TRANSIENT EFFECTS OF FIRE FLOW
ON DISTRIBUTION SYSTEMS

by

Muhammad Shahid Nadeem Mian

A thesis submitted in conformity with the requirements
for the degree of Master of Applied Science
Graduate Department of Civil Engineering
University of Toronto

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0-612-58670-7
To my parents

Muhammad Niamat Ullah and Amna Bibi
Abstract

Transient Effects of Fire Flow on Distribution Systems
M.A.Sc. 2001
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University of Toronto

Fire flows have a significant impact on the design and operation of a water distribution system. Transient pressure surges are initiated by the operation of a hydrant and by the development of an abnormal water demand for fire fighting. Fire flows can create additional localized stress on the distribution network and may induce undesirable areas of low pressures, which may be either transient or sustained in nature.

Many factors contribute to the intensity of transient surges including the size and layout of the network, the location of the operating hydrant, the hydrant valve operation, the supply or delivery rate, the presence of multiple hydrant operations, the water carrying capacity of the network, and the pipeline material.

The Insurance Services Office method provides a good estimate for fire flow rates and many agencies endorse this method. The role of computer simulation and optimization helps greatly to evaluate all aspects of a distribution system.
Acknowledgements

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My thanks to Zhiqiang Zhang, for supplying me material on simulation and optimization.

I appreciate Barry Kendall, Gay Saunders, and Sophie Greco, from office of the Fire Marshal, for helping to search literature and video films on fire fighting.
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Chapter 1

Introduction

1.1 Introduction

Water is a common fire-extinguishing agent. Water suppresses fire by cooling, smothering, emulsifying and diluting burning materials. An attempt is made here to gather and compile information on the amount of water needed for fire suppression, its effect on water distribution systems, and the adequacy and reliability of the required quantity of water for a community. This study does not cover the specifics of water supply requirements for automatic extinguishing systems. Public water supply systems are designed to supply normal domestic demands and also to provide water to fire hydrants for use by fire departments. The demand of water for fire suppression varies both with time and with location.

Adequacy of supply refers to the delivery of an acceptable quantity of water for fire fighting while other consumers are on line. Reliability reflects the ability of a distribution system to constantly deliver an adequate quantity of water. Typical components of a water distribution system include supply and storage facilities, treatment plants, pumping stations, water mains, hydrants, valves and service lines. The system components affect the overall performance of the distribution system. In this report, however, evaluation of water supply components against the criteria of adequacy and reliability is limited to operational performance and only indirectly includes design and construction.

Recent advances in computer technology and software developments are creating new opportunities for improving water utility plans and for managing water distribution systems. The role of computer simulation and optimization helps greatly to evaluate a distribution system. Optimization techniques are used to find a combination of decision variables that minimize an
objective function subject to constraints. A brief discussion on hydraulic reliability modeling is also included in the present study.

Transient analysis of pressure and flow in piping systems caused by fire flow is a concern because of potential for interruption in supplying water. There are limits on the minimum and maximum pressures for a distribution network to avoid pipe breakage or failure of other system components because of adverse hydraulic situations arising within the distribution system. The basic hydraulic equations describing the phenomena are non-linear differential equations and cannot be solved directly. A numerical procedure is needed to solve the governing differential equations.

Transient pressure surges in water supply distribution systems may occur due to changes in the flow demand, opening and closing of a fire hydrant main valve or the failure of a system component. The analysis of transient conditions caused by fire flow and evaluation of adequacy of distribution network are necessary for the design and operation of the water supply to a community.

1.2 Objectives

There are concerns about hydraulic conditions in distribution systems as a result of withdrawal of additional water for fire suppression. The objectives of this study are to provide an overall view on planning for a public water supply systems, to evaluate adequacy and reliability of a water distribution system and to identify transient surges in a pipeline for the severity of hazards. The specific objectives of the study are:

- Quantification of needed fire flow, including a thorough review of fire flow requirements from the literature.
- Evaluation of the steady state distribution system hydraulic performance to supply the required fire flow, including meeting delivery pressure requirements while fighting fires.
- Identification of hydraulic conditions (transient effects in particular) in a distribution system as a result of the sudden application of a fire flow at a fire hydrant.
1.3 Significance

In all but the largest distribution systems, the required fire flow is usually a major factor for determining the amount of water to be stored, the sizes of the mains in the network and other system components, and the pressure to be maintained. A lot of capital is needed to construct, maintain and upgrade water distribution and transmission systems\(^{[14,51]}\). Thus, consideration of fire fighting requirements is a significant part of distribution system design and analysis.

Fire is responsible for a number of life losses, injuries, property damage and interruption of water supply to users. In 1994\(^{[13,18]}\), there were 667,000 fire incidences in Canada and 2,045,500 in USA. These fire accidents resulted in 367 deaths in Canada and 4,275 deaths in USA and 2,470 civilian injuries in Canada and 27,250 civilian injuries in USA. The property damage due to these fires was $1.152 billion in Canada and $8.151 billion in USA.

Water is cheap and readily available in most places but becomes quite expensive if it has to be handled and stored in large quantities for adequate fire supply. Thus, an accurate and realistic estimate is needed for the amount of water that a community needs for fire fighting. Evaluation of different conventional methods could be helpful to compare various methods for the estimation of fire flow depending upon the level of fire threat. The evaluation of various conventional methods are also helpful to find design flows, to evaluate an existing system, for insurance rating purposes, or for maintenance and extension decisions relating to the existing system.

Determining of capability of the distribution system to supply the needed fire flow at all sections throughout the community is important to evaluate the system. The determination of system capacity is also helpful to decide which part of the system is inadequate and where reliability may or may not be compromised.

Transient pressure surges may accompany the operation of a hydrant or be associated with the sudden development of an abnormal water demand for fire fighting. Transient effects are threats against system adequacy and reliability because of their possible damage to system components. The determination of safe limits of water withdrawal rate duration and other precautionary measures can also be helpful in many ways. Despite this, transients have seldom been included in
the assessment of distribution systems for fire flows. For this reason, this thesis gives particular emphasis to this topic.

1.4 Thesis Organization

Following this introductory chapter, the next chapter overviews the water requirements for fire protection. Conventional methods used to estimate needed fire flow for a community are described and compared.

The components that make up a water supply system and distribution system performance measures against fire flow are identified in chapter 3. A brief discussion on hydraulic reliability modeling is included in chapter 4. Theoretical considerations for unsteady flow in water supply networks are summarized in chapter 5. Chapters 6 and 7 discuss case studies of distribution networks particularly transient analysis due to fire flow from a distribution system. Chapter 8 summarizes the conclusions of this study.
Chapter 2

Water Requirement for Fire Protection - Conventional Methods

There are a number of methods of estimating fire flows for fire suppression. The evaluation of different methods is helpful to estimate design flows for specific duration at a fire hydrant(s). Since transient pressure surges can be initiated by the operation of a fire hydrant, the evaluation of different methods is necessary to find fire flows that are expected at a fire hydrant. The current chapter reviews requirements for fire suppression relating to flow rate for required duration and thus provides a basis for subsequent chapters.

2.1 Introduction

Water is used as a primary extinguishing agent because of its availability, low cost and ability to absorb heat. The major useful property of water is its capacity to cool burning fuels to a temperature below ignition point\[^{12,32,63}\]. The water requirements for fire fighting include the rate of flow, the residual pressure required at the point of withdrawal, and the required duration of flow. Water, although cheap in most places, becomes quite expensive if it has to be handled and stored in large quantities for adequate fire supply. Therefore, the amount of water that a community needs for fire fighting should be carefully estimated.

Water works in urban areas are certainly widely used for the purposes of drinking and sanitation. However, water has also been used for extinguishing fires since time immemorial. The inclusion of water for the fire protection of cities is comparatively recent. Although, in most cities, domestic and industrial water need is greater than required for fire protection, in small towns the water requirements for fire fighting often exceed other requirements. In large cities, however, fire frequency may result in simultaneous fires.
A number of methods have been developed to estimate water supply requirements for fire fighting. The objective of this chapter is to compare a variety of these published methods. For comparison, the methods are divided into time periods, i.e., methods developed till 1911 (early techniques) and after 1911 (traditional techniques). A few methods to estimate fire flow for fire fighting are developed recently and these are Illinois Institute of Technology Research Institute (IITRI) method, Insurance Services Office (ISO) method, and Ontario Interim Water Supply Guideline method. The methods are also compared on the basis of population and number of fire streams, water flow and fire area, and total quantity of water required and fire area.

2.2 Water as an Extinguishing Agent

Water suppresses fire by cooling, smothering, emulsifying and diluting burning materials. Water is normally applied to a fire area in the form of jet or spray. The capacity for rapid removal of heat is greater for a water spray than for a water jet. However, the reach of water jet to the fire area is better than for a water spray, depending on the pressure and flow rate at the nozzle, nozzle design and its angle of elevation\(^\text{[40,70]}\).

The water applied to a fire serves at least four purposes:

- It removes the heat produced by the fire, thereby cooling burning fuels to a temperature below which they cease to burn\(^\text{[12,32,46,63]}\).

- It absorbs the heat of fire when it changes from liquid to gaseous state\(^\text{[63]}\), if enough stream is generated, air is displaced or excluded (smothering\(^\text{†}\))\(^\text{[46]}\).

- It cools material that has not been ignited\(^\text{[32,63]}\).

- It blankets unignited material from the oxygen required to initiate and sustain combustion\(^\text{[12,46]}\).

\(^\text{†}\) In cases where oxygen is produced while a burning material decomposes, smothering is not possible.
2.3 Adequacy

An adequate supply of water is necessary to provide fire protection and suppression. An inadequate fire protection system is potentially more objectionable than no system at all\textsuperscript{[11]}. A water supply system is considered to be adequate if it delivers the required fire flow at all points throughout a particular municipality with the consumption recommended by the appropriate authority such as the Ministry of the Environment.

\textit{Guidelines from Ministry of the Environment:} The publication \textit{Guidelines for the Design of Water Distribution Systems, 1984\textsuperscript{[7]}}, states that a water supply systems should be designed to satisfy the greater of either of the following:

- Maximum daily consumption + fire flow demand; or,
- Peak rate consumption (maximum hourly demand).

National Fire Protection Association\textsuperscript{[12]} (NFPA) as well as American Water Works Association\textsuperscript{[11]} (AWWA) recommends the same criterion as mentioned above. The needed fire flow varies with the size of the municipality, because the chance of multiple fires at any time and nature of development varies. Although the Ministry of Environment prepares guidelines for design of water distribution systems\textsuperscript{[7]}, the magnitude of the fire flow allowance is the responsibility of the municipality.

The different demands (for various duration) on the system and their effects on the capacity of the system for fire protection must be determined. Modern motorized fire apparatus can make available a stream with high pressure from ordinary water system where adequate water volume is provided\textsuperscript{[12]}. It is generally recommended that a minimum residual pressure of 20 psi\textsuperscript{5} (14 m of water) be provided at hydrants\textsuperscript{[11,7,11,12]}. The adequacy of the distribution system is further discussed in chapters 3 and 4.

\textsuperscript{5} Minimum acceptable residual pressure of 20 psi (14 m) is a typical tradition, however, is not a hard and fast rule. A higher pressure might be required for a community with rapid changes in elevations. Other systems may require low pressures with no adverse effects like, under sized supply lines, inadequate system storage, absence of looped supply, etc.
2.4 Different Techniques for Determining Fire Flow Requirements

In this discussion on water requirements for fire protection, no distinction is made for the ownership of the system, whether public or private.

_Definition of Required Fire Flow:_ The water flow required for fire protection is the rate of flow (at residual pressure of 20 psi or 14 m and for specified duration) that is necessary to control a major fire in a specific structure[5,11].

The required or needed fire flow can be calculated for individual buildings or groups of buildings. It is usual practice to divide a community into blocks of roughly homogeneous blocks. A brief history of the needed fire flow calculations (for non-sprinkled properties) is presented in the following section:

2.4.1 Herbert Shedd Method, 1889

The starting point for computing water for fire protection was to estimate the number of hose streams required for fire fighting. Herbert, 1889, tried to standardize a stream with a discharge of 200 US gallon per minute (13 l/s) and found it adequate for fire protection. His proposed number of required fire streams against population of the community are as follows[12]:

<table>
<thead>
<tr>
<th>Population</th>
<th>Number of Fire Streams Required</th>
</tr>
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<tbody>
<tr>
<td>5,000</td>
<td>5</td>
</tr>
<tr>
<td>10,000</td>
<td>7</td>
</tr>
<tr>
<td>20,000</td>
<td>10</td>
</tr>
<tr>
<td>40,000</td>
<td>14</td>
</tr>
<tr>
<td>60,000</td>
<td>17</td>
</tr>
<tr>
<td>100,000</td>
<td>22</td>
</tr>
<tr>
<td>180,000</td>
<td>30</td>
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</table>
2.4.2  J. T. Fanning Method, 1892

Fanning, 1892, recommended streams at a pressure of 54 psi (38 m) at water withdrawal points. The number of streams required for fire fighting for a city or town are given in the following table:\[12]:

Table 2.2: Fire Flow by Fanning

<table>
<thead>
<tr>
<th>Population</th>
<th>Number of Fire Streams Required</th>
</tr>
</thead>
<tbody>
<tr>
<td>4,000</td>
<td>7</td>
</tr>
<tr>
<td>10,000</td>
<td>10</td>
</tr>
<tr>
<td>50,000</td>
<td>14</td>
</tr>
<tr>
<td>100,000</td>
<td>18</td>
</tr>
<tr>
<td>150,000</td>
<td>25</td>
</tr>
</tbody>
</table>

The figures started at seven streams for a population 4,000 and went up to 25 streams for a population of 150,000.

2.4.3  John R. Freeman Method, 1892

Freeman, 1889\[40\], when doing the fundamental work on water flow through hose and nozzles, defined the fire stream as one with a discharge of 250 gpm (16 l/s) at a pressure of 40 to 50 psi (28 to 35 m).

In contrast to earlier work, Freeman, 1892 (reported in Some Fundamental Considerations in Determination of a Reasonable Return for Public Fire Hydrant Services\[60\]) suggested two standards; a minimum and a maximum. The formulae for Freeman’s curves are:

\[
Q_{\text{min}} = 250 \left( 1.7\sqrt{P} + 0.03P \right) \]

\[
Q_{\text{max}} = 250 \left( \frac{P}{5} + 10 \right) \]
where $P =$ population in thousands, and

$Q =$ fire demand in US gallon per minute (gpm).

Freeman argued that valuable buildings might require a large number of streams irrespective of a smaller population.

He pointed out that there is a fundamental difference between systems designed to supply domestic water needs and those that include fire protection. For ordinary water supply, small pipes were sufficient for water distribution, but larger pipes were required for concentration of the supply to the fire stream. He asserted that fire supply should be added to maximum domestic consumption.

He also pointed out that hydrants should be placed with reference to being able to concentrate streams at certain block rather than on the basis of lengths on the street mains. His extensive work on hose streams\(^{[40]}\) shows that long hose lines reduce the water which can promptly be delivered on a fire. He suggested that flow of water to the hose streams should be supplied from a reliable source for a period of not less than 6 hours.

2.4.4 Kuichling Method, 1897

Kuichling concluded that number of streams required for fire fighting can be estimated from:

$$J = 2.8\sqrt{P} \quad \text{.................................................................2.2}$$

where $J =$ number of jets used, and

$P =$ population in thousands.
The amount of water that required for fire fighting, on the basis of fire stream of 250 gpm (16 l/s), is:

\[ Q = 700\sqrt{P} \] ............................................................2.3

in which, \( Q \) = fire demand in US gallon per minute (gpm).

2.4.5 National Board of Fire Underwriters (NBFU) Method, 1911

Metcalf et al.\[^{[60]}\] prepared a formula for fire flow based on surveys by the engineers of NBFU in over 90 cities during the period from 1906 to 1911; the result is

\[ Q = 1,020\sqrt{P(1 - 0.01\sqrt{P})} \] ............................................................2.4

This formula has been established upon the analysis of fire data for the communities ranges from a population of 7,500 to 500,000. In 1948, the NFBU (Fire Protection Handbook, Tenth Edition), stated that this formula is applicable up to a population of 200,000. The formula is similar to Kuichling’s proposal, relating the number of fire streams in terms of the square root of the population in thousands. The NBFU supplemented fire flow figures by a statement of the number of hours for which the flow must be delivered. The required fire flow duration based on experience are\[^{[11]}\]:

<table>
<thead>
<tr>
<th>Required Fire Flow (gpm)</th>
<th>Duration (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,000</td>
<td>65</td>
</tr>
<tr>
<td>1,250</td>
<td>80</td>
</tr>
<tr>
<td>1,500</td>
<td>95</td>
</tr>
<tr>
<td>1,750</td>
<td>110</td>
</tr>
<tr>
<td>2,000</td>
<td>125</td>
</tr>
<tr>
<td>2,250</td>
<td>140</td>
</tr>
<tr>
<td>2,500 to 12,000</td>
<td>160 to 760</td>
</tr>
</tbody>
</table>
According to NBFU, the total consumption during a fire is related to the water storage capacity of the water supply system. However, most fires do not burn for the periods mentioned in the Table 2.3. In a city, fire flow should be maintained for the required duration from stored water. This rule is not intended for use where the flow is less than 250 gpm (16 l/s) for a minimum duration of 2 hours\textsuperscript{[12]}. 

In the above formulae, the flow requirement is related, in the rough way, to the population. It was recognized by Huston\textsuperscript{[49]} in 1948, that there is a need for estimating fire flow requirements for buildings outside the downtown districts. The above formulae are not effective, because of change in population growth development\textsuperscript{[26]}. Also outlying complexes (hospitals, schools, government buildings, business centers, etc.) were of such sizes and importance that, in many cases, they need more inclusive methods for fire flow calculations.

In 1942, the NBFU also published schedule for grading cities and towns for use in insurance rating\textsuperscript{[11]}.

**2.4.6 Thomas Method, 1959**

*Thomas, 1959*\textsuperscript{[69]}, carried out statistical analysis on the amount of fire flow in United Kingdom. His equations for number of streams and time duration are:

\[
J = 0.1\sqrt{A} \tag{2.5(a)}
\]

\[
t = \sqrt{A} \tag{2.5(b)}
\]

where 

\(J\) = number of jets used, 

\(t\) = time in minutes, and 

\(A\) = area of fire (ft\(^2\)).
Thomas studied the extinction of 48 fires ranging the ground plan area from 300 to 600,000 square feet. He concluded that the amount of combustibles involved in the different fires is much less important than the fire area.

2.4.7 Iowa State University (ISU) Method, 1967

The ISU method related fire flow to building volume, as obviously an important factor in fire fighting. The ISU method was based on experiments at the Fire Extension Service at Iowa State University and statistical analyses were also done. The resulting equation for fire flow is:\(^{[64]}:\)

\[
Q = \frac{V}{100}
\]

where \( Q \) = fire demand in US gallon per minute (gpm), and \( V \) = volume of space in cubic feet.

ISU found that combustion of fuel depends on the availability of oxygen and vaporization of applied water into steam. The capability of water vaporization displaces oxygen from the microenvironment and thus helps smothering. To determine optimum rates of water requirement for fire fighting, time-temperature curves were analyzed.

This method uses a more theoretical approach, since it assumes that the entire space is involved in the fire, and gives the amount of water needed to deplete the oxygen when water is vaporized into steam by the heat of fire. It is important to note that no consideration is given to combustibility of construction or contents.

In this method the entire volume of the structure including volume of basements, rooms, verandas, and other concealed spaces is accounted for. Therefore, the use of this equation for large open buildings, like warehouses, yields quite large fire flows.
2.4.8 Illinois Institute of Technology Research Institute (IITRI) Method, 1968

IITRI method of estimating fire flow rates was based on the analysis of 134 fires in the several occupancy types in the Chicago area. The fire flow equations, obtained through curve-fitting analysis of the available data, are\[^{55}\]:

**Residential Occupancies:**

\[ Q = -9 \times 10^{-5} A^2 + 50 \times 10^{-2} A \] .............................................. 2.7(a)

\[ T_c = 5 \times 10^{-3} A + 14 \] .................................................. 2.7(b)

\[ W = 3 \times A + 160 \] .................................................. 2.7(c)

and

**Non-Residential Occupancies:**

\[ Q = -1.3 \times 10^{-5} A^2 + 42 \times 10^{-2} A \] ................. 2.8(a)

\[ T_c = 5 \times 10^{-3} A + 46 \] .................................................. 2.8(a)

\[ W = 15 \times A + 28,000 \] .................................................. 2.8(c)

where

- \( Q \) = fire demand in US gallon per minute (gpm),
- \( A \) = area of fire (ft\(^2\)),
- \( T_c \) = fire control time (minutes), and
- \( W \) = quantity of water (US gallons).

The fire control time \((T_c)\) was defined as the time from the start of fire fighting operation to the time when control is established. The control was described as the state where the major flames have been subdued, and the fire no longer is increasing in size. This fire control time does not include final extinguishment and overhaul time. Equations 2.7(a), 2.7(b) and 2.7(c), residential fires, are limited to conditions where \( A \) is between 200 and 5000 square ft. Equations 2.8(a), 2.8(b) and 2.8(c), non-residential fires, are limited to conditions where \( A \) is between 1000 and 30,000 square ft.
The IITRI formulae are based only on the fire area. It was also pointed that tactical procedure of fire fighting, construction type, occupancy, etc. might influence the application rate of water use.

For comparison purposes, laboratory experiments were also conducted. Experimental analyses indicated that fire-fighting technique was the key parameter in determining the amount and rate of water used. The fire rates of 134 actual fires observed were approximately double the rate used in the laboratory.

**Guidelines:** The key factor is to obtain a formula that could be used, as a guide for calculating the flow needed to extinguish a fire in a single building or a single fire area within a building. The following methods namely Insurance Services Office Method and Ontario Interim Water Supply Guideline Method, are formulas and guidelines.

### 2.4.9 Insurance Services Office (ISO) Method

**ISO-1972:** ISO 1972 attempted to determine if a correlation exists between the needed fire flow and the total floor area. ISO obtained data from actual fire fighting incidents. The final equation for flow after minor adjustment for field experience is:

\[ Q = 18C \sqrt{A} \]

where
- \( Q \) = fire demand in US gallon per minute (gpm),
- \( C \) = coefficient related to the type of construction, and
- \( A \) = the total floor area in the building being considered (square feet).

The ISO procedure is a guide, which accounts for the type of construction, occupancy, exposure of the building and other parameters. Additional flow can be added or credits can be given based on occupancy, automatic sprinklers if any, and exposure of the building. The flows determined by
this method are generally considered a good estimate. Its current version is briefly described below.

**ISO-1980:** ISO 1980, improved their method of 1972 and documented it in the publication *Fire Suppression Rating Schedule*. They proposed the following formula:

\[
NFF_i = C_i O_i \left( X + P \right)_i
\]

where

- \( NFF_i \) = needed fire flow,
- \( C_i \) = construction factor = \( 18F \sqrt{A_i} \), with;
  - \( F \) = coefficient related to the class of construction, and
  - \( A_i \) = effective area
- \( O_i \) = occupancy factor, and
- \( X_i \) = exposure factor, \( P_i \) = communication factor, such that;

\[
(X + P)_i = 1.0 + \sum_{i=1}^{n} (X_i + P_i), \text{ where;}
\]

\[
n = \text{number of sides of subject building.}
\]

The construction factor depends upon the construction of the structure under consideration; the occupancy factor depends upon the combustibility of the material in the building; and the exposure factor depends upon the extent of exposure from and to adjacent structures.

The various portions of the building were analyzed separately and then combined for a total fire flow requirement. According to ISO, the lower and upper limits of NFF are 500 gpm (32 l/s) and 12,000 gpm (760 l/s) respectively. In case of automatic sprinklers, the NFF (i.e., needed for the sprinkler system) is converted to 20 psi (14 m) residual pressure, with a minimum flow of 500 gpm (32 l/s). Fire flow duration should be two hours for a NFF up to 2,500 gpm (160 l/s), and
three hours for a NFF of 3,000 to 3,500 gpm (190 to 220 l/s). A NFF greater than 3,500 gpm (220 l/s) should have a four-hour fire flow duration for individual property fire suppression.

The fire flow duration published in *Fire Protection Handbook, National Fire Protection Association (NFPA), 1991*[^12], are given in Table 2.4.

<table>
<thead>
<tr>
<th>Required Fire Flow (gpm)</th>
<th>Duration (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2,500 or less</td>
<td>2</td>
</tr>
<tr>
<td>3,000 to 3,500</td>
<td>3</td>
</tr>
<tr>
<td>4,000 to 4,500</td>
<td>4</td>
</tr>
<tr>
<td>5,000 to 5,500</td>
<td>5</td>
</tr>
<tr>
<td>6,000 to 7,000</td>
<td>6</td>
</tr>
<tr>
<td>7,000 to 8,000</td>
<td>7</td>
</tr>
<tr>
<td>8,000 to 9,000</td>
<td>8</td>
</tr>
<tr>
<td>9,000 to 10,000</td>
<td>9</td>
</tr>
<tr>
<td>10,000 to 12,000</td>
<td>10</td>
</tr>
</tbody>
</table>

ISO[^5] claims that the NFF reasonably coincides with the actual flow required to suppress a fire in a real-life situation. Certainly this method considers building construction, occupancy, adjacent exposed buildings, and communication paths between buildings etc.

Public and private protection at properties is individually evaluated and may vary from the community’s classification. This method has achieved widespread recognition in the fire protection community[^12].

### 2.4.10 Ontario Interim Water Supply Guideline Method, 1995

The Office of the Fire Marshal, the Ministry of Housing and Water Supply Adequacy Committee[^15,19] has developed a guideline for determining an adequate fire protection water supply. The purpose of this guideline is to estimate water supply requirement for life safety and exposure protection, with limited property protection. Property protection is a primary expectation of the building owner. They argued that for property protection, or where there is a
possibility of environmental contamination from a fire, the building should be protected by a supplementary fire protection, such as a sprinkler system.

A minimum water supply volume must be immediately available to the fire department to control fire growth until the building is safely evacuated, to prevent the fire from spreading to adjacent buildings, to provide limited measure of property protection, and to protect adjacent buildings where environmental contamination from a fire is possible. This minimum supply of water volume can be calculated by:

\[ q = K V S_{Total} \]  .....................................................2.11

in which, \( q \) = minimum supply of water volume in liters,
\( K \) = water supply coefficient,
\( V \) = total building volume in cubic meters, and
\( S_{Total} \) = total of spatial coefficient values from property line exposed on all sides
  \[ = 1.0 + [(S_{Side1} + (S_{Side2} \ldots \ldots + (S_{Side})] \leq 2.0. \]

This method takes into account compartmentalization of buildings, combustibility of construction and occupancy, and evacuation responses by the building occupants, through the factor \( K \). This method also accounts for exposures of the building faces, through the \( S_{Total} \) factor.

The minimum fire protection water supply volume as estimated by Equation-2.11 are limited to not be less than the water supply needed to provide the minimum flow rate of \( q/30 \) (liter per minute) for a duration of 30 minutes. A minimum accepted pressure of 140 KPa (20 psi; 10 m) is designated in this guideline.

2.5 Comparison of Different Techniques

Comparison between various techniques for fire flow computations is not easy, because:

- In some techniques flow was reported as number of streams used with different flow rates. In some other techniques flow was reported in gpm.
The fire flow data generally did not include the time during which the flow was used.

In early formulae, the flow requirement is related to the population. More recent developments involve the nature of development within the community.

It is difficult to determine whether the amount of water used was actually needed.

For simplicity, the comparison is divided into two groups:

*Early Techniques (until 1911):* In the early techniques, the flow requirement is related to the population in a community. Table 2.5 shows a brief comparison between various techniques:

<table>
<thead>
<tr>
<th>Populations Thousands</th>
<th>Shed 1889</th>
<th>Fanning 1892</th>
<th>Freeman 1892</th>
<th>Kuchling 1897</th>
<th>NBFU 1911</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-4</td>
<td>2-3</td>
<td>3</td>
<td>4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5-10</td>
<td>5</td>
<td>6-8</td>
<td>6</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>10-20</td>
<td>7-10</td>
<td>6-12</td>
<td>9</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>20-40</td>
<td>10-14</td>
<td>8-15</td>
<td>12</td>
<td>17</td>
<td></td>
</tr>
<tr>
<td>40-50</td>
<td>14-14</td>
<td>12-18</td>
<td>18</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>50-60</td>
<td>17</td>
<td>15-22</td>
<td>20-30</td>
<td>22-28</td>
<td></td>
</tr>
<tr>
<td>60-100</td>
<td>22</td>
<td>18-28</td>
<td>28-36</td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>22</td>
<td>18-28</td>
<td>28-36</td>
<td></td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>25</td>
<td>28-34</td>
<td>34-44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>180</td>
<td>30</td>
<td>34-48</td>
<td>44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>30-50</td>
<td>40-48</td>
<td>48</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Reprinted (Reference [12])

Table 2.5, shows that the general trend is for the required number of streams to gradually increase with increase in population of a community. Shed, 1889, used a 200 gpm (13 l/s) stream, recommended that a community of 5,000 persons needed about five streams. In a city of 180,000 the number of streams required for fire protection could be about 30 such streams. Fanning, 1892, used stream with 54 psi (38 m) pressure and suggested seven streams for a community of 4,000 and going up to 25 streams for a community of 150,000. Freeman, 1892, used a discharge of 250
gpm (16 l/s) at 40 to 50 psi (28 to 35 m) pressures, suggested lower and upper limits depending upon the value of the building. His fundamental formulae recommends two to three streams for a population of 1,000 up to 30 to 50 streams at a population of 200,000. He also proposed arrangements of fire hydrants for the fire protection of a community. Kuichling, 1897, proposed that the number of streams required was 2.8 times the square root of the population in thousands. NBFU, 1911, based on an extensive survey, proposed a formula that was commonly used for many years as a guide to determine the flow requirements for fire fighting. The NBFU suggested that duration of 4 hours stream is required for a 1,000 population up to a maximum of 10 hours for 6,000 or more. The formula (Equation 2.4) was not applicable to the cities over population 200,000 where the peak use rate governs the system’s capacity. The NBFU also suggested that for a population over 200,000, a flow of 12,000 gpm (760 l/s) plus 2,000 to 8,000 gallons (7,570 to 30,280 liters) additional water for another fire, for a 10-hour duration is required. This sets a maximum of 20,000 gpm (1260 l/s) of fire flow for any city.

Traditional Techniques (after 1911): The comparison here is further divided into sections, since the situations to which the calculation of required fire flow is applied vary greatly. Early techniques (until 1911) were based on population but more recent techniques (after 1911) are based on fire area and other parameters (e.g., material/type of construction, occupancy, exposure of adjacent structure etc.).

1. Alternative Construction Scenarios: Consider a building 20,000 ft² (1,860 m²) in size, one story 10-ft (3-m) high, ordinary construction and combustible occupancy. To estimate fire flow needed to control fire within this structure, the building is analyzed with different methods (Table 2.7).

The ISO method involves the exposure situation of adjacent structures in calculating the needed fire flow, which involve the separation distance, the construction of the exposed building wall, and a length-height value. The communication factor reflects the potential for fire to spread through open or closed communication passageways between buildings. Considering the following three scenarios for exposure building (adjacent building) and communication factors:
**Scenario-1:** 30-ft (9-m) apart, three stories, masonry construction, 150-ft (45-m) exposed wall, open communications of combustible construction.

**Scenario-2:** 10-ft (3-m) apart, one story, frame construction, 100-ft (30-m) exposed wall, semiprotected enclosed openings of combustible construction.

**Scenario-3:** 60-ft (18-m) apart, four stories, masonry construction, 100-ft (30-m) exposed blank wall, no communications.

Table 2.6 shows the variation in water requirement under the above mentioned scenarios. The exposure factor reflects the need for additional water to reduce the exposure to adjacent buildings.

<table>
<thead>
<tr>
<th>Method</th>
<th>Fire Flow Requirements, gpm (l/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Scenario-1</td>
</tr>
<tr>
<td>Insurance Services Office, 1980</td>
<td>3,000 (190)</td>
</tr>
<tr>
<td>Ontario Interim Method, 1995</td>
<td>2,020 (130)</td>
</tr>
</tbody>
</table>

The ISO method also considers the status of the fire department equipment and experience of the personnel involved.

Table 2.7 shows that Thomas, ISU, and Ontario Interim methods yield low fire flow requirement, but IITRI and ISO methods estimate higher fire flow requirements.

<table>
<thead>
<tr>
<th>Method</th>
<th>Fire Flow Requirements (gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thomas, 1959</td>
<td>2,100</td>
</tr>
<tr>
<td>Iowa State University, 1967</td>
<td>2,000</td>
</tr>
<tr>
<td>Illinois Institutes of Technology Research Institute, 1968</td>
<td>3,200</td>
</tr>
<tr>
<td>Insurance Services Office, 1980</td>
<td>3,000/3,500/2,500</td>
</tr>
<tr>
<td>Ontario Interim Method, 1995</td>
<td>2,020/2,020/1,450</td>
</tr>
</tbody>
</table>
2. *Alternative structure sizes:* A one story building of ordinary construction, but varying in size, is analyzed by different methods for needed fire flow. Figure 2.1 shows comparison of different methods for fire flow estimation against variation in size of the building.

The IITRI method estimates the highest value of NFF if the effective area is greater than about 4,000 ft² (370 m²); by contrast, the ISO method gives the smaller values. IITRI method shows that the value of NFF starts decreasing after a certain size of a structure, and yields negative values after 32,300 ft² (3,000 m²) size. It is clear that the relationship (IITRI method) is not intended for the decreasing range.

![Figure 2.1: Comparison of Fire Flow Rates](Image)

The Thomas method and the ISU method are in reasonable agreement. The curves for NFF coincide well for larger fire areas, while deviation increases as fire area decreases. In general, the Thomas method requires a larger flow rate.

The Ontario Interim method and the ISU method give consistent lowest (theoretically ideal) values of NFF, but these values may not always be practically accurate.
The ISU method is more theoretical but does not consider the combustibility of the construction material and occupancy. The ISU method was based on the ability of water to absorb heat and to expand many times when converted to steam. ISU considers only the volume of the building to be filled with steam. This method requires large fire flows for large open buildings. This method is extremely easy to apply because the volume of a structure is simple to compute.

The ISO and IITRI methods give bigger values of NFF compared to Thomas, ISU, and Ontario Interim methods. The maximum estimated fire flow by ISO method is 12,000 gpm (760 l/s) and minimum is 500 gpm (32 l/s).

The ISO method is one of the most comprehensive and widely recommended methods for estimating fire flow requirements, because it considers many factors including construction type, occupancy material, exposure, communication etc. This method was designed for insurance rating purposes and provides guidance for estimating fire flow requirements for specific structures.

The Ontario Interim guideline gives water supply (during early stages) to support occupant evacuation of a building during a fire and to prevent fire from spreading. This method considers property protection but as a secondary objective.

Total Required Quantity of Water: Figure 2.2 shows fire flow duration for different methods. The number of hours during which the required fire flow should be available varies from 2 to 10 hrs, as indicated by NBFU and NFPA. The time duration values, as given by the NBFU, NFPA, and Thomas methods are significantly higher than the values by the ISO and Ontario Interim methods. However the total required quantity of water increases with the increase in area for each method (Figure 2.3).

The curves shown in figures 2.2 & 2.3, for Thomas method, are calculated using equations 2.5(a), 2.5(b). It is assumed that the flow rate of water per jet is 145 gpm (9 l/s). Fire flow duration estimated by Thomas’s method is in agreement with ISO’s method.

The IITRI method and Thomas method give almost same total required quantity of water for fire areas. The ISO method reasonably agrees with IITRI and Thomas methods for total required quantity of water for fire fighting. In Ontario Interim method, values of flow duration and total
required quantities of water for fire areas are consistently smaller than the values by other methods.

---

**Figure 2.2:** Fire Flow Duration

---

**Figure 2.3:** Total Required Quantity of Water
2.6 Summary

Water is the most important component of fire protection, since, water has a remarkable ability to absorb heat. A certain quantity of water must be provided for fire fighting.

Estimation of the needed fire flow (NFF) by various early investigators including Shedd, Fanning, Freeman, Kuiching, and National Board of Fire Underwriters (NBFU) formulae are based on community’s population and are gross approximations. The techniques developed by later workers including Thomas, Iowa State University (ISU), and Illinois Institute of Technology Research Institute (IITRI) methods for NFF are no longer based on community’s population but do not consider construction type, occupancy, exposure, etc. The Insurance Services Office (ISO) guidelines were developed for insurance grading purposes and are widely in use. The ISO method provides guidance for NFF for specific structures and considers building construction, occupancy, adjacent exposed buildings, and communication paths between buildings. The Ontario Interim method provides water supply for early stages of a fire and consider life safety and exposure protection but with limited property protection.

In North America, estimates of water required for fire protection in communities of various size and types are usually based on the standards of the American Water Works Association (AWWA) and on Canadian Fire Underwriters Association (CFUA). The AWWA recommended ISO method and consider it as a comprehensive method. Both AWWA and ISO suggest that 3,500 US gpm (220 l/s) is the upper limit to be provided, and that large facilities (or those with several hazards) should be individually analyzed with a maximum of 12,000 US gpm (760 l/s).


The ISO method generally provides a good estimate for specific structures while evaluating fire flows. The ISO guidelines are not intended to address fire flow needs when designing water supply systems for newly developing municipal areas. For planning purposes other guidelines and predictions from experience should also be considered in the evaluation of NFF.
Chapter 3

Adequacy and Reliability of a Distribution Network for Fire Flows

Adequate and reliable supply of water is necessary to provide fire protection and suppression. Distribution networks continuously provide domestic water for consumption. The distribution system needs to provide additional water for fire suppression on demand. Sudden loading on the system due to fire flow raised the question of adequacy and reliability.

The performance of a water distribution system includes the concepts of adequacy (related to pressure and flow requirements), dependability (considering transient effects, service interruptions, inoperable valves and hydrants, main breaks, power outage etc.) and efficiency (related to water conservation and energy use). The objective of this study is to determine the capability of the distribution networks to supply uninterrupted needed fire flow (NFF) at all sections throughout the community.

The adequacy and dependability of a distribution system is to fulfill customer requirement under the widest range of likely operating conditions. Traditionally, the distribution system design is based on minimization of cost factors respecting some simple hydraulic constraints[31]. The dependability (measure of the performance of the water distribution system) involved[21]:

- interaction between the piping system, distribution storage, distribution pumping and system appurtenance such as pressure reducing valves, check valves, etc,
- reliability of the individual system components,
- spatial variation of demands in the system,
- temporal variation in demands on the system,
- hydraulic performance at critical locations in the distribution, etc.
Although there are some standards for evaluating distribution system performance\[^{14}\], the diversity of objectives makes it difficult to address overall performance at any one moment. The following section is a description of the distribution network in relation to fire fighting with emphasis on adequacy and reliability. The present study does not emphasize directly design standards but reviews the performance measures of a distribution network relating to fire flows. The identified credibility criterions are mostly taken from *Grading Schedule* by Insurance Services Office (ISO), and American Water Works Association (AWWA) and Fire Underwriters Survey (FUS) standards. Dependability of a distribution system against transient effects is not discussed in this chapter but is described in chapters 6, 7 and 8.

*Computer Simulation and Optimization*: The evaluation of distribution system performance concerning adequacy and reliability is complicated since structural, hydraulic, and water quality parameters are inter-linked\[^{14}\]. The structural conditions can affect both hydraulic and water quality performance. The hydraulic conditions, such as dead ends, can result in water quality problems. In addition, analysis of complex loops, grids and compound grid systems are often beyond the computational capacity of humans.

Recent advances in computer technology and software developments are creating new opportunities for improving water utility plans and for managing water distribution systems. The role of computer simulation and optimization is essential for the evaluation of a distribution system and is discussed in chapter 4.

### 3.1 Maximum Consumption Rate

It is likely that a fire could occur in a community when usage of water is at a high level, therefore, a water distribution network should be capable of delivering a NFF during the period of high (near maximum) water consumption. The Ministry of the Environment\[^{7}\] and AWWA\[^{11,12}\] recommend that the system should be capable of satisfying the greater of either, maximum daily consumption plus fire demand, or maximum hourly consumption. The AWWA states that average water consumption is about 143 US gallons per person per day (540 litters per capita per day).
The maximum daily consumption is typically 1.5 times the average daily consumption\cite{7,12}, but appropriate local values should be used.

In most cities, the peak hourly demand exceeds the maximum daily plus fire flow demand and therefore is the controlling factor in the supply system. However, in smaller communities, the maximum daily flow plus fire flow demand is the controlling factor.

### 3.2 Fire Flow and Duration

The needed fire flow and the required duration of fire suppression are discussed in chapter 2.

According to the ISO method\cite{5,48} fire flow varies up to 12,000 US gallon per minute (760 l/s) where a fire hydrant should supply a minimum capacity of 500 US gallon per minute (32 l/s) at 20 psi (14 m) residual pressure. Fire flow duration should be two hours for a NFF up to 2,500 gpm (160 l/s), and three hours for a NFF of 3,000 to 3,500 gpm (190 to 220 l/s). A NFF greater than 3,500 gpm (220 l/s) should have a four-hour fire flow duration for individual property fire suppression.

AWWA Manual M31\cite{11} and National Fire Protection Association\cite{12} (NFPA) recommended fire flow estimated by the ISO method. Their noted fire flow durations are varied from 2 to 10 hours and are presented in Table 2.4.

For adequacy, the available flows at hydrants should be above required minimum flows, i.e.:

\[
Q > NFF 
\]

where \( Q \) = flow available at hydrant, and \( NFF \) = needed fire flow.
For distribution system adequacy[^7,11,12]:

\[ Q > Q_{\text{req}} \]  \hspace{5cm} \text{(3.1(b))}

in which, \( Q \) = flow available, and \( Q_{\text{req}} \) = required flow for normal maximum day use plus fire flow.

### 3.3 Minimum Pressure

The principal requirement is the ability to deliver sufficient quantity of water from fire hydrants to fire department pumpers. A minimum pressure of 20 to 35 psi (14 to 25 m) is accepted in North America[^14]. It is generally recommended that a minimum residual pressure[^f] of 20 psi (14 m) be provided at all hydrants[^5,7,11,12,14,15,17,48]. This 20 psi (14 m) pressure can overcome friction loss and avoid cavitation in water mains or suction hose between the hydrant and a fire department pump[^17,48]. Certain topographical changes may require different pressure zones in a distribution system to prevent excessive pressures.

For adequacy, the available residual pressure should be above required residual pressure, i.e.:

\[ P > P_{\text{min}} \]  \hspace{5cm} \text{(3.2)}

in which, \( P \) = pressure available, and \( P_{\text{min}} \) = minimum required pressure.

High-pressure water flow networks may be adopted under certain circumstances. Yet, the reliability of fire flow in such systems can still be questioned because of greater probability of flow interruptions due to pipe rupture or other failures. Adequacy and reliability of such systems can be achieved but may be uneconomical due to high construction and maintenance cost. In

[^f]: Refers to the pressure at a given fire hydrant when a calculated amount of water is flowing from the fire hydrant.
North America there are no universally specified requirements for minimum and maximum pressures in water mains under normal operating conditions. It is recommended that water systems should develop their own goals for minimum water pressure in mains\[14\].

3.4 Water Supply System Components

The three components of a water supply system required to evaluate water supply adequacy are the supply works capacity, the water main capacity, and the distribution of the fire hydrants. In this section, the ability of the water system to deliver NFF at all sections throughout the community is reviewed.

Before discussing water supply system components and their arrangement, it is appropriate to describe type of water supply systems. There are two basic types of water supply systems, gravity systems and direct pumping systems.

**Gravity Reservoir System:** Untreated water is taken from a source and passes through the treatment plant components typically to filter, to chlorinate, and to control the pH factor. Treated water then usually flows to a holding tank or reservoir. Water from the holding tank flows into the distribution system on demand under gravity.

This system is most reliable because there is minimal use of mechanical parts. This system may combine with a pump station to supply water from a source at lower elevation to an elevated tank. In any case the treated water flows by gravity through the distribution system which increases reliability.

**Direct Pumping System:** In this kind of system, direct pumping transport water. Additional pumps may be required on line to meet fire flow demands. The adequacy and reliability of the water system are generally dependent on the pumping facility.

Often, water distribution networks use some combination of gravity and direct pumping systems. Typically, when water is pumped directly into the distribution system, the quantity of water exceeds the demand feeds into a storage facility.
3.4.1 Supply (Water Sources) Works Capacity

Obviously, an adequate amount of water should be delivered to the distribution system mains. Water sources include rivers, canals, streams, bays, pounds, wells, cisterns, or other similar sources. A number of factors have an impact on the supply capacity including minimum storage of water, location and capacity pump stations, purifying systems, and emergency supplies. These factors are discussed in more detail in the following sections:

- **Minimum Storage of Water**

  Every water distribution network has water storage component(s). Demands of water vary from time to time for residential, business, industrial, and fire flow consumption. An adequate supply system is constantly supplying water as needed by the consumer. The average daily minimum water storage should be maintained\[^{5,17}\]. The most extreme conditions are not as serious as a total interruption of the supply. The extreme dry weather conditions are not required to be taken as representative of normal ability. Only the average capacity of wells during the most favourable nine-month period should be counted on\[^{17}\].

  Since storage of water usually fluctuates, therefore, to determine minimum water storage available, it is necessary to maintain accurate records on storage facilities including holding tanks and standpipe tanks. In addition to quantity of water available, the rate of delivery of water to the distribution system from the storage during fire flow period is critical\[^{17}\]. Further, it is unreliable to depend upon a single well even where records are favourable\[^{17}\].

  For design, minimum storage available at a source is evaluated on the basis of extended period simulation.

- **Pump Stations**

  In a distribution system, there may be pumping of potable water from treated water supplies directly into the distribution system (or to elevated or standpipe tanks) or untreated water from an impounding source to a treatment plant. The process of water supply to the
distribution network is generally limited by both water storage and pumping capacity. Therefore, effective capacities of a pump station should be accounted for when delivering at normal and at peak operating rates\(^5\). In a pumping station there may be multiple pumps in parallel or in series or in combinations of both. The arrangement of pumps (including standby pumps) depends upon circumstances like pressure applications, variation in water delivery rate, maintenance and reliability concerns, etc. When determining the credibility of a pumping station, the following considerations should be accounted for:

- The total pumping capacity is the sum of all pump facilities available\(^5\).
- When there are more than one pump lifts\(^2\) in a series, the effective pump capacity is the capacity of the lift with the lowest total capacity\(^5\). In case the same pumps operate in two or more lifts, each lift shall be accounted for with the lowest total capacity\(^5\).

Other reliability considerations are:

- Pumping capacity should be sufficient to maintain maximum daily consumption plus NFF for the required duration when the two most important pumps are out of service\(^17\).
- There may be an electric power outage due to a failure in any power line or at the time of repair or replacement of a transformer, switch, control unit or other device. Power supply to pumps should be arranged such that the delivery of maximum daily flow plus NFF for the required duration should be maintained during a period of two days\(^17\).

- **Water Treatment Facilities**

  Although not directly the subject of the current study, related rules exist for treatment plants and other facilities. Water is forced to pass through the treatment facilities to purify and to meet health standards. This treated water is ready to enter the distribution system. When

\(^2\) lift is the vertical distance between the surface of water in static water source to the center of the suction of the pump that is to draft from the source.
determining credibility of a treatment plant the following considerations should be accounted for:

- The total filter capacity is the sum of all the filter capacities when the filter arrangement discharges directly into the distribution system\(^{[5]}\).

- Filters should be able to operate at a reasonable overload capacity based on records\(^{[5,17]}\). A 25 percent overload capacity of a treatment plant is considered enough under favourable operating conditions\(^{[17]}\).

- For cases when filter capacity is limiting due to pump inadequacy, consider the filters as a pump limit condition\(^{[48]}\).

Reliability of a treatment facility can be influenced due to out of service of at least one filter or other unit, to the reduction of filter capacity by turbidity, to freezing or other condition, to cleaning basins, and to dependability on automatic operating and control appurtenances.

- **Emergency Supplies**

  The ability of an emergency supply can be added to the effective supply if there are emergency water supplies available to use. These supplies can be a cross connection to industrial, military, or to an adjacent community water system, or to separate water storage normally unavailable to the distribution system, or to fire department supply. Such cross connections increase both the adequacy and reliability of a distribution system.

  Emergency water that moves through distribution system should meet health regulations\(^{[48]}\). If a second source is non-potable water, adequate measure should be taken to prevent the public water supply from becoming contaminated. When determining the credibility of emergency supplies, the following considerations should be accounted for:
- The total emergency capacity is the sum of all emergency supplies available at certain section of the system\textsuperscript{[5]}, and

- Additional credit may be given for emergency supplies that come in automatically when needed\textsuperscript{[5]}.

**Distribution Storage**: The source, including distribution storage, must be sufficient to fulfill combined fire and domestic needs that may call at any time. Storage of water in elevated reservoirs can make it possible for the system to furnish water in a period during which the supply works may be inoperative.

Mathematically, the supply works capacity (water that enters the distribution system) is\textsuperscript{[5]}:

\[
SWC = [(MS + PU + FL + EM) - MDC] + SS + FDS \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots 3.3
\]

in which, \(SWC\) = supply works capacity,
\(MS\) = minimum storage,
\(PU\) = pumping system capacity,
\(FL\) = capacity of treatment facilities,
\(EM\) = emergency supply capacity,
\(MDC\) = maximum daily consumption rate,
\(SS\) = supplemental suction supply, and
\(FDS\) = fire department supply.

This formula considers factors relating to the adequacy of the supply works, but not water supply reliability.
3.4.2 Water Mains Capacity

Normally a distribution network comprises primary distribution pipes heading from supply works, secondary feeder pipes forming major loops, and distribution pipes laid along individual streets. When determining the credibility of a water main, the following considerations of flow characteristics should be accounted for:

- Two or more primary feeders should run by separate routes from the source of supply to the high value areas of the community[12,17]. This will provide mutual support and reliability in the system[17].

- To increase the capacity and reliability of the supply, secondary feeders should be arranged in loops so as to give more than one directions of supply to any point[11,17,65]. This looping should preferably be not more than 1000 meters between mains[17].

- Distribution grids should be designed so that the maximum demand may be placed upon them without serious reduction in the system heads[165]. This grid should consist of pipes at least 150 mm in size and arranged so that the longest length should not exceed 200 meters[17]. For lengths longer than 200 meters, larger size pipes should be used. Since stagnant water could result in water quality problems, dead ends should be avoided wherever possible[12,14].

- A pressure increase of more than 25 percent due to source changes associated with fire flows should be avoided to reduce the breaks[17].

- Supply mains should be equipped with isolation valves at one and one half kilometer intervals. Supply mains should also be equipped with air valves at high points and blowoffs at low points[11,17].

The general arrangement of valves at strategic locations and intervals should be considered with respect to the time required to isolate breaks. Valves should be maintained in good operating conditions with inspection frequency of once a year[17].

The hydrant flow test is the standard procedure for evaluating the capacity of water mains to meet fire suppression needs. Although field measurements of pressures and flows reflect the hydraulic
performance of water distribution systems at the time of measurement, yet, values can be calibrated under different conditions. This test is discussed in the next section.

The fire department plans should be based on the assumption that there may be interruptions of several hours in case of break in a distribution main, and several days in the case of main feeders\[^{11,12}\].

### 3.4.3 Fire Hydrant Distribution and Capacity

A hydrant is a device that provides outlet(s) from a water source (distribution system) to supply pressurized water for fire fighting. The two types of fire hydrants in general use are the *dry barrel* and the *wet barrel hydrants*. In the dry barrel hydrant, there is a drain valve at the base of the barrel allowing residual water to drain out after every use of the hydrant. The dry type hydrant also known as *frost proof hydrant* is most commonly used in North America. The dry and wet hydrants usually have a compression valve at each outlet, but they may have another valve in the bonnet that controls the water flow to all outlets.

*Distribution:* Fire hydrant spacing is determined by the anticipated available flow and distance from the building site to the hydrant with consideration of its use by fire department\[^{5,12}\]. As a general rule, hydrant spacing should not exceed 800 ft between hydrants, depending upon field situations. In congested areas hydrant spacing of 500 ft or less is recommended by the National Fire Protection Association (NFPA)\[^{12}\].

The ISO\[^{5}\] procedure, developed for insurance rating purposes, as summarized in Table 3.1 recommends maximum distances from the hydrant to a structure.

<table>
<thead>
<tr>
<th>Credit Fire Flow (US gpm)</th>
<th>Distance From Building (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 1,000</td>
<td>Within 300 ft of location</td>
</tr>
<tr>
<td>Up to 670</td>
<td>Within 301 to 600 ft of location</td>
</tr>
<tr>
<td>Up to 250</td>
<td>Within 601 to 1,000 ft of location</td>
</tr>
</tbody>
</table>
The distance is measured by the length of laid hose from the hydrant to the building.

Where hose lines are intended for use directly at the hydrants, there should be at least enough hydrants to make two streams available at each part of the building\cite{12}.

**Type, Size and Installation:** It is unlikely that all fire hydrants in a community are of the same type with the same branching connections. Fire hydrants should conform to AWWA standards or Underwriters’ Laboratories of Canada listing\cite{17}. Hose threads, cap and operating nuts on outlets should conform to Provincial standards. The (ISO insurance rating) credit points for hydrant type, size and installation are summarized in Table 3.2\cite{5}.

**Table 3.2: Number and Size of the Outlets at a Hydrant**

<table>
<thead>
<tr>
<th>Hydrant Size, Type and Installation</th>
<th>Credit Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>With 6 inch or larger branch, and a pumper outlet, with or without 2.5 inch outlets</td>
<td>100</td>
</tr>
<tr>
<td>With 6 inch or larger branch, no pumper outlet but two or more 2.5 inch outlets, or with small foot valve or with small barrel</td>
<td>75</td>
</tr>
<tr>
<td>With only 2.5 inch outlet</td>
<td>25</td>
</tr>
<tr>
<td>With less than 6 inch branch</td>
<td>25</td>
</tr>
<tr>
<td>Flush type</td>
<td>25</td>
</tr>
<tr>
<td>Cistern or suction point</td>
<td>25</td>
</tr>
</tbody>
</table>

**Hydrant Markings:** The uniform color coding of hydrants are recommended by NFPA\cite{12}. These marking are based on water flow available from hydrants, Table 3.3.

**Table 3.3: Hydrant Markings**

<table>
<thead>
<tr>
<th>Class</th>
<th>Flow (US gpm)</th>
<th>Color of Bonnets and Nozzle Caps</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1,000 or greater</td>
<td>Green</td>
</tr>
<tr>
<td>B</td>
<td>500 to 1,000</td>
<td>Orange</td>
</tr>
<tr>
<td>C</td>
<td>Less than 500</td>
<td>Red</td>
</tr>
</tbody>
</table>
This marked rating is based upon the flow rates available at 20 psi (14 m) residual pressure, when testing individual hydrant during a period of average water consumption.

_Flow Test:_ Water supply test data for each hydrant within the system must be known, since, what is ultimately available to the fire department is critical to the fire protection. The available residual pressure data for various delivery rates (250, 500, 750, and 1000 US gpm) is useful to determine the type of hydrant operation, which is to be used to transport water to the fire attack point(s). For details see _Hydraulic Flow Tests Results_[71], _AWWA Manual17_[6], and _Fire Protection Handbook_[12].

The hydrant flow test data is also useful for determining the hydraulic adequacy of the distribution system[5].

_Inspection and Condition:_ In the _Rating Schedule_[5] it is noted that the inspection and condition of fire hydrants should be in accordance with _AWWA Manual-M17_[6]. The primary function of inspection is to test all hydrants at regular intervals and maintain operating condition. The frequent inspection[3] and operating condition of fire hydrants as credited by ISO is[5]:

\[ CIC = \left( \frac{HI}{100} \right) \times 3 \]

in which, \( CIC = \) credit for inspection and condition,

\( HI = \) hydrant inspection, and

\( HF = \) hydrant condition.

Maximum reliability credit of this item means a community needs to inspect all fire hydrants twice a year and maintain all fire hydrants such that there is no leakage and hydrants open easily[17,48]. This inspection should include operation at least once a year[17]. Hydrants should be situated with outlets at least twelve inches above the grade and be painted in visible colors.

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[3]: Frequency of inspection is the average time interval between the three most recent inspections.
3.5 Service Interruptions

The reasons for service interruptions are main breaks, pump failure, malfunctions of valves and hydrants, and the like. Interruptions may also be due to scheduled maintenance, rehabilitation or replacement of water mains or an appliance. Such interruptions should be measured in terms of both the duration of interruptions and the number of customers affected. The service interruption performance measure is calculated as\(^{(14)}\):

\[
\frac{\sum_i (N_{out,i} T_{out,i})}{N_{req} T_{req}}
\]

in which

- \(i\) = index for interruption periods,
- \(N_{out,i}\) = customer equivalents affected by interruption period \(i\),
- \(T_{out,i}\) = duration of interruption period \(i\),
- \(N_{req}\) = total number of customer equivalents, and
- \(T_{req}\) = required period of service.

Water main breaks are a major concern that affects the adequacy of the distribution system because it directly causes service interruptions and strongly influence customers. Inoperable valves and hydrants impact the customers of a water system indirectly, e.g., an inoperable valve may make it impossible to isolate a section of the system in response to a water main break. It is important to maintain accurate location and working condition records of hydrants and important valves. Hydrants need more attention because of severe consequences that may result from a failure\(^{(14)}\).

The reliability of the water supply works should be evaluated after considering factors like, minimum yield, frequency and duration of drafts, condition of intakes, danger from earthquakes, floods, forest fire, ice formation, silting up or shifting of river channels, etc. Where the value of the property is higher or functioning critically, a level of reliability may be justified.
3.6 Automatic Sprinkler System

Installation of an automatic sprinkler system is a practical way to reduce NFF. Individual buildings can be evaluated and credited for adequacy and reliability of NFF. Total water supply and delivery rates are based on potential needs and not the wants of individuals. Sprinkler buildings are not considered in the evaluation of communities fire flow requirements.

Usually sprinkler systems demand higher water pressures from municipal water distribution systems\textsuperscript{15}. Sprinkler systems are not discussed in the present study.

3.7 Efficiency

Efficiency of the distribution system refers to as the conservation of water and energy. It includes unaccounted for water, main leaks and pumping efficiency.

Leakage is a significant component of unaccounted-for water and represents a major waste of a resource. Excessive use of energy for pumping can be a symptom of distribution system problems like tuberculation, main breaks, leaking gaskets and stuffing boxes, worn-out impellers, etc. Both leakage and pump inefficiency can also result in increased operating expenses\textsuperscript{14}. Efficiency and water quality is not further discussed in the present study.

3.8 Summary

There are concerns over reliability in a distribution system because of inadequate fire flow and pressure, interrupted supply, and water quality problems. A dependable water system of adequate capacity for all sections in a community is necessary. Although difficult to quantify in detail, the arrangement of the supply works, pumping facilities, water demands, etc. can affect adequacy and reliability of the supply system.

When the water delivery capacity is less than the NFF, it is important to know which distribution network component is limiting the fire flow. It is useful to evaluate the areas of a community that are unprotected because of inadequate water supply. Several water supply system components are
discussed in this chapter; however, evaluation is limited to operational performance and only indirectly includes design and construction.

Water can be transferred between the water systems of adjacent communities or between a private system and the public system by operating valves under emergency situations. Fire flows are very important in a distribution system. Fire flows can govern the size of the pipe used in many locations.

Fire flow tests of fire hydrants should be conducted to measure the flow rate at 20 psi residual pressure. These measured flows can be used to evaluate water system adequacy level when comparing to NFF.

The fire department needs to know the availability of the various water flow rates from a fire hydrant, and at what pressure these flows are available. The residual pressures for various water delivery rates are used to determine the type of hydrant operation, the size of supply line(s), engine rated capacity, etc. Consideration should also be given to the accessibility of the fire department.

In general, supply works capacity should be greater than water mains capacity, the water mains capacity should be greater than hydrants capacity, and the hydrants capacity should be greater than needed fire flows (i.e., supply works capacity > water mains capacity > hydrants capacity > needed fire flows).
Chapter 4

Computer Modeling for
Adequacy and Reliability of a Distribution Network

Computer modeling is required to evaluate transient effects due to fire flows and to analyze
distribution system performance concerning adequacy and reliability. Since transients can make a
distribution system inadequate and unreliable, computer modeling approaches of simulation and
optimization concerning adequacy and reliability of a distribution system are included in this
study and are briefly discussed in this chapter. This chapter also discusses possible water delivery
operations at fire hydrants that fire department utilizes. Chapter 5 reviews the modeling of
hydrant operations as boundary conditions. Chapters 6, 7 and 8 analyze transient conditions
linked with opening, closing, and other behavior of fire flows at the hydrants.

4.1 Introduction

Recent advances in computer technology and software are creating new opportunities for
improving water utility plans, and for managing water distribution systems. The greatly
expanding potential applications of computer models to distribution piping systems resulted in the
development of various software approaches including relational database management systems\(^1\)
(RDBMS), computer aided drafting\(^2\) (CAD), geographic information systems\(^2\) (GIS), customer

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\(^1\) the RDBMS systems are able to store and maintain all network asset data and allow easier access to data,
flexibility in data modeling, and a substantial decrease in data storage and redundancy requirements with
independence from physical storage.

\(^2\) CAD and GIS systems store and retrieve utility maps and associated descriptive facility information.
These systems enable the model input and output data to be developed and displayed in graphical form,
generation of accurate and detailed data, verification of data integrity, and the production of high quality
engineering maps.
information systems\(^3\) (CIS), maintenance management systems\(^4\) (MMS), laboratory information management systems\(^5\) (LIMS), supervisory control and data acquisition\(^6\) (SCADA) systems, energy management system\(^7\) (EMS), etc.\(^{[28,47]}\).

These applications of computer models enhance the productivity and effectiveness of distribution networks. These models can process and store data relevant to network modeling applications. The operational computing models also benefit direct integration, support intersystem applications, and streamline water distribution operation and decision making activities.

**Hydraulic Simulation:** Hydraulic calculations are based on the concepts and principles of hydraulics and fluid mechanics expressed in the form of a hydraulic model that incorporates a sequence of pre-defined physical accounting procedures. A hydraulic simulation model formalizes the linkages between variables that affect the distribution of flow and pressure in a water distribution network. Typically, it represents a compromise between theoretical complexity and practical reality to the point where the model can be used as a tool for predicting changes in the design variables that affect the performance and viability of a distribution system. Examples of distribution system simulation models are KYPIPE, CYBERNET, EPANET, TransAM, WATNET etc. Simulation models are commonly used for trial and error process of testing various combinations of pipes, tanks, valves, transient and water quality analysis, etc.

**Hydraulic Optimization:** The alternative to the simulation trial and error approach is to search for the best solution based on an optimization routine. Optimization models are created to find the values of the decision variables that minimize (or maximize) the value of the given objective function while satisfying a given set of constraints\(^9\).

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\(^3\) CIS systems contain data on the physical characteristics of service to the customer as well as customer billing and water consumption data.

\(^4\) MMS systems can maintain information on all water system facility characteristics that are used for model construction and management.

\(^5\) LIMS systems store water quality characteristics of the distribution system and automate a number of common laboratory functions.

\(^6\) SCADA systems provide real-time and historical operational data for all remote facility sites.
Typically simulation programs determine heads and flows at nodes and pipes in the network, while optimization programs that determine pipe sizes, pump heads, pipe locations etc. and optimize a particular objective (i.e., lowest cost solution).

Various approaches for evaluating adequacy and reliability of distribution systems are discussed in chapter 3. This chapter summarizes the simulation and optimization modeling of a distribution system. A brief discussion on hydraulic reliability modeling that accounts for uncertainties is also included. Derivations for methods of calculating the adequacy and reliability by numerical means are not discussed.

Hydrant operation, when delivering water for fire suppression, is vitally important when determining substantial pressure changes in distribution systems. Typical fire hydrant operations are also included in this study.

4.2 Computer Simulation and Optimization

The evaluation of distribution system performance concerning adequacy and reliability is complicated because structural, hydraulic, and water quality parameters are inter-linked\(^\text{[14]}\). The structural conditions can affect both hydraulic and water quality performance. The hydraulic conditions, such as dead ends, can result in water quality problems. Complex loops, grids and compound grid systems are often beyond the capacity of human analysis.

The main objective of optimization is to reach an optimum with the minimum possible computations and with the maximum guarantee so that the solution is really a global optimum of the formulated problem\(^\text{[58]}\).

4.2.1 Deterministic Solution Techniques

Hydraulic simulation and optimization models provide a powerful tool for determining the hydraulics of a water distribution system. A large number of such models have been reported in

\(^7\) EMS software compile pertinent electricity charges as well as demand and energy consumption data for use in determining optimal system operational policies.
the literature. There are many optimization techniques like gradient techniques, linear programming, non-linear programming, enumeration and dynamic programming, simulated annealing, genetic algorithms, and hierarchical decomposition.

Researchers are presently linking simulation programs and optimization programs. The development and history of the subject can be found in references [10], [14], [45] and [66]. Linear programming is one of the early network optimization attempts. Brief introductions to enumeration, nonlinear programming and genetic algorithms are provided next.

**Enumeration:** Complete enumeration techniques simulate every possible combination of the decision variables. Enumeration can select the optimal solution, i.e., cheapest cost network that satisfies the pressure constraints. In many cases complete enumeration requires a considerable (and sometimes prohibitive) amount of computer time and memory. It is difficult to enumerate completely when combinations of variables becomes large\[^66\]. A number of researchers\[^41,66\] proposed techniques for selective enumeration (partial enumeration), but the global optimum may be eliminated in the process\[^41\].

**Nonlinear Programming:** Network constraints include the continuity equations, head losses around loops, pressure limitations, pipe diameters, external demands, optimal location of a booster pump or flow regulating valve, etc. Linear programming cannot include many of these constraints. Although there is a limitation on the number of constraints and size of the network, there are many nonlinear optimization packages MINOS\[^38\], GINO\[^57\], GAMS\[^26\], etc. When planning a network, pipe diameter is the most cost concern variable. Some models consider the diameter as a continuous variable while others\[^38\] directly determine the discrete size. El-Bahrawy and Smith\[^38\] applied the MINOS model to round off the pipe sizes to commercial diameters.

Su et al.\[^67\] used generalized reduced gradient (GRG) technique and include reliability constraints. In their model reliability is defined as the probability of the design pressure being maintained at appropriate nodes in the system, given the possibility of some pipes being unavailable because of breakage.
Duan et al.\textsuperscript{[37]} used GRG, KYPIPE and RAPS models and developed a general optimization model that can include pumps, tanks and their locations. They also included multiple loading conditions.

**Genetic Algorithms:** Genetic algorithms (GAs) represent an efficient global method for non-linear optimization problems. These algorithms are based on natural selection and the mechanisms of population genetics\textsuperscript{[42]}. Researchers obtain near-optimal rather than true optimal solution to a network problem using a rule-based procedure to update and improve the previous best solution\textsuperscript{[22]}.

These algorithms share favorable attributes of Monte Carlo techniques over local optimization methods. GAs do not require either linearizing assumptions or calculation of partial derivatives and are able to avoid numerical instabilities associated with matrix inversion\textsuperscript{8f}. GAs are introduced in their original form followed by different improvements that were found necessary for their effective implementation in the optimization of water distribution networks.

The GA techniques require set(s) of decision variables and are represented by a coded string\textsuperscript{9f} of finite length\textsuperscript{[34,66]}. This coding string is analogous to the structure of a chromosome in a genetic code\textsuperscript{[34]}. The GA evaluates the trial solution and computes a measure of worth or *fitness* for the string.

Researchers\textsuperscript{[27,53,58,66]} reported a genetic algorithm with three working elements: reproduction, crossover, and mutation. GA often finds the global optimum in relatively few evaluations compared to the size of the search space. Simpson et al.\textsuperscript{[66]} compare optimization by the stochastically based method (of genetic algorithms) with nonlinear method. He mentioned that GAs do not necessarily guarantee global optimum solution but rather a near-optimal solution.

\textsuperscript{8f} genetic algorithms need only objective function, not trend, derivative, or other auxiliary data. Also genetic algorithms search from a population of points, not from a single point\textsuperscript{[37]}.

\textsuperscript{9f} genetic algorithms work with coding set(s) of the variables and not with the variables themselves. Traditionally, binary coding has been used to specify the mapping. However, the improved genetic algorithms uses a Gray code representation\textsuperscript{[37]}.
4.3 Modeling for Adequacy and Reliability

As discussed in chapter 3, the reliability of a water distribution system is concerned with hydraulic failure, structural failures (also called mechanical failure), and efficiency and water quality problems. Hydraulic failure is the system failure due to delivered flow and pressure being inadequate at demand points. Mechanical failure is the system failure due to pipe breakage, pump failure, power outage, control or flow regulating valve failure etc. The reliability measures (definition and calculation) for a distribution network is not straightforward. Customers want adequate supply of water at all times at an adequate pressure while planners want networks that require little expenditure on construction and operation. Increase in reliability of water supply increases cost\(^{[45]}\). Also several different measures of performance may be necessary to define reliability\(^{[72]}\). Both adequacy and reliability modeling are therefore multi objective in nature. The issue is to optimize cost while setting reliability objectives at an acceptable level.

4.3.1 Nature of Failure

A higher water demand (i.e., fire flow) than the capacity of a network results in a hydraulic performance failure. Flow demands are stochastic and are most appropriately defined by a probability distribution rather than by a single precise deterministic value; therefore, the probability of flow requirements (real time demand forecasting) greater than the design flow must be evaluated. Water demands are also time dependent, i.e., demands can increase due to unexpected increases in population growth or change in lifestyle. For instance, if demands do not exceed the design value, the capacity of the network itself may decrease due to increase in friction with aging.

Clearly, the probability of demands exceeding the capacity of the network has both short term as well as long term implications. Other water supply system components like supply sources works, treatment facilities, etc. have similar complications and can thus cause hydraulic performance failure.

Other system performance failures are mechanical failures and are associated with components like pipes, pumps, valves etc. The hydraulic capacity of the network is reduced when a network
component fails or is isolated for service and the network is unable to meet pressure and/or flow requirements. If the system is designed with sufficient redundancy, the failure of the component does not necessarily cause failure of the network. The potential of component failure to reduce network capacity must be addressed in the reliability analysis. The probability of the event that cause failure or loss of service, and the magnitude of the impact of failure should be incorporated in the reliability measure. The period of time in which a network is able (unable) to meet the demands is one aspect of reliability.

Water quality performance deals with physical and chemical parameters such as turbidity, chlorine residuals or trihalomethanes. The reliability framework of water quality performance needs evaluation of specific variables across the network over an extended period of time. A transport model is necessary to estimate the parameter values.

4.3.2 Modeling Approaches

Bouchart and Goulter[23] formulated reliability of volume deficits arising from demand variation failure and from component (pipe) failure. They did not address the impacts of the volume deficit on the rest of the network due to removal of one of the components (i.e., pump or valve failure). The technique was mainly concerned with volumetric deficit. It did not account for the ability of a network to meet flow demands at reduced pressures.

Cullinane et al.[33] made a formulation based on availability, i.e., percentage of time when the pressure in the network is lower than a preset level. Their formulation recognizes the relationship between flows and pressures explicitly. However, they do not consider the duration or probabilities of the events. Their formulation does not consider the stochastic nature of the demands.

Goulter[45] mentioned a case-by-case method of reliability determination using simulation models. He defined a series of cases or set(s) of demands and network configurations. The network is modeled for each of the cases to determine pressure and flows. The ability of the network to meet the demands over the complete set of cases can then be evaluated. The system component failures are handled through adjusting the network configuration. For example, the loss of a pump, pipe,
valve, can be addressed by the removal of the relevant element from the network prior to hydraulic simulation. The concern is whether or not the sub-set of combinations contains all the critical combinations and in particular does it contain the worst case.

Jowitt and Xu[50] proposed a method that predict inadequacy followed by component failure without the requirement for a full hydraulic simulation. Their method reduces some of the computational burden of case-by-case method of hydraulic simulation.

Duan and Mays[37] drive eight different reliability measures while incorporating a number of simulation features. Bao and Mays[21] used a Monte Carlo simulation approach to measure reliability. The hydraulic reliability and time series was generated by modeling the probability distribution of the demand, pressure head and pipe roughness.

Goulter[44] applied an analytical approach called graph theory and described the performance of the network in terms of shape and connectivity between nodes, particularly source nodes and demand nodes. He incorporated hydraulic performance into reliability approaches. Hydraulic performance aspects cannot be associated with layout of the network by this theory.

Heuristic techniques are being used recently to assess and evaluate reliability of water distribution networks. Lansely et al.[56] used a redundancy measure[56] that was derived from entropy theory. Park and Liebman[62] quantify redundancy measure based on the expected shortage of flow due to failure of individual pipes. Martinez et al.[58] mentioned that the decision variables are considered simultaneously, but are grouped in levels. To make a decision in the upper level, all sub-optimums in lower level must be previously considered and then transferred to the upper level. The heuristic techniques are conceptually simple to address the complicated problems and also help to apply different optimization techniques in each level. The heuristic models cannot guarantee the convergence for general problems.

\(^{10f}\) redundancy is the existence of alternative paths. The basis of their redundancy measure is that the entropy theory is able to measure flexibility and diversity in the system, therefore, reflect the redundancy of the network as it contributes to reliability.[56].
Tanyimboh and Tabesh\textsuperscript{[68]} derived \textit{hitherto intuitive source head method} for calculating reliability of a distribution network. They used full demand satisfaction with reliability as the expectation of the ratio of available flow to required flow. The estimated reliability values are probabilistic. They accounted for the random nature of system component failures. The methodology is simple and straightforward to implement and demonstrated to be computationally efficient.

4.4 Hydrant Operation

Hydrant operation is the water delivery from a fire hydrant to the fire attack point with or without the use of fire apparatus. The modeling of hydrant operations as boundary conditions are included to analyze transient conditions linked with opening, closing, and other behavior of the flow outlets. The hydrant boundary condition is reviewed in chapter 5. The following are possible water delivery operations from fire hydrants that a fire department utilizes:

- **Hydrant operation without in-line pumping**: Water delivery operations that are related to transport of water from a fire hydrant to a fire attack point while no pumper is equipped to increase pressure within the hose line. Source pressure (residual pressure at a hydrant at a deliver rate) is overcoming all losses due to friction in hose, fittings, and elevation.

  \textit{Straight stream hydrant operation}: A supply line hose operation in which the direction of the hose lay is from the fire attack point towards the hydrant. Source pressure is the residual pressure at a hydrant at a delivery rate.

  **Hydrant tanker fill operation**: A tanker fill operation is the process of transferring water from a hydrant to movable tanker(s) of fire apparatus.

- **Hydrant operation with in-line pumping**: The in-line pumping is an operation in which an engine (mobile fire pumper) is used to boost the pressure within a hose line.
**Straight stream hydrant operation:** A supply line hose operation in which the direction of the hose lay is from the hydrant towards the fire attack point. This is the most common water transporting method from a fire hydrant to the fire attack point(s). The engine is positioned just beyond the fire structure, but away from the fire hydrant.

**Reverse hydrant operation:** A supply line hose operation in which the direction of the hose lay is from the fire attack point towards the hydrant. This method is used to maximize the water delivery rate from the hydrant. The engine is positioned close to the fire hydrant.

**Relay hydrant operation:** A water transport operation in which engine(s) relay water through an open or closed relay to other engine(s) from a hydrant. An open relay involves an open (to atmosphere) portable water holding tank that provides a static water supply for the next engine in the relay. But in closed relay operation water is relayed from the discharge of one engine through a hose line to the intake of another engine.

**Tandem pumping operation:** Tandem pumping operation occurs when a fire pump being supplied by a supply line supplies water to another fire pump through a supply line connected from an intake of the first pump to another intake of the second pump.

**Hydrant tanker fill operation:** A tanker fill operation is the process of obtaining water from a hydrant to movable tanker(s) of fire apparatus. This method is used in all situations where a hydrant is used as a tanker fill source. In cases where the tanker fill rate exceeds the water supply available from a hydrant a portable open tank is used. Water from the hydrant is discharged to this portable tank without the provision of an engine, and then water is forced from portable tank to mobile tankers by using a fire engine.

- **Multiple hydrant operation:** There are other possible operations like

  **Multiple fires in a network area:** It is likely that there are multiple fires at the same time in a large community. There is more chance that these hydrants are far apart from each other.

  **Multiple hydrants used to suppress a fire:** There may be a situation where single hydrant is unable to supply NFF. Starting and stopping times of multiple hydrant valves may vary. There is more chance that these hydrants are close to each other.

  **Multiple hydrants join with one manifold:** There may be a situation where single hydrant is unable to supply NFF. Starting and stopping times of multiple hydrant valves will be the same. There is more chance that these hydrants are close to each other.
These hydrant operations may be used individually or in any combination depending upon circumstances like required delivery rate. There is possibility of two simultaneous major fires in large cities or smaller places with high fire incidence\textsuperscript{[17]}.

\textsuperscript{[35]}In straight stream hydrant (also called forward lay hydrant) operation, the source pressure is the residual pressure available at the hydrant at the delivery rate. Pumping may or may not be involved in forward lay hydrant operation. In reverse lay and relay pumping hydrant operations, the source pressure is the residual pressure at the hydrant at the delivery rate plus the pump discharge pressure at the delivery rate. Generally straight stream hydrant and relay pumping hydrant operations require outlet pressure of 20 psi, but, reverse lay hydrant operation require outlet pressure well above this pressure value. More explanation of each hydrant operation can be found in \textit{Water Supply Command}\textsuperscript{[35]}.  

4.5 Summary

The objective of an optimization technique is to find a combination of decision variables which minimizes an objective function subject to constraints. Although various approaches for optimum design and operation of a water distribution system have been reported in the literature, the present optimization models are also criticized for the lack of practical and general applicability. Diversity of design and operation situations makes it difficult to produce an all-purpose optimal design package for water distribution systems. Some problems can be handled by a linear programming, others require dynamic programming or complex search techniques like genetic algorithms.

The techniques based on genetic algorithms use natural selection rules that guide the evaluation process. Genetic algorithms are often proposed as a mean of global optimization while keeping a range of good solutions to avoid being drawn into a false local optimum.
Flow conditions in pipes may vary when a sudden change in fluid velocity creates a significant change in fluid pressure, commonly referred to as a waterhammer or surge condition. Waterhammer can damage pumping and other facilities and can cause pipes to rupture. However, waterhammer can often be avoided by designing and/or operating these systems such that unfavorable changes in water velocity are minimized.

The following section provides some background that is required to understand transient conditions in networks that convey water or fuel. The transient conditions in a system can be represented mathematically by a set of differential equations. To solve these differential equations a numerical solution procedure, the method of characteristics, is considered. A computer program TransAM, which is used for simulation purposes in the present study, is also described briefly.

5.1 Introduction

Transient conditions in water supply distribution networks may occur due to changes in the flow demand, opening or closing of fire hydrants, starting or stopping of pumps, changes in the setting of control equipment or the failure of a system component. The analysis of these conditions is necessary for the design and operation of water supply system for a community\cite{20,29,52,74}.

Transient flows in water supply networks are divided into two analysis approaches, slow varying and rapidly varying, depending upon time rate of change of flow\cite{29}. An extended-period simulation approach has been used for slow varying flows. This approach involves a sequence of steady states subject to varying boundary conditions. The Lumped-system approach considers the flowing media as solid mass and any flow change is assumed to travel at infinite speed. This
approach includes only variation with respect to time in a single pipe while changes in the flow conditions along the pipe is neglected. Waterhammer analysis, or the distributed-system approach, is applicable to both slow and rapid changes. This approach is usually used for rapid changes and is in more detail.

The distribution system approach involves a set of partial differential equations with compressibility effects influencing describes system behavior.

5.2 Governing Equations

Steady, quasi steady, unsteady incompressible (i.e., hydraulic pressure surge), and unsteady compressible (i.e., waterhammer) flows are all governed by the basic differential equations for transient flow. Governing equations describing unsteady flows use the momentum or energy conservation and continuity principles to model the hydraulics of an element. The one-dimensional form of these equations are summarized below and elaborated in the next section\textsuperscript{[29,30,74]}. The momentum and continuity equations are often written

\[
\frac{\partial Q}{\partial t} + \frac{Q}{A} \frac{\partial Q}{\partial x} + gA \frac{\partial H}{\partial x} + RQ|Q| = 0 \quad \text{.................................................................5.1}
\]

\[
\frac{\partial H}{\partial t} + \frac{Q}{A} \frac{\partial H}{\partial x} + \frac{a^2}{gA} \frac{\partial Q}{\partial x} = 0 \quad \text{.................................................................5.2}
\]

in which, \( Q \) = rate of discharge, 
\( t \) = time, 
\( g \) = acceleration due to gravity, 
\( A \) = pipe cross-sectional area, 
\( H \) = piezometric head, 
\( x \) = distance along the centreline of the conduit,
\[ a = \text{acoustic wave speed}^{\ddagger}, \]
\[ R = f(2DA), \text{ and} \]
\[ f = \text{Darcy-Weisbach friction factor}^\S. \]

These equations (5.1 & 5.2) represent unsteady, uniform, compressible flow in which the flow, fluid density, and pressure head varies with both time and space.

### 5.3 Method of Characteristics: A Solution Procedure

Equations 5.1 and 5.2 are nonlinear hyperbolic partial differential equations. An analytical solution of these equations is not available, therefore, a numerical solution is desired. Many researchers use the numerical procedure called the method of characteristics\(^{[20,29,52,59,74]}\), other numerical procedure such as the finite-difference method\(^{[29,74]}\) is also in use.

The method of characteristics transforms these partial differential equations into a system of ordinary differential equations. These ordinary differential equations are valid only along the characteristic lines.

If partial derivatives are represented by subscripts, the equations 5.1 and 5.2 can be written in terms of the variable of interest, \(H\), and \(V\):

\[ V_t + VV_x + gH_x + \frac{fV|V|}{2D} = 0 \] ............................5.3

\[ H_t + VH_x + \frac{a^2}{g}V_x = 0 \] ............................5.4

\(^{\ddagger}\) describes the elasticity of the pipe wall, contained fluid, and support conditions, and is assumed constant for each conduit.

\(^\S\) Has traditionally been assumed constant and equal to that determined under steady state condition for each pipe; a number of more sophisticated approaches are currently being investigated.
in which \( H = H(x,t) \), and \( V = V(x,t) \) = instantaneous average velocity. By neglecting acceleration terms, \( VV_x \), and \( VH_x \), multiplying equation 5.4 by an unknown multiplier \( \lambda \) and adding to equation 5.3, and after rearranging:

\[
\lambda \left[ H_x \frac{g}{\lambda} + H_t \right] + \left[ V_x \frac{a^2}{g} + V_t \right] + \frac{fV|V|}{2D} = 0 \tag{5.5}
\]

Since dependent variables \( V \) and \( H \) are both functions of \( x \) and \( t \), total derivatives can be written as:

\[
\frac{dH}{dt} = H_x \frac{dx}{dt} + H_t \tag{5.6}
\]

\[
\frac{dV}{dt} = V_x \frac{dx}{dt} + V_t \tag{5.7}
\]

The unknown multiplier is defined as:

\[
\frac{1}{\lambda} = \frac{dx}{dt} = \lambda a^2 \tag{5.8}
\]

Based on equations 5.6, 5.7 and 5.8, equation 5.5 can be written as:

\[
\frac{g}{a} \frac{dH}{dt} + \frac{dV}{dt} + \frac{fV|V|}{2D} = 0 \tag{5.9}
\]
if \[ \frac{dx}{dt} = a \] \hspace{1cm} \text{5.10}

and \[ -\frac{g}{\alpha} \frac{dH}{dt} + \frac{dV}{dt} + \frac{fV|V|}{2D} = 0 \] \hspace{1cm} \text{5.11}

if \[ \frac{dx}{dt} = -a \] \hspace{1cm} \text{5.12}

Figure 5.1: x-t Grid for Method of Characteristics

Equations 5.9 and 5.11 are valid only along the positive and negative characteristics lines (C\(^+\) & C\(^-\)), Figure 5.1. Numerical integration must be performed along C\(^+\) and C\(^-\). Considering x-t grid such that \(\Delta x = \pm a \Delta t\), the finite-difference form of these equations gives:

\[ H_p = C_p - B_p Q_p \] \hspace{1cm} \text{5.13}

and \[ H_p = C_M - B_M Q_p \] \hspace{1cm} \text{5.14}
in which the constants of integration, with inclusion $\varepsilon$, are equal to\textsuperscript{[52]}:

\[ C_p = H_A + Q_A \left[ B - R|Q_A|(1 - \varepsilon) \right] \] \hspace{1cm} 5.15

\[ B_p = B + \varepsilon R|Q_A| \] \hspace{1cm} 5.16

\[ C_M = H_B + Q_B \left[ B - R|Q_B|(1 - \varepsilon) \right] \] \hspace{1cm} 5.17

\[ B_M = B + \varepsilon R|Q_B| \] \hspace{1cm} 5.18

where $\varepsilon = \text{linearization constant and varies from 0 to 1 to exist for monotonic variations of discharges}$\textsuperscript{[52]}. At point $P$ internal to a pipeline, $H_P$ and $Q_P$ can be calculated from equations 5.13 and 5.14.

Since at boundaries only one equation is available, therefore additional equations are needed to include the boundaries. In addition initial boundary conditions must also be known at points $A$ and $B$ to evaluate characteristic constants (equations 5.15 through 5.18).

5.3.1 Initial (Steady State) Conditions

Before starting the transient computations, initial steady state heads and discharges must be determined for the entire network. These may be calculated by using a steady state analysis on the basis of boundary conditions at the ends of branches of the network. The other procedure is to run transient computations from arbitrary initial conditions subject to the appropriate boundary conditions. This computation should be continued for a long enough time until conditions converge to a steady state.
5.3.2 Boundary Conditions (Node Equations)

Many workers\cite{20,29,52,74} describe ordinary boundary conditions of a water distribution network. In the present study the boundary conditions for valves discharging to the atmosphere (for fire hydrants operation) and for junctions having several incoming and outgoing pipes are described.

Branching Junction: Let $N_1$ be the number of pipes for which the flow direction is towards the node, and $N_2$ be the number of pipes where flow direction is away from the node. The designation of the inflow and outflow from the node (or junction) is with respect to the initial steady state conditions. Requisite number of equations will be $2(N_1+N_2)$, since for each pipe (at node) there are two unknowns, $H_p$ and $Q_p$. Assume $Q_{ex}$ is the known external flow.

For the inflow and outflow pipes the equations 5.13 and 5.14 can be rearranged as $(N_1+N_2)$ equations:

\[
Q_{p_i} = \frac{H_{p_i}}{B_{p_i}} + \frac{C_{p_i}}{B_{p_i}}, \quad i \in N_1 \quad \ldots \quad 5.19
\]

\[
-Q_{p_j} = -\frac{H_{p_j}}{B_{p_j}} + \frac{C_{p_j}}{B_{p_j}}, \quad j \in N_2 \quad \ldots \quad 5.20
\]

Suppose there is no head loss at the junction, and the difference in velocity heads in different pipes is neglected. Therefore, all $H_p$ values at the junction for inflow and outflow pipes are equal and give $(N_1+N_2-1)$ equations:

\[
H_{p_k} = H_{p_k} = H_{p_k} = \ldots = H_{p_k}, \quad k \in (N_1 + N_2) \quad \ldots \quad 5.21
\]
One last required equation is the continuity equation for the junction:

\[ \sum_{i \in N_i} Q_i - \sum_{j \in N_j} Q_j - Q_{\text{ext}} = 0 \] .......................... 5.22

The unknown values of \( H_p \) and \( Q_p \) for each pipe at the junction can be calculated through equations 5.19 to 5.22. Details can be found in many references\(^{[20,29,52,74]}\).

**Valve Discharging to Atmosphere**: Boundary conditions for large leakage, opening and closing of a fire hydrant, pipe rupture, discrete nodal demand, etc., can be represented by externally discharging valve\(^{\ddagger}\).

A constant head reservoir boundary condition can be applied, in which:

\[ H_p = \text{Elevation of the valve or leak orifice above the datum} \] .......................... 5.23

\[ Q_p = Q_{\text{ext}} \] .......................... 5.24

To account for loss of head through the valve, a valve discharge relation can be included in the associated pipe that is connecting the boundary to the network, detailed in references\(^{[20,52]}\).

### 5.4 The Computer Program: TransAM

The TransAM\(^{[59]}\) computer program has been developed by Dr. Bryan W. Karney and others to predict *waterhammer and mass oscillation* in networks that convey fluids such as water or fuel. Some typical applications for TransAM include analysis of pumping facilities, and water distribution systems. The program determines time varying flow and head (transients) in a

\(^{\ddagger}\) if a storage tank is closed to the atmosphere, it is called a pneumatic tank or an air chamber\(^{[20]}\).
network that may include pipes, valves, pumps, pump-turbines, surge tanks, leakage from pipelines, and junctions arranged in any reasonable configuration. Such transients are generated due to any variation in the operation of hydrant or valve within the network, or due to changes in the head or discharge at boundaries of the network. The rotational speed, torque, and power of pumps are computed along with the system hydraulic variables. TransAM also generates various graphic plots and can be used for creating time history plots from simulations.

The main TransAM simulation program uses method of characteristics for calculating time-varying flows and pressures throughout the network that is being modeled.

5.5 Summary

Water supply distribution networks experience transient loading resulting in waterhammer and mass oscillation. The design and operation of such systems requires analysis of these unsteady conditions (due to fire flow and other loading). Transient conditions in a distribution system are represented by one-dimensional nonlinear hyperbolic partial differential equations.

A closed form solution of partial differential equations is not available. A numerical procedure, method of characteristics has commonly been used for the computation of pipeline transients. For a numerical solution initial and or boundary conditions must be given in addition to the basic physical laws.

For simulation purposes the TransAM program is selected because it performs dynamic simulation of fluid distribution networks comprising many components such as pipes, valves, pumps, surge tanks, and junctions. The program calculates time-varying flows, pressures, and heads throughout the network.
Chapter 6

Effect of Fire Fighting Supplies on Distribution Systems
Case Study of a Simple Network

Transient forces due to fire flows can rupture pipelines or fittings and can interrupt water supplies. Therefore, the design and operation of a water utility to a community require the evaluation of transient effects caused by fire flows. This chapter presents a case study of a simple network and evaluates transients caused by various circumstances including change in flow demand, opening and closing of a fire hydrant main valve, etc. Chapter 7 analyzes transient conditions associated with a larger network. A summary of results and comparison of the case studies are presented in chapter 8.

6.1 Introduction

Water hammer is the effect of pressure changes that accompany a sudden change in the velocity of water flowing in a pipeline. Transient pressure surges may be initiated by the operation of a hydrant or by the sudden development of an abnormal water demand for fire fighting. The determination of these parameters is particularly useful for the design and maintenance of a distribution network.

Two example networks are analyzed to illustrate the application of transient theory to a distribution system supplying water for fire flows. One example is a simple network in this study, since behavioral response caused by hydrant(s) operation can easily be integrated. This chapter discusses transients in a simple network caused by fire flows. Chapter 7 discusses the transients in a larger network for fire flows.
The effect of hydrant operation is simulated when there is a normal course of flow. This variable can also be used to determine the flow rate that can be safely achieved at a specific hydrant and has both design and operation applications. The program TransAM is used for simulation purposes.

It is assumed that all system components like water sources, mains, controlling valves, pumps or boosters in the network are fulfilling system requirements and functioning normally under imposed transient loading. Transient effects on these system components due to fire flow are not directly discussed. The present study concentrates on pressure changes within the system due to pressure surges. It also explores pressure reduction within the system due to the possibility of excessive water withdrawal rate from the system for fire fighting. In addition, some consideration is given to adequacy and reliability due to pressure surges and reduction in water pressure within the system.

This chapter presents some simulations for a simple network and evaluates transients caused by various circumstances associated with fire flows from a hydrant. Transient forces due to fire flows can rupture pipelines or fittings and can make a distribution system inadequate and unreliable. The simulation results are helpful for indicating hydraulic conditions in a distribution system when water is being withdrawn for fire fighting. These results can be valuable to design, operate and extend a water utility for a community.

**Flow Control:** It is assumed that a magnitude of flow can be allowed to exit a distribution system such that a specified pressure condition will be maintained. The location (a hydrant) at which the flow exit the system is denoted as junction node and the external demand at that location is treated as NFF for fire suppression.
6.2 Network-1

A simple hypothetical network example is selected to emulate the transient effect that could occur within the network.

The network consists of 7 pipes and 7 nodes forming one loop, and two water supply sources (wells), Figure 6.1. The network includes one external pump, and there is one hydrant at both nodes [6] and [7].

<table>
<thead>
<tr>
<th>Pipe</th>
<th>C</th>
<th>Diameter (inch)</th>
<th>Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>110</td>
<td>6</td>
<td>1000</td>
</tr>
<tr>
<td>2</td>
<td>110</td>
<td>9</td>
<td>1000</td>
</tr>
<tr>
<td>3</td>
<td>120</td>
<td>6</td>
<td>1000</td>
</tr>
<tr>
<td>4</td>
<td>120</td>
<td>6</td>
<td>1000</td>
</tr>
<tr>
<td>5</td>
<td>130</td>
<td>9</td>
<td>1000</td>
</tr>
<tr>
<td>6</td>
<td>130</td>
<td>9</td>
<td>1000</td>
</tr>
<tr>
<td>7</td>
<td>130</td>
<td>6</td>
<td>1000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Junction</th>
<th>Elev. (ft)</th>
<th>Ext. Flow (US gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1000</td>
<td>800</td>
</tr>
<tr>
<td>2</td>
<td>1000</td>
<td>400</td>
</tr>
<tr>
<td>3</td>
<td>1000</td>
<td>–</td>
</tr>
<tr>
<td>4</td>
<td>1000</td>
<td>–</td>
</tr>
<tr>
<td>5</td>
<td>1000</td>
<td>–</td>
</tr>
<tr>
<td>6</td>
<td>1000</td>
<td>Hydrant</td>
</tr>
<tr>
<td>7</td>
<td>1000</td>
<td>Hydrant</td>
</tr>
</tbody>
</table>

Figure 6.1: Example Network-1

To simulate a fire flow from the distribution network it is assumed that there is a valve at each hydrant, and that each valve can be operated to discharge water (to atmosphere). While the network was supplying demand flow under steady state conditions, the hydrant valve starts discharging a specified quantity of water flow for fire fighting.
6.2.1 Assumptions

- A wavespeed of 1100.0 ft/sec (representative of plastic pipe) is assumed for all 7 pipelines, and there is no variation in pipeline elevations.

- Each pump is associated with a dummy discharge valve, which remains open as long as the network is in working condition. The pump station is given by some assumed but appropriate values of rated discharge, rated head, rated speed, initial speed, rated efficiency, and inertia. These characteristic values are:

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Pump P</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rated discharge, cfs</td>
<td>3.35</td>
</tr>
<tr>
<td>Rated head, ft</td>
<td>277.0</td>
</tr>
<tr>
<td>Rated speed, rpm</td>
<td>1000</td>
</tr>
<tr>
<td>Initial speed, rpm</td>
<td>1000</td>
</tr>
<tr>
<td>Efficiency</td>
<td>0.80</td>
</tr>
<tr>
<td>Roter inertia, lbf-ft²</td>
<td>700.0</td>
</tr>
</tbody>
</table>

- At node [7] there is a valve within the hydrant, which can discharge to atmosphere. The network was supplying demand flow under steady state conditions, meanwhile the valve starts discharging a flow of 1000 US gpm (2.228 cfs).

- The valve takes 10 seconds to open fully with linear operation. This assumption is based on watching videotapes on fire fighting operation at Fire Marshal Library².

- Some of the graphs showing transient pressures in this thesis are not strictly realistic since the negative pressures appeared are sometimes far below the vapor pressure of the fluid. Moreover, little attention has been paid to the issue of pipe strength or rating. Clearly, the large negative pressures are not physically possible. However, this does not directly take such results away from their utility. The transient simulations were being forced to run under these conditions so the results between different simulations in these hypothetical systems can be easily compared. Thus, the unrealistic pressure values are simply intended to identify the relative intensity of transients between different systems on a common basis.

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² Fire Science Library and AV Resources Centre, Office of the Fire Marshal, Place Nouveau Building, 5775 Yong Street, 7th Floor, North York, M2M 4J1, Canada.
In areas where transients are produced rapidly, the pressure may be reduced to the vapor pressure of the flowing fluid. This reduction of pressure can produce vapor cavities in the flow or may cause the water column to separate. Rejoining of these separated water columns or collapse of the cavities can result in a large pressure rise.

6.2.2 Simulation Scenarios

The following scenarios are considered to simulate a fire flow from a hydrant:

- **Case a:** 1) The hydrant valve at node [7] starts discharging a flow of 1000 US gpm (2.228 cfs).
  2) The hydrant valve at node [6] starts discharging a flow of 1000 US gpm (2.228 cfs) while the valve at node [7] is not operating.

- **Case b:** Hydrant valve (at node [7]) in operation including opening and closing of valve.

- **Case c:** Hydrant valve (at node [7]) in operation with valve opening time of 0.0001, 5, 15, 35 and 75 seconds, with hydrant flow remaining unchanged.

- **Case d:** Hydrant valve (at node [7]) in operation with the flow rate of 500, 1000 and 1500 US gpm, while valve operation duration remains unchanged (i.e., 10 seconds).

- **Case e:** Operation of multiple hydrants where valve opening time may or may not vary.

- **Case f:** Transient effects as influenced by the water carrying capacity of the network by increase of pipeline size (throughout the network).

- **Case g:** Transient effects as influenced by the pipeline material (wavespeed).

- **Case h:** Effect of valve losses on limiting heads.

**Case a:** 1) The hydrant valve at node [7] starts discharging a flow of 1000 US gpm (2.228 cfs).

Figure 6.2 shows two sections of the system through pipelines (5)-(3)-(1)-(2) and (5)-(6)-(7) profiling limiting heads due to transition effects. Figure 6.3 shows head variations at selected
Figure 6.2: Pipe and Head Profile - Hydrant at Node [7] in Operation

Figure 6.3: Head Variation at Selected Nodes due to Operation of a Hydrant at Node [7]
nodes [3], [4], [5] and [6]. These sections and nodes of the system are in accordance with Figure 6.1.

It is clear from Figure 6.2 that there are significant transient effects within the network due to opening of a hydrant at node [7]. At node [3] there is no observed transient effect, since this location represents a constant head reservoir. At node [4] there is a pump contributing water supply to the network and the change in pressure is considerable. At node [7] the transient effect is greatest, since the hydrant at node [7] has single water supply feeder. It also shows that in pipeline (7) there is generation of vacuum conditions. This vacuum can either collapse the water line due to the difference in external and internal pressure or might burst the water line due to rejoining of separated water columns. It also shows that the section of the network that has pipeline (7) is not reliable and has insufficient capacity.

Case a: 2)- The hydrant valve at node [6] starts discharging a flow of 1000 US gpm (2.228 cfs) while the valve at node [7] is not operating.

Figure 6.4 shows two sections of the system through pipelines (5)-(3)-(1)-(2) and (5)-(6)-(7) profiling limiting heads due to transition effects. Figure 6.5 shows head variations at selected nodes [3], [4], [5] and [6].

It is evident from Figure 6.4 that the transient effect within pipeline (7) is considerably less compare to the previous case when hydrant at node [7] was working. Pipeline (7) is adequate to sustain lower bound transient pressures. The positions of different operating hydrants within the network have different transient effects.

It is also clear that a pipeline that has a hydrant at a dead end is prone to suffer greater transient effects as compare to the one that has a hydrant at a loop.
Figure 6.4: Pipe and Head Profile - Hydrant at Node [6] in Operation

Figure 6.5: Head Variation at Selected Nodes due to Operation of a Hydrant at Node [6]
Case b: Hydrant valve (at node [7]) in operation including opening and closing of valve.

Figures 6.6, 6.7 and 6.8 shows head variations at selected nodes [3], [4], [5] and [6] against hydrant valve operation. It is clear from the Figures 6.7 and 6.8 that as the valve opens the transient pressure in the pipeline dives down and as the valve closes the transient pressure in the pipeline jumps up. It also shows that the rise and fall of transient pressure crosses over the final steady state pressure that prevails after a certain period of time.

Quick opening of a valve can either collapse a water line due to the difference in external and internal pressure or can burst a water line due to rejoining of separated water columns caused by negative pressures. Sudden closing of a hydrant valve can burst a water line if transient pressures exceed the pipeline rating pressure.

Figure 6.6: Head Variation at Selected Nodes due to Opening and Closing of a Hydrant
Figure 6.7: Head Variation at Selected Nodes due to Opening of a Hydrant

Figure 6.8: Head Variation at Selected Nodes due to Closing of a Hydrant
Case c: Hydrant valve (at node [7]) in operation with valve opening time of 0.0001, 5, 15, 35 and 75 seconds, with hydrant flow remaining unchanged.

Figures 6.9(a) through 6.9(e) shows head variation at node [7] due to variation in hydrant valve opening operation times of 0.0001, 5, 15, 35 and 75 seconds. Figures B.1, B.2 and B.3, presented in Appendix B, show head variations at selected nodes [3], [4], [5] and [6].

It is clear from Figures 6.9(a) through 6.9(e) that increasing of operation time results in decreasing of transient effects in the network. The concave shape of curve in Figure 6.9(d) and 6.9(e) indicates that opening pattern at the end of operation has greater transient influence than at the beginning of operation. Figure 6.9(f) summaries the effects of valve operation time as indicated in Figures 6.9(a) through 6.9(e). It is also clear from Figure 6.9(f) that transient effects are minimized when valve opening operation duration is greater than 15 seconds.

Since opening and closing time of valve can significantly harm water supply system, the valve motion should be controlled. Hydrants are now being so designed that they require a minimum number of full turns on their operating shafts to open or close their main valves.

Case d: Hydrant valve (at node [7]) in operation with the flow rate of 500, 1000 and 1500 US gpm, while valve operation duration remains unchanged (i.e., 10 seconds).

Figures 6.10, 6.11 and 6.12 show head variation at selected nodes [3], [4], [5] and [6] due to flow rates of 500, 1000 and 1500 US gpm.

It is clear from Figures 6.10, 6.11 and 6.12 that an increase in water withdrawal rate at a hydrant increase the difference between lowest pressure values experienced during this effect and final steady state pressure value, thus increasing transient effects. It also shows that increase in water withdrawal rate results in lowering the final steady state pressure value, thus making the system inadequate to supply NFF.
Figure 6.9: Head Variation at a Selected Node due to Variation in Hydrant Valve Opening Operation Time
With the present application, transient effects do not pose a greater threat than the adequacy of the system because of lowered pressures caused by excessive rate of water withdrawal. The transient effect can be controlled by installing special valves (modulating valves, pressure reducing valves, etc.) or by changing valve closure pattern. If the system needed to make adequate, it may be necessary to change pipeline size and or to increase pumping capacity etc. The changing of pipeline and or increasing pumping capacity may be expensive or even not feasible in the existing system.

Figure 6.10: Head Variation at Selected Nodes Where Flow Rate = 500 US gpm
Figure 6.11: Head Variation at Selected Nodes Where Flow Rate = 1000 US gpm

Figure 6.12: Head Variation at Selected Nodes Where Flow Rate = 1500 US gpm
Case e: Operation of multiple hydrants where valve opening time may or may not vary.

Figures 6.13, 6.14 and 6.15 show head variation at selected nodes [3], [4], [5] and [6] due to the operation of multiple hydrants simultaneously. Figures 6.14 and 6.15 show head variations when multiple hydrants (two in this case) are not opened concurrently. The alternate hydrant opens after the steady state condition prevails caused by the opening of the first hydrant.

It is evident from Figures 6.13, 6.14 and 6.15 that multiple hydrants operation poses a greater threat to the system due to transient effects and other effects due to greater water demand. Although the probability of simultaneous multiple fires is less. Multiple hydrants operation is necessary for simultaneous multiple fires but the probability of such a scenario is less.

Figure 6.13: Head Variation at Selected Nodes due to Operation of Multiple Hydrants Opened at Same Time
Figure 6.14: Head Variation at Selected Nodes due to Operation of Multiple Hydrants Opened at Different Times (hydrant at [6] starts after hydrant at [7] started)

Figure 6.15: Head Variation at Selected Nodes due to Operation of Multiple Hydrants Opened at Different Times (hydrant at [7] starts after hydrant at [6] started)
Figure 6.13 shows transient effects where multiple hydrants open at the same time, e.g., where two hydrants are joined through a manifold and open simultaneously. This situation may arise when there is a big fire in the area or filling a tanker for subsequent use. This situation poses greater transient effects when comparing with the operation of multiple hydrants at different times (Figures 6.14 and 6.15) while experiencing same excessive flow demand. Figures 6.14 and 6.15 also show that alternate opening of hydrants at different locations has different transient effects. The alternate opening of hydrants causes less effective transients since the response is divided into two parts compared with the simultaneous opening of hydrants (Figure 6.13).

The probability of multiple fires is low in this case because system represents a smaller area with less NFF customers. Although the chances of multiple fires in a smaller network area are low but are of concern; certainly a failure in this state could be devastating.

**Case f:** Transient effects as influenced by the water carrying capacity of the network by increase of pipeline size (throughout the network).

Figures 6.16(a) through 6.16(e) show head variation at node [6] due to variation in pipeline diameter while the operated hydrant is located at node [7]. Size of all the pipelines in the network are increased by 0, 10, 25, 50 and 100%. Hydrant operation pattern, normal demand and value of the fire flow remains unchanged. Node [6] is selected as observation point, because flow concentration is of concern and at node [6] there are three connecting pipelines. Figures B.4, B.5, B.6, B.7 and B.8, presented in Appendix B, show head variations at selected modes [3], [4], [5] and [6].

It is clear from Figures 6.16(a) through 6.16(e) that increasing of network's fluid flow capacity results in slight decreasing of the intensity of transient effects within the network. The transient effect is slowly subdued as compared with the increase in fluid flow capacity. The individual capacity of a pipeline is a multiple of diameter to the 2.63 power, Hazen-Williams equation. It is evident from the Figure 6.16(f) that the difference in final steady state head (prevailed after cease of transient effect) and the lowest pressure value (experienced during transient effect) changes slowly with the increase in pipeline diameter, while the difference in maximum head
Figure 6.16: Head Variation at a Selected Node due to Increase in Pipeline Size (Diameter)
(experienced during transient event and is equal to initial steady state pressure) and final steady state head changes rapidly.

Increase in water carrying capacity of the network will not make the system completely reliable, since potential for water hammer remains.

**Case g**: Transient effects as influenced by the pipeline material (wavespeed).

Figures 6.17(a) through 6.17(d) shows head variation at selected nodes [6] and [7], for a flow of 1000 US gpm and for a hydrant opening operation time 10 seconds, due to different pipe materials.

In Figure 6.17(a) and 6.17(b) the transient effect for a steel pipeline exceeds transient effect for plastic pipeline. In case of steel pipeline the net reduction of HGL exceeds that of the plastic pipeline, whereas the transient period (from start to cease) for steel pipeline is less compared to plastic pipeline. The HGL for steel pipeline shows many peaks before it reached lower limiting value. Figures 6.17(c) and 6.17(d) shows that limiting heads are not much varied with the increase in pipeline wavespeed.

Figures B.9 and B.10 shows head variation at selected nodes [6] and [7], for a flow of 1000 US gpm and for hydrant opening operation times of 1 and 5 seconds. Figures B.9, B.10 and 6.17 shows that transients decreased with the increase in valve operation time. Figures B.9(c) and B.9(d), for hydrant opening operation time of 1 second, shows that limiting heads are varied tremendously with the increase in pipeline wavespeed and this variation in nonlinear.

Pipe material has a significant impact on the hydraulic transients. The pressure wavespeed in concrete and steel is much higher than in PVC or polyethylene. With a lower wavespeed, the expected pressure rise is lower. Pressure waves are reflected at major discontinuities in the pipeline, including at exists and entrances, changes in diameter, and branches or tees. Reflected waves can cancel or coincide with subsequent waves and affect the pressure fluctuations in the pipeline.
Figure 6.17: Head Variation at Selected Node(s) Against Wavespeed (Valve Opening Operation Time = 10 Seconds)
It is clear that steel or concrete pipeline and minimal hydrant valve closure time have hazardous transients to the network. A hydrant operator (fireman) is less likely to consider pipeline material but is more likely to know pressure surges caused hydrant operation time (though important).

**Case h:** Effect of valve losses on limiting heads.

To account for loss of head through a valve, typically a valve discharge relation is included in the associated pipe that is connecting the boundary to the network. Figure 6.18(a) through 6.18(c) compare limiting heads with and without valve losses at all the valves discharging to atmosphere. There is no significant difference in limiting heads with or without considering valve (discharging to atmosphere) losses in the simple network.
Figure 6.18: Effect of Valve Losses on Limiting Heads
Chapter 7

Effect of Fire Fighting Supplies on Distribution Systems
Case Study of a Larger Network

Transient pressure surges due to fire flows within larger networks have different characteristics as compared with simple networks. This chapter discusses behavioral response caused by hydrant(s) operation within a larger network. Transient effects within a simple network are analyzed in chapter 6. The two examples of simple and larger networks for the evaluation of transients caused by fire flows are compared in chapter 8.

7.1 Network-2

A worked network example is selected to seek the transient effect that could occur within the network. An input file containing physical data of the system was created to use the program TransAM, and is presented for reference in Appendix A.

The network consists of 29 pipe segments and 20 nodes forming 11 loops, and two water supply sources (wells), as shown in Figure 7.1. The network includes two external pumps and one internal (booster) pump.
Figure 7.1: Example Network-2
7.1.1 Assumptions

- A wavespeed of 1100.0 ft/sec is assumed for all 29 pipelines.

- Pump stations data are not directly available. Each pump is associated with a dummy discharge valve, which remains open as long as the network is in working condition. The pump stations are described by some assumed appropriate values to rated discharge, rated head, rated speed, initial speed, rated efficiency, and inertia. These characteristic values are:

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Pump 1</th>
<th>Pump 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rated discharge, cfs</td>
<td>6.01</td>
<td>3.35</td>
</tr>
<tr>
<td>Rated head, ft</td>
<td>454.0</td>
<td>277.0</td>
</tr>
<tr>
<td>Rated speed, rpm</td>
<td>1000</td>
<td>1000</td>
</tr>
<tr>
<td>Initial speed, rpm</td>
<td>1000</td>
<td>1000</td>
</tr>
<tr>
<td>Efficiency</td>
<td>0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>Rotor inertia, lbf-ft²</td>
<td>700.0</td>
<td>700</td>
</tr>
</tbody>
</table>

- At node [7] there is a valve within the hydrant, which can discharge to atmosphere. The network was supplying demand flow under steady state conditions, meanwhile the valve starts discharging a flow of 1000 US gpm (2.228 cfs).

- The valve takes 10 seconds to open fully with a linear change in effective gate area.

- Some of the graphs showing transient pressures in this thesis are not strictly realistic since the negative pressures appeared are sometimes far below the vapor pressure of the fluid. Moreover, little attention has been paid to the issue of pipe strength or rating. Clearly, the large negative pressures are not physically possible. However, this does not directly take such results away from their utility. The transient simulations were being forced to run under these conditions so the results between different simulations in these hypothetical systems can be easily compared. Thus, the unrealistic pressure values are simply intended to identify the relative intensity of transients between different systems on a common basis.
7.1.2 Simulation Scenarios

The following scenarios are considered to simulate a fire flow from a hydrant:

- **Case a**: The hydrant valve at node [7] starts discharging a flow of 1000 US gpm (2.228 cfs).
- **Case b**: Hydrant valve (at node [7]) in operation with valve opening time of 0.0001, 5, 15 and 35 seconds, while hydrant flow remains unchanged (i.e., 1000 US gpm).
- **Case c**: Hydrant valve (at node [7]) in operation with the flow rate of 500, 1000, 2000 and 2500 US gpm, while valve operation duration remains unchanged (i.e., 10 seconds).
- **Case d**: Operation of multiple hydrants where valve opening time may or may not vary.
- **Case e**: Operation of a hydrant located near a booster pump.
- **Case f**: Effect of valve losses on limiting heads.

**Case a**: The hydrant valve at node [7] starts discharging a flow of 1000 US gpm (2.228 cfs).

Figure 7.2 shows three sections of the system through pipelines (8)-(7)-(6)-(28), (11)-(12)-(13)-(27) and (8)-(25)-(29)-(20) profiling limiting heads due to transition effects. Figure 7.3 shows head variations at selected nodes [18], [8], [7] and [16]. These sections and nodes of the system are in accordance with Figure 7.1.

Figure 7.3 indicates that there are negligible transient effects at selected nodes in the system. The lowest pressure experienced is the final steady pressure prevailed after valve opening, and acted as control point in this case.

It is clear that in Figure 7.2 the upper bound (maximum pressure experienced during operation) is equal to steady state pressure head before opening a hydrant at node [7] and the lower bound (minimum pressure experienced during operation) is equal to final steady state pressure head prevailed after opening the hydrant. Figure 7.2 shows no negative pressure in the pipelines (6), (7), (8), (11), (12), (13), (20), (25), (27), (28) and (29). The system works perfectly except at the suction side of the booster pump combining the pipelines (25) and (29), where the lower bound
Figure 7.2: Pipe and Head Profile – Hydrant at Node [7] in Operation
approaches the pipeline elevation. At node [18], at the end of pipeline (27), there is a pump contributing water supply to the network and the change in pressure is not abnormal, Figure 7.2(b). Neither transient effect nor negative pressure is observed within the system.

The system behaves adequately and reliably in this case, since the system is relatively large, properly designed and well looped, and is not imposing higher water demand.

**Figure 7.3:** Head Variation at Selected Nodes due to Operation of a Hydrant at Node [7]

*Case b:* Hydrant valve (at node [7]) in operation with valve opening time of 0.0001, 5, 15 and 35 seconds, while hydrant flow remains unchanged (i.e., 1000 US gpm).

Figures 7.4 and 7.5 show head variation at selected nodes [18], [7], [8] and [16] when valve
opening operation times are 0 and 10 seconds. Figures C.1, C.2, C.3 and C.4 are presented in Appendix C and show head variations for 0.0001, 5, 15, and 35 seconds of valve opening operation time.

It is evident from Figures 7.4 and 7.5 that the opening pattern of valve at node [7] has not attained any importance against transient effects, even if the valve opens quickly. Transient pressure peak values are within steady state pressures before and after operation of the valve at node [7].

It is clear from Figures 7.4 and 7.5 that the system is reliable in this case, since transient effect is negligible for the opening pattern of the valve at node [7]. This behavior of valve operation pattern is contrary as compared with network-1 where opening and closing time of the valve can significantly harm water supply system (Figures 6.9, B.1, B.2 and B.3).

![Graph: Head Variation at Selected Nodes due to Hydrant(s) Operation](image)

Figure 7.4: Head Variation at Selected Nodes Where Hydrant Valve Opening Operation Time = 0.0001 Seconds
**Case c:** Hydrant valve (at node [7]) in operation with the flow rate of 500, 1000, 2000 and 2500 US gpm, while valve operation duration remains unchanged (i.e., 10 seconds).

Figures 7.6, 7.7, 7.8, 7.9 and 7.10 show head variation at selected nodes [18], [7], [8] and [16] due to flow rates of 500, 1000, 1500, 2000 and 2500 US gpm.

It is clear from Figures 7.6, 7.7, 7.8, 7.9 and 7.10 that increase in water withdrawal at a hydrant decrease pressure value (final steady state pressure in this case), thus decreasing adequacy to supply NFF. Figure 7.11 shows head variation at node [7] due to the variation of fire flow rate. The upper pressure (initial steady state pressure) remains unchanged while lower water pressure (final steady state pressure in this case) tends to be linear with the increase in withdrawal flow rate.
Figure 7.6: Head Variation at Selected Nodes Where Flow Rate = 500 US gpm

Figure 7.7: Head Variation at Selected Nodes Where Flow Rate = 1000 US gpm
Figure 7.8: Head Variation at Selected Nodes Where
Flow Rate = 1500 US gpm

Figure 7.9: Head Variation at Selected Nodes Where
Flow Rate = 2000 US gpm
Figure 7.10: Head Variation at Selected Nodes Where Flow Rate = 2500 US gpm

Figure 7.11: Head Variation at a Selected Node [7] due to Variation of Fire Flow Rate
Figure 7.12 shows three sections of the system through pipelines (8)-(7)-(6)-(28), (11)-(12)-(13)-(27) and (8)-(25)-(29)-(20) profiling limiting heads due to fire flow of 1500 US gpm at node [7]. It is evident that at fire flow of 1500 US gpm lower bound approach pipelines (6), (7), (8), (20), (25), (28) and (29), hence entire system cannot work as expected when the flow rate exceed 1500 US gpm.

The transient effect does not pose a greater threat rather the adequacy of the system because of lowered pressure caused by excessive amount of water withdrawal rate. The transient effect does not act as a controlling factor. If the system needs to be improved, it may be necessary to change the pipeline size and or to increase pumping capacity etc. Changing of pipeline and or increasing pumping capacity may sometimes be expensive or practically difficult.

**Case d:** Operation of multiple hydrants where valve opening time may or may not vary.

Figures 7.13 through 7.16 show head variation at selected nodes [18], [7], [8] and [16] due to the operation of multiple hydrants simultaneously. Figures 7.14, 7.15 and 7.16 show head variations when multiple hydrants (two in this case) are not open at the same time. The alternate hydrant opens after the steady state condition prevails caused by the opening of the first hydrant.

It is evident from Figures 7.13 through 7.16 that multiple hydrant operation poses greater threat to the system due to lowering of pressure head rather than transient effects because of effective value of greater water demand. It is also clear from Figures 7.13, 7.14 and 7.15 that multiple hydrants operation poses almost the same resultant effect when located closely irrespective of the valves operation pattern. When comparing Figures 7.15 and 7.16 it is evident that operation of multiple hydrants located apart results in a little less resultant effect. In Figure 7.16 hydrants in operation are different than those in Figure 7.15, but the flow rate (1000 US gpm from each hydrant) and observation points (Nodes [18], [7], [8], and [16]) are the same.

Figure 7.13 shows transient effects where multiple hydrants open at the same time, e.g., where two hydrants are joined through a manifold and open simultaneously. This situation of opening multiple hydrants may arise when there is a big fire in the area or filling a tanker for subsequent use.
Figure 7.12: Pipe and Head Profile for Flow of 1500 US gpm at Node [7]
Figure 7.13: Head Variation at Selected Nodes due to Operation of Multiple Hydrants Opened at Same Time

Figure 7.14: Head Variation at Selected Nodes due to Operation of Multiple Hydrants Opened at Different Times (hydrant at [7] starts after hydrant at [8] started)
Figure 7.15: Head Variation at Selected Nodes due to Operation of Multiple Hydrants Opened at Different Times (hydrant at [8] starts after hydrant at [7] started)

Figure 7.16: Head Variation at Selected Nodes due to Operation of Multiple Hydrants Spacing Apart
Network-2 covers greater area with more number of NFF customers, and thus the chances of simultaneous multiple fires are relatively greater as compared with network-1. Multiple hydrants operation is necessary for simultaneous multiple fires but the overall probability of such a scenario is less.

**Case e: Operation of a hydrant located near a booster pump.**

Figure 7.17 presents pipe and head profiles of a hydrant operation located at different places near a booster pump. Figure 7.17(a) through 7.17(c) shows a section of the system through pipelines (8), (25), (29), and (20) due to fire flow of 1000 US gpm at nodes [7] and [8], and at the midpoint of pipeline (25). It is evident from Figure 7.17(a) (hydrant at node [7]) that the lower bound approaches first at the part of pipeline that is at suction side of the booster pump before any other part of the pipeline (8)-(25)-(29)-(20). It is clear from Figures 7.17(b) that the lower bound crosses pipeline as hydrant at node [8] is in operation (located at junction nearest to the suction side of booster pump). Figure 7.17(c) (hydrant at mid point of the suction line (25)) tells that crossing of pipeline by lower bound is a little faster as compared with the case when hydrant at node [8] operates (Figure 7.17(b)).

It is noted in Figure 7.1 that pipeline (25)-(29) containing the booster pump joins two junctions that combine two or more pipes and the size of the pipeline is relatively large (12 inches). The introduction of vacuum pressure into the suction line can be avoided with an alternate design as shown in the following Figure (a), but it will increase the possibility of excessive higher pressure in the delivery side as shown in Figure (b).
Figure 7.17: Pipe and Head Profile for Different Locations of an Operating Hydrant Concerning Booster Pump
Operation of a hydrant located near a booster pump can increase the likelihood of cavitation, since pump might attempt to boost more water than the system can support (in inadequate system).

When pressure decreases, the boiling point of water is reduced which in turn increase the ability to vaporize, thus allowing water to vaporize and form vapor pockets. Within the pump (if centrifugal) two pressure zones called high-pressure zone and low-pressure zone are created. When there are vapor pockets, high-pressure zone forces the vapor pockets back into the liquid state. This forward and backward operation results in increased friction loss and decreases efficiency and can cause extensive pump damage. The damage can be so significant that it may cause the pump to cease to operate; thus an inadequate system can also be an unreliable one.

The cavitation within the water main can combine many small vapor pockets and forces the water column to separate. The rejoining of this separated water column may create excessive pressure and can burst the pipeline. Excessive withdrawal of water from the system creates negative pressure and can cause even low-pressure break, or high-pressure break due to rejoining of separated water columns. Thus an inadequate system can also be an unreliable one although there are no transient effects.

Case f: Effect of valve losses on limiting heads.

Figures 7.18(a) through 7.18(c) compare limiting heads with and without valve losses that were considered at all the valves discharging to atmosphere. Unlike the simple network (Figure 6.18), the large network shows a significant change in limiting heads. It is clear that the inclusion of valve losses produces a less hazardous effect from the limiting heads. It also shows that it is usually conservative to neglect valve losses.
Figure 7.18: Effect of Valve Losses on Limiting Heads
This chapter presents overall conclusions and summary of this study. Two example networks (a simple network and a larger network) were analyzed for behavioral response caused by hydrant(s) operation and are presented in chapters 6 and 7. This chapter also compares and analyzes results of case studies and illustrates the application of transient theory to a distribution system supplying water for fire flows. The results presented in this chapter are not based on elaborated analysis of a fair number of networks.

8.1 Summary of Case Studies

The present study analyzed two example systems and evaluated transient pressure response caused by hydrant operation. Transient pressure surges are initiated by the operation of a hydrant and by the development of an abnormal water demand for fire fighting. The simulation determines the flow rate that can be safely achieved at a specific hydrant. The program TransAM was successfully used to simulate example networks.

Rate of Water Delivery: Abnormal transient effects start when the water delivery rate exceeds a certain amount of flow. When the delivery rate at the hydrant is within that flow rate, pressure transients are not to exceed the safe limits of the system. The simulation determines the flow rate that can be safely achieved at a specific hydrant.

In the present study, transient effects increase with the decrease in network size. The large network under study is less likely to undergo abnormal transient surges when increase in withdrawal rate at a hydrant decreases pressure from initial to the final steady state. The increase in flow rates sometimes lower the pressure to a negative value within the network and may limit
satisfaction of the needed fire flow (NFF). It is also observed that the upper pressure (initial steady state pressure) often remains unchanged while lower water pressure (final steady state pressure) tends to be linear with the increase in withdrawal flow rate. Transient effects seldom pose a greater threat compared to the adequacy of the system because of lowered pressures caused by large rates of withdrawal. The transient effect may not act as a controlling factor in larger networks contrary to the simple network (network-1) where transient surges are often the culprits.

**Location of an Operating Hydrant:** The location of an operating hydrant affects the system response. The position of different operating hydrants within the network has different transient effects. In the present study, transient effects increase with the decrease in network size.

Some parts experience more transient surges than others, e.g., a pipeline joining a constant head reservoir is safer than the suction and delivery lines of a booster pump. A hydrant at a dead end poses greater transient effect as compared with a hydrant at looped section. The pressure changes in well looped and relatively large distribution system are often low but may still be of concern.

**Opening Operation Time and Closing Operation Time of Hydrant Valves:** When a hydrant valve opens to discharge water, the transient pressure drops and as the valve closes the transient pressure in the pipeline increases, often rapidly. Quick opening of a valve may result in negative pressures within the pipeline while sudden closing of a hydrant valve can cause pressure which exceed the pipeline rating. In some ways, the opening of a hydrant valve poses a more elaborate requirements on the system than closing of a hydrant because opening of a valve results in imposing water demand on the system and thus thrusts more load on pumping, mains, filtration, and water reserves; closing a hydrant primarily stresses the conduit itself. The larger systems tend to experience smaller effects of valve operation compared with the simple pipe systems.

It is concluded that hydrant operation period has significant effect on the system. Thus, a hydrant valve should not be open suddenly nor should it be closed rapidly. Slow operation of a hydrant main valve can reduce the likelihood and extent of water hammer problems in many ways.
**Multiple Fires:** The chances of multiple fires are low but are of concern. In the case of a larger system (network-2) the effect could be abnormal and is due to lowering of pressure head without undergoing transient surges. This effect was kept almost equal for different cases including hydrants operating at the same time and at different times, and when the location of operating hydrants varies. This behavior may not be similar for very large networks with many supplying sources. The smaller network (network-1) shows considerable surge variation when subject to multiple hydrant operations.

**Material of Pipeline:** Pipe material has a significant impact on the hydraulic transients associated with hydrant actions. The pressure wavespeed in concrete and steel is much higher than in PVC or polyethylene. With a lower wavespeed in a pipeline (PVC or polyethylene), the pressure rise is reduced. Pressure waves are reflected at major discontinuities in the pipeline, including at exists and entrances, changes in diameter, and branches or tees. Reflected waves can cancel or coincide with subsequent waves and affect the pressure fluctuations in the pipeline. It is clear that steel or concrete pipeline and minimal hydrant valve closure time may introduce hazardous transients to the network as compared with a hydrant on PVC or polyethylene pipeline network. A hydrant operator (fireman) is less likely to consider pipeline material but is more likely to know pressure surges caused by hydrant operation time (though important).

**Head Losses Associated with Valves:** Head loss associated with valves can reduce the transients. A simple network (network-1 with few valves) does not show significant changes in limiting heads but a larger network (network-2 with many valves) shows considerable changes in limiting heads. The difference between maximum and minimum pressures experienced is less in larger networks as compared with simple networks. Head losses help to cease transients.
8.2 Limitations

There are some limitations in this study;

Firstly, although some exceptions are considered, it is generally assumed that outflow from a hydrant is 1000 US gpm (63 l/s) with linear valve opening operation lasting for 10 seconds.

Secondly, simulation calculations assume that the current supplies from the system remain unchanged when fire flow demand is imposed on the system. This is possible when all other water outlets are equipped with flow rate controllers and agitators but this does not happen. When hydrant is opened to deliver NFF the other supplies from the system are also reduced because of pressure reduction within the network. The reduction in supply may be very small at individual outlets. This assumption also shows that the estimated transient effect is more than the actual one.

Thirdly, the calculations are aimed at transient effects upon distribution system with hydrant(s) at chosen point(s) delivering fire flow. It is difficult to estimate whether or not this particular point in time is really representative. It is also difficult (or at least tedious) to estimate that the imposed NFF is a maximum or a minimum or perhaps an intermediate value.

Fourthly, the hydrant is modeled as an externally discharging valve but this does not happen in every fire fighting operation. In many situations there is a fire engine equipped with the hydrant by a suction hose and fire engine boosts water (if required) and work as flow rate controlling equipment. The withdrawn water is forced to pass through a nozzle, attached at the end of delivery hose, discharging in the form of high velocity water stream or spray. The assumption of fixed flow-discharging valve holds well in this case, since hydrant delivers controlled flow, and the losses are covered by the fire pump. The applicability of this assumption is justified as long as the hydrant acts as a pressure source, but the assumption may not hold when pressure in the pipeline becomes negative while the pumper continues with drafting operation.

Fifthly, a hose line bears greater elasticity than the distribution lines and is attached externally to the hydrant at the time of fire fighting. In case when a hose line containing a stream or spray nozzle at the other end is equipped with hydrant, and after the opening of hydrant main valve the control is shifted from hydrant valve to the nozzle. It may be helpful to figure out pressure-surges...
in the distribution system by the operation of this nozzle. If this approach is adopted it will be required to model the hydrant operation (boundary condition) accordingly.

Lastly, it was not verified that the pumps used within the distribution systems were working at their rated capacity, and the available pressure is not reduced. Inflow from water sources through pumps is not quantified against availability. Although it is assumed that the system components are working properly, there is a possibility of cavitation due to pressure zoning that may adversely effect the system components.

8.3 Other Conclusions and Summary

Water is the most important component of a fire protection system. Water has a remarkable ability to absorb heat and a certain quantity of water must be provided for fire fighting. There are many methods to evaluate fire flow requirements for fire protection. The Insurance Services Office (ISO) developed guidelines for insurance grading purposes that considers building construction, occupancy, adjacent exposed buildings, and communication paths between buildings. The ISO method generally provides a good estimate for specific structures when evaluating fire flows. The ISO guidelines are not intended to address fire flow needs when designing water supply systems for newly developing municipal areas. For planning purposes other guidelines and predictions from experience should also be considered in the evaluation of needed fire flow (NFF).

A dependable water system of adequate capacity for all sections in a community is necessary. The arrangement of the supply works, pumping facilities, water demands, etc could affect adequacy and reliability of the supply system. Adequate supplies of water are measured in terms of pressure and flow. Reliability is a measure of consistency of service. Hydraulic, structural, and water quality performance measures are necessary to be considered for evaluating dependability. The structural performance measures include service interruptions, mains breaks, and hydrants and valves operability. Water quality performance deals with physical and chemical parameters such as turbidity, chlorine residuals or trihalomethanes. The reliability framework of water quality
performance needs specific variables across the network over an extended period of time. A transport model is necessary to estimate the parameter values.

The role of computer simulation and optimization is beneficial to evaluate a distribution system. Optimization techniques are created to find a combination of decision variables which minimizes an objective function subject to constraints. Although various approaches for optimum design and operation of a water distribution system have been reported in the literature, the present optimization models are also criticized for the lack of practical and general applicability. Diversity of design and operation situations makes it difficult to produce an all-purpose optimal design package for water distribution systems. Some problems can be handled by a linear programming, others require dynamic programming or complex search techniques like genetic algorithms.

Transient conditions in water supply distribution networks may occur due to changes in the flow demand, opening or closing of fire hydrants, starting or stopping of pumps, changes in the setting of control equipment or the failure of a system component. The analysis of these conditions is necessary for the design and operation of water supply systems for a community.

For simulation purposes the TransAM program is selected because it can perform dynamic simulation of fluid distribution networks comprising many components such as pipes, valves, pumps, surge tanks, and junctions. The program calculates time-varying flows, pressures, and heads throughout the network.

This study does not support findings of earlier workers that when there is sufficient quantity of water available for domestic use there is usually sufficient water to provide useful supply for fire fighting. Water demand for fire fighting does not have any direct relation to the size and kind of distribution system supplying water. A small town can have a big fire and big fires make unusual demands on water system. Smaller systems are able to deliver only a fraction of water that may be needed for fire fighting. The major cause that hampers the ability of the small systems to provide adequate and reliable water supply is primarily due to the transient effects caused by sudden loading of abnormal water demand. These findings might hold well for a large distribution system, since larger systems pose less threat against adequacy and reliability, but the
chances of inadequacy remains due to reduction of fluid pressure caused by abnormal water demand for fire fighting.

Service interruptions are mainly the result of structural problems in a distribution system. Chances of developing a negative pressure within the network might result in the collapse of a pipeline or other system component.

The reliability of a distribution system may be dependent on hydrant operating conditions. During hydrant operations, the transient pressure can be much higher than the pipe rating and on the other hand, transient pressure may fall to a value as low as the vapor pressure of the water. Such circumstances can lead to pipe rupture or pipe collapse.

Overall, fire flow requirements are an important part of the loading of a water distribution system. Under both steady state and transient conditions, fire fighting loads can create large and important forces on the distribution network. Thus, these hydraulic demands need to be accounted for in rational distribution system design, operation, analysis and optimization.
References


Appendix A

Sample Data File

* The network includes two externals and one booster pump. Pump station data are not available.
* It is assumed that each pump is associated with a dummy discharge valve, which remains open * as long as the network is in working condition.
* To simulate a fire flow from a hydrant it is assumed that at node [7] there is a valve, which can * discharge to atmosphere. The network was supplying demand flow under steady state * conditions. Meanwhile, the valve starts discharging a flow of 1000 US gpm (2.228 cfs).
* A wavespeed of 1100.0 ft/sec is assumed for all 29 pipelines.

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| NLBC (node list) |
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| TDUR  0.0
| other PUMP INFO
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| 5.8373  INITIAL DISCHARGE
| 454.0  RATED HEAD
| 1000  RATED SPEED
| 1000  INITIAL SPEED
| .8000  RATED EFFICIENCY
| 700  INERTIA (WR2:lb-ft^2)

| END OF PUMP STATION DATA - PUMP-1

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| 0  0  0  1  3  3
| TAU CURVE POINTS
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| PUMP DATA
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| 277.0  RATED HEAD
| 1000  RATED SPEED
| 1000  INITIAL SPEED
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152.0 RATED HEAD
1000 RATED SPEED
1000 INITIAL SPEED
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700 INERTIA (WR2:lb-ft^2)

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<th>QWTF</th>
<th>QV1</th>
<th>QV2</th>
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**SET0 SET1 SET2 NSQ NENSQ MMQ**

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**Q CURVE POINTS**

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<th>PUMP STATION VALVES</th>
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<table>
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<th>VALVE LIST (by ordinal input number)</th>
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</tr>
<tr>
<td>1       l</td>
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<td>1       l</td>
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| End of Data File |
Appendix B

Figures Network-1

Figure B.1: Head Variation at Selected Nodes Where Hydrant Valve Opening Operation Time = 0.0001 Seconds
Figure B.2: Head Variation at Selected Nodes Where Hydrant Valve Opening Operation Time = 5.0 Seconds

Figure B.3: Head Variation at Selected Nodes Where Hydrant Valve Opening Operation Time = 10 Seconds
Figure B.4: Head Variation at Selected Nodes due to Increase in Pipelines Diameter by 0 %

Figure B.5: Head Variation at Selected Nodes due to Increase in Pipelines Diameter by 10 %
Figure B.6: Head Variation at Selected Nodes due to Increase in Pipelines Diameter by 25%.

Figure B.7: Head Variation at Selected Nodes due to Increase in Pipelines Diameter by 50%.
Figure B.8: Head Variation at Selected Nodes due to Increase in Pipelines Diameter by 100%
Figure B.9: Head Variation at Selected Node(s) Against Wavespeed
(Valve Opening Operation Time = 1 Second)
Figure B.10: Head Variation at Selected Node(s) Against Wavespeed (Valve Opening Operation Time = 5 Seconds)
Appendix C

Figures Network-2

Figure C.1: Head Variation at Selected Nodes Where Hydrant Valve Opening
Operation Time = 0.0001 Seconds
**Figure C.2:** Head Variation at Selected Nodes Where Hydrant Valve Opening Operation Time = 5.0 Seconds

**Figure C.3:** Head Variation at Selected Nodes Where Hydrant Valve Opening Operation Time = 15.0 Seconds
Figure C.4: Head Variation at Selected Nodes Where Hydrant Valve Opening Operation Time = 35.0 Seconds
Appendix D

List of Symbols and Abbreviations

\[ A \] Pipe cross-sectional area;
\[ A \] Area of fire (structure) in square meters;
\[ A \] Total floor area in a building being considered (square feet);
\[ A, A_t \] Effective area;
\[ AWWA \] American Water Works Association;
\[ a \] Acoustic wave speed;
\[ B \] A parameter;
\[ B_M \] Pipe characteristic constant correspond to characteristic line \( C' \);
\[ B_P \] Pipe characteristic constant correspond to characteristic line \( C'' \);
\[ C, C_i \] Construction factor;
\[ C^*, C' \] Characteristics lines;
\[ CAD \] Computer aided drafting;
\[ CFUA \] Canadian Fire Underwriters Association;
\[ CIS \] Credit for inspection and condition;
\[ CIC \] Customer information systems;
\[ C_M \] Pipe characteristic constant correspond to characteristic line \( C' \);
\[ C_P \] Pipe characteristic constant correspond to characteristic line \( C'' \);
\[ d \] A small quantity or incremental change;
\[ D \] Inner diameter of pipe;
\[ EM \] Emergency supply capacity;
\[ EMS \] Energy management systems;
\[ F \] Coefficient related to the class of construction;
\[ f \] Darcy-Weisbach friction factor;
\[ FDS \] Fire department supply;
\[ FL \] Capacity of treatment facilities;
\[ FUS \] Fire Underwriters Surrey;
\[ G \] Fire flow; volumetric flow rate;
\[ g \] Gravitational acceleration;
**GA**
Genetic algorithms;

**GIS**
Geographic information systems;

**gpm**
US gallons per minute;

**GRG**
Generalized reduced gradient;

**H**
Piezometric head; hydraulic gradient level (HGL);

**H**
Instantaneous head; \( H = H(x,t) \);

**HF**
Hydrant condition;

**HI**
Hydrant inspection;

**\( H_P \)**
Elevation of valve or leak orifice above datum;

**\( H_P \)**
Piezometric head at node \( P \) correspond to \( x \)-t grid, at time \( t+\Delta t \);

**\( H_x \)**
Derivative of instantaneous head with respect to \( x \); \( dH/dx \);

**\( H_{x} \)**
Derivative of instantaneous head with respect to \( x \); \( dH/dx \);

**i**
Subscript; index for interruption periods;

**i**
Subscript; index of set \( n \) (number of sides of subject structure); \( i = 1, 2, \ldots, n \);

**\( i, j, k \)**
Subscripts; index of pipes in sets \( N_1, N_2 \) and \( N_1+N_2 \) respectively;

**IFSTA**
International Fire Service Training Association;

**IIT**
Illinois Institute of Technology;

**ISO**
Insurance Services Office;

**ISU**
Iowa State University;

**\( J \)**
Number of jets (streams);

**K**
Water supply coefficient;

**\( KPa \)**
Kilo Pascal; Kilo Newton per square meter;

**LIMS**
Laboratory information management systems;

**l/min**
Liters per minute;

**l/s**
Liters per second;

**m**
Meter;

**MMS**
Maintenance management systems;

**MS**
Minimum storage;

**n**
Number of sides of subject building;

**\( N_1 \)**
Set of pipes discharging towards the node;

**\( N_2 \)**
Set of pipes discharging away from the node;

**NBFU**
National Board of Fire Underwriters;

**NFF, NFF_i**
Needed fire flow;

**NFPA**
National Fire Protection Association;
\[ N_{out,i} \] Customer equivalents affected by interruption period \( i \);
\[ N_{req} \] Total number of customer equivalents;
\[ O_i \] Occupancy factor;
\[ P \] Population in thousands;
\[ P \] Pressure head;
\[ P, P_i \] Communication factor;
\[ P_{min} \] Minimum pressure head; minimum required pressure under maximum daily demand and fire flows or minimum required pressure under normal operating conditions;
\[ P_{si} \] Pounds per square inch;
\[ PU \] Pumping system capacity;
\[ Q \] Fire flow; volumetric flow rate;
\[ Q_A \] Rate of discharge at node \( A \) correspond to \( x-t \) grid, at time \( t \);
\[ Q_B \] Rate of discharge at node \( B \) correspond to \( x-t \) grid, at time \( t \);
\[ Q_{exi} \] Known external flow;
\[ Q_{max} \] Maximum fire flow;
\[ Q_{min} \] Minimum fire flow;
\[ Q_p \] Rate of discharge at node \( P \) correspond to \( x-t \) grid, at time \( t+\Delta t \);
\[ Q_{req} \] Required flow rate; greater of either, maximum daily consumption plus fire demand or maximum hourly consumption;
\[ q \] Volume of water in liters;
\[ R \] A factor; \( f(2DA) \);
\[ RDMBS \] Relational data base management systems;
\[ SCADA \] Supervisory control and data acquisition systems;
\[ SFPE \] Society of Fire Protection Engineers;
\[ SS \] Supplemental suction supply;
\[ S_{side1}, S_{side2}, S_{side n} \] Special coefficients from property line exposed to sides 1, 2 and \( n \) respectively;
\[ S_{total} \] Total of spatial coefficient values from property line exposed on all sides;
\[ SWC \] Supply works capacity;
\[ t \] Time;
\[ T_c \] Fire control time; time from the start of fire fighting operation to the time when control is established (excluding final extinguishment and overhaul time);
\[ T_{out,i} \] Duration of interruption period \( i \);
\[ T_{req} \] Required period of service;
\( V \)  
Volume of space in cubic feet;

\( V \)  
Total volume of building in cubic meters;

\( V \)  
Instantaneous average velocity of flowing fluid; \( V(x,t) \);

\( V_t \)  
Derivative of instantaneous average velocity with respect to \( t \); \( dV/dt \);

\( V_x \)  
Derivative of instantaneous average velocity with respect to \( x \); \( dV/dx \);

\( W \)  
Fire flow in liters per second;

\( W \)  
Quantity of water in US gallons;

\( x \)  
Distance along the centerline of the conduit;

\( X, X_i \)  
Exposure factor;

\( \Delta \)  
An incremental change;

\( \Delta t \)  
Time step correspond to \( x-t \) grid;

\( \Delta x \)  
Pipe reach length;

\( \partial() \)  
Partial fraction of \( () \);

\( \varepsilon \)  
Belong to;

\( \lambda \)  
A multiplying factor;

\( \Sigma \)  
Summation;

\( \varepsilon \)  
Friction term; linearization constant.