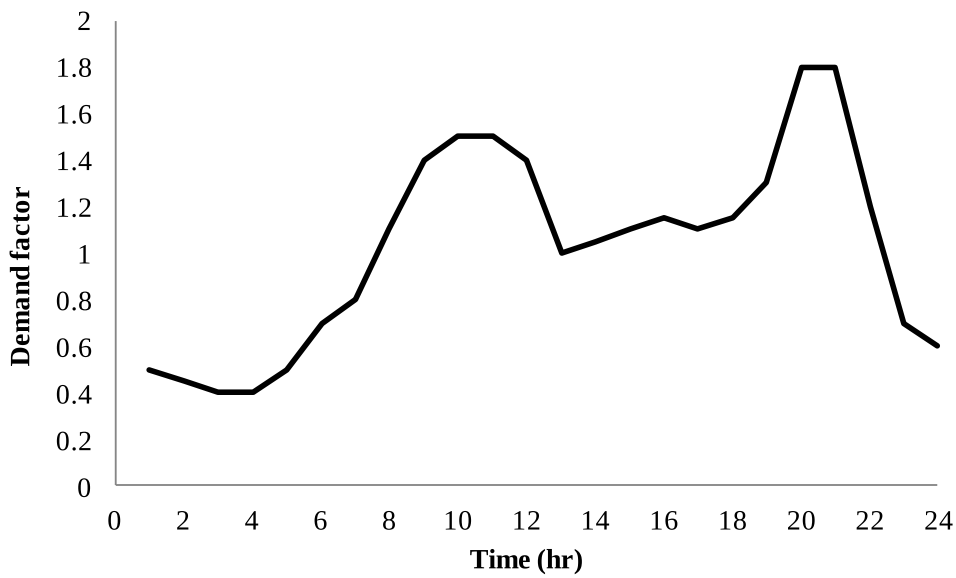


Figure 6.5. Diurnal demand pattern for water consumption

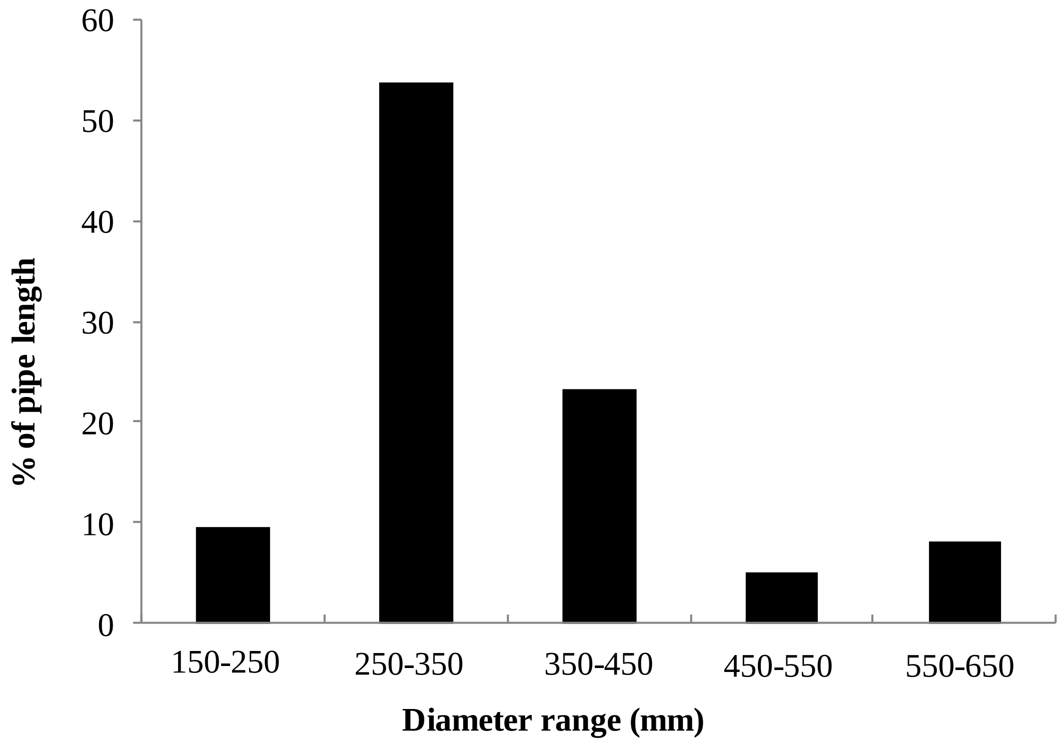


Figure 6.6. Percentages of pipe length in different diameter categories

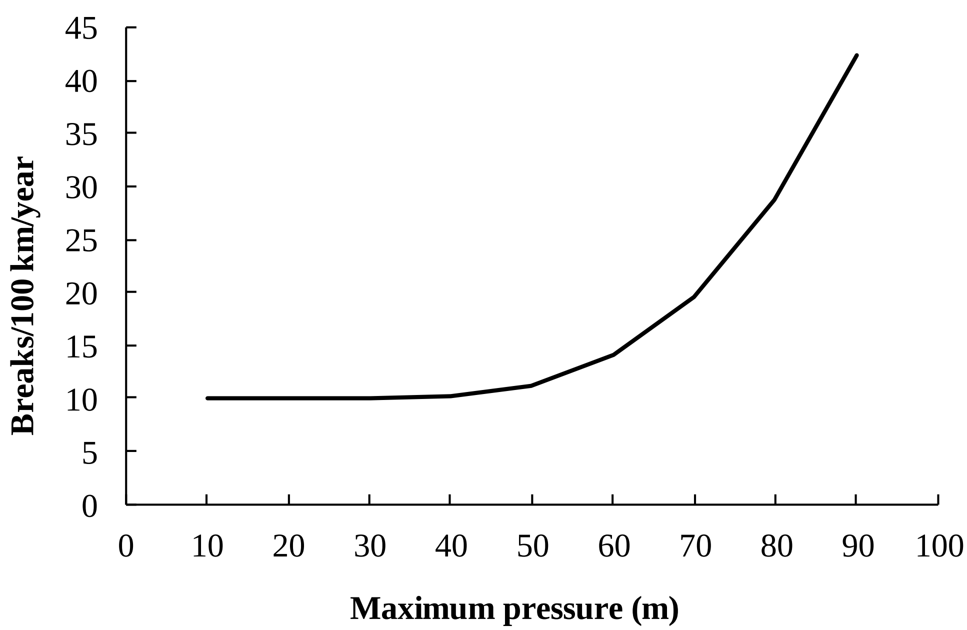


Figure 6.7. Pipe break rate function (adapted from Lambert et al. 2013)

To determine the number of runs required in an MCS, samples of different sizes of uncertain parameters were first generated. Then, the mean and standard deviation of each sample were computed. The results revealed that the statistics of samples and simulation results change very slightly when the size of samples exceeds 1000, therefore 1000 was selected as the appropriate sample size. The mean of the normal distribution for a demand is set to be the base demand value specified at each node and the standard deviation is set to be 10% of the mean (therefore, the coefficient of variation C_v = 0.10). The pipe roughnesses are also assumed to follow Gaussian distributions with means equaling the corresponding initial values specified for each pipe and standard deviations equaling 5% of the corresponding mean values. This 5% is selected based on the fact that the generated roughness coefficients would fall between 70 to 130, which is the expected range of variation of roughness coefficients of pipes during their service lives. The C_v of 0.10 for demands is selected following the same reasoning. All nodal demands are considered to be independent of each other. Also, nodal demands and pipe roughnesses are assumed to be independent of each other. Because real water systems in developed countries have reliabilities greater than 0.999 (Bao and Mays 1990), the size of pumps for the case study are selected to achieve the nodal reliability of 100%.

For the first MCS, the MPC of the Hamilton system is set at 30 m. The simulation results (in the form shown in Table 6.1) were used to determine the frequency distribution of the maximum pressure. Multiplied by the break rate function, the frequency distribution of the maximum pressure was transformed to the frequency distribution of pipe break rates. In Figure 6.8, the resulting histogram and cumulative distribution function (CDF) of pipe break rates of the Hamilton system are shown. The expected pipe break rate for this case as determined from the resulting histogram or the CDF is 25 breaks/100 km/year. From the CDF curve depicted in Figure 6.8, the probability of the break rate being less than 20 breaks/100 km/year was found to be 0.11.

One of the main objectives of WDS design is to ensure that pressures across a network are always between the minimum and maximum acceptable limits. The average maximum or minimum pressure in the Hamilton distribution system over MCS runs are

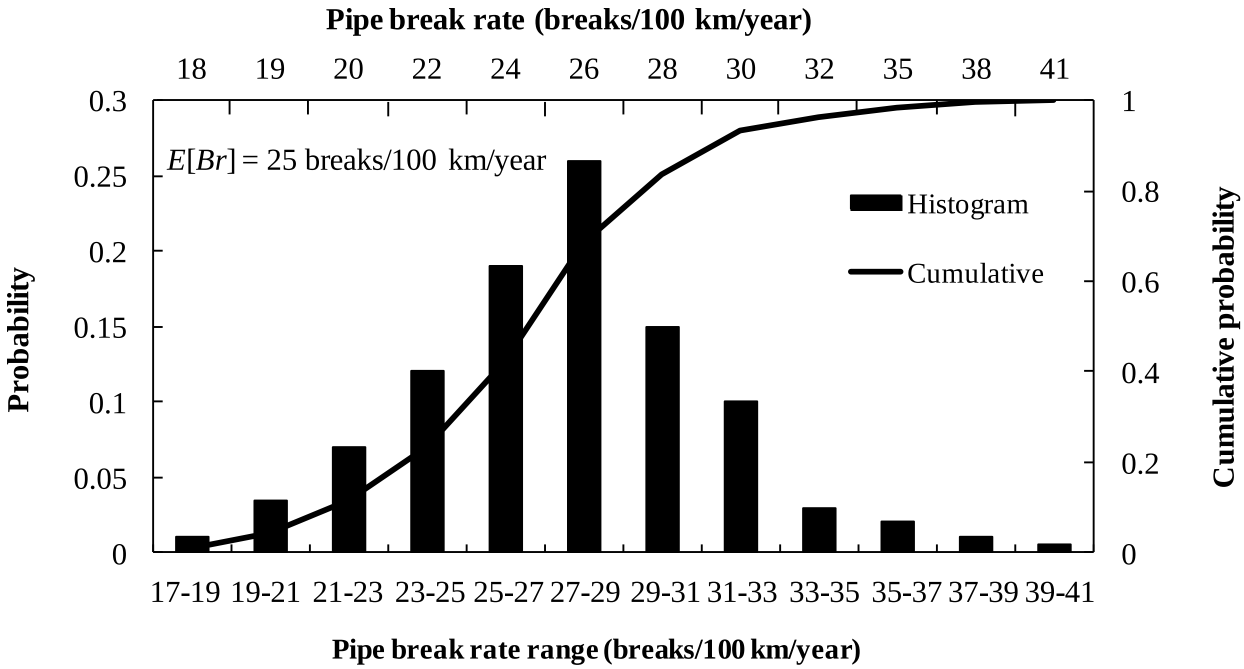


Figure 6.8. Histogram and CDF of the pipe break rate of the Hamilton network

also calculated. The average maximum and minimum pressures were found to be 76.5 m and 43 m, respectively. The value of the average minimum pressure clearly indicates that although an MPC of 30 m is enforced for all simulations, the average minimum pressure within the system is still much higher than the MPC considered at the design stage. This finding may motivate utilities to extend pressure management to include strategies that decrease the operating pressure by reducing pressure standards. This issue has not been investigated yet.

6.7 Expected Pipe Break Rates, System Pressure, and MPC

The implicit objective of enforcing an MPC is to provide adequate flow to control fire that might erupt anywhere in the network, to possibly prevent low or negative pressures during transient events, and to ensure customer satisfaction with the prevention of low pressure events. In order to achieve reasonable operating conditions, different local standards for water pressures have been set, e.g., the MPC is 14 m in most provinces of Canada while it is 20 m and 10 m in Australia and the UK, respectively (Ghorbanian et al. 2015b). There are no universally acceptable or established rules or guidelines for the specification of MPCs (Ghorbanian et al. 2015b). Thus, there may be the possibility of revising the MPC for some WDSs. Reduction in the MPC may cause consumer complaints and make the system more susceptible to low/negative pressures during transient events; however, the beneficial effects of lowering MPC include decreases in water demands, energy use, leakage, and the frequency of pipe breaks. Thus, although it may not have been part of the original intent, there is a connection between the MPC and the pipe break rates. Quantification of this connection would be helpful to highlight the benefits of MPC reduction. A case study in the US indicated that about one-third of 36 utilities considering pressure management practices maintained average pressures greater than 50 psi at low pressure locations, suggesting potential for pressure reduction through pumping at lower heads (LeChevallier et al. 2014). Investigation about the consequences of MPC reduction on consumer satisfaction and hydraulic performance of WDSs is definitely needed but is beyond the scope of this paper.

The simulation results considering two values of MPC for the Hamilton network are summarised in Table 6.2. As expected, higher MPC results in higher expected pipe break rates because the expected break rate is assumed to be a monotonically increasing function of pressure. The results in Table 6.2 also suggest that when the MPC is reduced from 30 m to 20 m, the average minimum pressures, average maximum pressures, and the expected pipe break rates would decrease by 24.4%, 12.4%, and 24%, respectively. Clearly from Table 6.2, the probability of the break rate being less than 20 breaks/100 km/year increases from 0.11 to 0.91 when the MPC is reduced by 1/3.

Figure 6.9 depicts the CDFs of maximum pressures for the Hamilton distribution system when two values of MPC are considered. As expected, reduction in the MPC causes a decrease in the system pressures during low flow conditions. The probability of the maximum pressure being less than 70 m is 0.9 when the MPC is 20 m while this probability is 0.1 (which means that the probability of pressure being higher than 70 m is 0.9) when the MPC is set to be 30 m. This indicates the risk of significantly increasing pressures all across the system that will be caused by the use of high MPC. Another concern about the design of WDSs is how frequently low pressures occur. Although the

Table 6.2. Expected pipe break rates, average minimum and maximum pressures, and probability of the break rate being less than 20 breaks/100 km/year for two values of MPC

Indicators	MPC = 30 m	MPC = 20 m
Expected pipe break rate (breaks/100 km/year)	25	19
probability of the break rate being less than 20 breaks/100 km/year	0.11	0.91
Average maximum pressure (m)	76.5	67
Average minimum pressure (m)	43	32.5

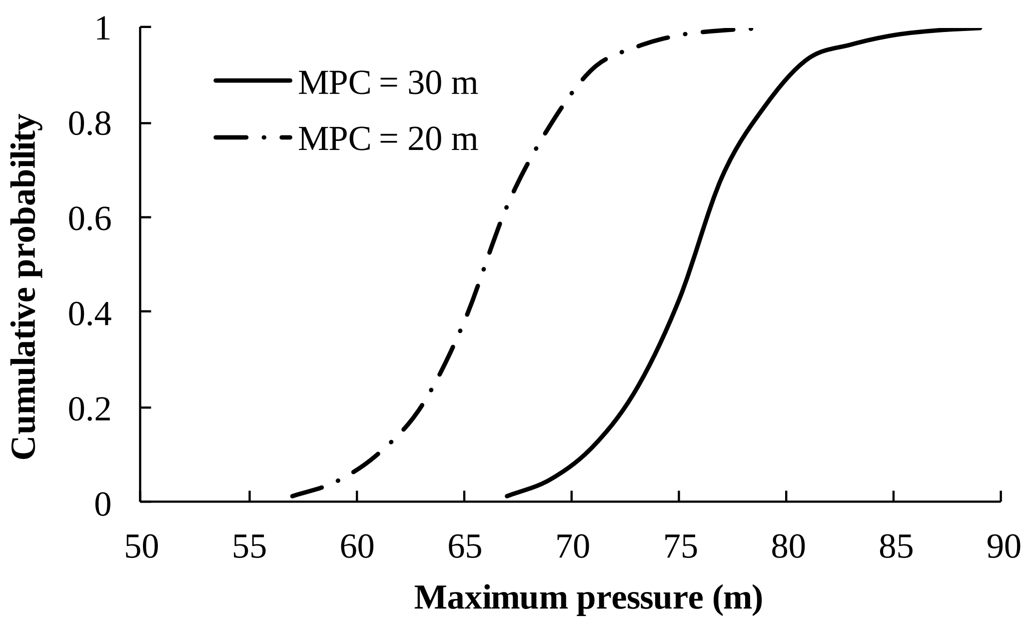


Figure 6.9. CDFs of maximum pressures for two values of MPC

MPC is enforced in the design of WDSs, low pressures close to the MPC may not occur that frequently as is usually expected. Quantification of the frequency of occurrence of low pressures is important but no such studies have been conducted. In Figure 6.10, the probability distribution of the minimum pressure occurring within the 24-hr simulation period is depicted for the Hamilton network. In the simulation, the MPC is set to be 30 m. It was found that the probability of occurrence of the minimum pressure between 30 m and 32 m is only 1%. Figure 6.10 clearly shows that for the majority of the time, minimum pressures are much higher than the MPC of 30 m in the Hamilton network. This confirms that the pressure in a WDS may be considerably higher than what is required most of the time. However, design standards require that the MPC is met all across the network even though the actual occurrence of pressures as low as the MPC is very rare.

6.8 Shortcomings and possibilities

At the present, the above-presented approach for determining the expected pipe break rates associated with system operating pressures remains theoretical because of the scarcity of field data. If a relationship between pipe break rates and maximum operating pressures can be established for different physical and environmental conditions of WDSs, development of economic models would be possible to determine the financial benefit of reducing pressures. This gives support to the argument for collecting data in different networks in order to determine the exact form of the break rate function. Although many large municipalities are now collecting demand and pipe break data with the SCADA systems, the system pressure also need to be recorded in order to determine the pipe break rate function. A more realistic approach would be to estimate the leakage time-instant and then the pressure at the leakage location is estimated using a network model. An estimate of the leakage time-instant is useful for diagnosis purposes as it may clarify the leakage causes (Boracchi et al. 2013). Also to determine whether a pipe break occurrence is due to high pressures, field work is required for determining whether the failure is due to internal or external causes. Even more heartening is the fact that municipalities are setting up geographic information systems (GIS) to integrate networks' data for improved visualization and graphical querying of the implicit and explicit

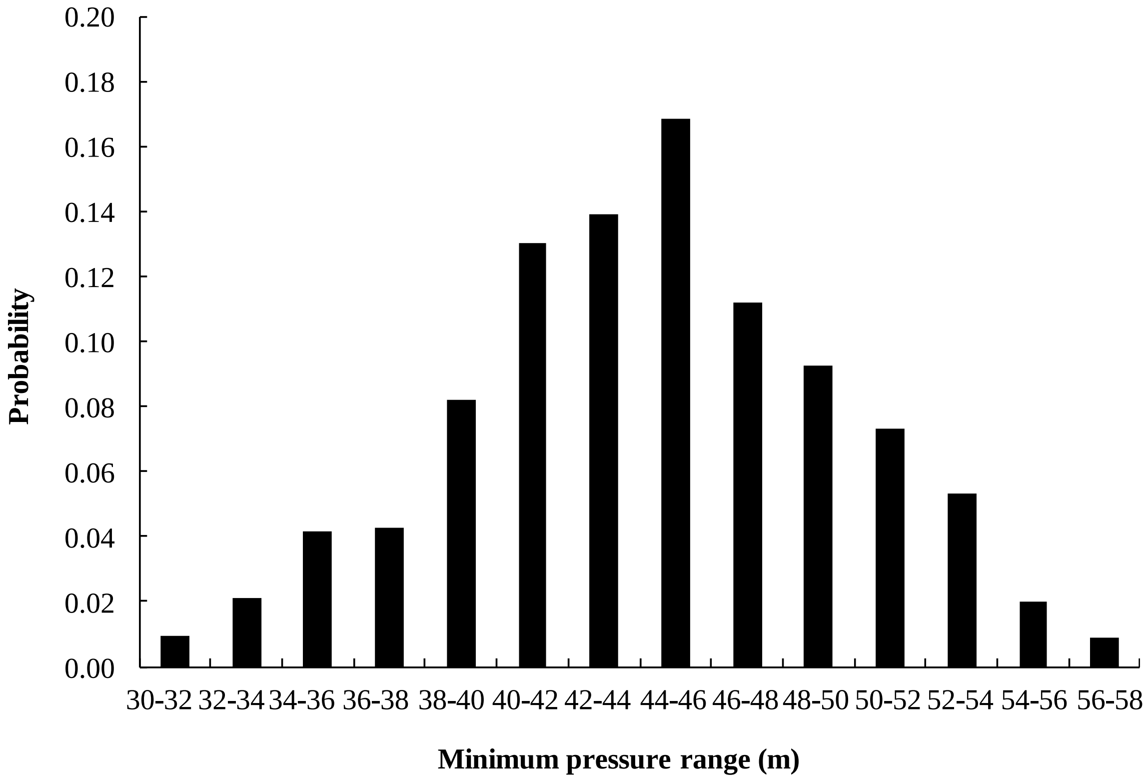


Figure 6.10. Probability distribution of the minimum pressure of the Hamilton network

knowledge accumulated during the maintenance and management of WDSs. With the extensive capabilities of GIS, it is possible to display the breaks occurring in any given event together with the corresponding system pressure and then eventually determine the break rate function. Indeed, the savings in repair costs associated with pipe breaks justify the time, cost, and resources needed to collect the required data.

6.9 Conclusions

High operating pressure increases the frequency of pipe breaks. This paper presented a probabilistic approach to quantifying the expected pipe break rates in WDSs. The probabilistic approach considering uncertain demands and pipe roughnesses is applied to compute the expected pipe break rates in the Hamilton network. The expected pipe break rates for the Hamilton network are estimated to be 25 breaks/100 km/year. When the

MPC is reduced by 33%, the average minimum pressures and average maximum pressures would respectively decrease by 24.4% and 12.4%.

For the case study presented in this paper, the probability of the maximum pressure being less than 70 m increases from 0.1 to 0.9 when the MPC is reduced by 33%. The frequency of occurrence of low pressures is quantified in this paper and it is shown that low pressures (pressures that are close or equal to the MPC) occur very infrequently. These findings may motivate water utilities to rethink about pressure standards. The expected pipe break rates defined in this paper can be used as an indicator for the design of WDSs and can also be easily incorporated into an optimization scheme in order to minimize the expected pipe break rates.

A similar probabilistic approach was used by Filion et al. (2007b) to introduce the stochastic design method quantifying the expected annual damages associated with low and high pressure hydraulic failures in WDSs. In this paper, however, the context differs from what has been studied by Filion et al. (2007b). Only steady state pressures are considered here and dynamic pressures caused by hydraulic transients or water hammers are not considered. Of course, uncontrolled transient pressures can significantly affect pipe breaks and the related surge pressures should eventually be incorporated into the proposed probabilistic approach as well. To achieve this, several challenging studies need to be performed including (1) establishing the relationship between the magnitude and occurrence frequency of high transient pressures and the magnitude and occurrence frequency of the maximum and/or minimum operating pressures during a typical day as calculated in the proposed MCS; and (2) obtaining the frequency distribution of the maximum pressures including both calculated steady-state maximum pressures and the transient pressures determined based on the relationship established in Step (1). Step (1) requires extensive observed transient data and likely extensive network transient modeling studies considering all routine and permitted transient events during a typical period of operation. Currently available data sets are typically insufficient or too short-term for the completion of Step (1). Use of extensive transient modeling – combined with careful calibration studies – is required for the completion of Step (1) and subsequently the estimation of the expected pipe break rates may be investigated in future studies. The challenge is that the specifics of system responses and of individual pipe breaks are inevitably complex and that pipe breaks often arise from a combination of interacting factors.

Chapter 7

The Link between Transient Surges and Minimum Pressure Criterion in Water Distribution Systems

This chapter is based on the paper entitled “The Link between Transient Surges and Minimum Pressure Criterion in Water Distribution Systems.” by Vali Ghorbanian, Bryan Karney, and Yiping Guo presented in the *Pipelines Conference: Recent Advances in Underground Pipeline Engineering & Construction*, ASCE, August 23-26 (2015), Baltimore, Maryland, USA.

The MPC is used to ensure a minimum delivery pressure when designing WDSs. This criterion is established by political jurisdictions and is different around the world. A low value of the MPC may reduce water consumption (e.g., faucet, showers, and lawn watering) and also lead to efficient operation through reduced energy use, leakage, and frequency of pipe breaks. However, if this criterion is too low, the system may be more susceptible to low pressure failures, either hydraulic (e.g., an inability to supply the required flow) or safety related (e.g., an increased risk of intrusion events and pipe bursts associated with hydraulic transients). Thus, although it may not have been part of the original intent, there is a direct connection between MPC and transients that should not be ignored. This chapter looks specifically at the role of MPC and how it affects system response in transient conditions in order to raise awareness about issues that can arise from changes in MPC. First, the definition of MPC and the possible effects of changes in the MPC on WDSs during transient events are briefly explained. Then, two case studies are developed to demonstrate the role that MPC plays in transient conditions. The results show, not surprisingly, that using surge control strategies is more efficient than increasing the MPC in order to prevent unwanted surge pressures.

7.1 Introduction

WDSs are designed to provide safe drinking water for domestic consumption. These systems must also provide an adequate supply of water, at an acceptable pressure, to deal with routine and emergency conditions, including fire flow requirements. The standard design approach requires that pressure at any point in the system is maintained within a range whereby the maximum pressure is not exceeded so that the likelihood of pipe burst is greatly reduced and the minimum pressure is always maintained in order to provide adequate flow for expected demands. Indeed, the MPC is generally established to ensure the supply of adequate demand to consumers and possibly, although this is seldom explicit, to prevent low/or negative pressures during transient events. The MPC is established by political jurisdictions in each country or region and its value changes somewhat around the world. For example, in the most provinces in Canada, the MPC is 14 m but in Australia and the UK, the MPCs is 20 m and 10 m, respectively (Ghorbanian et al. 2015b). Having different MPCs naturally implies that water pressure delivered to customers might be deemed high enough in some countries while the same delivered water pressure in other countries is considered unacceptable. The benefits of reducing the MPC may include decreasing demands, e.g., faucet, showers, and lawn watering, and also improving system performance, i.e., reduction in energy use, leakage, and the frequency of pipe breaks. However, on the negative side, lowering this criterion may result in consumer complaints and make the system more susceptible to low/negative pressures during transient events. Therefore, there is a link between transient pressures and the MPC that cannot be completely ignored.

Indeed, low MPC can put the system at risk during transient events: a risk to the pipeline, to its associated hydraulic devices and to those in their vicinity, and a risk of water contamination and thus to human life. Reduction in the MPC may allow the occurrence of vapor pressure in a transient event, which can lead to column separation in pipeline systems, particularly at specific locations such as closed ends and at high points or knees (changes in pipe slope). In a column separation process, two or more liquid columns are separated by a vapor cavity and then, after wave reflection, the sudden velocity change caused by the rejoining liquid columns, or when one liquid column collides with a closed end, tends to cause an instantaneous rise in pressure (Wylie and Streeter 1983; Chaudhry 1987). This pressure rise travels as a wave through the entire pipeline and often forms a severe load for individual pipes and supporting structures. Although water column separation and collapse is not common in large networks, this does not eliminate the risk. Another impact of lowering the MPC is the increase in the risk of an intrusion event associated with hydraulic transients. Contaminants may intrude into a WDS through a variety of pathways including submerged air valves, leak points, faulty seals, joints, and service connections when the pressure is low/negative (Thomason and Wang 2009). A low/negative pressure may be initiated by a pump power failure, a valve closure/opening, or demand variations. Gullick et al. (2004) monitored pressures for 43 sites in 8 WDSs and reported 21 negative pressures that lasted less than 3 minutes mainly caused by pump shutdowns. Clearly, neither negative pressures or water column separations are wanted in pipeline systems and should be prevented to the extent practical either by employing surge control strategies or by increasing the steady state pressure. If transient pressures were better controlled using surge control techniques, the system become less vulnerable to the value of MPC; in this context, designers could sometimes reduce the MPC, and still be in a better condition. This chapter explores how the MPC affects transient pressures and briefly reviews how destructive transient pressures may be controlled to limit down surge pressures to an acceptable limits even when the MPC is relatively low.

7.2 The Role of MPC in Transient Pressures

A MPC is generally specified in WDSs design to achieve safe, reliable, and economic operation. However, rapid flow changes during transient events generate propagating pressure waves, which have both positive and negative phases as shown in Figure 7.2. The pressure fluctuations in Figure 7.2 are produced by a sudden valve closure (i.e., with a closing time of 2 seconds) located at the downstream end of the pipe, occurring in the simple system shown in Figure 7.1 (the unrealistic negative pressures in Figure 7.2 is interpreted in the next section). Pressure fluctuations during transient events often violate the regulation of minimum standard for water pressure (Figure 7.2). To some extent at least, pressure transients in WDSs are inevitable and often most significant at pump stations, control valves/hydrants, and in locations with low static pressures. To minimize a system's susceptibility to surge pressures and to efficiently control down surges to a minimum acceptable level, surge control strategies are often adopted.

Surge control strategies have been divided into three categories: design strategies, maintenance strategies, and operational strategies. Engineering and system design strategies include installing surge control devices, using larger diameter pipes, or different pipe materials. Devices such as surge anticipation valves, pressure relief valves, air release/vacuum valves, surge tanks, and air vessels are often used to control surge pressures in pipeline systems. As a part of maintenance strategies, repair practices are important for the safe and efficient operation of pipeline systems since deterioration of pipelines is a natural process. Pipeline deterioration often increases the number of pipeline bursts. Therefore, assessment of the pipelines' interior conditions employing, e.g., hydraulic transient models for quantifying levels of deterioration (Gong et al. 2013), can be useful for planning rehabilitation or identifying critical burst points. Operational practices include adjusting the settings of valves, the starting and stopping procedures of pumps, and operating procedures of fire hydrants. A reduced rate of flow change, achieved through slower valve action, proper hydrant operation, and the use of VFDs (Variable Frequency Drives, which is a system for controlling the rotational speed of pumps in response to changes in flow or pressure) or increased inertia in pumps, are all potentially effective solutions to many problems associated with surge pressures (Wylie and Streeter 1983).

Transient pressures can be controlled by the aforementioned techniques. Minimum transient pressures can be controlled by either using surge control strategies or

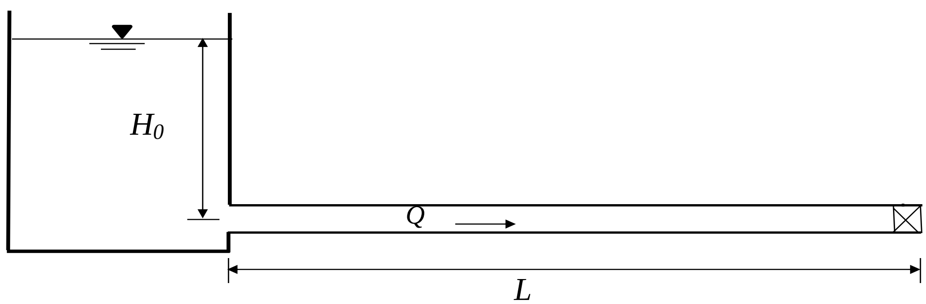


Figure. 7.1. Simple system configuration (water depth in the reservoir H_0 = 30 m; flow rate Q = 0.5 m^3/s, length L= 1000 m, pipe diameter D = 0.65 m, Darcy-Weisbach friction factor f = 0.015, and wave speed a = 1000 m/s)

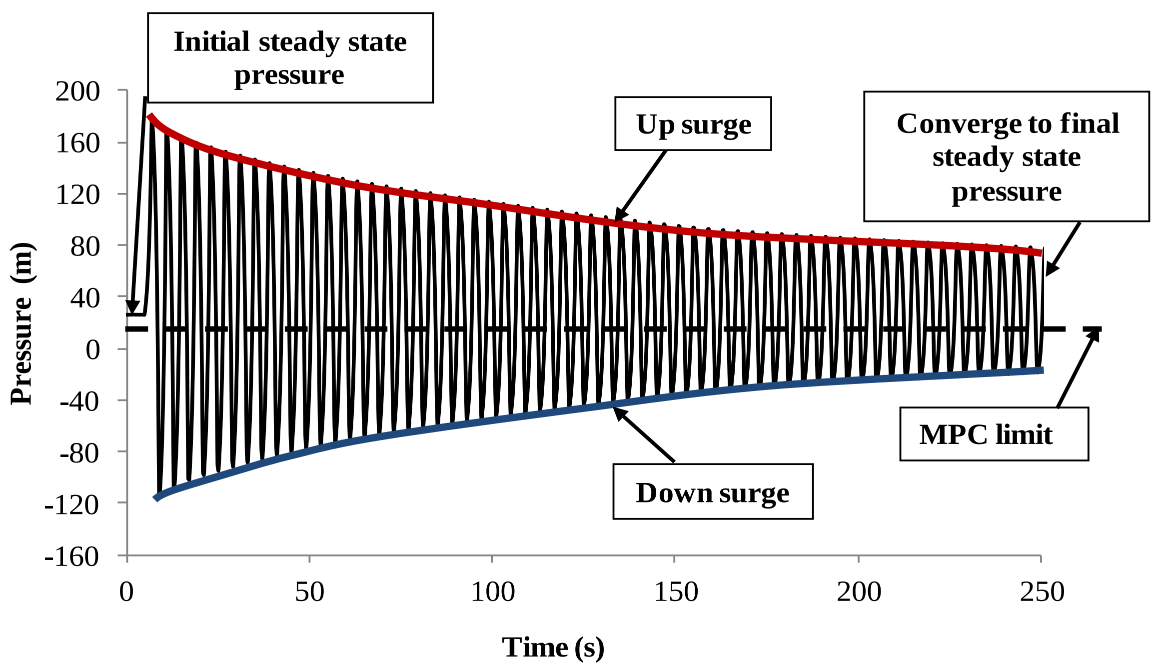


Figure 7.2. Minimum and maximum transient pressures

increasing the steady state pressure throughout the system. Two case studies are now presented to examine the impact of MPC on transient pressures. To assist in transient analysis, a transient model was developed using the method of characteristics (Wylie and Streeter 1983).

7.3 Case study 1: Series of Pipes

To explore and illustrate how the value of MPC affects the system response during transient events, the series of pipes shown in Figure 7.3 is considered. The length, wave speed, and Darcy–Weisbach friction factor for each pipe are 1000 m, 1000 m/s, and 0.015, respectively. For simplicity, the elevations of all nodes are set to be 0 m. The reservoir water level is 23.5 in case that the MPC is set to be 10 m at the most downstream node. To meet the higher MPCs at node 4, the reservoir level is increased. To introduce transient condition into this case study in a simple way, an almost sudden valve closure (1 s) at node 4 is initially considered. Figure 7.4 depicts the pressure envelopes throughout the pipeline system caused by the severe transient condition. The pressure in the pipes becomes unrealistically negative which needs to be either carefully interpreted, or the model should be improved by including column separation. Fortunately, however, this further complication is often not required, since the main role of the transient analysis is to simply identify whether there is a problem. Clearly Figure 7.4 shows that, no matter what values are plotted, sudden changes in the flow rate can induce powerful and destructive forces into a pipe system.

Pipe's strength is often considered as the primary resistance against up surge pressures, and this is related to pipe’s material, wall thickness, and general condition. To avoid destructive down surge effects, the valve must be operated slowly, and/or the steady state pressure can be increased throughout the system. Figure 7.5 shows the closure time of the valve, for the series of pipes system, against different MPCs in case that the down surge is intended to be maintained at 5 m. Not surprisingly, the valve closing time decreases as the MPC increases. If, in a system, the steady state pressure

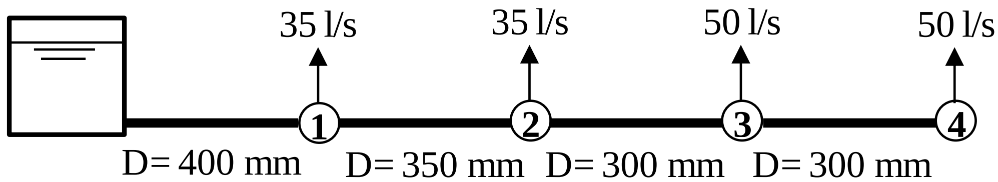


Figure 7.3. Series of pipes (Gupta and Bhave 1996)

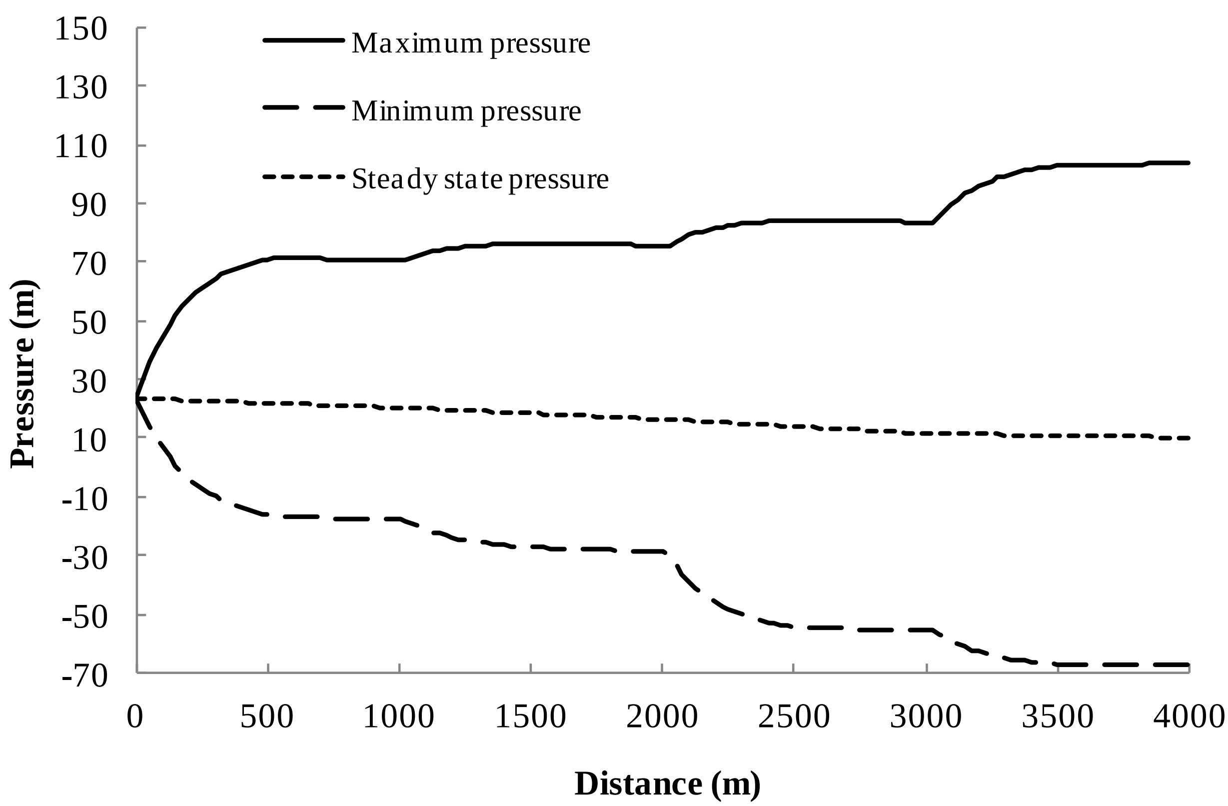


Figure 7.**4**. Transient response caused by the downstream valve closure

reduces, the valve closing time should be increased in order to increase the down surge pressure, thereby the minimum transient pressures are maintained at the desired level. Figure 7.6 illustrates the case where the steady state pressure is increased in order to raise down surge pressures. As illustrated, the down surge pressures still remain negative even the MPC is increased as much as 3.5 times. Therefore, increasing the MPC in WDSs design may not be as efficient as adopting minimal surge control strategies to avoid unwanted surge pressures during transient events. Clearly dramatic actions often have consequences even in systems with considerable pressure.

7.4 Case Study 2: The Hanoi Network

The second study network is shown in Figure 7.7; it was first studied by Fujiwara and Khang (1990) in order to develop their model for optimum design of the primary WDS of the city of Hanoi, Vietnam. The system is a gravity system that draws water from the reservoir at node 1. The system configuration and demands at each node are set according to Fujiwara and Khang (1990) and the results of network optimization presented in Savic and Walters (1997). All junctions are located at the same elevation (0 m). The reservoir head is 100 m to maintain the MPC of at least 15 m during fire flow events throughout the system. To introduce transient conditions into the system, hydrants at nodes 13 and 22 are put into use simultaneously representing a severe fire-fighting scenario. It is assumed that the fire flow requirement at the two nodes are both 0.25 m^3/s and each hydrant takes 2 s to be opened. For simplicity, the Darcy–Weisbach friction factors for all pipes are considered to be 0.015.

Figure 7.8 shows the transient response in the system at nodes 13, 22 and 32. As expected, there are significant transient effects within the network, i.e. losses of pressures, due to the openings of hydrants at nodes 13 and 22. Due to the increase in demands at nodes 13 and 22, a reduced pressure wave moves through the system. This wave is reflected from the upstream reservoir and then propagates back and forth in the system. As indicated in Figure 7.8, the pressure dropped at the non-fire flow node (node 32) confirming the idea that simultaneous operation of fire hydrants would increase the risk of loss of pressure in water networks. As can be seen from Figure 7.8, the pressure head falls below 15 m during the transient event although this value is enforced to be the MPC in the steady state design of the network.

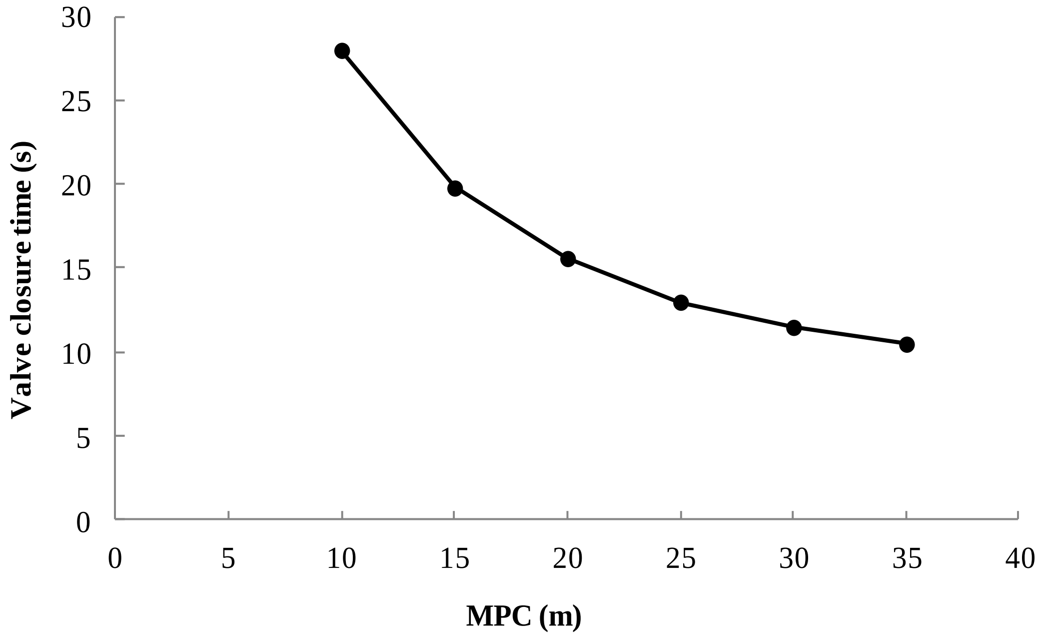


Figure 7.5. Valve closure time versus different MPCs

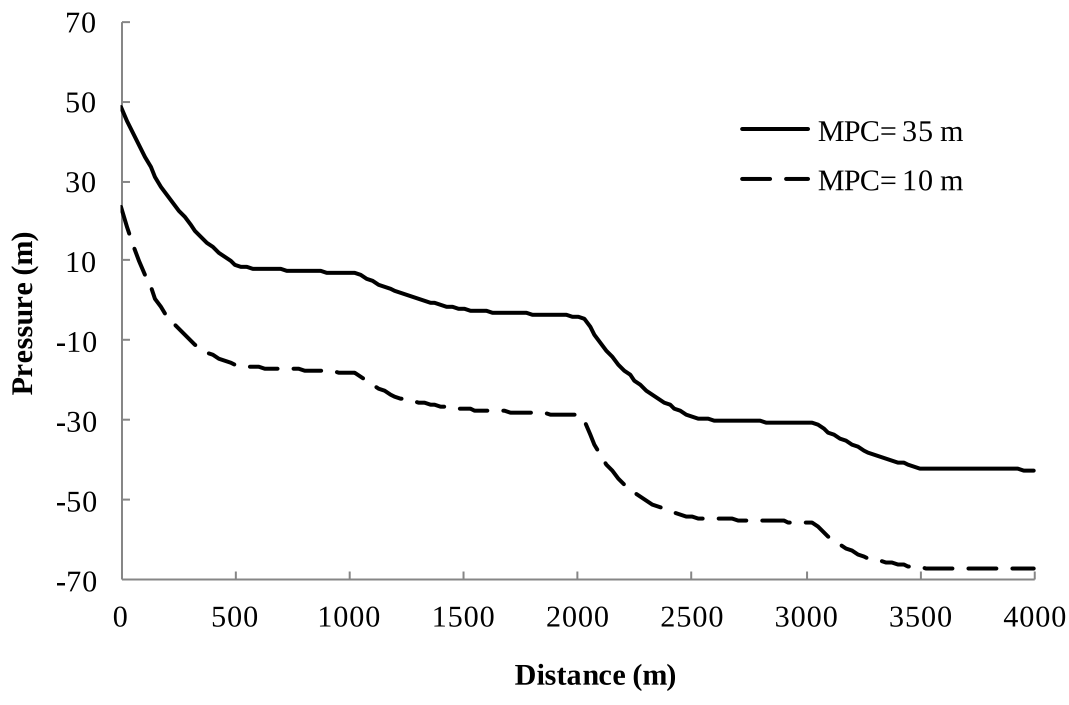


Figure 7.6. Down surge pressures under different MPCs

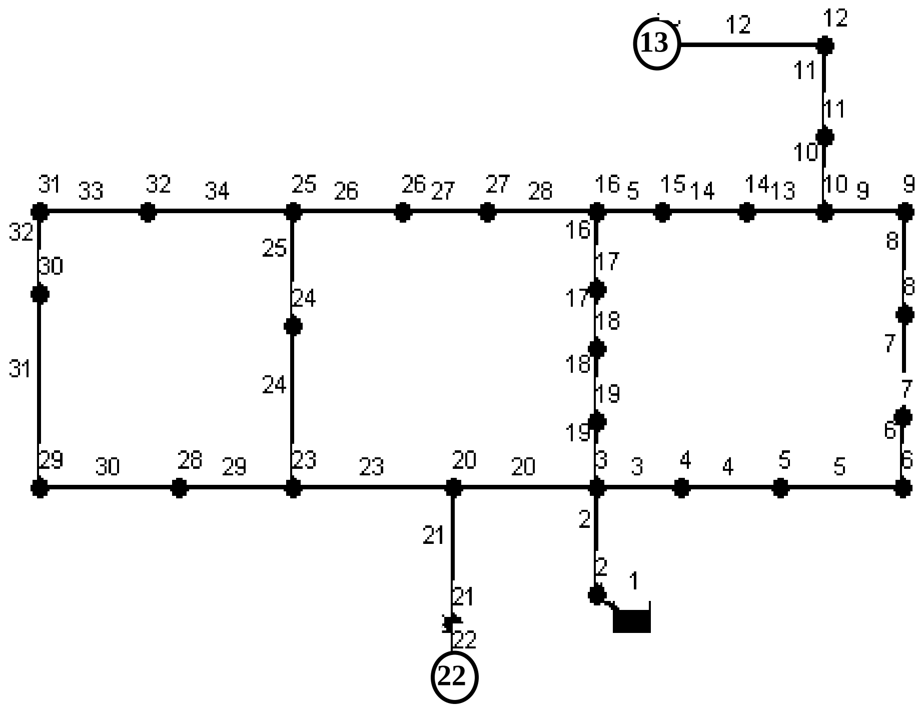


Figure 7.7. Hanoi network

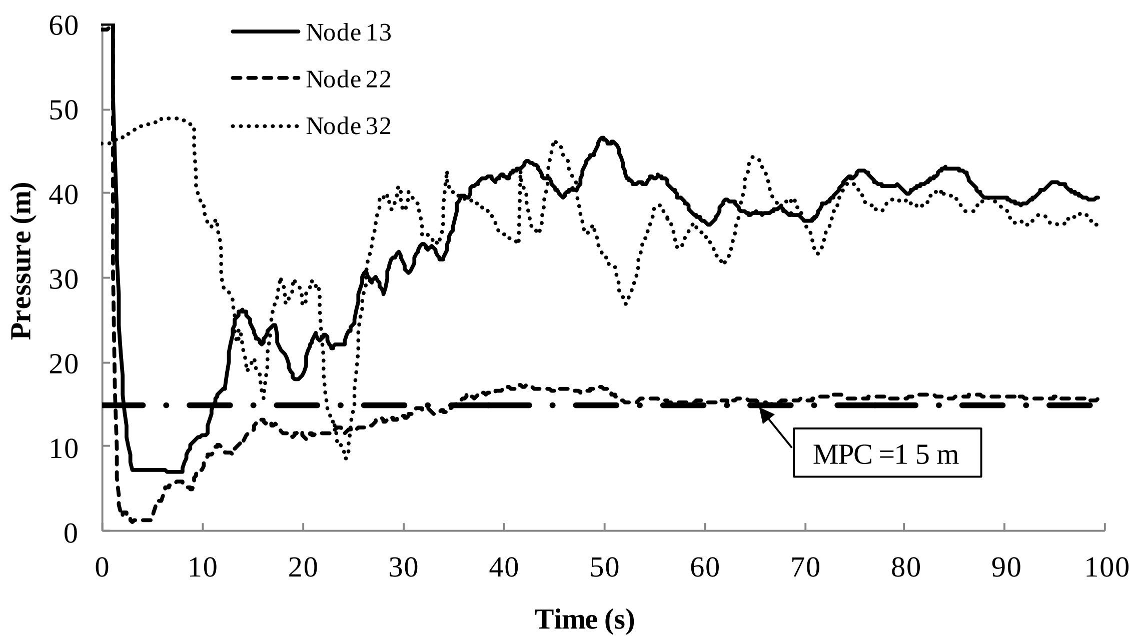


Figure 7.**8**. Surge pressure profiles due to hydrant operations

Much attention has been paid to the issue of controlling the operation speed of hydrants in order to prevent low pressures in a system. In this case study, we determined that in order to maintain down surge pressures at 15 m, the hydrants at nodes 13 and 22 should be gradually opened in 20 s and 55 s, respectively. Figure 7.9 depicts the transient pressures at nodes 13, 22, and 32 in case the opening time of the two hydrants is so extended. As shown in Figure 7.9, with the increase of the opening times of the hydrants, the down surge pressure can be maintained at the desired level (i.e., 15 m). It is possible to determine an approximate minimum safe value for the time to operate a valve in order to protect a system against destructive transient pressures (Wylie and Streeter 1983; Goldberg and Karr 1987). If $t > 2L/a$, where t (s) is the opening time of a valve, L (in metres) is the characteristic length of the network, and a is the wave speed for the pipes (in m/s), there can be a considerable reduction of surge pressures in water networks. The characteristic length of the network may be the sum of the pipe lengths from the source of the surge to the upstream reservoir or the energy source of the system.

However, determining a specified opening time for every hydrant is a challenging task since there are thousands of network configurations in which the characteristic lengths are different. Although fire crews have been trained on proper hydrant operation, this does not universally protect the system against low transient pressures. To make the system safe during hydrant operations, there should be a device for the control of the down surge pressures at the desired limit even if the fire crews try to open the hydrant as fast as they are able to. This device should be portable and can be quickly attached to a hydrant, it should also be able to control minimum transient pressures in different system configurations. This calls for more investigation for the development of a surge limit control algorithm so that the down surge is controlled within a predetermined level during hydrant operations.

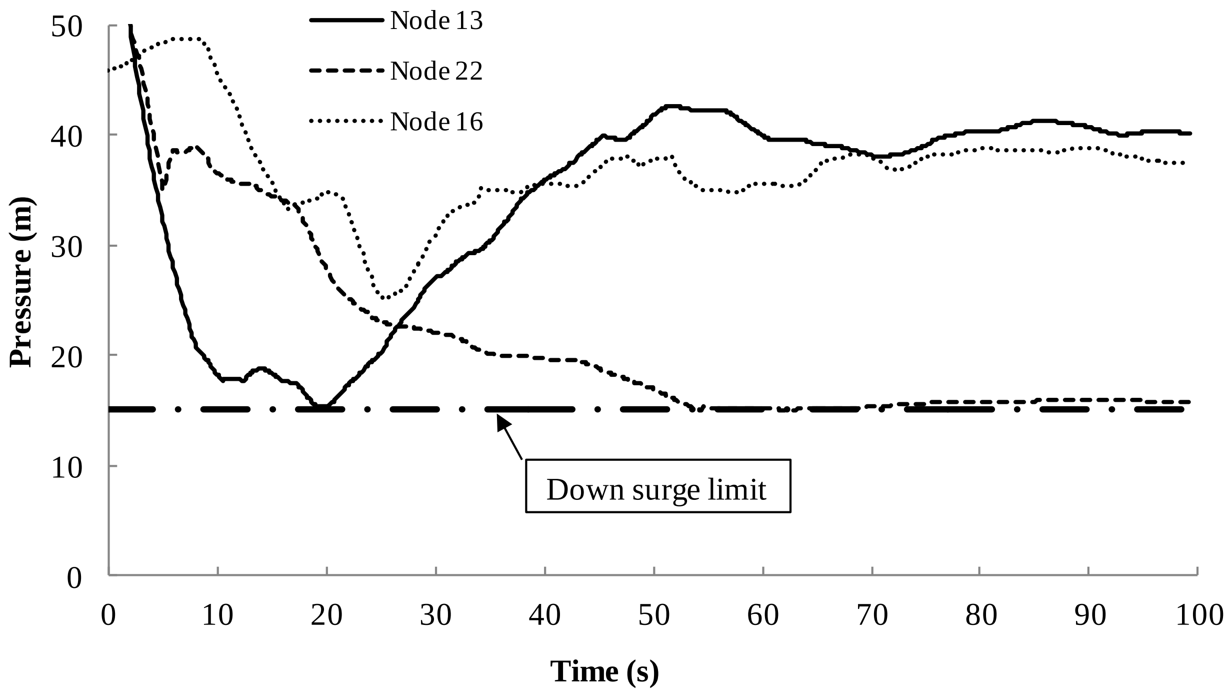


Figure 7.9. Transient Pressure profiles with the controlled opening of the hydrant

7.5 Conclusions

The role of an MPC is to lead to a reasonable design process and outcome. But as systems have aged, there is a desire to reduce the MPC to save energy and reduce the stress on pipeline systems. Lowering the MPC obviously often makes systems more susceptible to negative pressures and contaminant intrusions during transient events. MPC are often violated during transient events due to pressure fluctuations and some care might be needed to define exactly what MPC limits really mean. Consequently, there is an interesting link between transient pressures and the MPC that cannot be completely disregarded. The hydraulic transient response in WDSs is strongly sensitive to system specific characteristics. These destructive transient pressures can be controlled either using surge control strategies, with some of them involving design and operational considerations and some also including the addition of surge protection devices, or sometimes by increasing the steady state pressure throughout the system.

The results clearly show that sudden changes in flow rates can induce dramatic forces in a pipe system; forces that are quite capable of causing unacceptable consequences in operation and may even destroy equipment. Transient events can also put water systems at risk of loss of pressure even if systems are normally operated under high pressures. The results indicate that, not surprisingly, increasing the MPC in WDSs design may be an inefficient surge control strategy. The risk of loss of pressures due to simultaneous operation of fire hydrants can be controlled by extending the opening time of hydrants. However, determining a required opening time for every hydrant is a challenging task since there are many hydrants scattered at different locations of WDSs. Developing a surge limit control algorithm to control the down surge pressures during hydrant operations would seem to be a worthwhile task. This chapter highlights the notion that even those WDSs that are operated under low pressures have the risk of high pressure transients, but that transient pressures can be efficiently controlled using surge control strategies.

Chapter 8

Development of a Control Valve Algorithm to Limit Pressure Down-surges During Hydrant Operations

This chapter is based on the paper entitled “Development of A Control Valve Algorithm to Limit Pressure Down surges During Hydrant Operations.” by Vali Ghorbanian, Bryan W. Karney, and Yiping Guo that will be submitted to the *Journal of Hydraulic Engineering*.

Low pressure WDSs are often more vulnerable to low or negative transient pressures than high pressure systems. In design, an MPC is enforced for the safe operation of WDSs, i.e., for ensuring an ability to supply the required flows and/or pressures and to prevent direct contamination during operations. However, the MPC is often violated during transient events due to hydrant operations, especially when a hydrant is quickly opened. A portable control device which can be quickly attached to a hydrant is conceived here to control the transient pressures up to desired levels even if fire crews open the hydrant as fast as they would. This chapter explores the use of such a portable device for limiting the down-surge pressures by creating a down-surge control boundary in a pipe system during hydrant operations. This boundary is established using the portable control device to safely operate a hydrant in WDSs. In essence, the idea is to sense the pressure change at a hydrant location and then, according to these pressure changes, adjust the opening of the control valve so as to limit the down-surge, and thus the residual pressure, to a predefined level. This novel idea is explored here through numerical simulations and case studies. It is shown that using this device, down-surges can be effectively controlled in both a simple pipeline and a more complex water network during hydrant operations.

8.1 Introduction

WDSs are designed to meet demands and required fire flow under a sufficient pressure constraints. Although pressure is necessary to supply demands to consumers, it should be carefully managed and controlled because high and low values of pressure can both cause a risk to the system. Pressure across WDSs should be maintained within a reasonable range of maximum and minimum pressures to avoid pipe bursts and to ensure that water is supplied at adequate flow rates to all consumers. The goal for establishing pressure standards is to ensure the safe and reliable operation of WDSs. But, established rules or guidelines for specifying the appropriate MPCs for WDS designs are different around the world, e.g., in most provinces in Canada and most states in the US, the MPC is 14 m but in Australia and the UK, MPCs are 20 m and 10 m, respectively (Ghorbanian et al. 2015b). The maximum pressure standards are not set in most regions and just for most provinces of Canada, the maximum pressure is set to be 70 m, and the design standard requires that water mains are designed to withstand the maximum operating pressures in addition to transient pressures. There may be the possibility of reducing the MPC in order to achieve the benefits of decreasing demands and improving system performance, i.e., reduction in energy use, leakage, and the frequency of pipe breaks. However, low MPC makes the system more susceptible to hydraulic failures (e.g., an inability to supply the required flow) and to low/negative pressures during transient events. Therefore, although this is not addressed explicitly, there is a link between the MPC and hydraulic transient that we should be cautious about.

Transient pressures inevitably occur in WDSs when flows or pressures are changed, say when pumps are switched on or off or when valves and hydrants are operated. Any change in flow can result in a surge; however, the common causes of surges are the operation of pumps, valves and hydrants. Transients can introduce high and low pressures and rapid fluid accelerations into a piping system that may fracture or weaken the pipe or its supports. Contaminants can also intrude into a pipe through a leak when induced by transient low or negative pressures. Clearly, destructive transient pressures (high or low pressures) are unwanted in WDSs and should be eliminated to the extent practical either by employing surge control strategies or by increasing the steady state pressure. Ghorbanian et al. (2015a) concluded that using surge control strategies is more efficient than increasing the MPC.

If a system operates in a high pressure, the maximum surge pressure can be significant but it can be controlled within specified limits by well-chosen protection approaches (Simpson et al. 1994; Jung and Karney 2006). It has long been known that WDSs operating in low pressures can put the system at risk during transient events: the risk of an intrusion event and consequently the risk of water contamination, and the risk of column separation and sudden pressure rise due to rejoining of columns. Low/negative pressures occur more frequently in low pressure systems during transient conditions. These low pressures may be initiated by a pump power failure, a pipe replacement, a valve closure/opening, or rapid demand variations (i.e. hydrant operations). Besner et al. (2011) reported that negative pressure events lasting from 13 seconds to 28.6 minutes could be caused by rapid demand variations. Rapid flow changes during transient events generate pressure fluctuations which often violate the regulation of minimum standard for water pressures. But transient pressure fluctuations can be curtailed by surge control strategies. If down-surge pressures are moderated using surge control techniques, the system becomes less vulnerable to the value of MPC.

WDSs design typically requires that pressure be above a MPC when the network is experiencing a worst-case loading, i.e., the greater of maximum hourly demand or maximum daily demand plus a fire fighting demand at a critical node. Hydrant operations to provide the required fire flow induce transient pressures that may cause the MPC to be violated. Pressure surges produced by hydrant operations should be controlled by the slow opening of hydrant. Often, in order to protect systems against destructive transient pressures due to valve opening, an approximate minimum safe value for the time to operate a valve should be considerably greater than $2L/a$ (Wylie and Streeter 1983; Goldberg and Karr 1987), where L (m) is the characteristic length of the network (the characteristic length of the network is defined as the sum of the pipe lengths from the source of the surge to the upstream reservoir or the energy source of the system), and a is the wave speed for the pipes (m/s). However, determining a specified opening time for every hydrant is a challenging task since there are thousands of different characteristic lengths that need to be determined.

Although fire crews have been trained on proper hydrant operation, this does not protect the system against low transient pressures due to human errors. Therefore, a portable device that can be quickly attached to a hydrant would be useful for the control of the down-surge pressures. This device should be able to properly operate together with the hydrant even if the fire crews try to open the hydrant as fast as they are able to, and regardless of what the minimum steady state pressure is.

This chapter aims to explore how an anti-surge boundary can be designed and applied for transient protection, particularly at hydrant locations. Although having the same role in limiting transient pressures in pipe systems, this approach is different from a pressure sustaining valve or back pressure valve that modulates the valve opening to maintain the set point pressure corresponding to the locally sensed pressures (Hopkins 1988). With a pressure sustaining valve, if maintaining the set pressure in the upstream point is not possible, the valve will close completely which is not desirable for hydrant operations. The key issue in our conceived surge control device is to adjust the opening rate of the controlled valve so as to limit the down-surge, and thus the residual pressure, to a predefined level. First the basic components of the conceived control valve are described. Subsequently, the mathematical model of the control valve is developed and used to determine the requirement adjustment of valve opening in order to limit surge pressures. The key novel features in the control model are described later on. The developed anti-surge boundary is verified using a simple pipeline system; and finally, a network example involving a successful numerical application of the down-surge control model of the conceived device is presented.

8.2 Control Valve Components

The new control valve conceived in this study was developed for pressure control applications where no external power is available to operate the valve, and therefore the pipeline pressure is utilized to adjust the opening of the valve. The main components of the control valve are depicted in Figure 8.1. To ensure the accuracy of the pressures regulated by the control valve, five separate pilots are required. A pilot is a mini-valve which senses the pressure at the valve and actuates the control valve diaphragm in order to change the valve setting for maintaining the required pressure. There should be a total of five pilots to properly control the pressure at the control valve (Figure 8.1). The normal opening of the valve is controlled by pilot 1, this means that as long as the pressure is above the desired surge limit, the control device does not function and the valve would be conventionally open. When the pressure is lower than the pilot 1's spring setting (i.e., the set point H_{US} which is the upsurge limit), pilot 1 allows pilots 2 and 3 to activate. Pilot 3 causes the pipe connector to open. A valve actuator is the mechanism for opening and closing a valve, either manually or automatically. For the control valve, the actuator comprises of disks, pistons, springs, oil-filled pipes, and hydraulic supply, i.e., internal water pressure to move disks and the valve diaphragm. Pilot 2 controls the actuator movement according to internal pressures. Pilot 4 is activated when the pressure is lower than H_{LS} (the low surge limit), which allows pilot 5 to activate. Pilot 5 causes the valve diaphragm to close when the pressure is below H_{LS}, and then a positive surge is produced to boost pressures in the system. The reservoirs 1 and 2 are used for storing the released oil when the valve diaphragm is moving upward or downward. The feature *A* and its components are used in failure condition and will be explained in section 8.5.

The mechanism of pilot 2 in order to control the valve opening can be performed by adjusting the sizes of pilots' disk and the disk connected to the valve's diaphragm (Disk *a*). The rate of change of the opening of the valve diaphragm is proportionate to the rate of displacement of pilot 2's disk which causes the valve diaphragm to move. To clarify this presumption, consider Figure 8.2 depicting two pistons connected to one

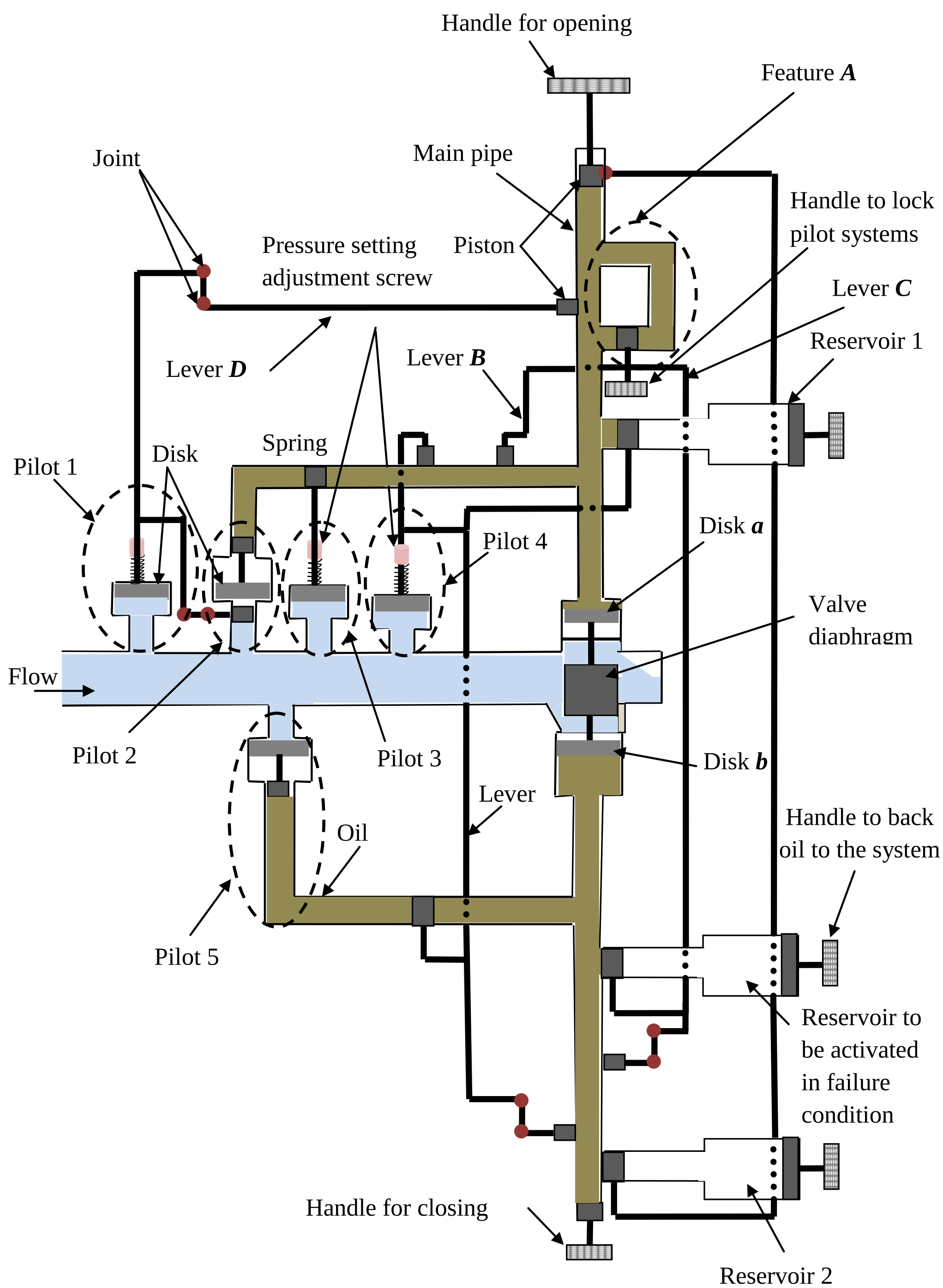


Figure 8.1. Control valve components

another with an oil-filled pipe. If an upward force, due to the internal pipe pressure, is applied to one piston (the left one in Figure 8.2), then the force is transmitted to the second piston (the right one) through the oil in the pipe connector according to

$$\frac{F_1}{A_1} = \frac{F_2}{A_2} \quad \text{and} \quad F_1 = PA_1 \tag{8.1}$$

where F_1 and F_2 are the forces applied to pistons 1 and 2; respectively, A_1 and A_2 are the areas of piston 1 and 2, respectively, and P is the internal pressure. The displacement of disk 2 is

$$X_2 = \frac{F_1 X_1}{F_2} \tag{8.2}$$

where X_1 and X_2 are the displacements of disks 1 and 2, respectively. If the areas of the two disks are equal, the applied forces and displacements of the two disks are the same. Note that experimental investigation is required in order to determine the exact value of displacement of the valve diaphragm, considering internal pressure and the sizes of the disks. Since oil is essentially incompressible, the efficiency is high and almost all of the applied force to the first piston can be transmitted to the second piston. The virtue of this system is that the connector pipe can be of any length and shape.

The control valve comprises a spring and a diaphragm with adjustable spring screws and/or specific spring selections to allow the actuator to function in response to particular forces. Springs conveniently convert mechanical force into mechanical motion (i.e., Hooke's Law: $F = k\, x_s$, F = force applied to spring, k = spring constant, and x_s = displacement of spring). The spring adjuster must be set for the proper set pressure, i.e. H_{US} and H_{LS}. Figure 8.3 shows how the spring adjustment can work in the conceived control valve. When the pressure drops below a pressure set point (say the transient pressure is below the spring setting of the middle disk in Figure 8.3), disc m will move downward, due to the energy stored in the compressed spring which causes the oil-filled pipe to be blocked by the upper piston (piston m). Then, the force transmitted by the left piston to the valve diaphragm is disconnected. Spring motion is linearly related to the

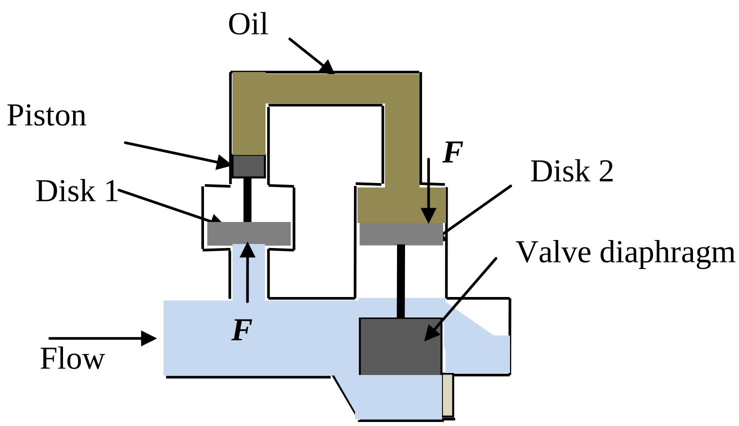


Figure 8.2. Valve displacement mechanism

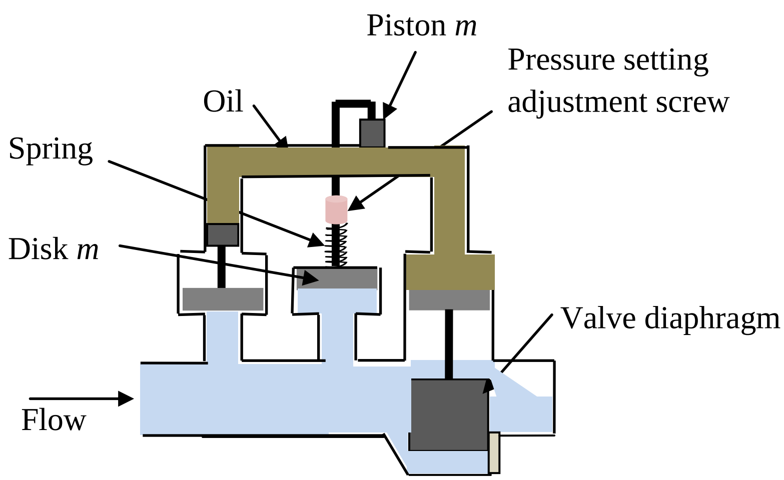


Figure 8.3. Spring adjustment for the control valve

applied force from the disk. Neglecting the weights of disks, the spring displacement for pilots 1, 3, and 4 in Figure 8.1 which activates pilots 2 or 5 can be determined as

$$x_s = \frac{P_s A}{k} \tag{8.3}$$

where A is the area of the disk; and P_s is the up or low surge limits (H_{US} or H_{LS}).

In the numerical simulation (Eqs. (2.13) and (8.5) through (8.8) explained in the next section), the control rule's parameters (i.e., k_c, k_r, ΔH_s, H_{LS}, and $H(t)$) indicate how the rate of τ (the dimensionless valve opening ratio) should be changed if the transient pressure is lower than H_{US}. However, in a physical system, as explained earlier, the rate of change in τ depends upon the instantaneous internal pressure which can be transmitted to the valve diaphragm by a pilot system. Therefore, the sizes of disks in a pilot system, the constant of springs attached to diaphragm, as well as the sizes of disks *a* and *b* can be determined based on the pressures that are lower than H_{US} and H_{LS} to apply appropriate force for proper displacement of the valve diaphragm. Obviously, these parameters must be determined based on a test experiment. Of course, the values of k_c and k_r can help in order to determine the initial guest for the sizes of disks and springs' constant.

8.3 Mathematical Model

To simulate the control device to prevent unwanted down-surges during hydrant operations, the extended MoC is used to determine the relationship between nodal heads and flows (Karney and McInnis 1992). The MoC provides a systematic way of calculating transient conditions within a pipeline. However, at each end of the pipe or at the control device an auxiliary relation between head and discharge must be specified. Such a head-discharge relation is referred to as a boundary condition. Since the hydrant functions as a valve in the system, the device attached to the hydrant for controlling surge pressures should act like a control device valve. Therefore, the valve discharge equation is employed here as the boundary condition. The valve equation defines the relationship between the flow passing through a valve, Q, and the head difference across the valve, ΔH.

$$Q = \tau C_v \sqrt{\Delta H} \qquad (8.4)$$

In Eq. (8.4) C_v is the valve coefficient which is conventionally calculated as $C_v = Q_0 / \sqrt{\Delta H_0}$ where Q_0 and ΔH_0 are the steady state flow and the head difference across the valve, respectively, when valve is full open; τ is the dimensionless valve opening ratio with $\tau = 0$ representing the no-flow case and $\tau = 1$ representing a fully opened valve. The opening and closing of valves can be represented if τ is a function of time. The opening or closing motion of a conventional valve is predefined while the opening of the control valve is automatically adjusted in response to the transient pressures which are desired to be controlled. In other words, τ at each time step is unknown and needs to be dynamically determined in the mathematical modeling of the operation of a control valve. In a real physical system, the pressures can be controlled by the proper operation of the control valve actuators, while in the numerical simulation of the control valve using the mathematical model, the control procedure is simulated by using a mathematical control rule in order to determine τ at each time increment.

To develop the mathematical model of the control valve, consider Figure 8.4 depicting a simple pipe at which a hydrant is installed at the most downstream end. Let H_P be the pressure head at the hydrant. The MoC solution for each pipe in a distribution network proceeds as explained in Eqs. (2.13) and (2.14). The control device is treated with equations similar to Eqs. (2.13) and (8.4). The characteristic equation at the hydrant is represented by the C^+ equation. Eq. (2.13) applies for the pipe at the boundary. The discharge flow of the hydrant is governed by the valve discharge equation which is used as the boundary condition. The flow through the valve corresponding to a specified τ can be determined as

$$Q = \frac{Q_0}{\sqrt{H_0}} \tau \sqrt{H_p} \tag{8.5}$$

where H_0 is the pressure head when valve is fully open.

Eqs. (2.13) and (8.5) should be solved simultaneously to determine H_p and Q (in Eqs. (2.13) and (8.5) Q and Q_p are the same). To control the down-surge pressure at a predefined limit and to prevent the oscillation around the low surge limit (i.e., H_{LS} in Figure 8.5), a surge control region should be identified so that the control valve opening is controlled in this region to prevent pressure from getting below H_{LS}. In other words,

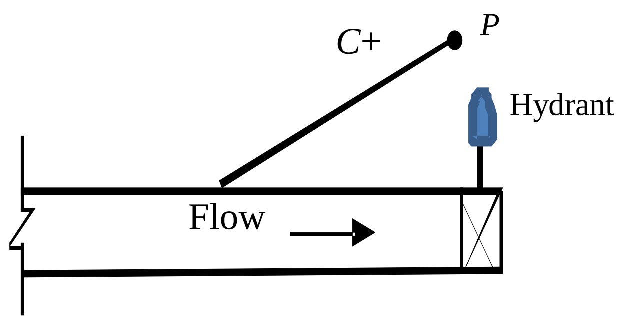


Figure 8.4. Representing the boundary condition

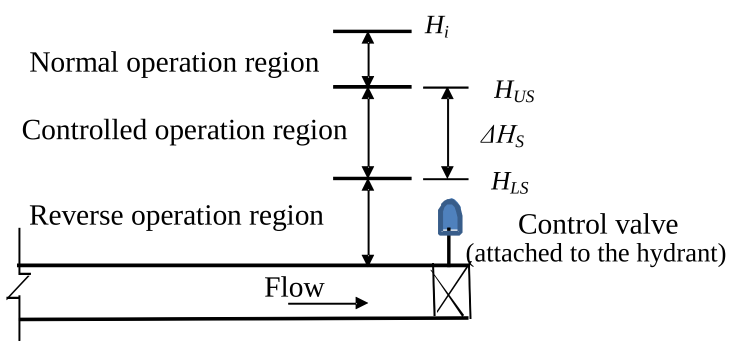


Figure 8.5. Surge control region (H_i is the initial steady state pressure)

instead of using a predefined opening motion, the valve opening is automatically adjusted in response to the pressure at the control valve. Consequently, τ in equation (8.5) is unknown for the control valve and needs to be dynamically determined by the characteristics of the valve and the pressure at specific time. Usually, a function referred to as the control rule should be established to adjust the valve opening in order to maintain transient pressures above a set point. In a physical system, the pressures can be controlled by proper operation of control valve actuators, while in the numerical simulation of the control valve, the control procedure is represented by using a control rule for determining the dimensionless valve openings (i.e., τ) at each time increment.

The control rule can be expressed by:

$$R_1 = \frac{H(t) - H_{LS}}{\Delta H_s} \tag{8.6}$$

$$R_2 = \frac{H(t) - H_{LS}}{H_{LS}} \tag{8.7}$$

$$\frac{d\tau}{dt} = \begin{cases} \text{Regular normal opening} & \text{if } R_1 > 1 \\ k_c R_1 & \text{if } 0 < R_1 \leq 1 \\ k_r R_2 & \text{if } R_1 \leq 0 \end{cases} \tag{8.8}$$

where $H(t)$ is the pressure head at time t; H_{LS} is the low surge limit; H_{US} is the upsurge limit; ΔH_s is the pressure difference of the low and upsurge limits; and K_c and K_r are controller parameters depending on system configurations and can be adjusted according to specific requirements. Figure 8.6 represents the control algorithm involved in the operation of control device. The hydraulic implication of the control algorithm can be explained as follows: when pressure $H(t)$ at the valve is above the set point H_{US} (i.e., $R_1 >$ 1), the valve would be conventionally opened. By contrast, if the pressure $H(t)$ at the valve begins to decline below the upsurge limit, H_{US}, (i.e., $0 < R_1 \leq 1$), the valve opening would be according to the control rules (Eqs. (8.6) to (8.8)). When pressure $H(t)$ at the valve drops below the low set point H_{LS} (i.e., $R_1 \leq 0$), the valve would start to close (at this stage, R_2 is negative, therefore, τ decreases) to produce the positive surge in order to increase pressures in the system. With these control laws, the automatic adjustment of the valve motion would be smooth and continuous.

In summary, for the control valve, the MoC equation (Eq. (2.13)), the valve discharge equation (Eq. (8.5)), and the controller equations (Eqs. (8.6) through (8.8)) constitute the mathematical model of the boundary condition in the pipe network. Therefore, the three unknown variables at the valve boundary (i.e., τ, H, Q) can be numerically solved using the finite difference method.

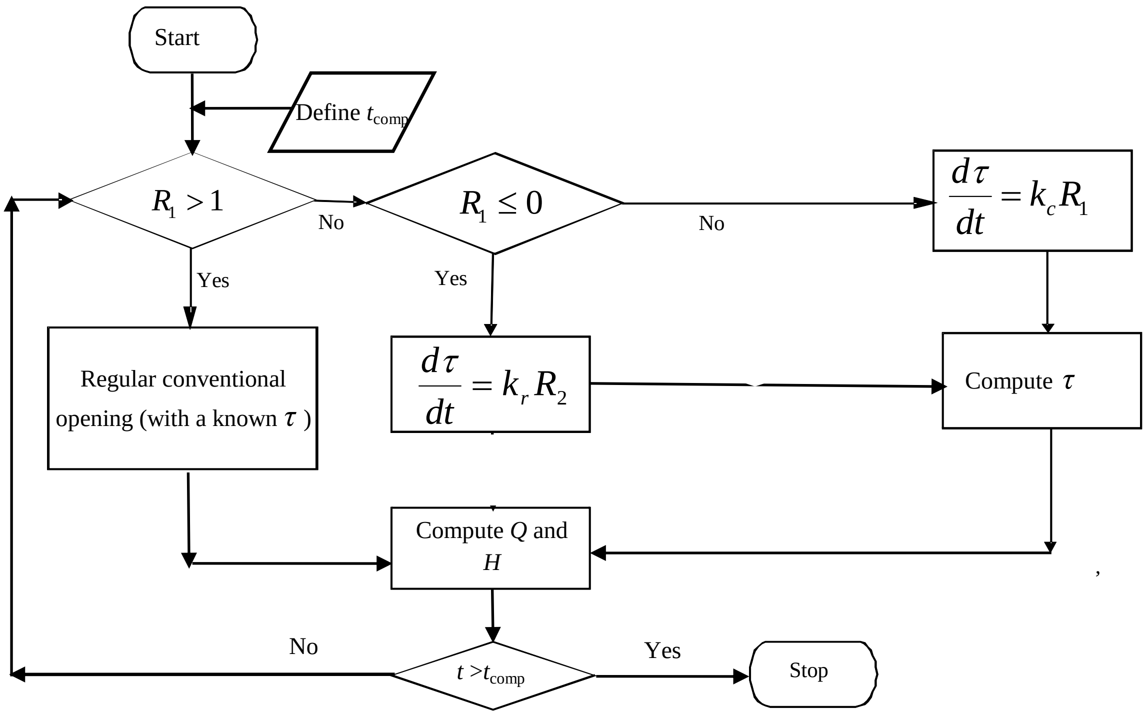


Figure 8.6. Control decision logic algorithm (t_{comp} is total computational time)

8.4 Model of Control Valve Opening

Based on the above developed mathematical model for the control valve, the physical consideration and mathematical model to establish the boundary condition representing the new control device are explained in this section. Practically, this new surge control device should be portable. The main idea is illustrated by a simple system shown in Figure 8.7. In such a system, it is assumed that a hydrant is installed at the most downstream end. Once the hydrant is opened, the pressure waves would propagate at speed a toward the upstream end along the pipeline. The wave would arrive at the reservoir within L/a second. To maintain down-surges above a predefined limit the control device adjusts the valve motion continuously and accurately. The pressure at the valve, $H(t)$, is tracked in this boundary model and the corresponding mathematical model for the control valve including the extended MoC (Eqs. (2.13) and (2.14)), the valve discharge equation (Eq. (8.5)), and the controller (Eqs (8.6) through (8.8)) are applied.

Once the up and low surge limits are established, the control device could adjust the valve motion continuously to ensure that $H(t)$ is always above the low surge limit. To illustrate the application of the mathematical model, it is assumed that in the system shown in Figure 8.7, the hydrant is opened to provide 0.5 m^3/s fire flow requirement. The low and upsurge limits are set to be 5 and 7 m, respectively. The reservoir has a constant head of 11 m and there is no initial flow in the system. To limit the down-surge pressure in the system, the constants of the controller are taken as $k_c = 2$ and $k_r = 2$.

Numerical simulation was conducted for the simple system shown in Figure 8.7. Results presented in Figure 8.8 show that the opening of the control valve was automatically adjusted in response to the pressure at the valve (the time needed for the normal opening of the valve is 3s; however, the time of the control valve opening is 5.4s). Figure 8.9 depicts the dimensionless transient pressure profile at the control valve and the dimensionless flow in the system indicating that the down-surge is controlled above a predefined level (H_{LS}). The control valve represented here is assumed to be spring actuated and undamped so that it responds instantaneously to changes in flow conditions.

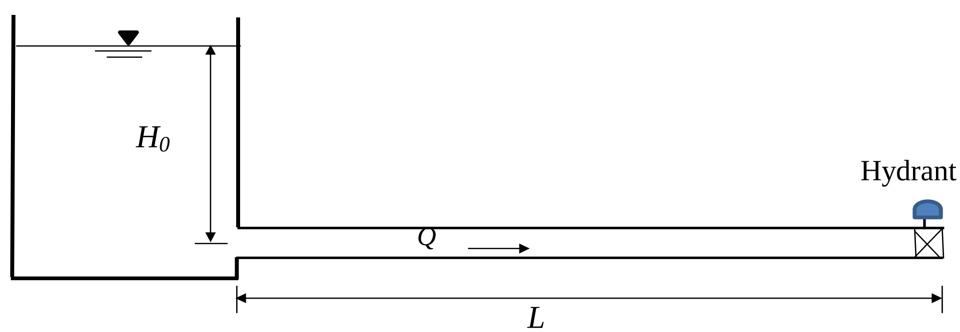


Figure 8.7. Simple system configuration (pipe length L=500 m, diameter D = 1.0 m, Darcy-Weisbach friction factor f = 0.015, and wave speed a =1000 m/s)

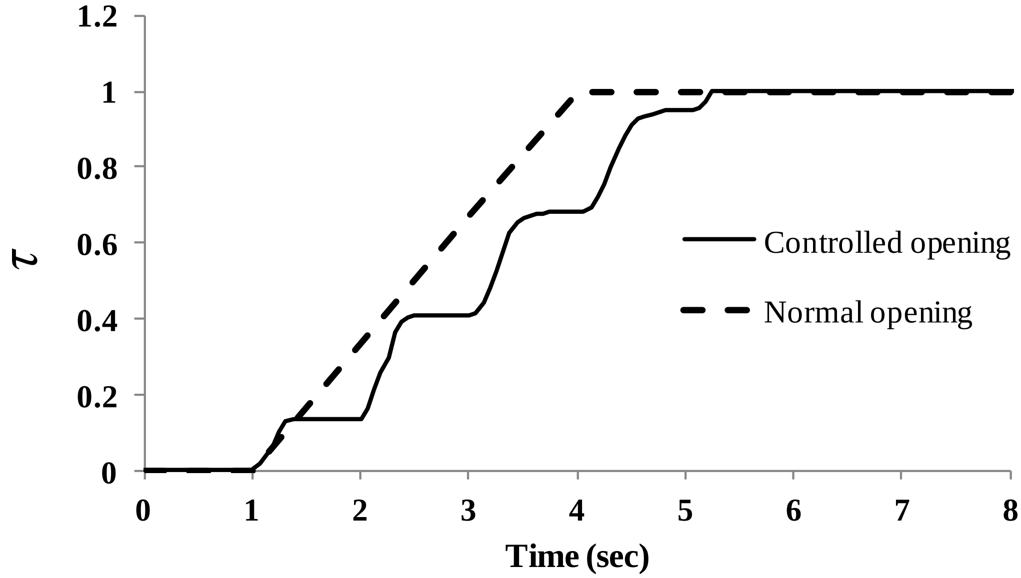


Figure 8.8. Normal and controlled valve opening ratio

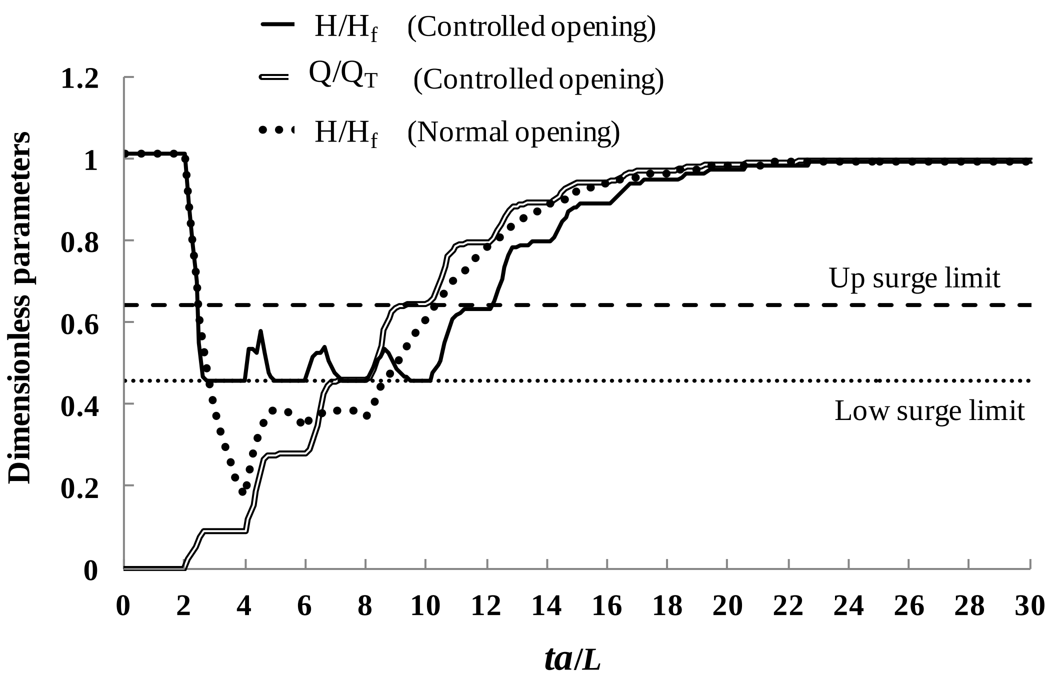


Figure 8.9. Transient pressures and flow response when the new boundary model is employed (t is time, H_f is the final steady state pressure, and Q_T is the total demand in the pipe)

8.5 Sensitivity to response time of the control device and safe-failure feature

If there is a discrete or finite response time to sudden and large pressure changes in the pipeline, the valve response may be slower than that of transient pressures. Therefore, the down-surge may not be controlled at the desired level and the control device may need more adjustments. Figure 8.10 illustrates how different lag times to activate the valve's actuator affect the pressure response of the system shown in Figure 8.7. As indicated in Figure 8.10, increase in the lag time t_l causes the minimum transient pressure to become lower than the specified H_{LS}. As can be seen from Figure 8.10, the minimum transient pressure remains unchanged when the lag time increases beyond a certain value (for this case study, the minimum transient pressure does not change when the lag time is 1s or longer). This threshold for the lag time is different for each system configuration and the time of the normal opening of the valve. Indeed, the unchanged minimum transient pressure (in Figure 8.10) is the minimum down-surge which could occur during the normal opening of the valve. In other words, the control valve does not function any more if the lag time is greater than a certain value. From this perspective, the example indicates how drastically the response of a physical system can be affected by assumptions about the characteristics of the control valve. The response time of the control valve is determined by the design and physical characteristics of both the pilot system and the valve itself.

The proposed control device may fail in controlling transient pressures and in the critical situation, it may cause the valve to be locked. If the control valve is locked, i.e., in the situation that lever *D* (in Figure 8.1) causes the main oil-filled pipe to be closed while the pressure is higher than the surge limit, the valve would be closed and all the oil-filled pipes used to transmit forces to the actuator are blocked. In these circumstances, a dangerous condition would occur: the valve could not be opened while it should have been opened during fire flow conditions. Thus, a fail-safe feature is required to prevent or mitigate unsafe consequences of the control valve's failure. Feature *A* coupled with the two connected levers *B* and *C* is proposed for the control device so that this feature which

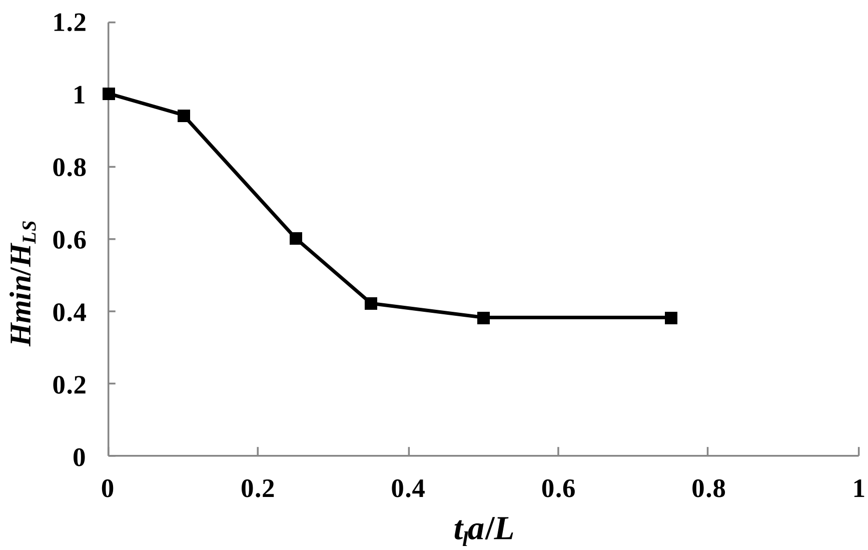


Figure 8.10. Responsive the control valve versus lag time (*Hmin* is minimum transient pressure and t_l is the lag time)

can be controlled manually is used to lock all the pilot systems under the possible failure conditions. Obviously, the valve would normally be operated in this case and pressure cannot be controlled at desired level.

8.6 An Example Pipe Network System

To illustrate the application of the current mathematical model to water networks, the system shown in Figure 8.11 is considered. The case study used here was studied by other researchers such as Wylie and Streeter (1983) and Boulos et al. (2005). The Darcy–Weisbach friction factor and wave speed are 0.02 and 1000 m/s, for all pipes, respectively. Nodes 3 and 6 have the external demands of 0.3 and 0.4 m^3/s, respectively. The elevations of all nodes are set to be 0 m. To perform more accurate transient analysis, pressure dependent demands (PDDs) are employed (Jung et al. 2009). For this purpose, the constant demands are replaced with the PDD formulation (Eq. 2.6) at demand nodes. In this chapter, P_{de} and n are assumed to be 15 m and 0.5. To induce transient condition into the system, a fire hydrant is assumed to open at node 7 in order to supply fire flow requirement of 0.7 m^3/s. The mathematical model developed in this chapter is employed as a boundary condition at node 7.

Figure 8.12 shows the dimensionless transient pressure profile at the control valve and dimensionless flow at node 7. As shown in the figure, the down-surge is controlled above a predefined level (H_{LS}). In this case study, H_{US} and H_{LS} are set to 19 m and 17 m, respectively. Figure 8.13 depicts the response of the control valve opening ratio to the pressure at the valve. As illustrated in the Figure, the time of the control valve opening is longer than the normal opening time indicating that the valve opening is adjusted so that the down surge pressure is maintained to be above the predefined limit. To highlight the effect of the control valve in maintaining down-surge pressures, the transient pressure profiles are depicted in Figure 8.14 for different opening times. As indicated in the Figure, the transient pressure cannot be controlled even the valve is uniformly opened in 30s, while down-surge pressures are controlled by employing the control device valve with an opening time of 29s.

8.7 Conclusions

Fire hydrant operations are common in WDSs and the system may experience unexpected low transient pressures if a hydrant is quickly opened. The creation of a down-surge control boundary through the carefully designed control valve is a new approach that has the potential to effectively control transient pressures in WDSs arising from hydrant operations. This is explored here by numerical simulation and a considerable potential of the proposed control valve for transient protection has been demonstrated in the numerical examples. As shown, the down-surge can be maintained above a desired level, during the opening of a valve, with the use of the proposed surge control algorithm. Such an algorithm provides a detailed prescription for the operation and conceptual design of the proposed control valve. The opening of the control valve is adjustable in response to the transient pressures to be controlled. Importantly, such a control valve could be readily portable and quickly attached to a hydrant by fire crews. The internal pressure of the pipe feeding to a hydrant is the force required to operate the control device; therefore, this device can limit the down-surge to be above a desired level with the function of the

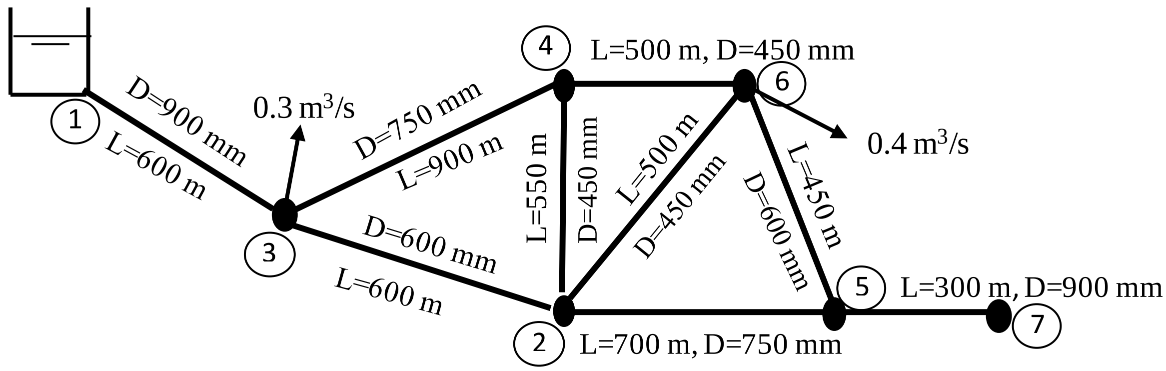


Figure 8.11. Example pipe network (*D* is pipe diameter and *L* is pipe length)

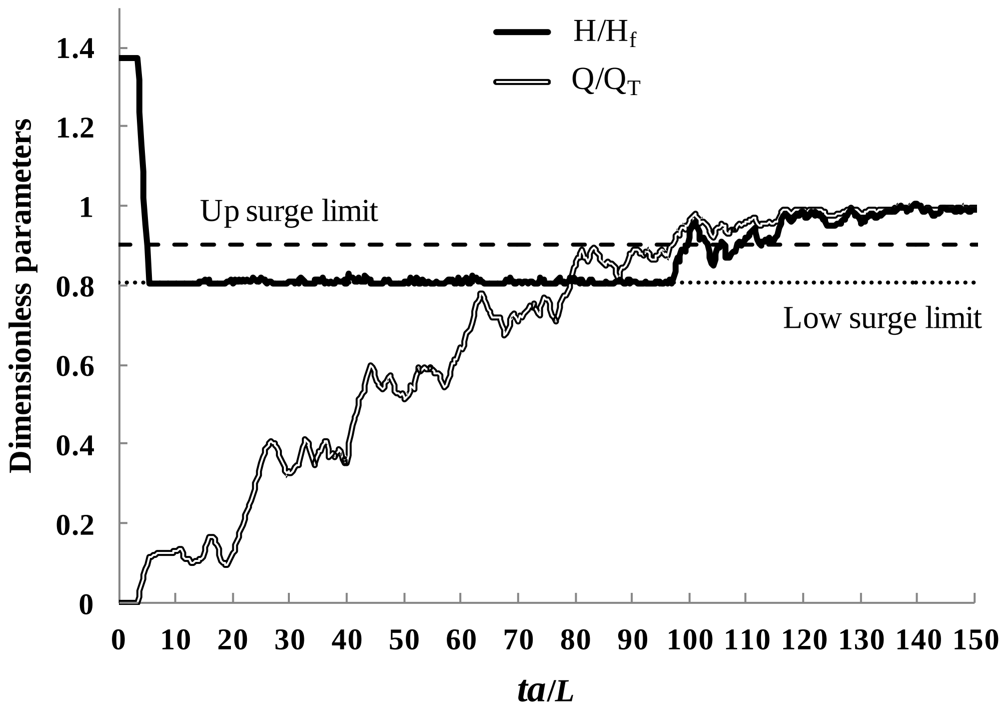


Figure 8.12. Transient pressure and flow response at node 7 (H_f is the final steady state pressure and Q_T is the total demand in the pipe)

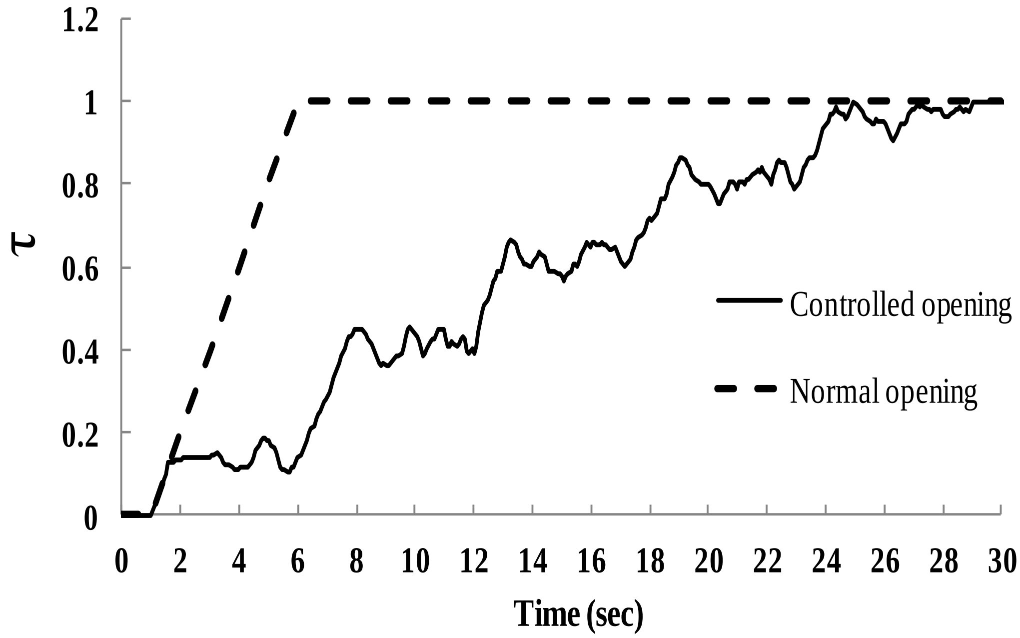


Figure 8.13. Comparison of normal and controlled valve opening

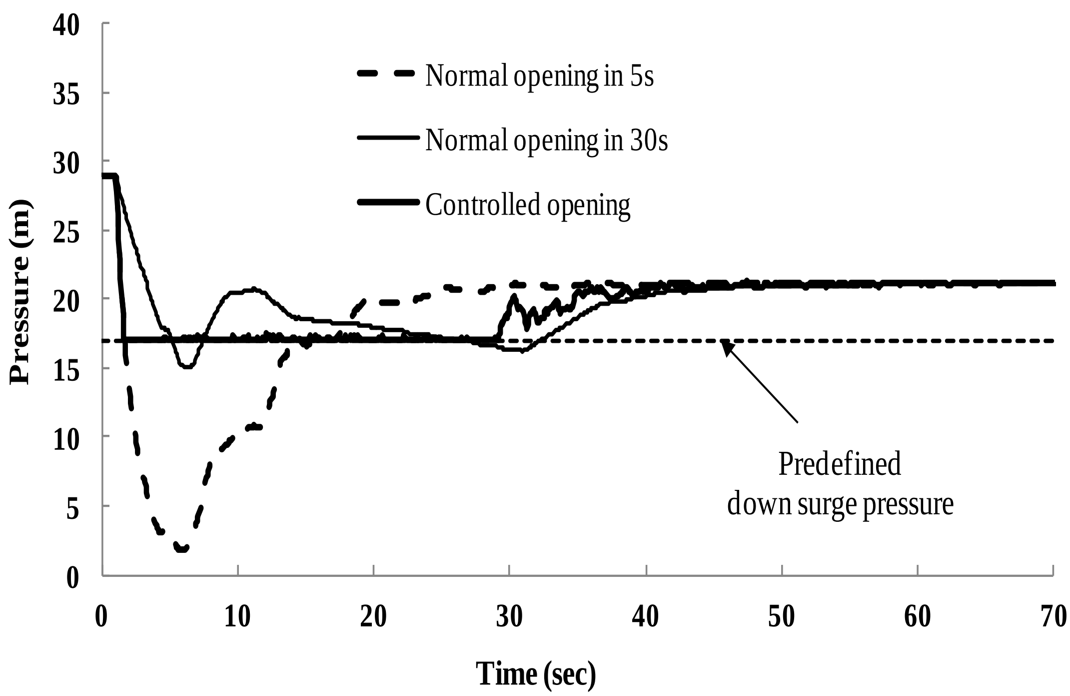


Figure 8.14. Transient pressures profiles for cases of different opening times and controlled valve opening

control device effectively independent of typical system characteristics. This attribute arises from the valve's novel diaphragm which can be displaced by a set of disks and springs. Experimental investigation is required to confirm these findings and to determine the required sizes of the disks, springs and other valve characteristics.

Chapter 9

System operation, energy consumption, and leakage

This chapter examines in greater depth some aspects of discussion which has been made in the paper “Minimum pressure criterion in water distribution systems: challenges and consequences.” (chapter 4) and the methodology used in the paper “Intrinsic Relationship between Energy Consumption, Pressure, and Leakage in Water Distribution Systems” (chapter 5). It does not introduce new research results, but elaborates on issues surrounding pump operations, the use of pump speed multiplier pattern in EPANET, tank operations, and extends the discussion of the operating points of pumps. Also, useful discussion is made on the effects of pressure on leakage. Because Chapters 4 and 5 need to retain the same structure as required by journals, these materials logically appear as a separate chapter. Journal article length constraints did not permit the inclusion of this material in the two manuscripts.

9.1. System Operation and Distribution Pressure

The operation of WDSs requires that system-wide pressures, flows, and tank water levels remain within acceptable limits. The operation of pumps and tanks affects the system operation the most. Pump operating costs make up a large proportion of the expenses of water utilities. It is therefore important to plan the operation of pumps to minimize energy consumption while maintaining the required standard of service and reliability. Most water utilities begin to use time-of-day energy pricing (in which the price of energy is highest in midday when water use is greatest), thus the goal would be to utilize maximum pumping capacity during off-peak period and consequently the cost of energy is minimized rather than the energy itself. Moreover, shifting the pump operation from the peak period to the mid and off-peak periods would increase pressures at night and early morning when demands are at the minimum and would result in more frequent pipe breaks and leakage. Hence, pump selection and operation are critical issues in operating WDSs.

Pumps are sized to handle required demands. The point of intersection between the pump characteristic curve and the system head curve, i.e., the operating point, represents the capacity at which the pump will operate (point 1 in Figure 9.1). For cases with storage tanks, the system head curve varies with fluctuations in tank levels, the operating point may change to points 2 and 3 in Figure 9.1 and pump selection becomes more critical. It is important to select the pumps so that they will operate within safe operating limits near the best efficiency point for both the high and low system head conditions. Operational controls are based on the time of day, i.e., pumps are programmed to turn on and refill a tank during off-peak periods; tank water level, i.e., pumps are set to activate when tanks drain to a specified minimum level and to shut off when tanks are refilled to a specified maximum level; or pressure in the system, i.e., pumps are set to turn on when pressures within the system drop below a desired value. Storage tanks provide service to meet fluctuating demands, to accommodate firefighting and emergency requirements, and to equalize operating pressures.

The desired volume of a storage tank is a function of the capacity of the distribution network, the location of the storage tank, and the use to which it is to be put into. The tank volume is determined on the basis of uniform rate (usually less than 24 hours, e.g., uniform 12-hour pumping), total 24-hr demand, and the amount of storage required for emergency and firefighting purposes (AWWA 1999). Regardless of the shape of the tank, three elevations are important for operation purposes: the maximum, minimum, and initial water elevations. The maximum and minimum elevations represent the highest fill level of the tank and the lowest the water level in the tank should ever be, respectively. The initial water level in the tank depends on the time of day at which the tank and pumps start to operate and the required volume of daily demand. To determine

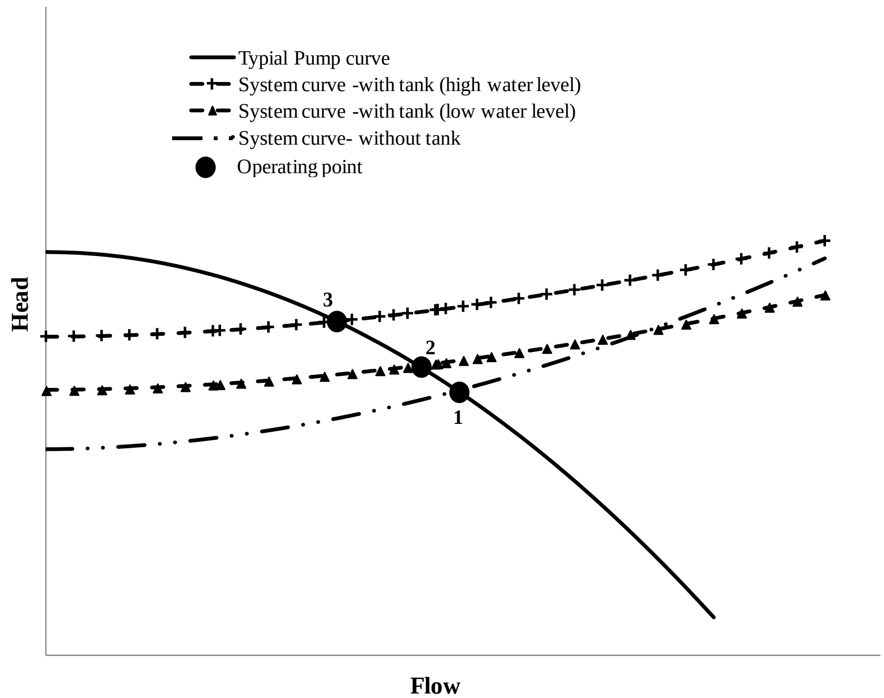


Figure 9.1. Typical system head curves and operating points for systems with and without storage tanks

the tank draining and filling characteristics and the trend of the water level in the tank, extended period simulations (EPSs) of 48 hours or longer are usually performed.

9.2 Scenarios

In order to illustrate the interactions between system operation, energy use/cost, and delivered pressure, two simple system configurations shown in Figure 9.2 are considered. The upstream reservoir has a constant head of 4 m and the elevation of the demand node is set to be 0. Hazen Williams' coefficient is considered as 110 for all pipes. A base demand of 250 L/s with a diurnal demand pattern depicted in Figure 9.3 is applied to the downstream node. To determine the energy costs, the base price of electricity is taken as \$0.11/kWh during the peak hours of the day and the price factors for other hours of the day is considered (Figure 9.3). The MPC is considered to be 35 m for all scenarios. To

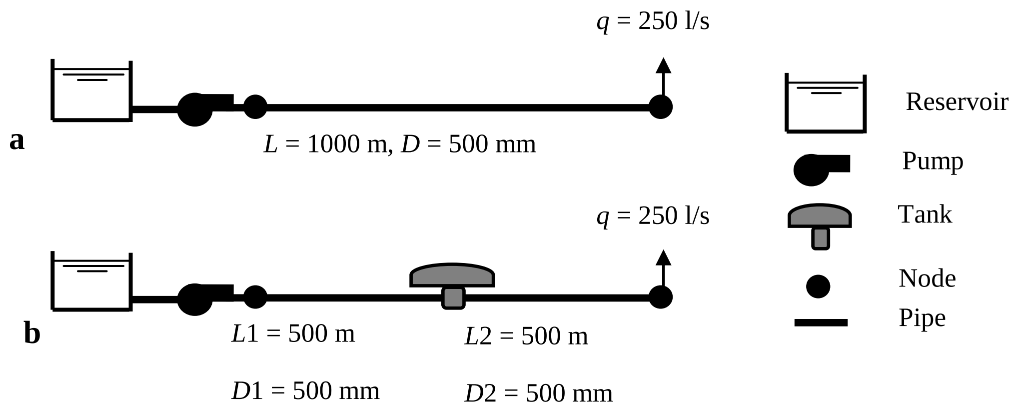


Figure 9.2. Single pipe system: (a) with and (b) without tank (*D* = pipe diameter; *L* = Pipe length; and *q* = nodal demand)

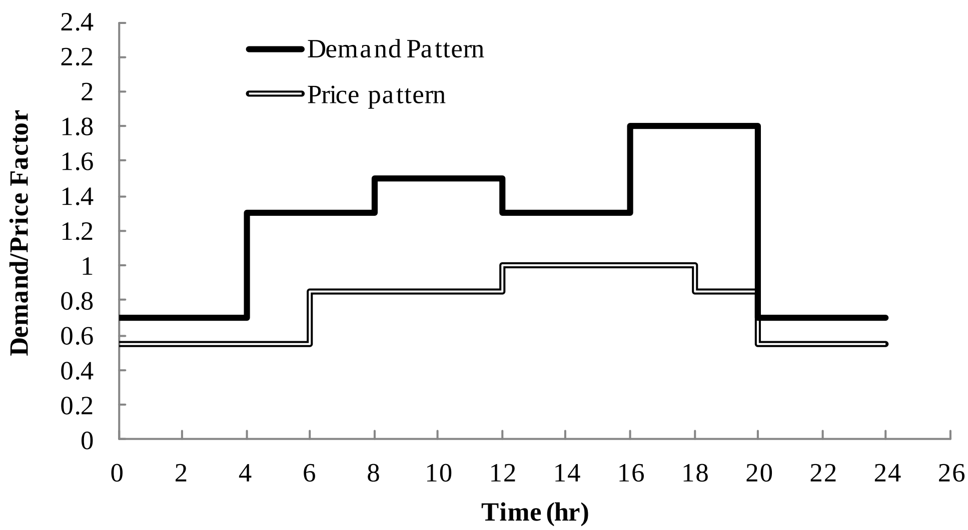


Figure 9.3. Diurnal demand pattern for water consumption and price pattern for energy cost

consider the pump efficiency, Eq. (5.5) is considered. The best efficiency is set to be 80% (that is assumed to be wire-to-water efficiencies, i.e., both motor and pump efficiencies) for all scenarios. Extended period simulations of 72 hours are performed to determine energy use, energy costs, and pressures at the most downstream node using EPANET2 (Rossman 2000). Table 9.1 shows the results of employing different pumping strategies to the systems shown in Figure 9.2. Energy use and energy cost range from 3964 kWh/day and 347 \$/day to 15440 kWh/day and 462 \$/day, respectively. Clearly from Table 9.1, it can be seen that the operating point is different for each scenario.

9.2.1 Scenario 1: 3 Pumps in Series

Centrifugal pumps can be put in series to increase pressure. A series arrangement also provides redundancy. In case of a pump failure, the pump design allows the flow to go through the defective pump and the remaining pumps can still deliver pressure. For the system shown in Figure 9.2(a), three identical pumps in series with pump characteristics defined by the curve $H = 24 - 4.8 \times 10^{-5}Q^2$ are considered, where H is in meters and Q is in liters per second. The effect of placing three pumps in series is dramatic in terms of

Table 9.1. Pump strategies, energy use and costs, operating points, and delivered pressures

Pumping strategy	Energy use (Kwh/day)	Energy cost (\$/day)	Operating point (*H* m, *Q* l/s)	Delivery head (at the most downstream node) (m)	
				Maximum	Minimum
1. 3 pumps in series	5519	462	18 m, 355 l/s	70	35
2. 3 pumps in parallel	3964	355	45 m, 160 l/s	44	35
3. Variable speed pump	4993	419	60 m, 330 l/s	54	35
4. Tank water level	8741	395	50 m, 400 l/s	51	35
5. Uniform 12-hour pumping	9652	347	60 m, 610 l/s	59	35
6. Uniform 6-hour pumping	15440	386	55 m, 610 l/s	53	35

excess pressure and energy cost which are 70 m and 462 \$/day, respectively. The scenario represents a case of high pressure, well over what requires.

9.2.2. Scenario 2: 3 Pumps in Parallel

Placing 3 pumps in parallel leads to a flatter combined characteristics curve that provides higher pressures at greater flows relative to the case with pumps in series. Three identical pumps in parallel defined by the characteristic curve $H = 60 - 5.8 \times 10^{-4}Q^2$ (H in meters and Q in liters per second) are considered for the system shown in Figure 9.2(a). This case indicates a noticeable improvement over Scenario 1 in terms of reduction in excess pressure and energy use and cost. Clearly, operating three pumps in parallel can improve the system performance while the energy consumption and energy cost are significantly lower than the case with pumps operated in series.

9.2.3 Scenario 3: Variable Speed Pump

Variable-speed pumps (VSPs) are frequently used in systems that do not have adequate storage. Their use increases the initial capital cost of pumping stations as well as maintenance expenses (Walski et al. 2007). The main advantages of VSPs are exploited when the operating conditions in the system are highly variable. For VSPs, as the pump speed changes, the pump curve is adjusted resulting in different operating conditions. For the system shown in Figure 9.2(a), the characteristic curve for the pump running at full speed is defined by $H = 80 - 1.8 \times 10{-}^{4}Q^2$, where H is in meters and Q is in liters per second. The pump speed, which can be controlled by a variable frequency drive (VFD), is set based on the time of the day. The pump is assumed to operate at full speed for the hours of 16-20 and the pump speed can be reduced to 0.85, 0.9, and 0.95 of the full speed for the hours of 0–4 and 20–24, 4–8 and 12–16, and 8–12, respectively. In the case of VSPs, the efficiency of the VFD should be considered in order to determine the total efficiency. This efficiency varies with the pump speed, drive manufacturer, and load. The total efficiency is determined by multiplying the efficiency of the VFD and the pump efficiency. The efficiency of the VFD is considered to be 0.927, 0.935, 0.948, and 0.96 for respectively the reduced speed of 0.85, 0.9, 0.95, and the pump operating at the full speed (Walski et al. 2007). Clearly from Table 9.1, this case shows an improvement over Scenario 1 in terms of reduction in excess pressure and energy use and cost but not as efficient as operating three pumps in parallel.

9.2.4 Scenario 4: Tank Water Level

Pumping into a system with storage tanks, whether that tank is an elevated tank or a ground tank on a hill, may represent an efficient operation in terms of energy cost. For a system including storage tanks, a more efficient and less costly constant-speed pump can be used to operate at its most efficient flow and pressure. For the system shown in Figure 9.2(b), the pumping strategy is considered on the basis of tank water level. The pump is set to be turned off when the tank level is at 15 m and to be turned on when the water level in the tank is 1 m. Note that the minimum water depth of 1m is not the very bottom of the tank since some storage is retained as an emergency water supply. For hypothetical systems studied in this chapter, the volume required for firefighting and emergency purposes is not computed and the minimum water level considered here is just an assumption. The tank is situated at the middle of the pipeline system (Figure 9.2(b)) and it has a cylindrical geometry with a diameter of 20 m (the volume of the tank is chosen to be one sixth of the total volume of the daily demand, i.e., 4380 m^3). The tank bottom elevation is at 37 m for supplying an MPC of 35 m. The effective height of the tank is considered to be 15 m for water level fluctuations. The pump characteristic is defined by the curve $H = 66 - 1.04 \times 10^{-4}Q^2$ (H in meters and Q in liters per second). This characteristic curve and the following characteristic curves are all selected to ensure that the pump operates close to the operating point. As shown in Table 9.1, in this case, the energy consumption, energy cost, and maximum delivery pressure are all at relatively high values compared to other scenarios.

9.2.5 Scenario 5: Uniform 12-hour Pumping

For the system shown in Figure 9.2(b), an uniform 12-hour pumping is adopted to pump water during the hours of the day from 0 to 8 and from 20 to 24. The aim of this strategy is to fill up the storage tank during off-peak periods and to supply demand from the tank during peak periods to minimize the energy cost. The tank has a cylindrical geometry with a diameter of 30 m. The tank bottom elevation is at 37.5 m to ensure the satisfaction of an MPC of 35 m. The height of the tank is considered to be 25.5 m for water level fluctuations and the initial water level is set to be 11 m based on the time of the day at which the tank and pumps start to operate and the volume of water demand at that time. The tank size is arbitrarily selected which is larger than what is used in practice in order to facilitate comparisons of all scenarios and to perform simulations. In real systems, the tank size depends on the volume of demand that should be supplied in a pressure zone that often is smaller than what is assumed in this chapter. The pump characteristic curve is considered as $H = 80 - 5.4 \times 10^{-5}Q^2$ (H in meters and Q in liters per second). The pump is selected to operate close to the desired operating point most of the times. This case shows the improvement in terms of reduction in energy cost. However, in terms of system energy use and excess pressure, this scenario is not as efficient as Scenario 2. Under this scenario, the delivery pressure is at the highest as compared to all the other scenarios since water level in the tank needs to be boosted during the pumping hours in order to provide the desired pressure at the most downstream node during pump shut-off times, thus high pressure is distributed throughout the system.

9.2.6 Scenario 6: Uniform 6-hr Pumping

The uniform 6-hr pumping scenario is included to the analysis to compare the result with the uniform 12-hour pumping and also to decrease the excess pressure in the system. The uniform 6- hour strategy is considered to pump water during hours of the day from 8 to 14 and 20 to 2. In the system configuration shown in Figure 9.2(b), the tank has a cylindrical geometry with a diameter of 27 m. The tank bottom elevation is at 36.5 m to supply the MPC of 35 m. The height of the tank is considered to be 18 m for water level fluctuations and the initial water level is set to be 11 m. The pump characteristic curve is selected to be $H = 73 - 4.9 \times 10^{-5}Q^2$ (H in meters and Q in liters per second). Although this scenario shows a reduction in excess pressure compared with the uniform 12-hour strategy, the system energy use and energy cost are greater than that scenario.

9.2.7 Comparison of Different Pumping Strategies

The results (Table 9.1) clearly show that the energy consumption, energy costs, and the maximum delivery pressure strongly depend on the operation strategy. Figure 9.4 depicts the delivery pressure at the most downstream node during a 72-hour simulation. As can be seen clearly from the results (Table 9.1 and Figure 9.4), the system that operates with a variable-speed pump can be reasonably better than the system with pumps in series but not as efficient as the system with parallel pumps. Figure 9.4 clearly shows that pumps operating in series provide high pressures to the system and also result in the maximum energy costs (Table 9.1). The parallel pumping strategy results in the minimum energy use, relatively low energy cost, and low excess pressure compared with other scenarios. Nonetheless, the system still needs a small storage tank for emergency and firefighting purposes. A major factor affecting the effectiveness of parallel pumping is the system head curve. A flatter slope on this curve for a given discharge reflects lower system head loss. Conversely, when the slope of the curve is steep, the ability of the pump to supply adequate flows is more limited by the piping of the system. The steepness of the system head curve determines the efficiency of running several pumps in parallel. If the system head curve is steep, the system operating with parallel pumps may be inefficient (Walski et al. 2007). Because of the technical and financial constraints on direct pumping, storage tanks may be presented in most real world distribution systems.

The storage scenarios result in more energy use although the energy cost is not as high as the system operates with series of pumps and variable speed pumps. The maximum delivery head, for three operating policies including storage tank, is also higher than that of the parallel pumping strategy. Operating systems with storage tanks require the operation of the network at high capacity during off-peak times or when the tank level is at the minimum set point. In order to increase the flow, the pressure needs to be increased and the head loss also increases, consequently, the energy use is increased. However, the energy cost depends upon the time of the day that can result in relatively

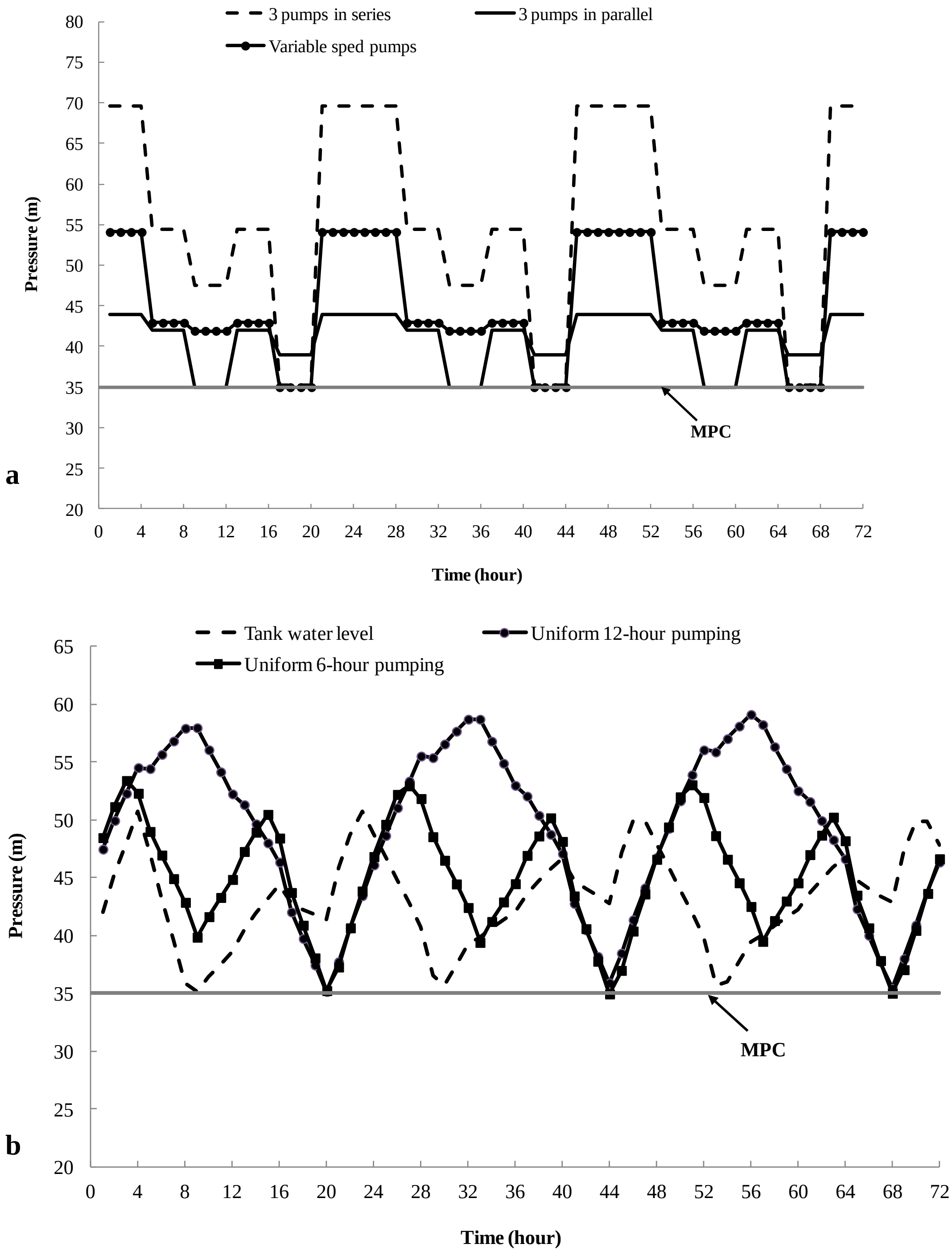


Figure 9.4. Pressure distributed in the system following different pumping strategies a) without tank and b) with tank

low cost. Therefore, the policy of storage tank operation seeks to minimize the energy cost rather than energy consumption. However, as mentioned earlier, because of the financial constraints and the required technical use (the steepness of the system head curve) of direct pumping strategies, pumping with storage tanks may still qualify as an efficient operation for some utilities.

To plan for system-wide optimization studies, other consideration including maximum delivery pressure is also required. Pressure management is now recognised as one of the most efficient and cost effective strategy for reducing excess pressure and leakage rate (Ulanicki et al. 2008). However, in this case, energy supply rarely changes. Additional benefits might be gained by extending pressure management to include strategies that decrease the energy supplied which can be achieved by reducing the pressure standards. While pressure standards are often set according to safety requirements during worst-case loadings, e.g., the greater of the maximum hour demand and the maximum day demand plus fire flow, the system would be over-designed during off-peak periods, and therefore the system faces an urgent challenge during off-peak periods. Indeed, a high value of the MPC requires relatively high system pressures which result in the increase in risks of pipe damage and leakage. Although it has been acknowledged that reduction in water pressure can influence the energy requirement and operation costs, there is an absence of literature regarding the consequences of changes in pressure standards, i.e., reduction in the MPC, that is beyond pressure management. Alterations to operational standards will undoubtedly affect the manner in which the system functions but if the impact of high MPC on WDSs could be better understood, the incentive to reduce it would likely be greater. This gives rise to the need for assessing the effectiveness of changes in pressure standards in order to obtain a clear picture of system performance under less stringent low pressure standards.

9.3. Energy Consumption and MPC

Quantifying the energy use while changing the MPC may be helpful to highlight the energy savings by decreasing the MPC. In water supply systems, most of the energy is consumed by pumping to provide the necessary heads and flows. The total head a pump needs to supply is called the total dynamic head (TDH) which includes (i) the head at the highest delivery point, (ii) the static head difference between the point of supply and the highest delivery point in the system, and (iii) the friction losses between the supply and delivery points. A pump must supply energy to lift water from a source to the point that satisfies a MPC and to overcome the frictional head loss along the pipe to ensure that the adequate demand reaches the downstream point.

The physical interaction that shows how a pump experiences reduction in MPC is illustrated in Figure 9.5. Two typical characteristic curves of centrifugal pumps are depicted in Figure 9.5. If the MPC is H_{c0} and total demand (the required demand, Q_d, and leakage rate, q_0) is $Q = Q_d+q_0$, at the downstream point, the total dynamic head is $H_{T0} = H_{c0}+H_{f0}$, where H_{f0} is the frictional head loss, as indicated in Figure 9.5(a). This point is denoted on the pump curve as operating point 1, which corresponds to the original impeller speed. The energy which must be supplied to the pump corresponds to point 1 in Figure 9.5(b). Now suppose that the MPC is reduced to H_c, under this condition, the pump has to supply a dynamic head of $H_T = H_c+H_f$, where H_f is the frictional head loss when pressure is reduced, to overcome the frictional loss across the pipe and to satisfy demand $Q = Q_d+q$, where q is the leakage rate when pressure is reduced at an MPC of H_c. This pumping requirement corresponds to operating point 2 in Figure 9.5(a) which is associated with the lower impeller speed. The change of operating point can be achieved by substituting with smaller pump that corresponds to a lower power curve with less energy consumption, as depicted in Figure 9.5(b).

If a lower value of MPC is to be considered, less power is required. Overall, the change from the higher pressure to the lower one results in a decrease in Break Horse power (BP). The net rate of pumping energy savings is simply equal to the difference in the power requirements between the two scenarios of the MPCs and is schematically represented by the difference between points 1 and 2 in Figure 9.5(b). The only response considered is the shift in the pump's operating point to bring a new pump with an equal

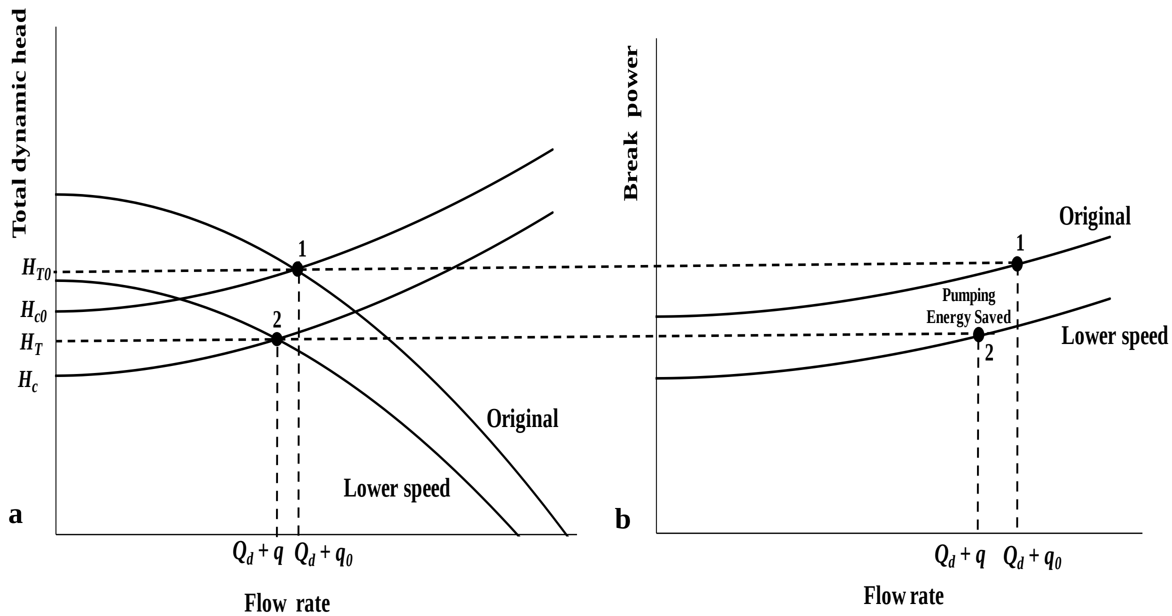


Figure 9.5. (a) Pump characteristics and system curves for typical centrifugal pumps; and (b) Brake Horse Power curves for low and original speed pump settings

efficiency into the revised scenario. Such an approach may be feasible for the existing systems where there is no pipe renewal (the pipe's friction is not changed) and pump efficiencies for the two scenarios of changing pressures are assumed to be equal. The necessary power required to achieve the MPC can be represented by pump curves of different shapes. Of course, pump operation is controlled by a unique discharge-pressure relationship. This means that a change in operating point is determined by both the flow and pressure. Because reducing the TDH implies that both the delivery head and flow from the leaky system are to be lowered, the characteristic curve corresponding to the smaller pump must contain the new operating point.

9.3.1. System Configuration

9.3.1.1 A Simple Pipe Line

To illustrate the fundamental behavior of distribution systems in response to changes in the minimum pressure standards, a simple system configuration shown in Figure 9.2(a) is considered first. To easily compare the relative reduced energy use against the relative reduction in MPC, the upstream reservoir level and the elevation of the demand node is both set to be 4 m. All simulations are performed with the use of EPANET 2. For leaky systems, the leakage rate is modeled by the value of the emitter coefficient and it is set to be 15% meaning that the total amount of water demand, in this case, is 287.5 l/s. For no-leak systems, if the MPC is reduced, the reduction in energy is simply proportionate to the amount of reduction in pressure

$$\frac{E_{save}}{E_i} = \frac{\Delta H}{H_{m0} + h_f} \tag{9.2}$$

where E_i is the initial energy use, E_{save} is the energy saving due to the reduction in MPC, ΔH is the amount of reduction in MPC, and H_{m0} is the initial MPC. Line *A* in Figure 9.6 shows the energy reduction response for the single pipe system without leakage. However, the actual energy saving is more than the amount indicated by line *A* because of leakage. If a system is leaking, reduction in pressure causes a decrease in both flow and pressure consequently the system energy use and leakage are reduced as indicated in Figure 9.6 by line *B* and curve *C*. But this is not all. Delivery pressure would also improve due to the reduced friction losses in the system as depicted in Figure 9.6 by line *D*, thus the system might now be operating in a better condition than it was. Therefore, the system performance could be improved by a reduction in MPC because of reduced system energy use, leakage, and head loss. As indicated in Figure 9.6, all lines and curves associated with all relative reductions lie below the 1:1 line indicating that the percent of decrease in reduction of energy use, leakage, and head loss is less than the percent of reduction in MPC.

The main assumption made to develop all lines in Figure 9.6 is that the pump efficiency is considered to be equal for all scenarios involving changes in MPC. However, if the pump efficiency changes, the relative reduction in energy use may become either more or less than what is indicated in Figure 9.6. This is confirmed by Figure 9.7 depicting the relative energy use for two different delivery pressures. The

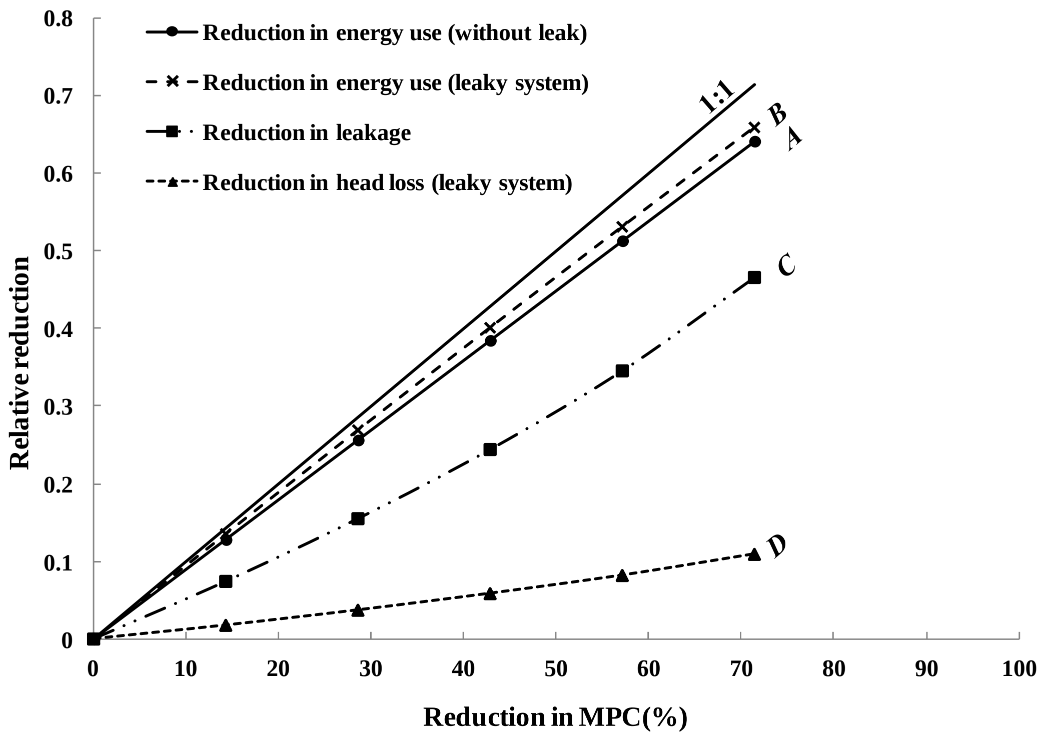


Figure 9.6. Relative reduction in energy use, leakage, and head loss as a function of reduced MPC

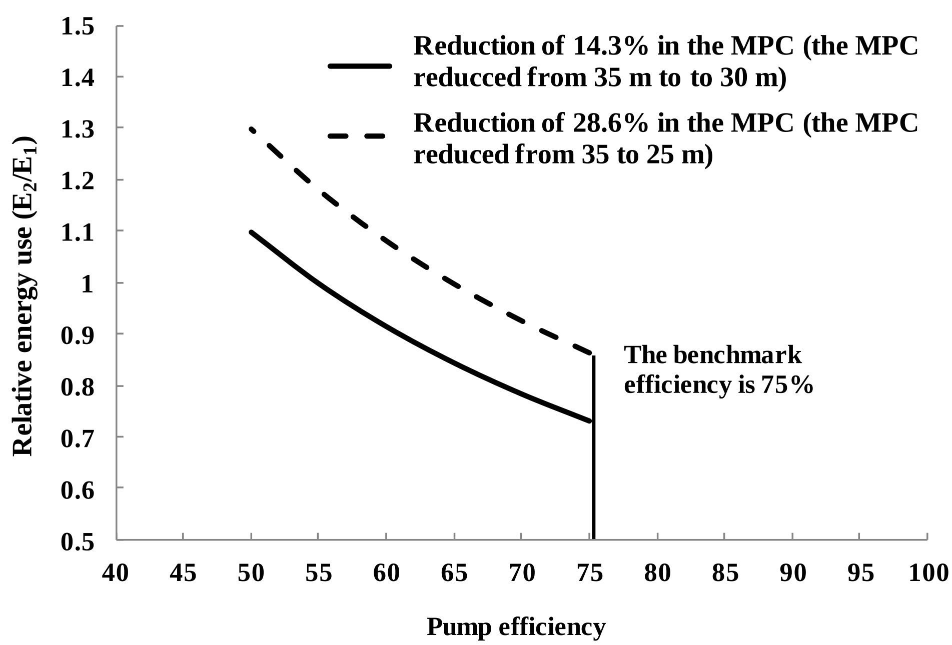


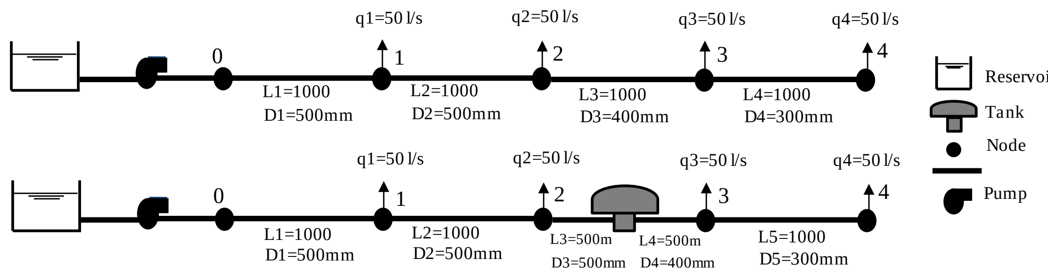
Figure 9.7. Effect of pump efficiency on relative energy use due to changes in MPC

system energy use could be increased ($E_2/E_1 > 1$, E_1 and E_2 are the energy use before and after the reduction of MPC, respectively) if the pump efficiency would be set to be 55% and 65% for the reductions of 14.3 % and 28.6 % in MPC, respectively.

9.3.1.2 Series of Pipes

Changes in energy use resulting from a reduction in the MPC depend upon a wide variety of factors including system topology, pipe characteristics, pump arrangement, and operating policies. To demonstrate the fundamental influence of changes in the MPC, a series pipe system with two configurations is established (Figure 9.8). To evaluate the effects of reduction in the MPC on energy use of the scenario with storage, a direct pumping configuration (no tank scenario) is also considered. Although, real and more complex networks are not necessarily represented by these basic systems, these simple systems have been selected to provide some insights about the effectiveness of changes in the MPC on energy consumption and maximum operating pressures. Water level in the upstream reservoir is set at 5 m and the elevations of all nodes are considered to be 0 m.

To examine the frictional loss effect on the system, two friction scenarios are considered, the low friction scenario with a C_h factor of 130, where C_h is the Hazen Williams's coefficient, and the high friction scenario with C_h = 70. In the configuration with storage, the tank is located between Nodes 2 and 3 and it has a cylindrical geometry with a diameter of 26 m. For the scenario in which the MPC is set to be 10 m, the elevation of the tank bottom is set to be 39 m and 55.5 m respectively for low and high friction regimes. Because the system is completely hypothetical, a high elevation for the tank bottom is assumed to satisfy the predefined MPC. In real systems, the elevation of the tank depends on the system topology and the topography of the surrounding areas. Since the residual pressure in the system with storage directly depends upon the tank level and pump characteristics, the tank bottom elevation and the pump characteristic curve are changed simultaneously so that the predetermined MPCs is supplied at Node 4 for different scenarios. For all scenarios, the height of the tank is considered to be 26 m and the initial water level is set to be 11 m.



D=pipe diameter; L=Pipe length; and q=nodal demand

Figure 9.8. Series pipeline system configurations

For the scenario where the MPC is 10 m, the system is considered to be driven by pumps defined by the characteristic curve shown in Table 9.2. To achieve higher MPC for other scenarios, in the system without storage, larger pumps are taken into account to supply higher head. The pump efficiency is considered according to Eq. (5.5) with the best efficiency point of 80% for all scenarios. All pumps for different scenarios are selected to operate close to the operating point most of the times. For the storage configuration, the uniform 12-hour pumping and for the no storage scenario, 3 identical pumps in parallel with on-off pump controls are considered. A base demand of 50 L/s with a diurnal demand pattern depicted in Figure 9.3 is assigned to all the demand nodes. The energy costs are determined considering the base price of electricity of \$0.11/kWh during the peak hours of the day and the price factors shown in Figure 9.3.

The performance of the two systems shown in Figure 9.8 is tested for all scenarios with MPCs changing from 10 m to 35 m using EPANET2 simulations. To model daily demand and tank level fluctuations and to ensure stationary pressure at nodes, extended period simulations of 72 hours are performed. Figure 9.9 shows daily total energy cost against changes in the MPC for the system configurations with both low and high friction regimes. As expected, energy costs decrease for all scenarios as the MPC is reduced. For each friction regime, energy costs are greater for the storage configuration compared with its no storage counterpart as the MPC changes. Although the pump is operating during the time of the day with low electricity price in the storage configurations, the system energy

Table 9.2. Pump characteristics curves

H_m(m)	System with storage		System without storage	
	Low friction	High friction	Low friction	High friction
10	H = 108-108Q^2	H = 188-188Q^2	H = 65-966Q^2	H = 114-1694Q^2

H_m is the MPC in the system in meters; H is pumping head in meters; and Q is pumping flow rate in m^3/s.

use is much more than what is incurred by the direct pumping configurations, therefore, the energy cost is greater than the system without storage. The trend of the mechanical flow energy supplied by the pump is similar to the energy cost curves for each system and scenario. In Figure 9.10, a plot of the daily energy consumption for all scenarios against the MPC is shown. As it is obvious, power requirements are greater for system with storage and higher friction pipes. Comparison of Figures 9.9 and 9.10 reveals that the energy use for the high friction system without storage is lower than its storage counterpart operating in the low friction regime but the energy cost of the no-storage configuration in high friction is greater than its storage counterpart under low friction regime. This highlights the role of friction in system energy use and effects of energy tariff and operating strategy on energy cost.

9.4. Pressure-Leakage Relationship

Leakage rate has long been known to be related to the internal pressure of the pipe at leaky locations. Thus, lowering the pressure throughout the pipeline systems causes leakage reductions. Colombo and Karney (2002; 2005) examined the impact of leaks on the energy consumption in water supply systems. They concluded that leaks increase operating costs in all systems and energy costs increase more than proportionately with leakage. Pressure management is regarded as an effective way to control the amount of water lost in WDSs. In the pressure management process, the factors related to losses are identified through calculation for minimum night flow conditions since most of the users are not active during the night and pressures are high throughout the systems (Walski et al. 2006a; Gomes et al. 2011; Campisano et al. 2012). Pressure management is usually

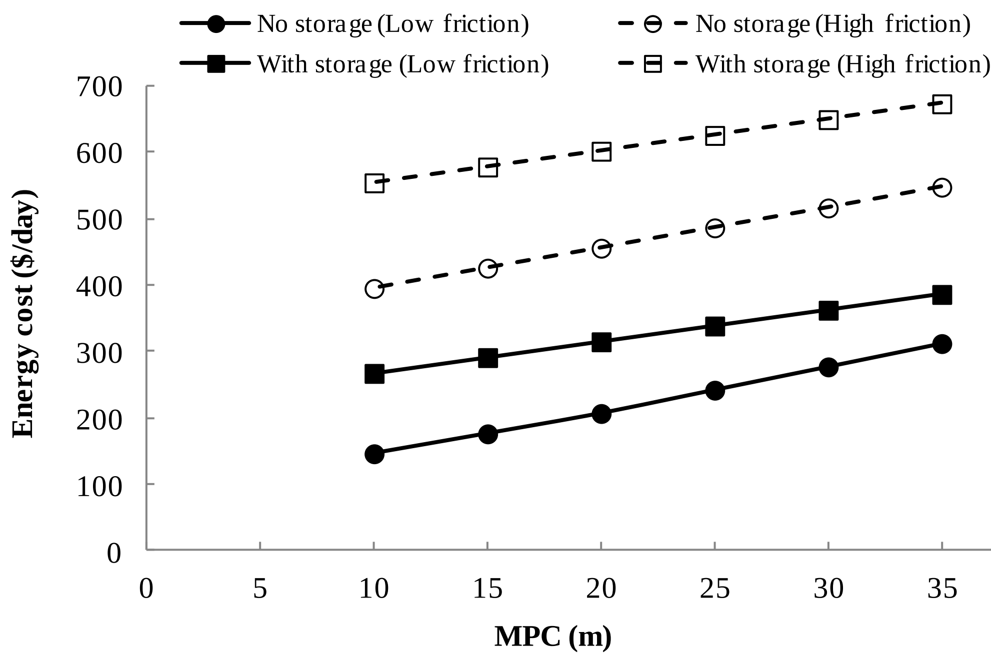


Figure 9.9. Energy costs for all scenarios

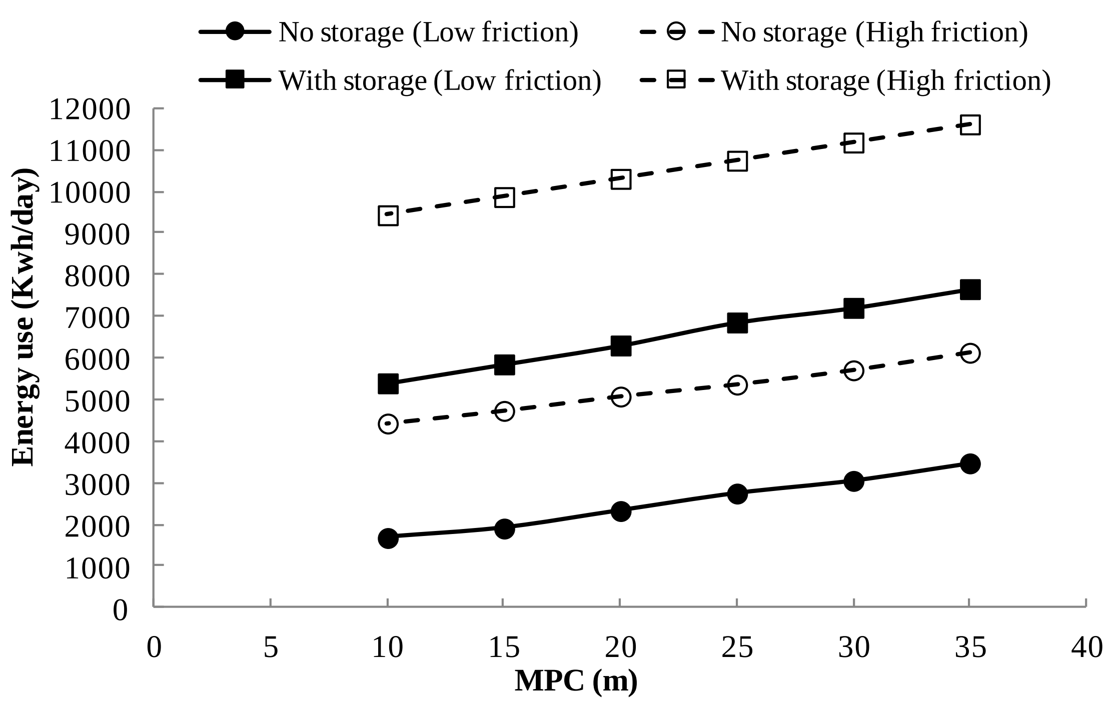


Figure 9.10. Energy consumption as a function of MPC

performed by installing pressure reduction valves in water networks and the number of valves and their locations should be optimized and calibrated in the pipe networks which are challenging tasks (Liberatore and Sechi 2009). Flow through leaks can be calculated as

$$q = CH^a \tag{9.3}$$

where q is the leakage rate, H is the pressure at the leaky location, C is the discharge coefficient, and a is the exponent that is traditionally assigned a value of 0.5, however, values from 0.52 to 2.59 are also recommended (Lambert 2000). As it is clear from Eq. (9.3), leakage varies nonlinearly with pressure and can be reduced with a decrease in system pressure. If a is considered to be 0.5, reducing the pressure by 30% would reduce leakage by 17%. Reduction in pressure may not only decrease leakage rate but also may reduce the rate at which new leaks occur (Lambert 2000).

The coefficient C in Eq. (9.3) can change depending on leak size, flow regime through the leak, and internal pressure of the pipe. In practice, accurate knowledge of leak sizes is highly unlikely, thus C in Eq. (9.3) is subject to uncertainty. To show the variation of leakage rate due to changes in discharge coefficient and pressure, the mean-centered first-order method (MCFOM) can be used to estimate the first and second moments (i.e., mean and variance) of leaks. In an engineering analysis, the MCFOM is employed to generate moments of a dependent variable in terms of a straightforward function of the first two moments of the independent variables when the probability distribution functions (PDFs) of the independent variables are not available. Therefore, the method is a practical alternative to approximate the mean and variance of dependent variables (Ang and Tang, 2007). To determine the mean and variance of the leakage rate, the function q is expanded as a multivariate Taylor series about the means of discharge coefficient and pressure. The expressions assuming that the discharge coefficient and pressure are statistically independent of each other are written as (Ang and Tang 2007)

$$E(q) \cong g(\mu_c, \mu_H) + \frac{1}{2}\left[\sigma^2{}_c\left(\frac{\partial^2 q}{\partial c^2}\right) + \sigma^2{}_H\left(\frac{\partial^2 q}{\partial H^2}\right)\right] \tag{9.4}$$

$$Var(q) \cong \sigma^2{}_c\left(\frac{\partial q}{\partial C}\right)^2 + \sigma^2{}_H\left(\frac{\partial q}{\partial H}\right)^2 \tag{9.5}$$

In Eqs. (9.4) and (9.5), $E(q)$ is mean of the leakage rate, g (μ_c, μ_H) is the value of the leakage rate when the discharge coefficient and pressure assume their respective mean values; $\partial^2 q/\ \partial c^2$ and $\partial^2 q/\ \partial H^2$ are the second order partial derivatives of function q evaluated at μ_c and μ_H, respectively. $\sigma^2{}_\mathrm{c}$ and $\sigma^2{}_\mathrm{H}$ are respectively the variances of the discharge coefficient and pressure, Var (q) is the variance of the leakage rate, $\partial q/\ \partial c$ and $\partial q/\ \partial H$ are the first order derivatives of q evaluated at μ_c and μ_H, respectively. Note that the Taylor series estimate of the mean in (9.4) is truncated after the second-order term and the Taylor series estimate of the variance in (9.5) is truncated after the first-order term. Figures. 9.11 and 9.12 show the mean and standard deviation of leakage against the coefficient of variation (c.o.v) of discharge coefficient and pressure. Figure 9.11 indicates that leakage decreases as pressure reduces and for each curve the variability in pressure does not significantly affect the leakage rate. Note that only the variance of pressure contributes to the mean value of leakage rate according to Eq. (9.4) because $\partial^2 q/\ \partial c^2 = 0$, thus in Figure 9.11, the mean value of leakage rate is plotted against the c.o.v of the pressure. Figure 9.12 shows the standard deviation of leakage rate for different values of c.o.v's of pressure and discharge coefficient. Clearly from Figure 9.12, the variability in both pressure and discharge coefficient affects the standard deviation of leakage rate. However, leakage is more sensitive to the uncertainty in the discharge coefficient, thus if the discharge coefficient has not been estimated accurately, the computed leakage rate from Eq. (9.3) is subject to significant error. This highlights a challenge in assessing the pressure management strategies, i.e., the behavior of leaks should be accurately modeled in order to accurately quantify the effect of pressure modifications.

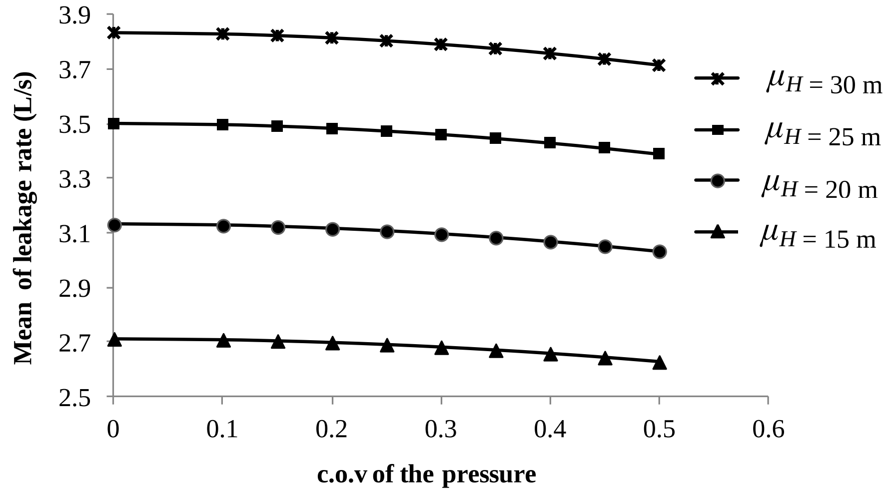


Figure 9.11. Mean of leakage rate as a function of the c.o.v of pressure ($\mu_c = 0.7$)

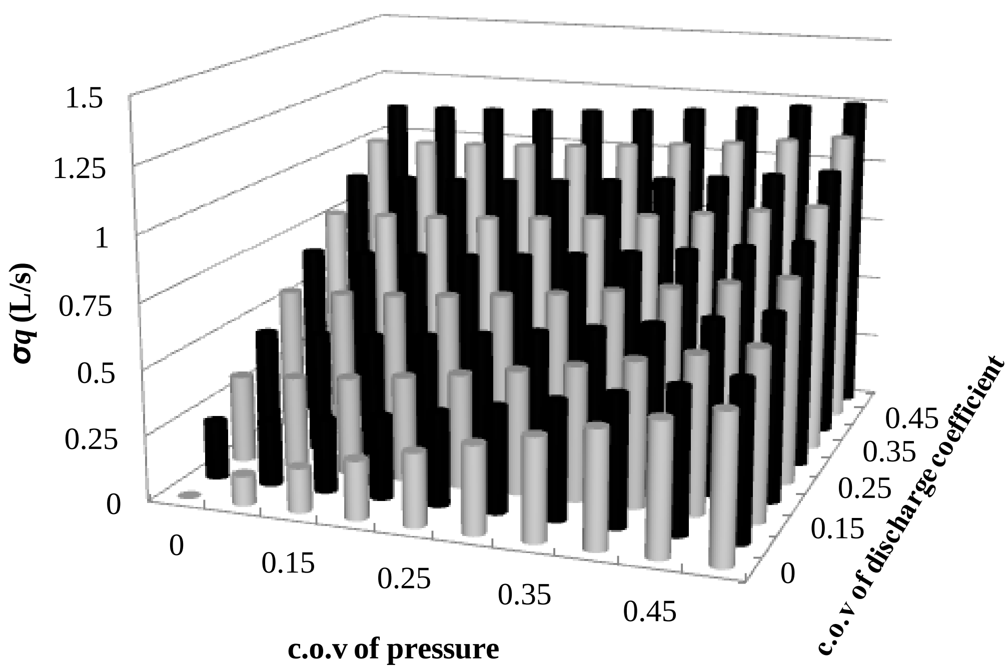


Figure 9.12. Standard deviation of leakage rate as a function of c.o.v's of pressure and discharge coefficient

While it is acknowledged that there is a link between leaks and internal pressures of water systems, the relation between the pressure standards and leakage is a recent issue. The priority here is to evaluate how changes in the MPC affect the water loss, energy requirement, and maximum operating pressure in leaky systems with and without storage. To assess the fundamental behavior of leaky systems in accordance with changes in the MPC, two hypothetical systems shown in Figure 9.8 are considered. Leakage rate is modeled as a percentage of demand although it has no revenue for municipalities and is not usable for customers (Colombo and Karney 2002). The performance of the aforementioned systems was tested for a low friction scenario, C_h = 130, at six different MPCs, ranging from 10 m to 35 m, using EPANET2 simulation. A low friction scenario is just considered here and it could be concluded that operating in the higher friction environment deteriorates the system performance in terms of leaks and energy consumption (Colombo and Karney 2005). Leaks at nodes, in EPANET2, are modeled with the use of the emitter feature in which the flow rate is considered to be a function of pressure at each node. Leakage rate is controlled by the value of the emitter coefficient and in the first stage of analysis, it is set to be 30% meaning that the total amount of water lost through the leak in a 24-hour cycle is 30% of the total daily demand volume. This establishes as the reference leakage when the MPC is maintained at 35 m in the system. Leaks then are computed for all the MPC scenarios.

For the two configurations shown in Figure 9.8, leaks are defined at nodes 1, 2, 3, 4 and at each node the same value of the emitter coefficient is assigned. For the scenario in which the MPC is 35 m, the pump characteristic curves are defined by $H = 122 - 72Q^2$ and $H = 69 - 567Q^2$ (H is in m and Q is in m^3/s) respectively for the storage and no-storage configurations. For other scenarios of reduction in MPC, a new pump curve and tank elevation are set so that the predetermined MPC at the most downstream node is achieved. For the storage configuration, the volume of tank is considered 30% greater than the no-leak system and the volume is held unchanged for other scenarios. The demand and price pattern as well as pumping strategy are taken into account identical to what was considered for the two configurations shown in Figure 9.8. Leakage for each scenario is determined as a percentage of the total daily demand volume. Pressures at each node and energy costs are determined by EPANET2.

Figure 9.13 shows the leakage percentage for the two configurations against the MPC. Clearly, a reduction in delivery head decreases leakage for both configurations. The leakage percentage for direct pumping is less than the storage configuration, similar to what was found in the study of Colombo and Karney (2005). The storage configuration distributes higher pressure in the system compared with the no-storage configuration since the water level in the tank needs to be boosted during the pumping hours to satisfy the pressure requirement at the most downstream node during pump shut-off periods. Therefore, the pressure at the other nodes becomes more than those in the no-storage configuration which results in more leaks. Figure 9.14 depicts the leakage reduction curves. For the no-storage configuration, the leakage reduction response is more than its storage counterpart in terms of reduction in the MPC. Energy costs are expected to be lower for the two configurations where reduction in the MPC is greater. Figure 9.15 which plots the decrease in energy costs against reduction in the MPC confirms this presumption. The two curves lie below the 1:1 line implying that the percent of decrease in energy cost is less than the percent of reduction in the MPC. From the results shown in Figures 9.14 and 9.15, a 30% decrease in the MPC causes a reduction of about 12.5% and 10% in leakage and 23.5% and 13% in energy costs for respectively the no-storage and storage configurations. Figure 9.16 shows the decrease in the maximum operating pressures in leaky systems against reduction in MPC. Unlike the no-leak system, changes in excess pressure are more noticeable for the no-storage configuration with respect to reduction in the MPC. From Figure 9.16, excess pressure decreased by 7.2% and 4.5%, for respectively the no-storage and storage configurations, due to a 50% reduction in the MPC.

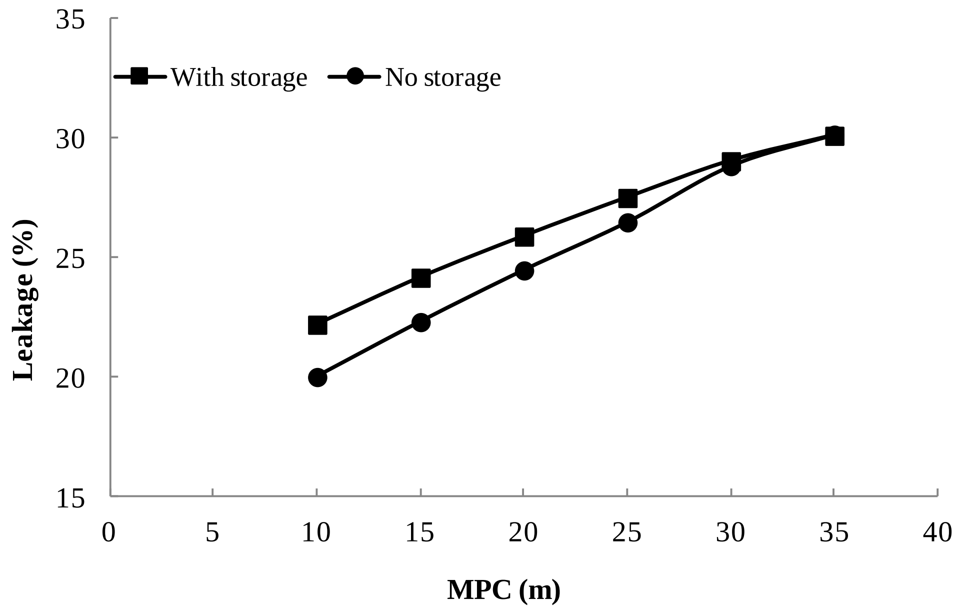


Figure 9.13. Leakage response to changes in MPC

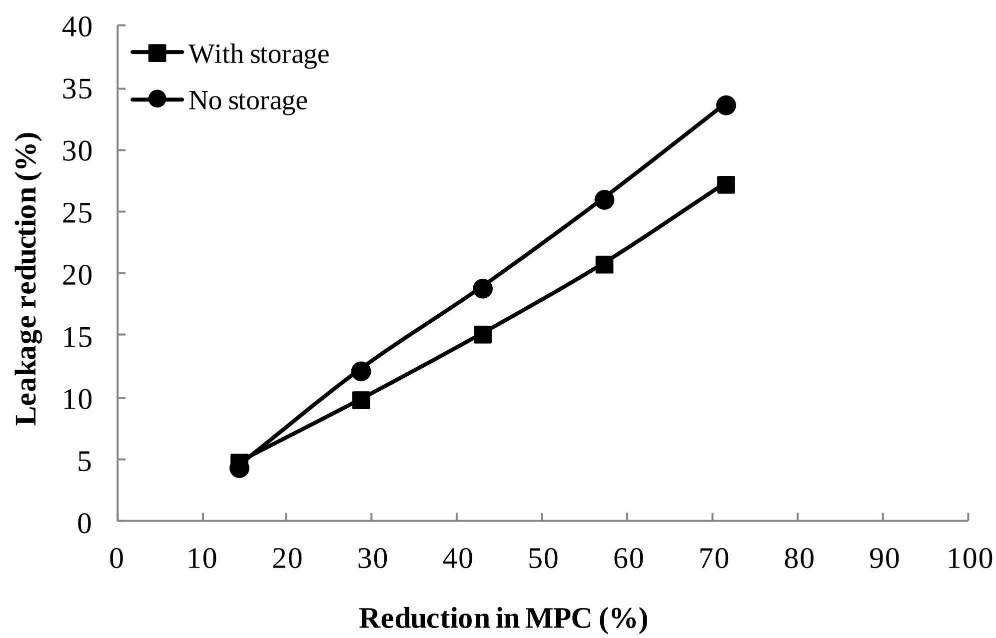


Figure 9.14. Leakage reduction response to reduction in MPC

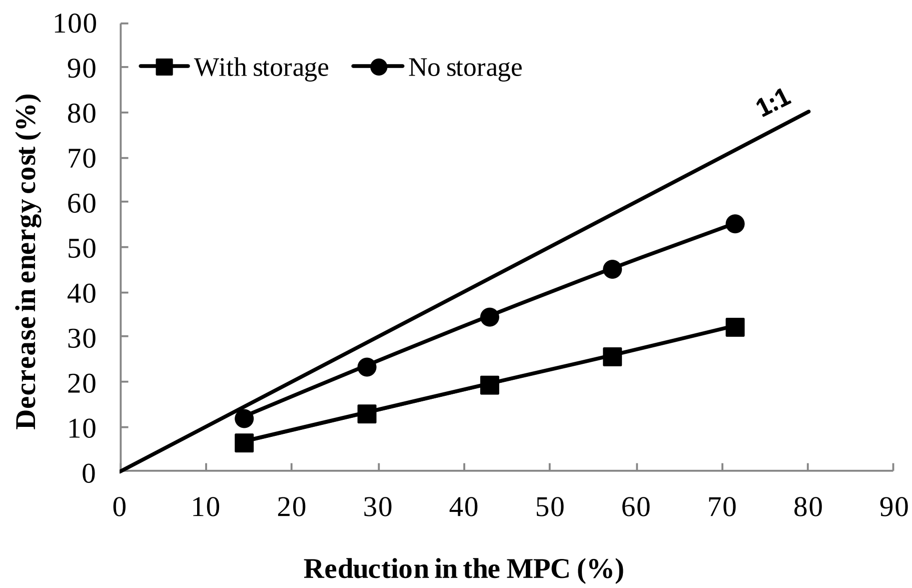


Figure 9.15. Energy cost response to reduction in the MPC in leaky systems

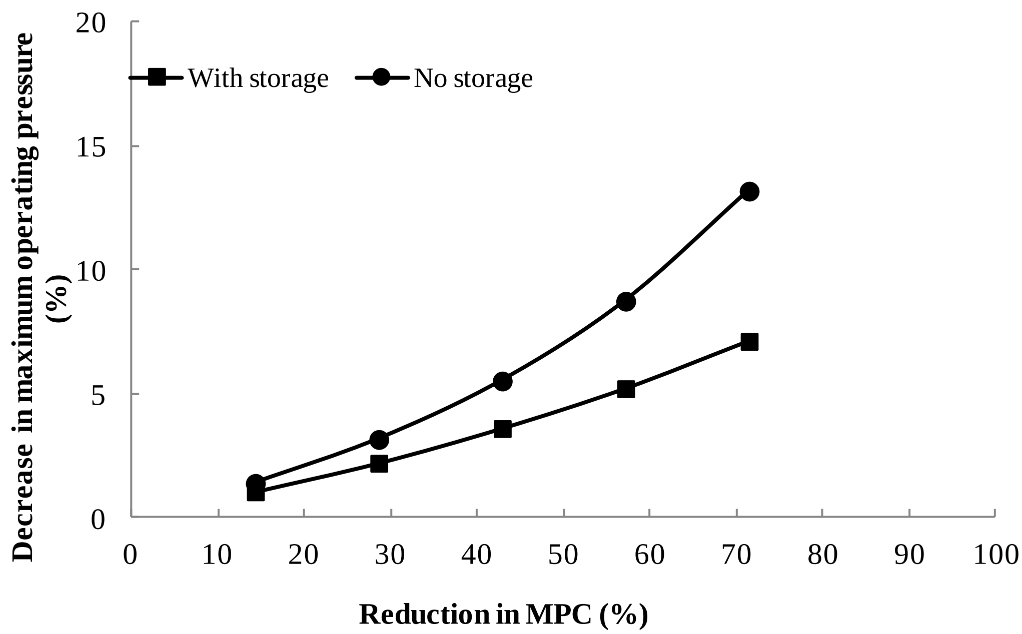


Figure 9.16. Decrease in maximum operating pressure against reduction in MPC in leaky systems

9.5 Summary

Operating pressure is a key factor influencing the energy use of WDSs. Control and reduction of operating pressure can help reduce energy expenses and leaks. For two simple systems considered in section 9.2, the inclusion of a storage tank may not be the most efficient operating policy because of high energy consumptions during the time when the pump needs to be turned on which distributes high pressures into the system. The results clearly indicate that the policy of tank operation tends to minimize the energy cost rather than energy consumption, consequently the environmental impact increases. The parallel pumping strategy in a system without storage contributes to low energy uses and costs as well as low excess pressures, of course in this case, a small storage tank is still required for emergency and firefighting purposes. Reduction in energy supplied, which could be achieved by decreasing the pressure standards, has a three-fold benefits of reduction in energy use, leak, and excess pressure.

The most important factors contributing to a system's energy response to changes in the MPC are the inclusion of storage in the system and the pipes' friction. For two series pipe systems considered in section 9.3.1.2, the difference in energy consumption between the two friction scenarios is evident. A 30% reduction in the MPC would decrease about 10% to 23% of energy consumption, and about 3.5% to 17.5% of excess pressures for the systems under study. Reduction in pressure standards decrease leaks, however the inclusion of storage capacity decreases the leakage reduction response to the decrease in MPC. For leaky systems (section 9.4), 30% decrease in the MPC causes a reduction of about 10% to 12.5% in leakage, 13% to 23.5% in energy costs, and 2.5% to 3.5% in excess pressure for the systems with pipes in series. The results of this chapter are based on hypothetical systems designed to highlight the potential impact of high system operating pressure on energy use and leakage. Reducing the MPC saves energy and decreases leaks; however, study to determine the appropriate degree of reduction in MPC needs to be continued.

Chapter 10

Summary, Conclusions, and Future Work

10.1 Summary

WDSs are built to safely deliver adequate quantities of drinking water to end users under sufficient pressures. Improved efficiency in the operation of WDSs is not a new objective but perhaps its urgency is especially acute given the growing concerns of resource scarcity, environmental degradation, the on-going challenges of urban growth, and deteriorated water supply infrastructures. As a part of this for better efficiency, there has been a lot of research about controlling pressure or pressure management in recent years. Some of these inquiries have been the direct result of the limitation of capital resources to build, operate, or rehabilitate WDSs. High and low pressures both put WDSs at risks. High pressure systems may cause more frequent pipe breaks and increase energy use and leakage. Low pressure systems cause consumer complaints, make the system more susceptible to negative pressures and possibly contaminant intrusions during transient events. Hence, pressure standards are required to ensure that safe, reliable and economic operations of WDSs are achieved. However, pressure criteria under which water is delivered to customers differ around the world.

This thesis has made two basic research contributions to the study of WDSs. First, it opens a window for debate between the tensions that reduction in pressure standards may create and the benefits that may be achieved by reducing pressure standards. An attempt is made to highlight the problematic issues associated with pressure and to illustrate the intrinsic relationships between pressure and other factors influencing the performance of WDSs. A more comprehensive picture of high and low pressure problems is provided, and a clearer understanding of the consequences and challenges may be achieved prior to making changes to the MPC. Second, by focusing on the linkage between pressure standards and transient pressures, how destructive transient pressures may be controlled to limit down surge pressures to an acceptable limit even with relatively low delivery pressures is illustrated. A novel idea was also developed in order to control pressure changes during hydrant operations.

10.2 Conclusions

The following is a list of main conclusions reached and contributions made in this thesis.

1- The need for pressure criteria and how these criteria may be violated or achieved were critically appraised. Pressure criteria certainly influence the performance of WDSs. While in WDSs design the attempt is to maintain pressures above an MPC, this criterion is often temporally violated during transient conditions. Some metrics were proposed to quantify the violation of MPC during transient events and they are useful to determine the intensity of transgression in WDSs. Considering more completely issues associated with pressure standards implies that there may be still room for a more thought and widespread debate on how pressure standards should be defined and interpreted in modern water supply practice.

2- The pros and cons of reduction in the MPC were explained. Reduction in the MPC has both benefits and costs. Reducing the MPC may cause decrease in water demands and leakage, and quite significantly, energy costs as well. However, if water pressure is reduced, booster pumps must be installed at buildings with heights that are more than the minimum supplied pressure; thus some of the savings are moved to building owners. Lowering the MPC can also reduce the probability of pipe breaks. Water quality will be affected by changes in the MPC but in existing systems, a change in the MPC does not affect water quality unless pipes are rehabilitated or storage volumes are changed. Reduction in the MPC causes the system to be under lower pressures and makes the system more vulnerable to low/negative transient pressures. Policies to control and avoid low pressure events, may not yet be fully linked to minimum pressure standards and how the MPC is enforced/ensured in WDSs is also an important issue.

3- The effects of changes in delivery pressure on system energy use and cost, leakage, excess pressure, and environmental impacts were examined. The decrease in head loss due to the reduction in pressure is more noticeable in systems with high leakage rates. Pressure reduction aimed at leakage reduction and energy saving is more effective in newer systems with smoother pipes. The inclusion of storage tank causes the system energy use to increase because of boosting pressures during off-peak times or when the tank level is at the minimum set point. The results clearly showed that pressure should be limited not only for the purpose of the usual benefits of leakage reduction and possible decrease in pipe bursts, but for the key reason that energy must be paid for, both financially and environmentally.

4- A probabilistic approach to quantifying the expected pipe break rates of WDSs was presented. The probabilistic approach considering uncertain demands and pipe roughnesses, was applied to compute the expected pipe break rates in the Hamilton network. The results show that higher values of MPC require relatively high system pressures and result in more frequent pipe breaks. The results also reveal that the frequency of occurrence of low pressure events is very small. These findings may motivate utilities to rethink about pressure standards. The expected pipe break rates defined in this thesis can be used as an indicator for the design of WDSs and can also be easily incorporated into an optimization scheme in order to minimize the expected pipe break rates.

5- The role of MPCs and how they affect system response in transient conditions was explored in order to raise awareness about issues that can arise from changes in steady state pressures. Two case studies were developed to demonstrate the role that delivery pressure plays in transient surges. The results showed that MPC are often violated during transient events due to pressure fluctuations and some care might be needed to define exactly what MPC limits really mean. Moreover, transient events can put water systems at risk of loss of pressure even if systems are normally operated under high pressures. The results also indicated that, not surprisingly, increasing the MPC in WDSs design may be an inefficient surge control strategy. However, the sensitivity to severe low pressure events decreases as the MPC increases, this confirms that a link between standards, operation and performance always exists. The results highlighted the notion that even those WDSs that are operated under low pressures have risk of high pressure transients, but those transient pressures can be efficiently controlled using surge control strategies. Considerable thoughts need to be given to what standards mean, how to design for their satisfaction, and what constitutes violations.

6- The risk of loss of pressures due to simultaneous operation of fire hydrants can be controlled by extending the opening time of hydrants. However, determining a required opening time for every hydrant is a challenging task since there are many hydrants scattered at different locations of WDSs. A portable control device, which can be quickly attached to a hydrant, is useful to control transient pressures even if fire crews open the hydrant as fast as they are able to. The use of such a portable device for limiting the down surge pressures was explored by creating a down-surge control boundary in a pipe system during hydrant operations. This idea was developed by numerical simulation and a considerable potential for transient protection has been demonstrated in the numerical examples. The results indicated that the down-surge can be maintained above a desired level, during the opening of a valve, with the use of the proposed surge control algorithm. The opening of the control valve is adjustable in response to the transient pressures to be controlled. Importantly, such a control valve could be readily portable and quickly attached to a hydrant by fire crews. The internal pressure of the pipe feeding to a hydrant is the force required to operate the control device; therefore, this device can limit the down-surge to be above a desired level with the function of the control device effectively independent of typical system characteristics.

10.3 Future Work

With a series of hypothetical systems, this preliminary study has shown that reduction in delivery pressure is useful in improving system performance through reduced energy use, leakage, and pipe break rates, additional testing on real networks is required. To carry out these additional studies, data on pressure should be collected from SCADA systems. The performance of the WDS can be evaluated and the collected data of pressure can be compared with the minimum pressure standards to determine to what extent low pressure events occur and answer to this important question: Do low pressures occur frequently in WDSs as is assumed in conventional design?

Given the generally poor conditions of water supply infrastructures and the stress on resources, redesign of WDSs is not only welcome but urgently needed. The major factors affecting WDSs design and operation are uncertainty in future water demands and pipes roughnesses. Making appropriate and long-term decisions is a challenge that requires long-term planning and management. More flexible WDS design approaches may be used to deal with uncertainty in short term period and to achieve long term goals. The aim here can be to evaluate the flexibility inherent to determine to what extent the performance of the system can be improved in the form of changing reliability levels as a result of changing network features, i.e., pipe diameters, tank volumes, and pumping capacities. The results can indicate the key influential parameters for the efficient improvement of the performance of the system considering uncertain design factors.

Flows needed for fire protection are often considerably greater than consumer water demands, thus design and operation of WDSs are significantly influenced by fire protection requirements. The impact of design and operational decisions may include increased costs to the utility to meet the minimum required pressure and potential water quality problems. The required fire flows are described in fire codes published by cities, counties, or other political jurisdictions. The needed fire flow (NFF) is the flow considered necessary for suppressing a major fire in a specific building. The required fire flow duration is 2 hours if NFF is equal or less than 2500 gpm (158 L/s) and it is 3 hours for NFF equals 3000 to 3500 gpm (189-221 L/s) (AWWA 2008). The estimated methods to determine NFF are subject to uncertainty. This uncertainties and its impact on the design and operation of WDSs may be studied in the future.

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Appendix A

Node and Pipe data for Anytown Network

This appendix outlines the data parameters of the Anytown network in this thesis. The node and pipe data in this appendix were taken directly from Walski et al. (1987). The system topology for pipes and system configuration are set according to Gessler's optimization (Gessler 1985).

A.1 Node data

The Anytown network comprises of 22 nodes. Node 10 is a clear well and Nodes 65, and 165 are elevated tanks with 250000 gallon (1136 m^3) capacity. According to Gessler's optimization, a tank with the volume of 800000 gallon (3032 m^3) is located at Node 150. There are 19 nodes supplying demand to the network users. Data on nodal elevation and demands for the year of 2005 were taken directly from Table 3 of Walski et al. (1987) and are shown in Table A.1. The demand pattern shown in Table A.1 was taken from Table 6 of the original paper.

A.2. Pipe data

The network comprises 40 pipes. The data for pipes were directly taken from Table 1 and table 7 (according to Gessler's optimization) of Walski et al. (1987). The pipe characteristics are shown in Table A.2.

Table A.1: Node data for Anytown network and demand pattern of daily water consumption

Node Characteristics			Demand pattern	
Node ID	Elevation (m)	Average day demand (Lps)	Time of day	Demand multiplication factor
20	6.1	31.5	12-3 a.m.	0.7
30	15.3	12.6	3-6 a.m.	0.6
40	15.3	12.6	6-9 a.m.	1.2
50	15.3	37.8	9-12 a.m.	1.3
55	24.4	37.8	12-3 p.m.	1.2
60	15.3	31.5	3-6 p.m.	. 1.1
65 (Tank)	65.6	-	6-9 p.m.	1
70	15.3	31.5	9-12 p.m.	0.9
75	24.4	37.8		
80	21.4	31.5		
90	15.3	63.1		
100	15.3	31.5		
110	15.3	31.5		
115	24.4	37.8		
120	36.6	25.2		
130	36.6	25.2		
140	24.4	25.2		
150	36.6	25.2		
160	36.6	63.1		
165 (Tank)	65.6	-		
170	36.6	25.2		

Table A.2: Pipe data for Anytown network

Pipe no.	Diameter (mm)	Length (m)	C-factor	Pipe no.	Diameter (mm)	Length (m)	C-factor
2	406	3660	70	44	457	1830	120
4	356	3660	120	46	203	1830	120
6	610	3660	70	48	203	1830	70
8	305	2745	70	50	610	1830	120
10	305	1830	70	52	203	1830	120
12	254	1830	70	54	203	2745	130
14	305	1830	70	56	203	1830	120
16	254	1830	70	58	406	1830	120
18	305	1830	70	60	356	1830	120
20	254	1830	70	62	203	1830	120
22	254	1830	70	64	203	3660	120
24	254	1830	70	66	203	3660	120
26	305	1830	70	68	305	1830	130
28	254	1830	70	70	305	1830	130
30	254	1830	120	72	152	1830	130
32	254	1830	120	74	356	1830	130
34	254	2745	120	76	152	1830	130
36	254	1830	120	78	305	30.5	120
38	254	1830	120	80	305	30.5	120
40	254	1830	120				
42	356	1830	120				

Appendix B

Node and Pipe data for Hanoi Network

This appendix outlines the data parameters of the Hanoi network in this thesis. The node and pipe data in this appendix were taken directly from Fujiwara Khang (1990). The system topology for pipes is set according to network's optimization presented in Savic and Walters (1997).

B.1 Node data

The Hanoi network consists of 32 nodes. Node 1 is the reservoir and no pumping facilities are considered. The fire flow requirements at the two nodes 13 and 22 are considered to be 0.25 m^3/s. There are 31 nodes supplying demand to the network users. Data on nodal demands was taken directly from Table 1 of Fujiwara Khang (1990). The demand data is shown in Table B.1.

B.2. Pipe data

The network comprises 34 pipes. The data for pipes were directly taken from Table 2 of Fujiwara Khang (1990) and Table 7 of Savic and Walters (1997). The pipe characteristics are shown in Table B.2. The Darcy–Weisbach friction factors for all pipes are considered 0.015.

Table B.1: Node and pipe data for Hanoi network

Pipe data			Demand data	
Link ID	**Length (m)**	**Diameter (mm)**	**Node ID**	**Demand (LPS)**
Pipe 1	100	1016	Junc 2	247.2
Pipe 2	1350	1016	Junc 3	236.1
Pipe 3	900	1016	Junc 4	36.1
Pipe 4	1150	1016	Junc 5	201.4
Pipe 5	1450	1016	Junc 6	279.1
Pipe 6	450	1016	Junc 7	375
Pipe 7	850	1016	Junc 8	152.8
Pipe 8	850	1016	Junc 9	145.8
Pipe 9	800	1016	Junc 10	145.8
Pipe 10	950	762	Junc 11	138.9
Pipe 11	1200	609.6	Junc 12	155.6
Pipe 12	3500	508	Junc 13	250
Pipe 13	800	508	Junc 14	170.8
Pipe 14	500	406.4	Junc 15	77.8
Pipe 15	550	304.8	Junc 16	86.1
Pipe 16	2730	508	Junc 17	240.2
Pipe 17	1750	609.6	Junc 18	373.6
Pipe 18	800	762	Junc 19	16.7
Pipe 19	400	762	Junc 20	354.1
Pipe 20	2200	1016	Junc 21	258.3
Pipe 21	1500	406.4	Junc 22	250
Pipe 22	500	304.8	Junc 23	290.2
Pipe 23	2650	762	Junc 24	227.8
Pipe 24	1230	508	Junc 25	47.2
Pipe 25	1300	406.4	Junc 26	250
Pipe 26	850	304.8	Junc 27	102.8
Pipe 27	300	508	Junc 28	80.6
Pipe 28	750	609.6	Junc 29	100
Pipe 29	1500	508	Junc 30	100
Pipe 30	2000	508	Junc 31	29.1
Pipe 31	1600	406.4	Junc 32	223.6
Pipe 32	150	304.8		
Pipe 33	860	304.8		
Pipe 34	950	508		