1-2009

Utilizing Available Residual Pressure in Fujairah Lines in Bypassing Al Ain Reception Station Considering Transients and Water Hammer Conditions

Hazem Bakri Al Naser

United Arab Emirates University
Deanship of Graduate Studies
M.Sc. Program in Water Resources

Theses Title:

Utilizing Available Residual Pressure in Fujairah Lines in Bypassing Al Ain Reception Station Considering Transients and Water Hammer Conditions

Author's Name: Hazem Bakri Al Naser

Supervisor: Dr. Walid El Shorbagy
Associate Professor
Department of Civil & Environmental Engineering
Faculty of Engineering, UAE University

United Arab Emirates University

January 2009

UTILIZING AVAILABLE RESIDUAL PRESSURE IN FUJAIRAH LINES IN BYPASSING AL AIN RECEPTION STATION CONSIDERING TRANSIENTS AND WATER HAMMER CONDITIONS

A Thesis submitted to the
Deanship of Graduate Studies
United Arab Emirates University

In Partial Fulfilment of the Requirements for
M.Sc. Degree in Water Resources

Examination Committee **Signature**

Dr. Walid Elshorbagy, Chair
Civil and Environmental Engineering Department
College of Engineering, UAE University
Al Ain, UAE

Dr. James Uber, External Examiner
Dept. of Civil & Environmental Engr
University of Cincinnati
736 Engineering Research Center
Cincinnati, Ohio 45221-0071

Prof. Mohsen Sherif, Internal Examiner
Civil and Environmental Engineering Department
College of Engineering, UAE University
Al Ain, UAE

Mohsen Sherif

United Arab Emirates University
January 2009

ACKNOWLEDGMENT

I wish to express my love and gratitude to my father, mother, brother and sister; for their endless support, through the duration of my studies.

This research project would not have been possible without the support of many people.

I wish to express my gratitude to my supervisor, Dr. Walid Al Shorbagy who was abundantly helpful and offered assistance, support and guidance.

Deepest gratitude is also due to Eng. Mohamed Sultan Al Mamari (TRANSCO) without whose assistance this study would not have been successful.

ABSTRACT

The objective of this study is to utilize the available residual pressure at Fujairah Transmission lines in order to save energy, operation and maintenance costs, and still deliver the required water to the end users of the distribution network. This has been achieved by bypassing the existing pumping stations (i.e. AARS Reservoir and pumping stations in addition to the local reservoirs and pumping stations of each area). The possibility of transients, surge, and cavitations occurrence was studied via a comprehensive transient simulation to make sure that the proposed bypassing will not have any impact on the upstream transmission system by any operating conditions of the distribution system. A number of water demand scenarios in each zone of the city, in which the available energy is usually wasted under the current operating conditions, were studied in the proposed bypass.

Suitable solutions were proposed such as Pressure Relieve Valves and Air Valves to eliminate or at least to minimize unacceptable performances. In addition, the remaining residual pressure in the line was ensured to be sufficient to deliver water to the end users in the studied areas. Finally, the annual savings including energy operation and maintenance costs was estimated to be AED 4.06 Million (about 1.11 USD) while the capital cost of the proposed bypass was estimated to be AED 6.6 Million (1.81 USD). Such saving is associated with about 1 year and 8 month's payback period.

This study will help in reducing the cost of potable water delivery to end consumers in Al Ain region by reducing operation costs, maintenance costs, spare parts, manpower, etc... through utilizing available residual pressure in the transmission lines. In addition, this study will save power consumption that can be utilized in other fields or alternatively reduce power generation. This may have a positive impact on our environment by reducing the amount of CO_2 generated during power production. Existing pumps that will be decommissioned can be utilized elsewhere and thus saves the cost of purchasing new pumps. Finally, this study will improve the potable water quality by reducing or even eliminating the stagnant time of water in the storage tanks. This is in line with the current national efforts trying to achieve full integration of the transmission with the downstream distribution systems.

List of Symbols

Σ	: Summation
Q	: Flow rate in pipe
Ha & Hb	: The total energy at nodes
Kl	: loss equation coefficient
IMGD	: Imperial Million Gallon per Day
n	: The exponent from the head loss equation
AARS	: Al Ain Reception Station
p	: Pressure (N/m^2, lb/ft^2)
γ	: Specific weight (N/m^3, lb/ft^3)
z	: Elevation at the centroid (m, ft)
V	: Velocity (m/s, ft/sec.)
g	: Gravitational acceleration constant (m/s^2, $ft/sec.^2$)
h_p	: Head gain from a pump (m, ft)
h_L	: Combined headless (m, ft)
HGL	: Hydraulic Grade Lines
EL	: Energy Lines
WL	: Water level (m, ft)
Qp^t	: The flow in pipe p at time t where the pipe p is connected to the tank
AT	: Tank area
EPS	: Extended Period Simulation
ρ	: Mass density of water
x	: Distance along the pipe centreline
H	: Pressure head (pressure/density)
f	: Friction factor
τ	: Shear stress at the pipe wall
a	: Acoustic wave speed
Ec	: Young's Modulus (Pa, PSI)
μ_p	: Poisson's Ratio,
Ev	: Bulk Modulus of Elasticity (Pa, PSI)
Tm	: Time of manoeuvre
t	: Time
L	: Length

D.I.	: Ductile Iron
FBE	: Fusion-banded epoxy
TRANSCO	: Transmission Company
AADC	: Al Ain Distribution Company
PRV	: Pressure Relieve Valve
AV	: Air Valves
Ri	: Inflow Resistance
Ro	: Outflow Resistance
FCV	: Flow Control Valve
PSV	: Pressure Sustaining valve

Table of Contents

List of Figures

List of Tables

CHAPTER I

INTRODUCTION

1.1 General Background

In the United Arab Emirates (UAE), Desalination plants are considered the main source of potable water. Main demands for water include domestic, forestry and agricultural uses. Water demand has historically grown at a slightly faster rate than electricity demand. The system-constrained supply of water from generation and desalination plants has increased from 182.9 Million Imperial Gallon per Day (MIGD) in 1998 to 483 MIGD in 2006. The total annual water supply from generation and desalination plants has increased from 66,772 MIG in 1998 to 176,457 MIG in 2006. Water is transmitted through main pipelines of 600mm to 1600mm diameter and pumping stations. Distribution to customers is carried out through main pipelines of less than 600mm diameter and, in some remote areas, through road tankers. Figure 1.1shows the water capacity versus production in UAE whereas Figure 1.2 shows water production and transmission facilities in UAE in 2006 [1].

Due to the low selling cost (close to zero) of desalinated water in the Emirate of Abu Dhabi, people used to consume the costly, high quality desalinated water in fields that require lower water quality such as irrigation, cleaning, gardening, car wash, etc... This results in high water consumption per capita (174 gallon/capita/day) in the UAE, which is considered one of the highest water consumption rates per capita in the world.

In order to secure sustainable fresh water, several strategic plans were set to supply fresh water under the pressure of water shortage. Al Fujairah-AlAin project (via twin 1600mm diameter) and Showaihat project (via twin 1200mm diameter) are two of these strategic plans used as new alternatives for Al-Ain region. Al Fujairah – Al Ain project consists of several Tap Offs connected to these lines (around 15 Tap Offs) to supply the demands of different areas along the lines.

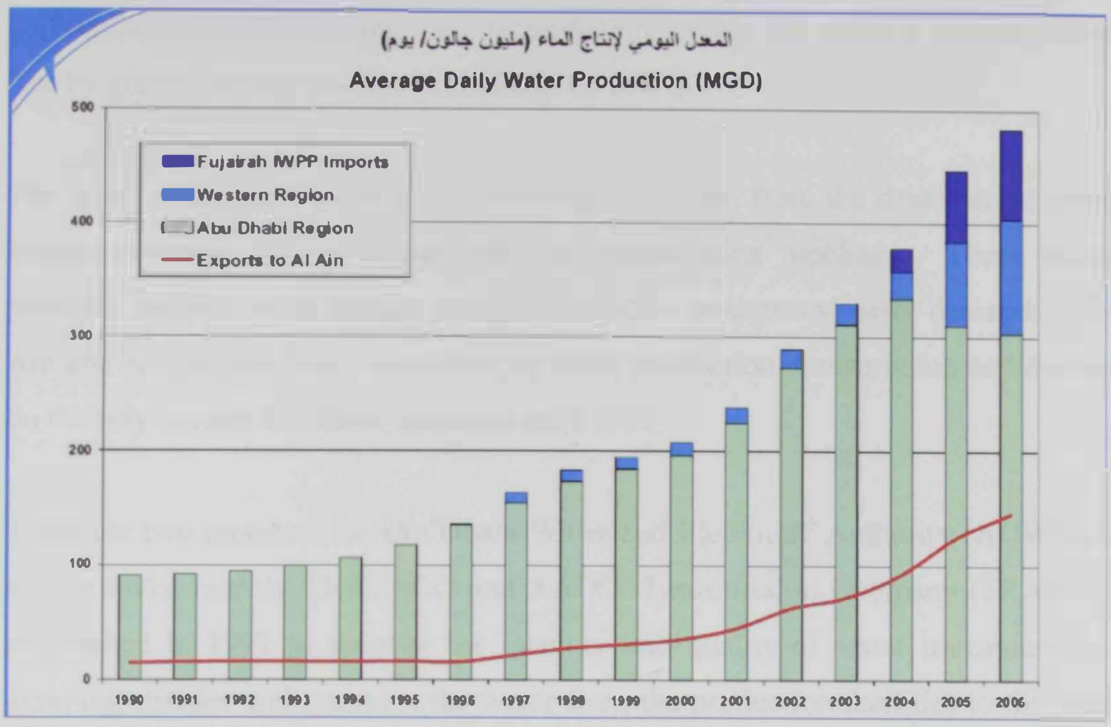


Figure 1.1 Water capacities versus production in UAE [1]

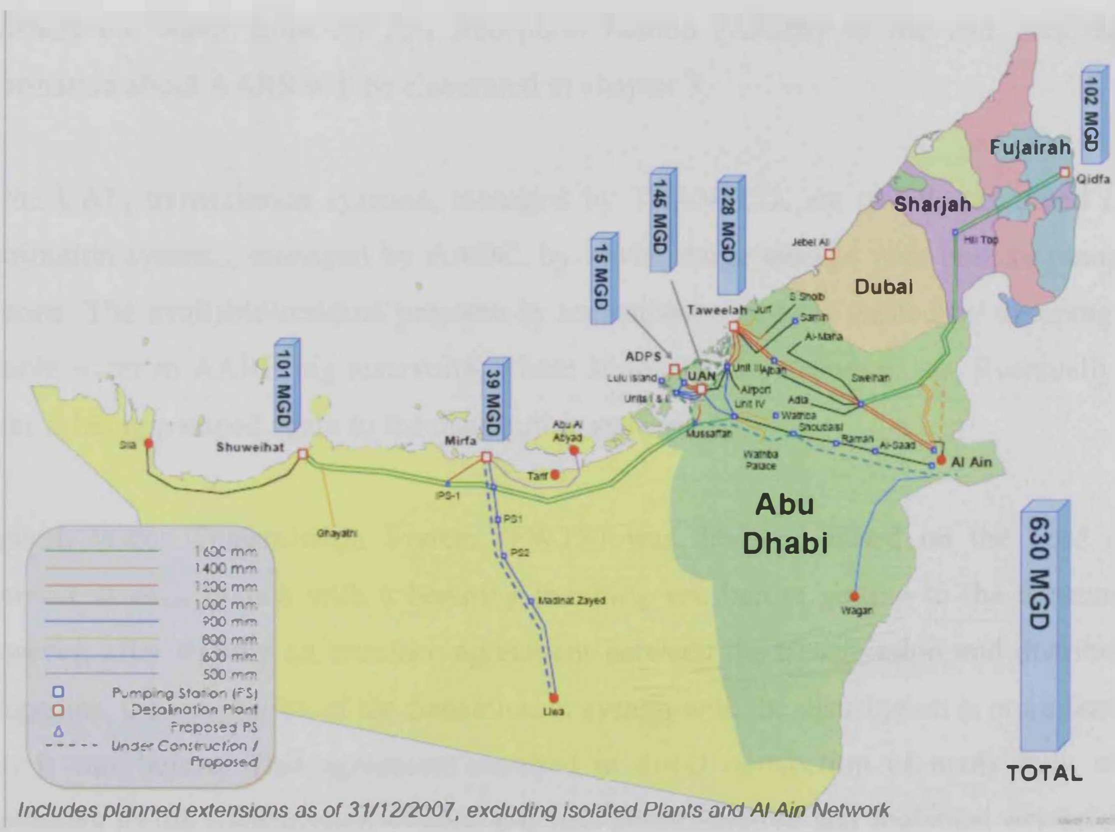


Figure 1.2: Water production and transmission facilities in 2006 in UAE [1]

The supplied water in these lines is produced from a desalination plant in Al-Fujairah then pumped to the top of nearby mountains, where an intermediate reservoir exists, through three working pumps and one standby, with 6,314 m^3/hr flow and 502-meter water column (mWC) head each. Following the intermediate reservoir, the water is provided through the line by gravity having average pressure of 65 mWC.

The water is supplied to far areas, reaching to 300km, from the desalination plants using booster-pumping stations along with the transmission pipelines. These transmission pipelines include water storage tanks to cover the emergency water demands in both Al-Ain and Abu Dhabi. More expansion on water production, transmission and distribution is on the way to cater the future demands until 2020.

There are two members in Abu Dhabi Water and Electricity Authority (ADWEA) related to this study, namely TRANSCO and AADC. Transmission Company (TRANSCO) was established in 1999 to monitor the quantity and quality of water transmission, operate pumping stations and transport the water from the production facilities to the distribution networks in Al-Ain Distribution Company (AADC). AADC was established to monitor the quantity and quality of the water distribution, operate distribution-pumping stations and transport the water from Al Ain Reception Station (AARS) to the end user. More information about AARS will be elaborated in chapter 3.

In the UAE, transmission systems, managed by TRANSCO, are usually separated from distribution systems, managed by AADC, by having large storage with booster pumping stations. The available residual pressure in transmission lines is wasted by dumping the potable water in AARC big reservoirs, where atmospheric pressure exists. Eventually the water is being pumped again to the distribution systems.

Fujairah Water Transmission System (FWTS) was designed based on the need of a reservoir at each branch with a boosting pumping set, before getting to the consumers. However, after signing an interface agreement between the transmission and distribution companies, the integration of the transmission system with the distribution is more feasible than it was before. This agreement resulted in direct connection of many bulk water consumers to the transmission system and less administrative and logistical obstacles. It also mandates certain criteria to be met for direct consumers' connections before operating them. At the interface point, special installations are needed such as double isolation valves, flow control valves, certified and regularly calibrated flow meters, pressure elements and water quality measuring elements for chlorine residual, PH, and conductivity.

1.2 Problem Statement, Work Principle and Objectives

The existing system contains two main systems (Transmission and Distribution). Transmission lines deliver the potable water from main desalination plant in Fujairah to AARS with arrival residual pressure of about 6.5 bars where the scope of work of TRANSCO ends and the scope of work of Al Ain Distribution Company (AADC) starts. The pressure is then killed when the water is dumped in AARS reservoirs. After that, the water is pumped again to the zone distribution pumping station with about 3 bars where the pressure is killed again and the water is dumped in their reservoirs. Finally, the water is pumped again with 3 bars to the distribution systems. Figure 1.3 depicts the existing system.

The possibility of transients, surge, and cavitations occurrence are studied via a comprehensive transient simulation to make sure that the proposed bypassing will not have any impact on the upstream transmission system by any operating conditions of the distribution system. A number of water demand scenarios in each zone of the city, in which the available energy is usually wasted under the current operating conditions, are studied in the proposed bypass. Suitable solutions are explained to eliminate or at least minimize unacceptable performances. In addition, it is required to ensure that the remaining residual pressure in the line is enough to deliver water to the end users in the studied areas. Finally, the amount of the saved energy is determined which will reflect the amount of energy and money saved taking into consideration the bypassing costs.

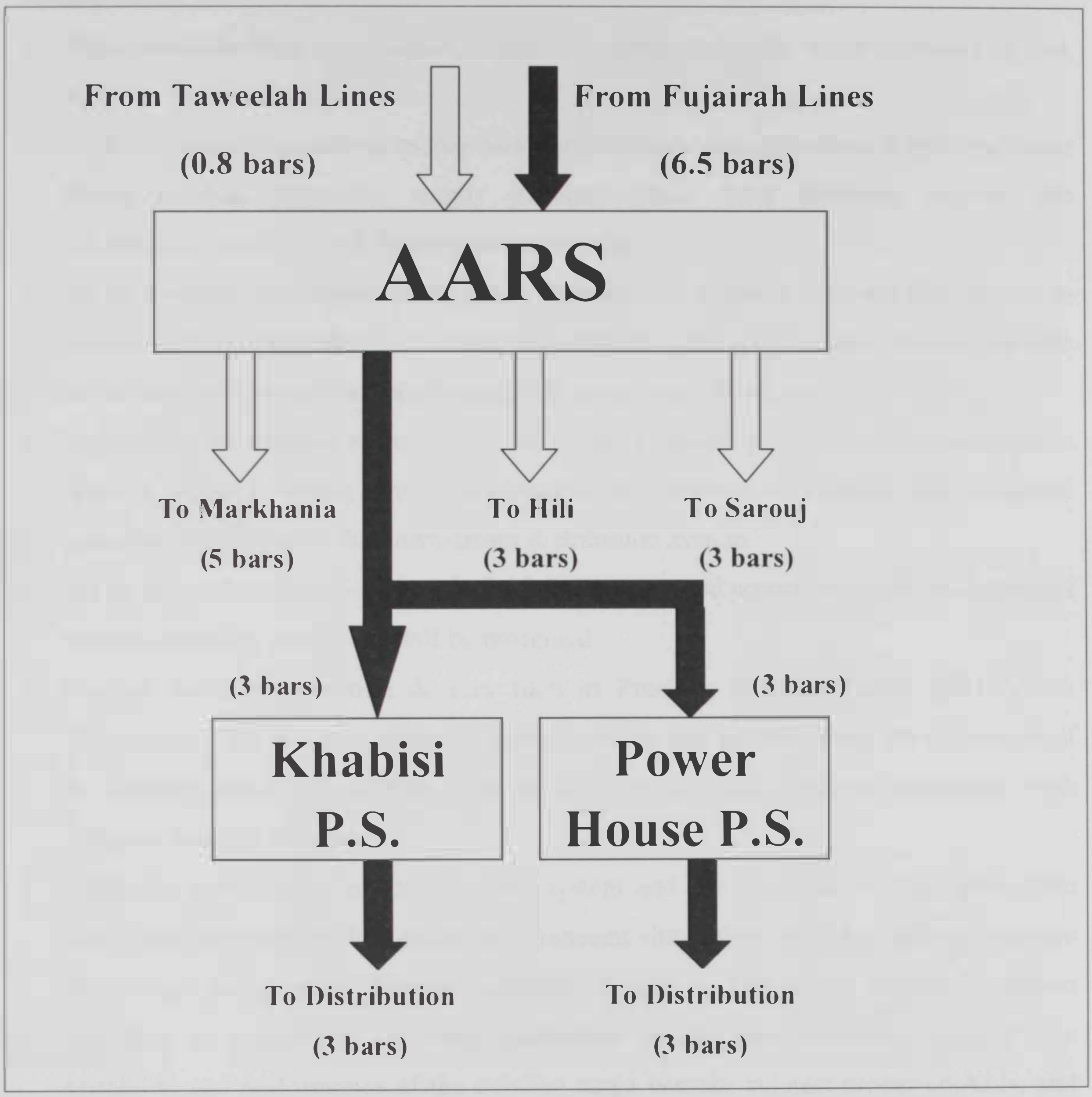


Figure 1.3: Existing system

1.3 Methodology

In order to achieve the objectives, several tasks are required and were carried out as summarized:

1. Understand the existing situation, supply scenarios, controls, water networks layout, etc…
2. Collect actual design and operating data such as pumps data, pipelines diameters, water flows, residual pressures, supply patterns, etc… from different sources like TRANSCO, AADC, field, or other element party.
3. Set up hydraulic simulation model using specialized simulation software (InfoWater) to simulate the existing situations using the collected data and compare the results with actual data such as residual pressure at each area, water flows, etc…
4. Study different demand scenarios for which the available pressure in the transmission lines is utilized, taking into consideration the impacts of normal and abnormal operation conditions of the downstream distribution system.
5. Set up hydraulic simulation models for the new proposed scenarios where the impact of several operation conditions will be presented.
6. Propose suitable protection devices such as Pressure Relieve Valves (PRVs), Air Valves, etc… for the new proposed system, taking into consideration the utilization of the existing protection devices with suitable settings and locations associated with different demand patterns.
7. Study the performance of the proposed system and the potential of surge generation due to the new changes by setting up a transient simulation using specialized software (InfoSurge) considering different operations scenarios. The output of this simulation will help in addressing the surge generation in the pipe networks, evaluate the suitability and performance of the existing surge vessels, suggest proper controls, and provide basis for recommendation of possible modifications when needed.
8. Estimate the power saving, operation costs, and maintenance costs in addition to the capital costs associated with new modifications.
9. Summarize all results, outcomes, recommendations, conclusion and suggestions, and provide a basic feasibility study for future detailed designs.

1.4 Study Impacts

This study will help in reducing the cost of potable water delivery to end consumers in Al Ain region by reducing operation costs, maintenance costs, spare parts, manpower, etc... through utilizing available residual pressure in the transmission lines. In addition, this study will save power consumption which can be utilized in other fields or alternatively reduce power generation. This may have a positive impact on the environment by reducing the amount of CO_2 generated during power production. Existing pumps that will be decommissioned can be utilized elsewhere and thus saves the cost of purchasing new pumps. Finally, the study will improve the potable water quality by reducing or even eliminating the stagnant time of water in the storage tanks. This can be achieved by providing new system that directly connects the transmission and the distribution systems.

1.5 Thesis Contents

This document includes six other chapters. Chapter 2 presents theoretical background and related case studies of hydraulics and transients. In addition, chapter two addresses some case studies of applied transients modelling in pressurized water network. At the end of this chapter, the simulation software used in this study (InfoWater and InfoSurge) is described. In chapter 3, a general overview of the existing situation is presented, and detailed information of AARS and related areas in particular are discussed. This includes demand patterns for each area, existing pumps and control fittings, and operational conditions. Chapter 4 presents setup the hydraulic simulation models including inputs; introduces the study of possible bypassing scenarios taking into account the potable water requirement in each area, and finally proposes the optimum bypass scenario. In addition, a simulation of current system using InfoWater software and model verification is elaborated. In Chapter 5, performance of the proposed bypass scenario is evaluated using InfoWater and InfoSurge software under different operation conditions (i.e. normal and abnormal operation conditions). Chapter 6 presents a cost analysis of bypass proposal with estimated payback period. Chapter 7 includes summery of tasks, findings, cost estimates, conclusions and recommendations of the study.

CHAPTER II

THEORETICAL BACKGROUND

Applying principles that are well-established and easy-to-understand result in designing a pipeline to operate under steady-state conditions. Placing the line into service might not be safe if no enough design provisions and operational procedures are provided. If the pipe friction, minor losses, or aging factors are not exact, this leads to the fact that the actual flow is just slightly different from the calculated flow. Engineers usually face serious problems with pipelines because of cavitation or unsteady flow conditions that occur either during filling the line, while making intentional changes or changes caused by power failure to pumps, accidental opening or closing of a valve, etc.

The most critical time in the life of the pipe, especially long lines over hilly terrain, can be the initial filling, pressurizing, and flushing out of the air. During filling operations, transient pressure generated can easily exceed the safe operating range of the pipe. It is more complicated to analyze a system to Figure out the type and magnitudes of possible hydraulic transients than to perform the steady-state calculations. A huge number of pipelines results into failure because some practicing engineers have not had adequate exposure to the analysis of unsteady flow topic, as it is a relatively advanced topic. Problems related to unsteady flows might be neglected [2].

In this chapter, theoretical aspects related to the hydraulic simulation of pressurized water networks are elaborated. This includes the following simulation conditions:

- Steady state conditions
- Gradual unsteady flow conditions
- Rapid unsteady flow conditions (Hydraulic Transients)

Following the above coverage, a number of case studies of applied transients modelling in pressurized water network are discussed then a brief introduction about the software packages utilized in this study is presented.

2.1 Steady State Conditions Simulation

When the flow and pressure do not change with time or maintain a state of relative equilibrium even after undergoing fluctuations or transformations, flow is considered steady. When the hydraulic conditions have reached equilibrium, steady flow hydraulic modelling can provide a snapshot of the conditions in water networks.

Below are the main equations used in simulating a steady state flow in pipe networks. These equations are:

- Continuity Equation (Conservation of Mass)
- Conservation of Energy
    - Energy Equation with Pumps
- System of Equations

2.1.1 Continuity Equation (Conservation of Mass)

A Continuity Equation is a differential equation that describes the conservative transport of some kind of quantity (water flow in our case). Equation 2.1 represents the continuity equation at a node where the total flow entering a node is equal to the total flow leaving the same node.

$\Sigma Qin = \Sigma Qout$ at a node (2.1)

Where:

Qin is the inlet flow rate

Qout is the outlet flow rate

2.1.2 Conservation of Energy

The low of conservation of energy states that the total amount of energy in any isolated system remains constant but cannot be recreated, although it may change forms, e.g. friction turns kinetic energy into thermal energy. Equation 2.2 describes the conservation of energy low.

$dz/dt = (Q_{in} - Q_{out})/A$ (2.2)

where

A is horizontal surface area of the reservoir z is the elevation of water surface in the reservoir above specific datum

t is time

Q_{in} is inflow into the reservoir

Q_{out} is outflow from the reservoir

Energy Equation in Case of Pumps

A pump head might be added to the pressure head, static head, and velocity head. The energy can be obtained in form of Bernoulli equation for steady-state flow by balancing the energy between two points (example points 1 and 2) in the system. Equation 2.3 describes the Bernoulli equation.

$$P_1/\gamma + z_1 + V_1^2/2g + hp = P_2/\gamma + z_2 + V_2^2/2g + h_L \quad (2.3)$$

Where:

p = Pressure (N/m^2, lb/ft^2)

γ = Specific weight (N/m^3, lb/ft^3)

z = Elevation at the centroid (m, ft)

V = Velocity (m/s, ft/sec.)

g = Gravitational acceleration constant (m/s^2, $ft/sec.^2$)

h_p = Head gain from a pump (m, ft)

h_L = Combined headless (m, ft)

Hydraulic Grade Lines (HGL) and Energy Lines (EL) are graphical forms of Bernoulli equation. Hydraulic Grade Line represents the piezometric head, or the sum of the elevation head (z), and the pressure head (p/γ). While Energy Line represents the total energy, (head) available to the fluid, which includes Hydraulic Grade and the velocity head ($V_2^2/2g$). Energy Line is the height to which, a column of water would rise in a pitot tube. At big reservoirs, where the velocity is essentially zero, the EGL is equal to the HGL, as can be seen in Figure 2.1and Figure 2.2.

2.1.3 System of Equations

System equation can be developed by applying the continuity equation at all nodes, and the energy equation at all pipes. System equation is developed and solved using suitable numerical techniques in order to determine either unknown nodal pressure or unknown pipe flows.

2.2 Gradual Unsteady Flow Conditions

Due to demand changes during the day, unsteady state conditions of water flows occur. Using Extended Period Simulation (EPS) approach (step-wise (quasi-steady state) dynamic simulation), such conditions can be modelled.

In order to solve an EPS, the following information is required:

- Operational controls
- Nodal demands as a function of time
- Physical description of the tank

EPS involves a series of steady state simulations with tank levels being updated between steady state analyses. Tank water level (WL) at any time (t) is described as follows:

$$WL_{t+\Delta t} = WL_t \pm (Qp^t \times \Delta t) / AT \qquad (2.4)$$

Where:

Qp^t is the flow in pipe p at time t where the pipe p is connected to the tank

AT is the tank area.

The positive sign is used in case of flow (Qp^t) is entering the tank; while the negative sign is used in case of flow; (Qp^t) leaving the tank. Although an additional relationship describing changes in tank levels is required for EPS, simultaneous quasi-linear equations can be solved for pipe flows and nodal heads for steady state and EPS. The time step between tank level changes in an EPS is typically hourly increment, which is acceptable under most normal operating conditions.

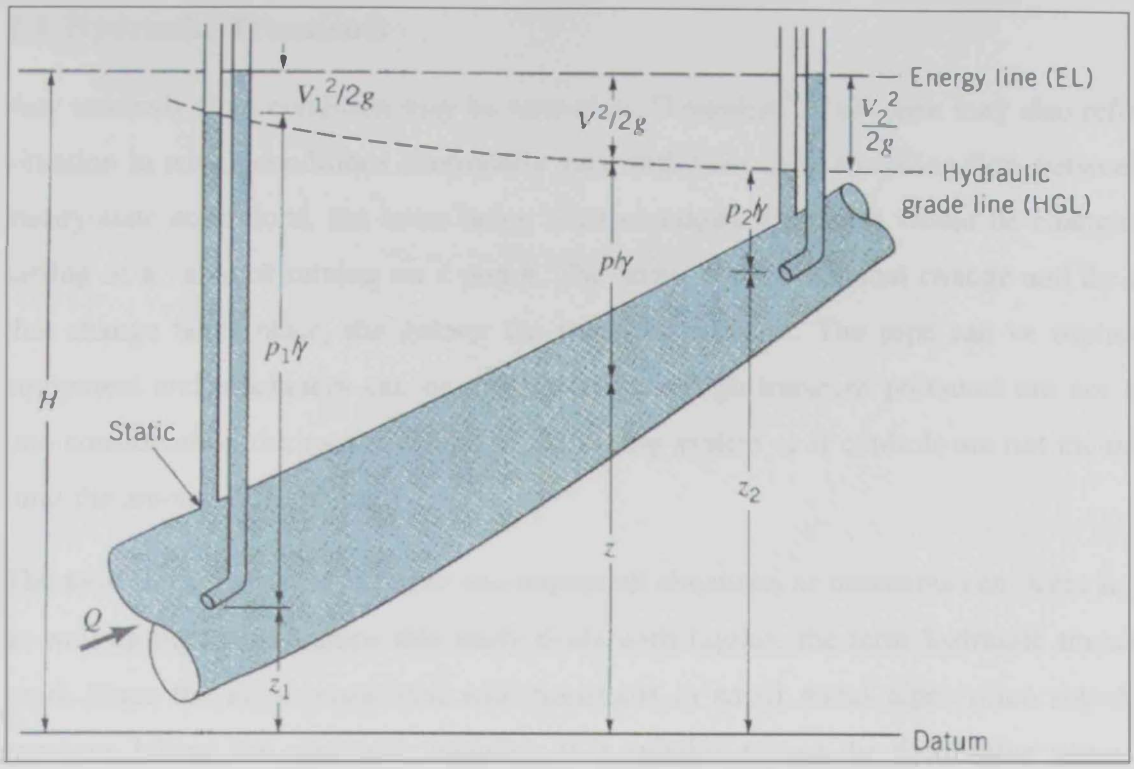


Figure 2.1: HGL and EL with pitot tube considering frictionless system [14]

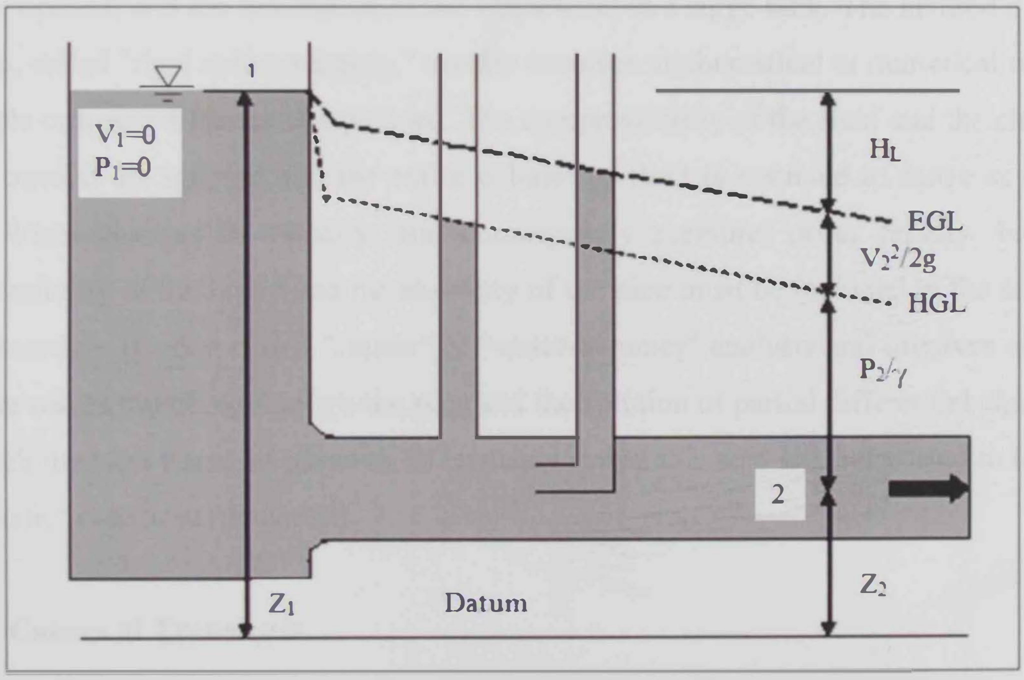


Figure 2.2: HGL and EL with pitot tube considering friction system

2.3 Hydraulic Transients

Any unsteady flow condition may be termed as "Transient". This term may also refer to a situation in which conditions continually vary with time or to transition flow between two steady-state conditions, the latter being most common. Examples would be changing the setting of a valve or turning on a pump. The larger the incremental change and the faster that change takes place, the greater the transient pressure. The pipe can be ruptured or equipment and machinery can be damaged, if the high transient pressures are not taking into consideration during the design of the piping system or if controls are not included to limit the amount of head rise.

The term fluid transient is used to encompass all situations as transients can occur in gases as well as liquids and since this study deals with liquids, the term hydraulic transient is used. Since the noise associated with transients in small metal pipes often sounds like someone hitting the pipe with hammer, it is suitable to use the descriptive name when water is referred to.

The term "surges" is used to refer to the transients which involve changes and that occur slowly. Examples to that would be an oscillating U-tube, establishment of flow after a valve is opened, and the fluctuation of the water level in a surge tank. The method of surge analysis, called "rigid column theory," usually involves mathematical or numerical solution of simple ordinary differential equations. The compressibility of the fluid and the elasticity of the conduit are ignored and the entire column of fluid is assumed to move as a rigid body. When changes in velocity, and consequently pressure, occur rapidly, both the compressibility of the liquid and the elasticity of the pipe must be included in the analysis. This procedure is often called "elastic" or "water-hammer" analysis and involves acoustic pressure waves travelling through the pipe and the solution of partial differential equations. Although the term transient refers to all unsteady flows, it is also generally used to identify the "elastic" case in particular [2].

2.3.1 Causes of Transients

Water hammer conditions can be generated due to many events. Some of the most common events are:

1. Improper filling, flushing, or removal of air from pipelines
2. Starting or stopping pumps.

3. Change of valve opening.
4. Operation of check valves, air-release valves, pressure-reducing valves, and pressure-relief valves.
5. Trapped air in pipelines
6. Change in power demand of hydraulic turbines
7. Pipe rupture

Transients occur in all pipelines and depend on its magnitude and the ability of the pipes to tolerate high pressures without damage whether the transient creates operational problems or pipe failure. For instance, an un-reinforced concrete pipeline may have a transient pressure allowance of only a few feet its operating pressure before damage can occur. For such situations, even slow closing of control valves or minor interruptions of flow due to any cause may create sufficient transient pressures to rupture the pipeline. On the contrary, steel pipelines can usually take relatively high transient pressures without failure. Therefore, it is important that every pipe system should have at least a brief transient analysis carried out to identify the possibility of serious transients and decide if a detailed analysis is required [2].

2.3.2 Equation of Motion

The equation of motion governs the unsteady flow. The summation of all forces acting on a mass of fluid in a given direction is equal to the product of the mass and the acceleration in the direction of the force. Figure 2.3 is a free-body diagram showing forces acting on a cylindrical segment of fluid. The forces include pressure forces acting on the two ends, friction or shear forces on the outer surface, and gravity. The equation of motion applied in the direction of the fluid motion x is

$$PA - \left(P + \frac{\partial P}{\partial x} \Delta x \right) A - \tau \, \pi \, D \Delta x + \rho \, g \, A \, \Delta x \, \sin \theta = \rho \, A \, \Delta x \frac{DV}{Dt} \tag{2.5}$$

Simplifying and dividing by Δx gives:

$$-\frac{\partial P}{\partial x} A - \tau \, \pi \, D + \rho \, g \, A \, \sin \theta = \rho \, A \frac{DV}{Dt} \tag{2.6}$$

It is normally assumed that the wall shear stress for unsteady flow is equal to the steady-state value at the same velocity. It is customary to work with a friction loss expressed as a loss of head over a reach of pipe rather than dealing with a distributed wall shear stress. By applying the equation of motion to Figure 2.3 this relationship can be developed for steady-state flow for the simplified case of θ = 0.

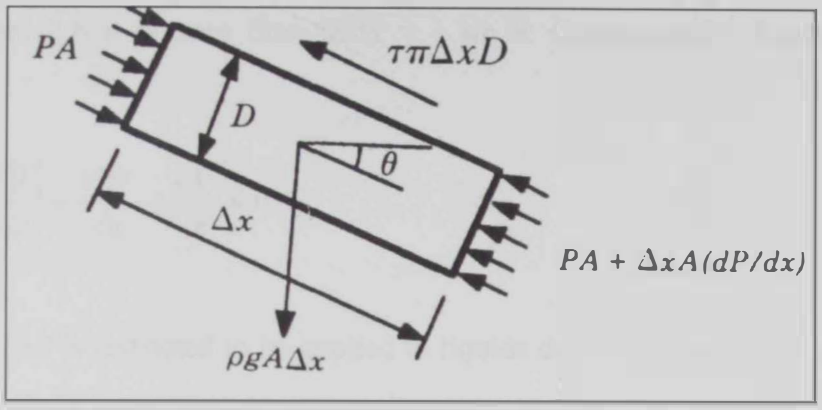


Figure 2.3: Free body diagram of fluid section [2]

$$(P_1 - P_2)A = \tau \pi D \Delta x = \gamma (H_1 - H_2)A \tag{2.7}$$

From the Darcy—Weisbach equation:

$$H_1 - H_2 = \frac{fLV^2}{2gD} = \frac{f\Delta x V|V|}{2gD} \tag{2.8}$$

in which f is the friction factor. The absolute value sign on V is to ensure that the friction force is always opposite to the direction of flow. Substituting Equation 2.8 into 2.7 and rearranging the shear stress term in Equation 2.6 is expressed as:

$$\tau \pi D = \frac{\rho f V|V| A}{2D} \tag{2.9}$$

Dividing Equation 2.6 by ρA, substituting in Equation 2.9 and expanding the total derivative of V, we get:

$$\frac{\partial P}{\rho \partial x} + \frac{fV|V|}{2D} - g \sin \theta + \frac{\partial V}{\partial x}\frac{dx}{dt} + \frac{\partial V}{\partial t} = 0 \tag{2.10}$$

This can be further simplified by using the piezometric head H= P/γ + z. The differentiation gives

$$\frac{\partial H}{\partial x} = \frac{\partial P}{\gamma \partial x} + \frac{dz}{dx} \qquad (2.11)$$

From Figure 2.6 it is seen that dz/dx = - sin θ. Consequently, Equation 2.10 can be reduced to

$$g\frac{\partial H}{\partial x} + \frac{fV|V|}{2D} + \frac{V\partial V}{\partial x} + \frac{\partial V}{\partial t} = 0 \qquad (2.12)$$

Equation 2.12 is restricted to be applied to liquids due to the use of the piezometric head. Therefore, it is important to remember not to use the pressure head for further computations and the piezometric head must be used in this case [2].

2.3.3 Equation of Continuity

It is stated in the law of conservation of mass applied to a control volume that the net mass flux through the control surface and the time rate of change of mass inside the control volume must be equal. When applying this to the present situation, it is noted that the control volume (in this case a short section of pipe) can increase in cross-sectional area and length due to increased pressure. It is assumed that adding length to the pipe is negligible for the development that follows. Hence, the continuity equation turns out to be

$$\rho AV - \left[\rho AV + \frac{\partial(\rho AV)}{\partial x}dx\right] = \frac{\partial(\rho A dx)}{\partial t}$$

or $$-\frac{\partial(\rho AV)}{\partial x}dx = \frac{\partial(\rho A dx)}{\partial t} \qquad (2.13)$$

Expanding both sides of Equation 2.13,

$$-\left(\rho A\frac{\partial V}{\partial x} + \rho V\frac{\partial A}{\partial x}dx + AV\frac{\partial \rho}{\partial x}dx\right) = \rho dx\frac{\partial A}{\partial t} + Adx\frac{\partial \rho}{\partial t} \qquad (2.14)$$

Rearranging and dividing by pAdx,

$$\frac{1}{A}\left(\frac{\partial A}{\partial t}+V\frac{\partial A}{\partial x}\right)+\frac{1}{\rho}\left(\frac{\partial \rho}{\partial t}+V\frac{\partial V}{\partial x}\right)+\frac{\partial V}{\partial x}=0 \tag{2.15}$$

The first two terms in parentheses are the total derivatives of A and p with respect to time, so the equation reduces to

$$\frac{1}{A}\frac{dA}{dt}+\frac{1}{\rho}\frac{d\rho}{dt}+\frac{\partial V}{\partial x}=0 \tag{2.16}$$

Next, replace dA/dt with structural properties of the pipe. Expressing the wave speed equation in differential form and rearranging,

$$dA=\frac{AD}{e\,E}dP=\frac{\rho\,g\,A\,D\,dH}{e\,E} \tag{2.17}$$

The first term of equation 2.16 then becomes

$$\frac{\rho\,g\,D}{e\,E}\frac{dH}{dt}$$

The second term can be expressed in terms of the bulk modulus and dH using the definition

$$K=\frac{dP}{d\rho/\rho}\ or\ \frac{d\rho}{\rho}=\rho g\frac{dH}{K}$$

The second term in equation 2.16 becomes

$$\frac{\rho g}{K}\frac{dH}{dt}$$

Substituting in to Equation 2.16 and rearranging,

$$\frac{dH}{dt}\left(\frac{1+KD/eE}{K/\rho}\right)+\frac{1}{g}\frac{\partial V}{\partial x}=0 \tag{2.18}$$

Using for wave speed, Equation 2.16 reduces to

$$\frac{dH}{dt}+\frac{a^2}{g}\frac{\partial V}{\partial x}=0 \tag{2.19}$$

This is the final form of the continuity equation, which will be solved together with the equation of motion (Equation 2.13) since they provide two equations in two unknowns H and V. The method of characteristics is the technique to be used to transform the partial differential equations into total differentials [2].

2.3.4 Method of Characteristics

The momentum and continuity equations are further interpolated and expressed in form of two independent total differential equations:

$$\frac{g}{a}\frac{dH}{dt}+\frac{dV}{dt}+\frac{fV|V|}{2D}=0 \qquad C^{+}\text{ equation} \tag{2.20}$$

$$for\,\frac{dx}{dt}=+a \tag{2.21}$$

$$\frac{g}{a}\frac{dH}{dt}-\frac{dV}{dt}-\frac{fV|V|}{2D}=0 \qquad C^{-}\text{ equation} \tag{2.22}$$

$$for\,\frac{dx}{dt}=-a \tag{2.23}$$

Equations 2.21 and 2.24 are called the C+ and C- compatibility equations.

This is a popular method of solving hyperbolic equations. It entails converting the two partial differential equations to ordinary differential equations then solving using an explicit finite difference method. One drawback of the method of characteristics is that the time step must be small to satisfy the Courant condition for stability [3].

It is important to mention that only the method of characteristics explicitly links the time step to the space step, giving this fixed rigid approach. The main drawback of the method of characteristics is that the time step used in the solution must be common (fixed) to all pipes. In addition, the method of characteristics requires the distance step in each pipe to be a fixed multiple of common time interval, further complicating the solution procedure. In practice, pipes tend to have arbitrary lengths and it is seldom possible to exactly satisfy both time-interval and distance-step criteria. This "discretization problem" requires the use of either interpolation procedures (which have undesirable numerical properties) or distortions of the physical problem (which introduces an error of unknown magnitude).

Finally, in order to satisfy stability criteria and ensure convergence, the method of characteristics requires a small time step. The stability criterion is developed by neglecting the nonlinear friction term and is referred to as the Courant condition. The Courant condition relates the computational time increment (Δt) to the spatial grid size (Δx). A numerical scheme is stable if and only if $|\Delta x| \geq a\,|\Delta t|$, in which a is the wave speed. In other words, the Courant condition requires that the numerical distance a wave propagates $|\Delta x|$ must exceed the physical propagation distance $a\,|\Delta t|$ [4].

This can be concluded in the following equations:

$$C^{+} : HP_I = CP - B\ QP_I \tag{2.24}$$

$$C^{-} : HP_1 = CP - B\,QP_I \tag{2.25}$$

$$CP = H_{I-1} + B\,Q_{I-1} - R\,Q_{I-1}|Q_{I-1}| \tag{2.26}$$

$$CM = H_{I+1} - B\,Q_{I+1} + R\,Q_{I+1}|Q_{I+1}| \tag{2.27}$$

In which,

$$B = \frac{a}{gA}, \tag{2.28}$$

and

$$R = \frac{f\Delta x}{2gDA^2} \tag{2.29}$$

Adding equations. 2.27 and 2.28 gives

$$HP_1 = \frac{(CP + CM)}{2} \tag{2.30}$$

2.3.5 Finite Difference Solution

The two partial differential equations of motion and continuity are converted by the characteristic methods into four differential equations, which are then expressed in a finite difference form. The derivatives in the governing equations are replaced with approximate difference quotients when finite difference and finite element techniques are used. By contrast, in the method of characteristics, only the nonlinear friction term must be approximated (which is typically done by a linear difference term). Explicit finite difference schemes also have significant restrictions on the maximum time step to achieve stable solution.

Implicit methods require a simultaneous solution for every unknown in the problem at each time step, although they usually overcome the stability limitations, resulting in excessive computational and memory requirements even for present day computers [4].

2.3.6 Boundary Conditions

Only certain types of elements may be specified at the boundaries, or end points of a system. These are the boundary elements such as reservoir elements, flow boundary elements, or surge tank elements. These provide information to the model, which is essential to the solution of any and all of the system equations. This information consists of the head value, discharge value, or a function relating head and discharge at each boundary point. In addition, during connectivity processing, the program is signalled to end an old branch or start a new branch by the boundary condition elements.

At a boundary, to assign a specified head value (or schedule of values) the reservoir element is used. This is representative of a headwater condition, tail water condition, or free outfall at the end of a system. This will probably be the most commonly used boundary element. The flow boundary element is used to assign a specified discharge value (or schedule of values) at a boundary. There is no simple physical analogy in this element.

However, it is very useful for representing a connecting portion of a system not to be explicitly modelled. For example, one branch of a system made up of parallel generating units might be modelled explicitly while the others, including penstocks, surge tanks, and turbines etc., are represented with a constant discharge. This saves both data preparation and computational expense. A function relating the head and discharge at the boundary is implicitly defined where a surge tank element takes place at the end of a branch [5].

2.4 Elastic and Rigid Column Theories

The Elastic column theory is based on the assumption that, wherever a disturbance occurs, the pressure wave that is created will propagate along the pipeline at a rapid, but nevertheless finite, rate. This results in the wave moving through the system, reaching specific points after a period of time (dependent on the wave celerity of the system and the location relative to the position where the disturbance was introduced), provided original steady flow conditions are experienced. It can be understood, therefore, that the propagation of the pressure wave (positive) results in compression of the fluid and the deformation of the pipeline as the pressure wave moves through the system. In applying the elastic theory to determine the magnitude of the transient pressure, the elasticity of the pipeline can be neglected (rigid pipe theory) or taken into account (elastic pipe theory) [6].

The rigid column theory describes unsteady flow of an incompressible fluid in a rigid system and is usually applicable to slower transient phenomena. The application of rigidity assumptions is limited to the analysis of surge. These assumptions result in simplifying solving the ordinary differential equation. The dynamic hydraulic of a rigid water body during the mass oscillation can be determined using Newton's Second Law of Motion:

$$\Delta H = f\,(L/D)(V|V|\,/\,2g) + (L/g)\,(\Delta V/\,\Delta t) \qquad (2.31)$$

where,

ΔH = change in head (m)

The hydraulic grade line can be established for each instant in time using the fundamental rigid model equation. The head loss between the two ends of the pipeline, which is also the head necessary to overcome frictional losses and inertial forces in the pipeline can be indicated using the hydraulic grade line.

Flow changes are directly proportional to the change in slope. i.e. when the flow reduced due to valve Closure, the slope reduced. On other hand, if flow increased by valve opening, the slope increased allowing the potential of vacuum conditions to occur. In general, the maximum transient head envelope calculated by rigid water column theory (RWCT) is a straight line as shown in Figure 2.4.

The rigid model applies to slower surge or mass oscillation transients as in "Wave Propagation and Characteristic Time". During mass oscillations, moderate changes in head occur slowly, allowing changes of the liquid density and/or elastic deformation of the pipeline to be neglected. The rigid model has limited applications in hydraulic transient analysis because the developed equations do not accurately model pressure waves caused by rapid flow control operations [7].

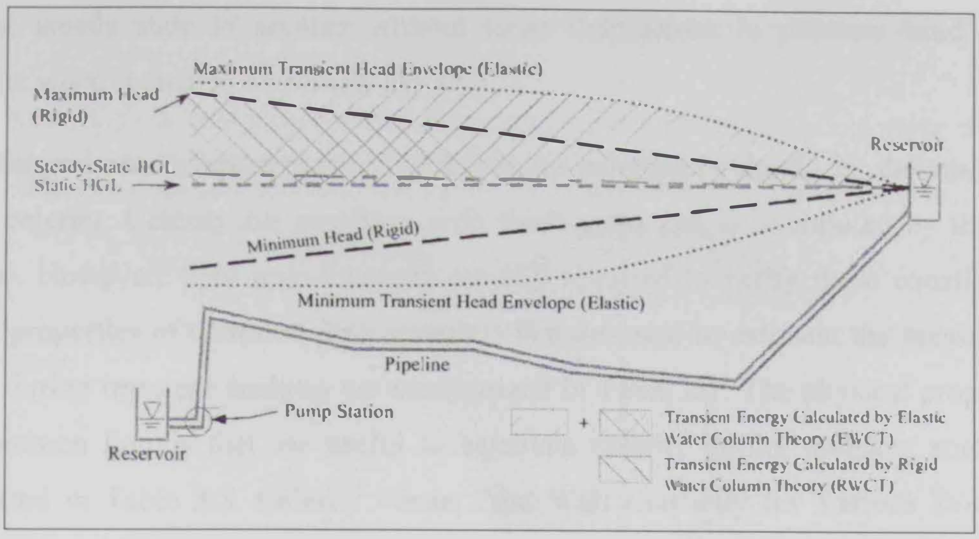


Figure 2.4: Static and steady HGL versus rigid and elastic transient head [7]

2.5 Column Separation Theory

Description of this problem considers the liquid pressure at a point in a pipeline, which is flowing full. Should this point pressure decrease, regardless of cause, to the vapour pressure of the flowing liquid, a vapour cavity will form. This vapour cavity is sometimes referred to as a vapour column. The phenomenon of vapour cavity formation is commonly referred to as a "column separation", meaning separation or interruption of the liquid column. It often occurs following rapid closure of a valve in a pipeline, which is flowing full. Nevertheless, column separation may also occur under other circumstances, for instance, following propagation of a transient wave of low pressure into an elevated portion of a pipeline flowing full. Such a low-pressure wave, for example, could be caused by a sudden pump failure in a liquid transmission or distribution system. The severe damage, which the high pressures associated with collapse of the vapour column, can inflict upon the pipe system causes column separation to be a problem of significant engineering concern.

2.6 Wave Propagation

In Hydraulic Transient, the disturbances in a fluid caused during a change from one steady state to another are described. The main reason of the disturbances is the pressure changes caused by the propagation of pressure waves throughout the distribution system.

With the sound velocity, these pressure waves continue to propagate until they are dissipated down to the level of the new steady state by the action of some form of damping or friction. One exception case where the flow undertakes a smooth transition from one steady state to another without large fluctuations in pressure head or pipe velocity is when regulated extremely slowly.

Before determining pressure wave velocity, some parameters should be determined first such as celerity. Celerity for pipelines with thick walls can be computed by theoretical equations. However, field investigations are still required to verify these equations. The physical properties of common pipe materials that are used to estimate the pressure wave velocity during transient analysis are summarized in Table 2.1. The physical properties of some common liquids that are useful to calculate celerity during transient analysis are summarized in Table 2.2. Celerity versus Pipe Wall Elasticity for Various D/e Ratios" provides a graphical solution for celerity given pipe wall elasticity and various diameter/thickness ratios is presented in Figure 2.5 [7].

In order to determine the pressure wave velocity in deferent pipe materials, Figure 2.6 is used. It shows the pressure wave velocity for water in round pipes versus the ratios of different diameters to thicknesses. These relations are defined for KR of 0.91 that is a constant coefficient accounting for the type of support for the pipeline such as the restraint against longitudinal pipe movement. The number to the right of curves indicates Young's Modulus, PSI x 10^6, Ec, value (Table 2.1), in PSI that was used to construct the curve.
Pressure Wave propagation with speed, a, can be determined in a system length, L, which is the longest path connecting a pump to a storage tank or reservoir as an approximation, and generated by a flow control, reaching the other end of the pipeline in a time interval equal to L/a seconds.

Table 2.1: Physical properties of common pipe materials [7]

Material	Young's Modulus, Ec Pa X 10^9	Young's Modulus, Ec PSI X 10^6	Poisson's Ratio, μ_p
Aluminium	69	10	0.33
Asbestos Cement	23-24	3.3 – 3.5	-
Cast Iron	80 – 170	11.6 – 24.7	0.24 – 0.27
Concrete	14 – 30	2.0 – 4.4	0.10 – 0.15
Reinforced Concrete	30 – 60	4.4 – 8.7	-
Ductile Iron	172	24.9	0.3
Polyethylene	0.7 – 0.8	0.10	0.46
PVC	2.4 – 3.5	0.3 – 0.5	0.49
Steel	200 – 207	29 – 30	0.3

Table 2.2: Physical properties of common liquids [7]

Liquid	Temperature (°C)	Bulk Modulus of Elasticity, Ev		Density	
		10^6 lbf/ft^2	GPa	Slugs/ft^3	Kg/m^3
Fresh Water	20	45.7	2.19	1.94	998
Salt Water	15	47.4	2.27	1.99	1,025
Mineral Oils	25	31 – 40	1.5 – 1.9	1.67 – 1.73	860 – 890
Kerosene	20	27	1.3	1.55	800
Methanol	20	21	1	1.53	790

2.7 Classifications of Flow Control Operations

The characteristic time is significant in transient flow analysis because it dictates which method is applicable for evaluating a particular flow control operation in a given system. The characteristic time for the pipeline is the total time required for the same wave to come back to its origin. It is calculated as 2L/a second. This time increment is used to determine the type of operation whether instantaneous, rapid, gradual, or slow as shown in Table 2.3. The rigid model provides accurate results only for surge transients generated by slow flow control operations that do not cause significant liquid compression or pipe deformation.

The operation is considered "instantaneous" if a flow control operation produces a velocity change in a time interval equal to zero (Table 2.3). The operation is considered "rapid" if the flow control operation produces a velocity change in a time interval less than or equal to a pipeline's characteristic time. If a flow control operations that occur over an interval longer than the characteristic time, the operation is considered "gradual" or "slow."

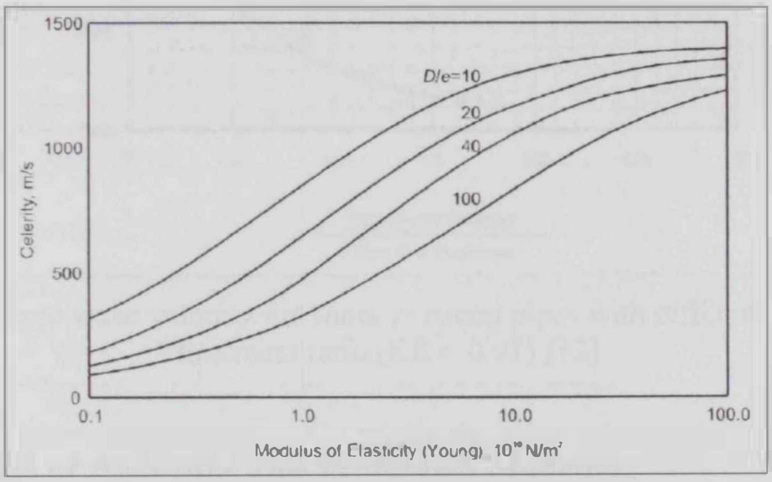


Figure 2.5: Celerity VS pipe wall elasticity for various D/e rations [7]

Table 2.3: Classification of flow control operations based on system characteristic time [7]

Time of Manoeuvre	Operation Classification
$T_M = 0$	Instantaneous
$T_M \leq 2L/a$	Rapid
$T_M > 2L/a$	Gradual
$T_M >> 2L/a$	Slow

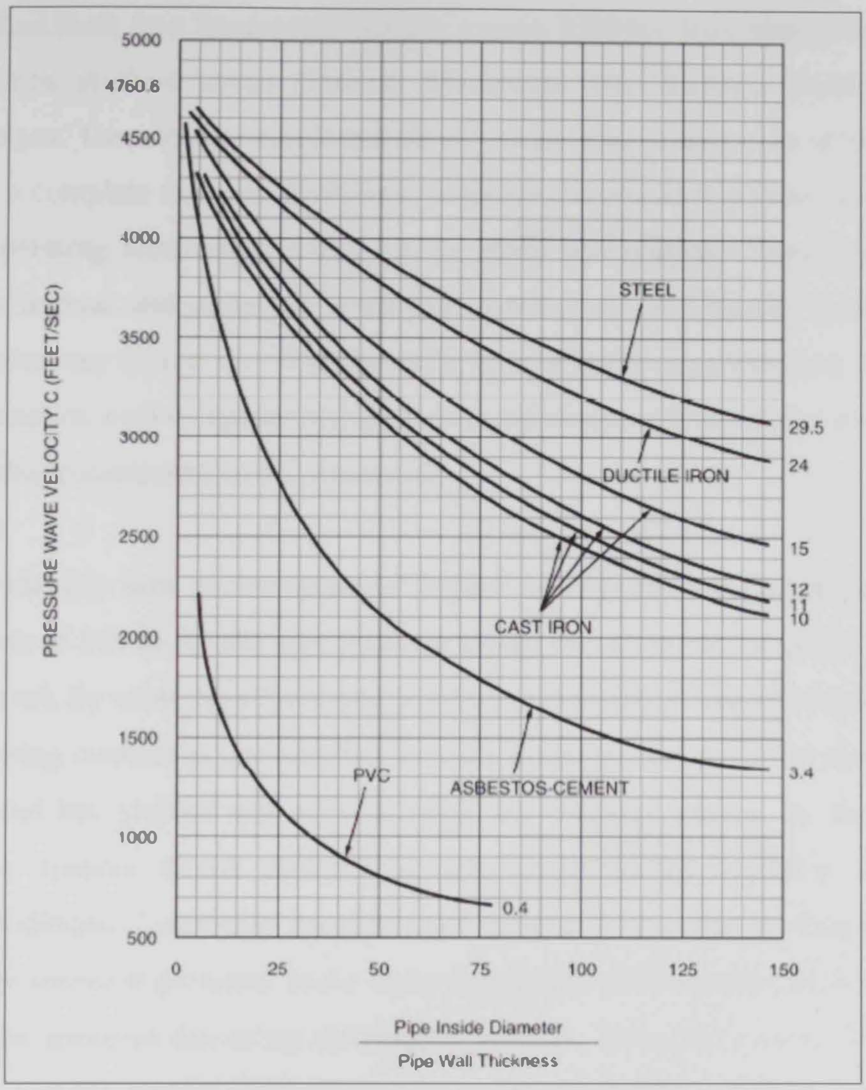


Figure 2.6: Pressure wave velocity for water in round pipes with different diameters/ wall thickness ratio (KR = 0.91) [12]

2.8 Case Studies of Hydraulic and Transients Modelling

In this section, four case studies were presented where transient's simulations were utilised. These cases are in deferent locations of the word. i.e. UAE, USA, and other locations.

2.8.1 Case 1: Pressure Utilization on Water Transmission Pipelines in Alain Region Considering Transient Conditions [8]

Al Mamari (2007) investigated the feasibility of bypassing a number of reservoirs and pumping stations in a small transmission system in AlAin City of United Arab Emirates. The system has a major transmission pumping station (AlAin Reception Pumping Station, or 'AARS') that hosts four fixed speed pumps, a main 1200mm line, storage reservoirs and boosting pumps at three zones (Dahma, Markhania and Maqam), besides few other connection pipes. Three surge vessels and air valves are also components of the system. By carrying out a complete transient analysis to evaluate the potential of pressure rises related to various operating scenarios, the goal of the study was realized. These scenarios were divided into normal and abnormal operating conditions. The typical demand changes occurring during any typical day determines the normal operating conditions. Rare and odd operating scenarios such as sudden valve closures/openings and pump trips are represented by the operating conditions that are abnormal.

In order to identify safe performance of bypass setting that was proposed, restricting pressure heads of 160 m for the pipe pressure rating and 60 m for the pump shutoff head were considered. By using InfoWater and InfoSurge, simulation results associated with the normal operating conditions showed that in such situations the proposed setting is safely performing and has yielded safe closure times for different valves. In the situation of sudden valve closures, unsafe performance was shown by the results of the abnormal operating conditions. Corrective measures are proposed in order to shield the system against severe transient pressures under these conditions. PRVs should be installed on the branches at the upstream side of the distribution network. When the system pressure is kept 100 m, it is possible to size and evaluate the proposed valves. A total of 847,355 UAE Dirham's per year was the estimated as savings in energy cost if the proposed bypass setting is adapted. After a payback period of one year and one month, these savings are found to be available in order to cover the capital cost of the needed bypasses and PRVs.

2.8.2 Case 2: Susceptibility of Potable Water Distribution Systems to Negative Pressure Transients [9]

In this study, surge models were developed for five different areas in United States of America (New Jersey, and New York). The pressure was monitored for around of two weeks in at least one susceptible location in two (2) different systems. Based on surge modelling results the pressure monitoring locations were selected. The data gathered from the field was then compared to model results, and accordingly the wave speeds and pump shutdown times were adjusted until a good match was found between the data of both the field and the model. For surge modelling, H_2OSURGE (MWHSoft, Pasadena, CA) was used. The results of simulations of transient pressure events were shown in the model output, and the analysis of the location and magnitude of low and negative pressure events under a variety of system conditions was also included.

The selected system was located in a relatively flat part of New Jersey and it was a medium-sized system, serving approximately 83,000 persons with 31,100 service connections. 89% of the connections were found to be residential, 9% were commercial, and less than 2% included industrial, fire and other customers demand. The distribution system operates as one pressure gradient with customers at elevations ranging from approximately 5 to 75 feet mean sea level (msl). Figure 2.7 shows the system model skeleton which includes the 18 pump stations that were active during a high demand day (23.6 mgd peak hour flow supplied) in 1999 and 7 elevated storage tanks. Pipe diameters in the model range from 2- to 16-inches. All pipes are ductile iron and no valves or hydro-pneumatic tanks in the system were found.

Summary of main findings for case II:

- When a complete loss of pumping power was simulated, it was predicted that the system would experience low or negative pressure transients.
- Areas of local elevation greater than 30 to 40 ft immediate surroundings, areas within 1 mile of an elevated tank and areas that were not well gridded were more susceptible to low or negative pressures
- In general, distribution system locations in close proximity to pump stations with average downstream velocities greater than 3 fps were most susceptible to low and negative pressures. Areas near pump stations that power on and off several times a day will be more susceptible to low and negative pressures than areas near pump stations that operate continuously.
- Negative pressures were not detected in the distribution system monitored. However low pressures (pressure < 20 psi) were measured in one location in System 2.
- Calibrated EPS models produce surge models that can adequately assess distribution susceptibility to low and negative pressures. However, the predicted pressures may be lower than observed in the field.

- In the system (wave speed = 2,500 ft/s), the magnitude of the surge predicted by the model was greater than the magnitude of the surge measured in the field, but the trend in model and field responses were similar.
- The main recommendations addressed in the study were:
  1. Increasing the time, in which transient producing events occur, can help reduce the impact of the event.
  2. Increasing the pump shut down time in the system reduced the magnitude of the transient produced by approximately 3 psi.
  3. Installing hydro-pneumatic tanks or air vacuum valves can be used to control the magnitude of the low / negative pressure transient (surge) once it has been created.

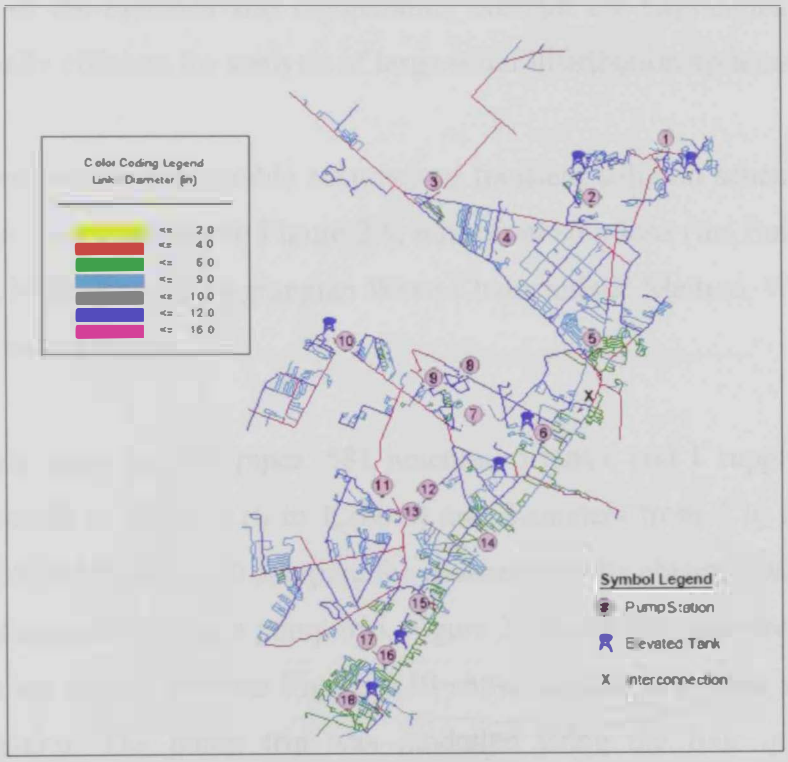


Figure 2.7: Hydraulic model for selected system (Case II) [9]

2.8.3 Case 3: Hydraulic Transient Guidelines for Protecting Water Distribution Systems [4]

The authors compared the formulation and computational performance of two numerical methods for modelling hydraulic transients in water distribution systems. One method is Eulerian-based. and the other is Lagrangian-based. The Eulerian approach explicitly solves the hyperbolic partial differential equations of continuity and momentum and updates the hyperbolic partial differential equations of continuity and momentum and updates the hydraulic state of the system in fixed grid points as time is advanced in uniform increments. The Lagrangian approach tracks the movement and transformation of pressure waves and updates the hydraulic state of the system at fixed or variable time intervals at times when a change actually occurs. Each method was encoded into an existing hydraulic simulation model that gave initial pressure and flow distribution and was tested on networks of varying size and complexity under equal accuracy tolerance. Results indicated that the accuracy of the methods was comparable. but that the Lagrangian method was more computationally efficient for analysis of large water distribution systems.

In order to demonstrate the comparable accuracy of transient solution schemes on actual, large, and complex system shown in Figure 2.8. numerical solutions (the Eulerian Method Of Characteristic. MOC. and the Lagrangian Wave Characteristic Method, WCM. solution scheme) methods were applied.

The network's main parts are 797 pipes. 581 junction, 5 tanks, and 1 supply pump. Pipe lengths varied from 20 to 4,200 ft (6 to 1,280m) and diameters from 4 to 24 in. (100 to 600mm). Figure 2.9 and Figure 2.10 compare the transient results obtained using MOC and WCM solution schemes following a pump trip. Figure 2.9shows the pressure transient just downstream from the pump, whereas Figure 2.10 shows results at a node some distance away from the pump. The pump trip was modelled using the four quadrant pump characteristics in the form developed by Marchal and co-workers (1985). A 20-ft (6m) length tolerance was used in the analysis, resulting from a time step of 0.0056 s.
The transient results, which were gathered using the Eulerian MOC and the Lagrangian WCM solution scheme, were matching.

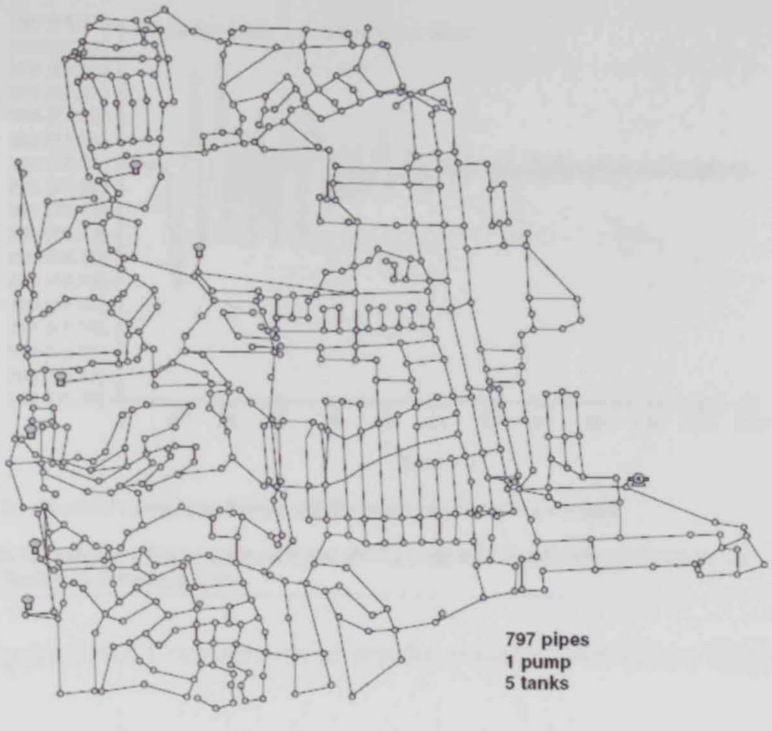


Figure 2.8: Pipe network schematic for studied system (Case III) [4]

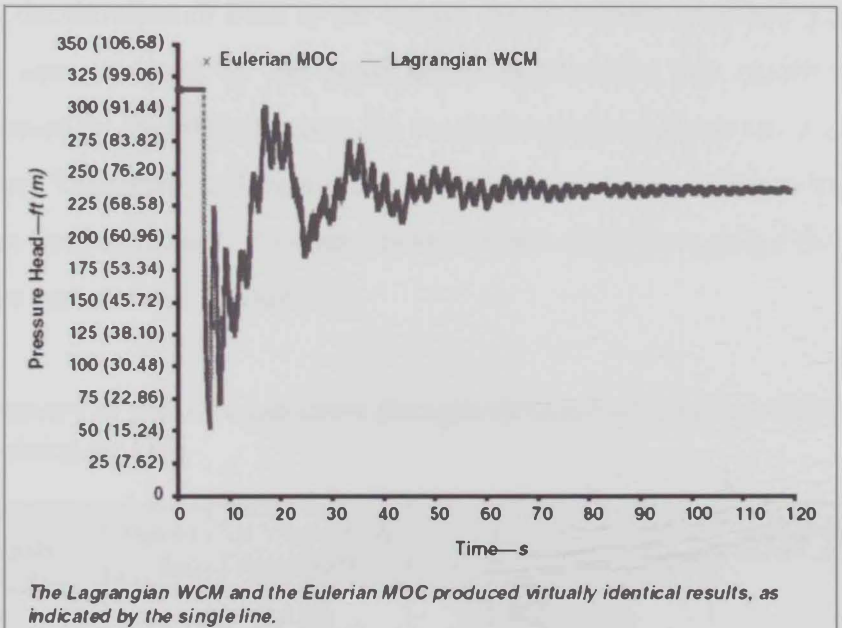


Figure 2.9: Comparison of results at pump (Case III) [4]

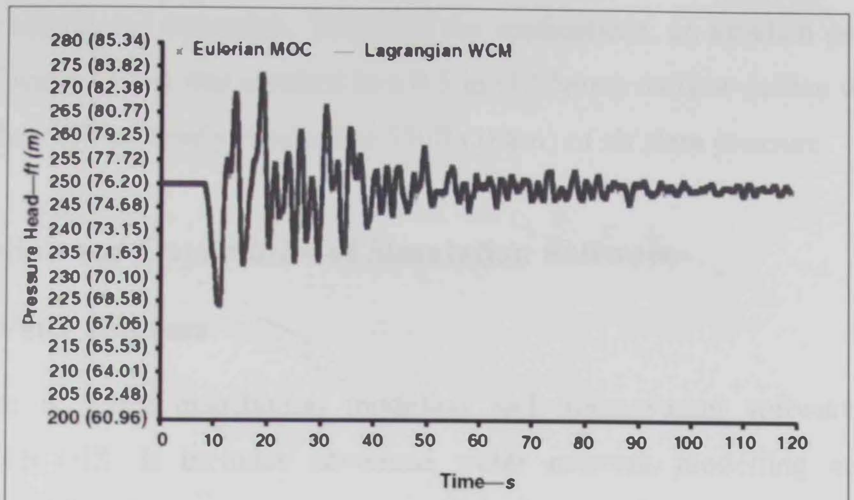


Figure 2.10: Comparison of results at node 3066 (Case III) [4]

2.8.4 Case 4: Pressure surges in pipeline system resulting from air releases [10]

The impact of final release of air through air valves, which produce a pressure surge, was tackled in this study. "Air slam" is the term that refers to this phenomenon, which results from the rapid deceleration of fluid at the instant the air is fully expelled. a pressure surge similar to the one produced by the rapid liquid deceleration that results from a valve closure is produced by this phenomenon. An excessive pressure surge can occur if the air is released too fast. Therefore, it is significant to design air release valves to avoid excessive pressure surges from occurring. Pressure changes when all air is expelled through different orifice sizes are summarized in Table 2.4.

Table 2.4: Summery of pressure increases through different-size orifices following expulsion air [10]

Office Size in.(mm)	Head in Air Valve (HA) feet of water (psi)	ΔH feet of water (psi)	ΔH Surge Analysis feet of water (psi)
4.0 (100)	0.059 (0.026)	236.4 (102.3)	237.8 (102.9)
2.0 (50)	0.825 (0.358)	228.4 (98.9)	218.1 (94.4)
1.0 (25)	4.690 (2.030)	111.6 (48.3)	120.6 (52.2)
0.5 (12.5)	7.810 (3.380)	33.4 (14.5)	42.3 (18.3)

The air compressibility within the air valve is taken into consideration while performing the surge analysis. An outflow orifice smaller than the inflow orifice is desirable to alleviate undue secondary pressure surges, which are caused by the final release of air, as shown in the considered examples. In one of the applications, an air-slam pressure of less than 100ft of water (30m) was resulted in a 0.5 in (12.5mm) outflow orifice whereas a 3 in (75mm) outflow orifice nearly resulted in 550ft (168m) of air slam pressure.

2.9 Description and Capabilities of Simulation Software

2.9.1 InfoWater Software

This software is water distribution modeling and management software. It is fully integrated with GIS. It includes advanced water network modelling and functional optimization with the latest generation of ArcGIS. In InfoWater, modeller can work in a single environment and using a single dataset while commanding GIS analysis and hydraulic modelling. Geospatial analysis, infrastructure management and business planning are delivered by this software in unparallel levels.

Several applications of water distribution systems can be resolved by InfoWater Model such as Water Quality Simulation, Real-time Hydraulic, Fire Flow Assessment and Leakage Control, Pump Scheduling, Energy Consumption Minimization, Capital Budgeting and Conservation Studies, Operational Study and Emergency Response, Master Planning and Infrastructure Rehabilitation, System Expansion and Improvement, Sampling Program Design, Satellite Treatment Identification, and New System Design and Facility Sizing.

The software allows in several commands such as creating, editing, modifying, running, mapping, analyzing, designing and optimizing network models, reviewing query, and display simulation results within ArcGIS. In addition to that, core features of ArcGIS are extended in InfoWater such as providing a comprehensive geospatial environment for complete network model construction, graphical editing, network simulation, results presentation, map generation, and data sharing and exchange. This integration between InfoWater and ArcGIS assists in delivering informed GIS solutions so that to meet the quality standards of drinking water, optimization of the system performance and capital improvements, and enhance operations as well [11].

2.9.2 InfoSurge Software

As a result of continuous efforts, research and development in transient flow analysis, InfoSurge software was developed. This is modelling software helps in making proper decisions by analysing water systems under transient conditions. In other words this software helps water utilities, by providing the required information, to analyze their systems and make decisions for transient conditions. Vapour pressure cavitation and column separation are explicitly modelled, allowing the effect of pressure surge, due to vapour cavity, collapse to be properly evaluated. One of the major characteristics of this software is that it is designed to compute, quickly and precisely, the pressures and flows throughout the network during transient operation and evaluate the alternate operational scenarios and surge protection devices as well.

In the water risk management and environmental protection plan it is significant to take into consideration the hydraulic transients and surge protection devices despite the fact that the abnormal transients may occur only once every year, or in case pipes break or when there is failure in power. Hydraulic systems usually operate at a stable condition of dynamic balance and knowing that normal transients may occur several times a day as pumps start or stop the changes in flow take minutes or hours

Shifting from one stable state to another causes disturbance which is described by hydraulic transients. The main elements of disturbance are the pressure changes caused by the propagation of pressure waves throughout the distribution system. These pressure waves continue to propagate, with velocity of sound, until they are dissipated down to the level of new stable condition by the action of some form of friction or damping. However, undertaking a smooth transition from one stable condition to another is possible without large fluctuation in pressure head or pipe velocity with extremely slowly regulation (example of very slow valve opening or closure).

By using Wave Characteristic Method in InfoSurge, the basic conventional equations of fluid mechanics for the transient flow of an incompressible fluid in a pipe network can be solved. This method is highly efficient and powerful and it simply provides the engineer with valuable insights into transient behaviour of their systems.

The good convergence properties of Lagrangian-type methods such as wave plan method, in addition to the distributed friction effects and inherent accuracy of Eulerian-type methods such as method of characteristics are both combined in wave characteristic method. Nevertheless, extensive calculations in minor steps and at numerous locations especially for larger systems are required to be carried out in Eulerian-type.

Lagrangian nature dominates the Wave characteristic method and it is based on tracking the movement and magnitude of pressure wave and reflected at the junctions between fixed length time steps. Pressure and flow time are computed for any point in the network by summing pressure and flow time histories with the time of the contributions of incremental waves.

Briefly, since the Lagrangian nature dominates the wave characteristic method, larger systems will be accurately analyzed in an expeditious manner. In addition, comparing to the Eulerian-type methods this one is less sensitive to the nature of the network and to the length of the simulation.

Responses of water distribution system to valve opening/closure, pump trip and changes in pump speed are provided in InfoSurge. Additionally, this software helps in identification of special protection measurements in order to avoid pipe breaks, pipe leakage. The water quality in the distribution system is improved as well. Several pressure and flow transients protection devices exist in InfoSurge such as Air Release/Vacuum Valves (2 and 3 stage valves), PRVs and Surge Anticipation Valves, Open and Closed Surge Tanks, Bladder and Hybrid (vented to admit air) Tanks, Feed Tanks (provide inflow to prevent cavitations), Bypass Lines and Check Valves.

InfoSurge Capabilities

Modelling the wide range of water hammer events, including transient cavitations and several surge protection devices is one of the strong software capabilities of InfoSurge.

InfoSurge has the ability to determine the transient pressures and flows variations for all positions in the water distribution system at different network elements such as pumps, reservoirs, pipe junctions, closed ends, pressure relieve valves, open surge tanks and feed tanks, closed surge tanks and bladder tanks, surge anticipation tanks, valves orifice and other reservoir elements, air valves, bypass lines and check valves. In addition, this software can determine the gas volumes for closed surge tanks and air vacuum valves and pump speed variations for pump trips. It supports both tabular and graphical reporting capabilities with high quality graphs, which can be exported, to other applications via Windows copy and paste tools. These reporting capabilities allow the user to determine the system's response to pump station power failures, valve closures, and pump speed changes. By avoiding the potential catastrophic effects of water hammer and other undesirable system transients this software helps in improving the reliability and safety of the water utility system.

Transient conditions considered in InfoSurge are valve opening and closing (variation in flow area), change in specified inflow or outflow, and pump start or shutdown (variable speed pump-pump trip). Furthermore, disturbance occurs when some elements undergo operational status changes (open or closed) due to conditions encountered during transient operation such as check valves, feed tank connections, bypass lines, pressure relieve valves and air valves.

When InfoSurge is coupled with InfoWater capabilities, it can solve several problems such as transient flow, air intake and release at discrete points, two-phase fluids (vapour and liquid) and two-fluid systems (air and liquid), overflows prediction at outfalls or spills to the environment more accurately, Network Risk Reduction by performing a hydraulic transient analysis using advanced surge protection, risk management of contamination during sub-atmospheric transient pressures, which can suck air, dirt and contaminants into the water distribution system or sub-atmospheric transient pressures which cause backflow of dirty water (pathogen intrusion) into the distribution system, risk reduction of water contamination during sub atmospheric transient pressures, during which groundwater and pollutants could be sucked into the pipe, and finally, risk reduction of transient-related damage to maximize operator safety and frequency reduction of service interruptions to customers [12].

CHAPTER III

SYSTEM DESCRIPTION

The general arrangement of AARS will be thoroughly discussed covering the design, operation and supply patterns in this chapter. This includes description of the existing system.

3.1 Description of Al Ain Water Reception Station (AARS)

AARS is located in the northern side of Al Ain near the airport. It is a large water reception station with 2 x 3 km dimensions and 253m elevation above sea level. AARS receives potable water of about 85.6 Imperial Million Gallons per Day (IMGD) from two main sources. The first source is a major desalination plant in Taweelah (Abu Dhabi), located on the Arabian Gulf (western coast of UAE), which is about 160 km away from Al Ain city. Taweelah lines consist of twin 1200mm diameter GRP pipes carrying about 49.3 IMGD to AARS. The approximate residual pressure at Taweelah line outlet (upstream AARS) is about 0.8 bar (as provided by TRANSCO) which is relatively small to bypass AARS pumping station. The second source is from another major desalination plant in Fujairah city, located on the Arabian Sea (eastern coast of UAE), which is about 250 km away from Al Ain city. Fujairah Lines consist of twin 1400mm diameter GRP pipes carrying 36.3 Imperial Million Gallons per Day (IMGD). The approximate residual pressure at Fujairah line outlet (upstream AARS) is about 6.5 bars (as provided by TRANSCO) which might be enough to bypass some of the downstream pumping station units. Fujairah Desalination Plant has recently become the main water source to AARS.

Al Ain reservoirs inlet mains have separate circuits to be supplied either from Taweelah, Fujairah or from both. The reservoirs outlet mains are arranged to feed the same pump house. Four main pumping groups currently exist in AARS. The first group is Al Markhania Group with about 38.1 IMGD supply. The second group is Al Khabisi Group with 18.8 IMGD supply. The third group is Hili Group with 13.8 IMGD supply. Finally, the fourth group is Sarouj Group with 9.9 IMGD supply. Figure 3.1 presents a schematic sketch describing AARS with the upstream pressure for those pumping groups. The balance 5 IMGD is used for other indirect connections to AARS, mainly farms.

AARS consists of the following main components, located inside a boundary wall of (1900m x 800m):

- Four concrete reservoirs of 5 MIG (22,730 m^3) capacity for each and 80m x 60m x 6m-dimensions.
- Four pumping groups contained inside the pumping house of 146m x 27m x 8m dimensions. Each pump group will be discussed separately.
- Four outlet pipelines (1200mm D.I. to Markahnia, 900mm D.I. to Hili, 900mm D.I. to Khabisi, and 900mm D.I. to Sarouj).
- Chlorination building for chlorine disinfection, where chlorine is injected at the inflow mains to the station as well as at the outlet mains [8].

3.2 AARS Pump Groups

As stated before, AARS consists of four main pump groups. These groups are:

- Markhania Pump Group
- Khabisi Pump Group
- Hili Pump Group
- Sarouj Pump Group

As summery for each pump is presented in Table 3.1.

Table 3.1: AARS pump groups data

Pump Group Name	No. of Pumps	Design Head (m)	Design Flow (m3/hr)	Efficiency (%)	NPSHR (m) (3% OROP)	Speed (RPM)	Power (kw)	Pump Flow IMGD	Total Pumps Flow IMGD	Average Flow Required IMGD
Markhania	3	47	3,938	90.3	3	827	558	20.79	62.37	38.1
Khabisi	3	64	1,253	85.5	3	1,600	255	6.61	19.84	18.8
Hili	3	56	1,350	88.1	4	1,190	234	7.13	21.38	13.84
Sarouj	2	69	1,053	85.4	3.3	1,185	187	5.56	11.12	9.9

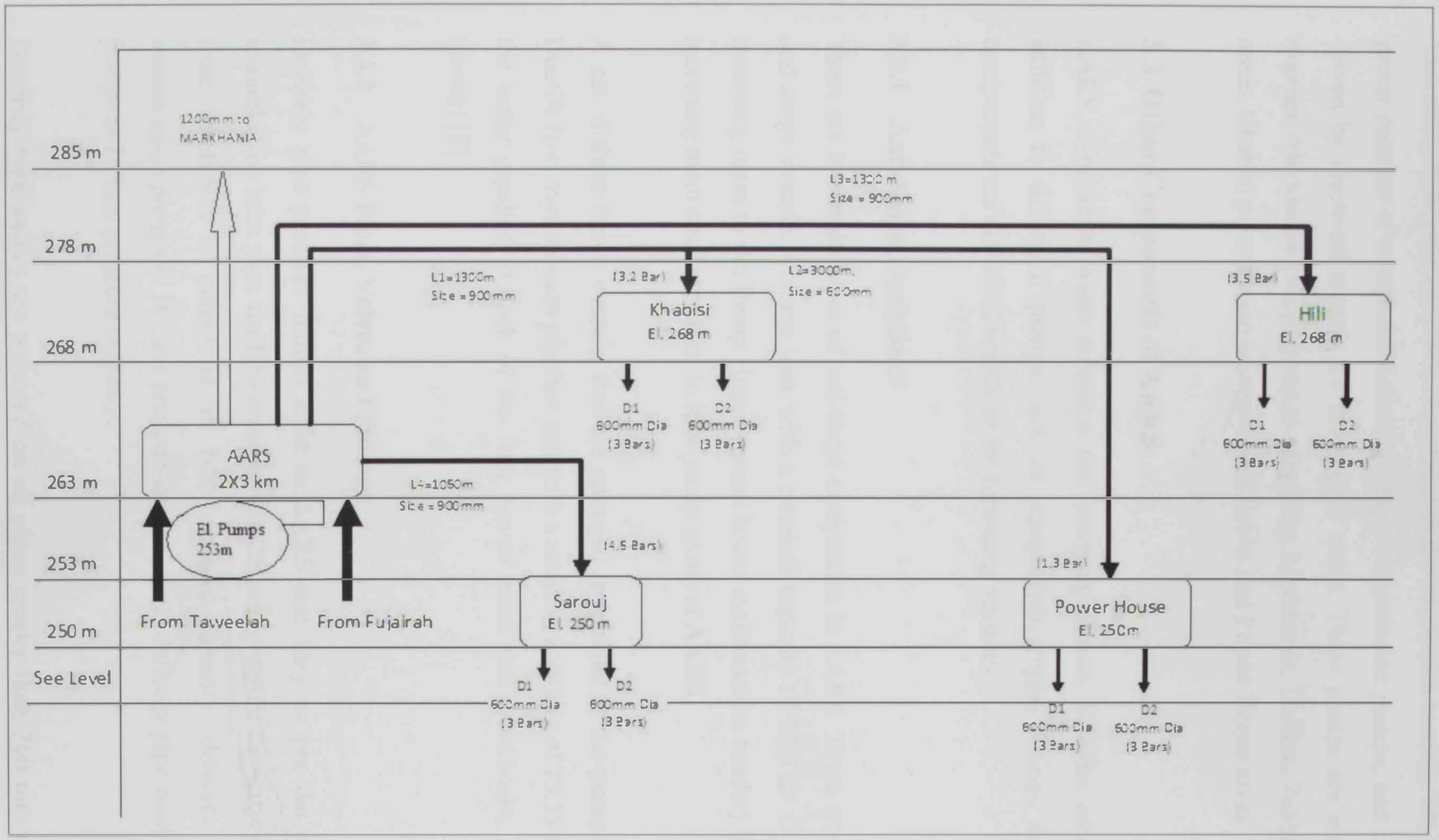


Figure 3.1: Schematic sketch describing AARS

It is essential to note that most of these pumps were installed more than 25 years ago. Therefore, 80% efficiency was considered in the simulation models. Moreover, each pump group consists of composed centrifugal type transmission pumps, and one standby pump driven by electrical motors for emergency cases. These pumps are arranged in parallel manner. Markhania pump group is supplying Markhania, Dahma, Zakher, and Al Wagan areas. Khabisi pump group is supplying Khabisi and Power House areas.

3.3 Other Components of AARS

AARS, a principle water reception and pumping station, includes other components in addition to the main pumps such as equipments, pipes, fittings and valves. These components are described briefly in the following sections.

3.3.1 Anti-Surge Equipment

There are two main types of anti-surge equipments in AARS. These types are surge tanks and surge vessels. A surge tank with a nominal capacity of 37.5 m^3 is provided on each incoming main to the Pump House (pump house main suction header) in addition to each incoming main suction header to four-pump group at AARS.

A set of three Surge vessels, that are arranged in parallel at the pumps' outlet 1200 mm Ductile Iron transmission pipeline and with a nominal capacity of 73.55 m^3, is provided on the water pipeline to each of the four served areas (i.e. Markhania, Khabisi, Hili and Sarouj) [8].

3.3.2 AARS Pipes, Valves and Fittings

Deferent pipe pressure ratings exist in AARS and vary as per the pipe location. For example, the inlet pipe work to reservoirs is 25 bars pressure capacity; suction pipe work from reservoirs to pumps is 10 bars pressure capacity; delivery pipe work from transmission pumps is 16 bars pressure capacity; and delivery pipe work from distribution pumps is 16 bars pressure capacity.

Butterfly type valves are provided on all pipes greater than 300 mm nominal diameter. Whereas Gate type valves are provided on all pipes less than or equal to 300 mm nominal diameter.

Under the maximum unbalanced pressure that may occur across the valve, all the valves are capable of being operated either electrically or manually. Electrically actuated section valves have been provided on the following:

- Upstream the inlet station flow meters,
- The two lines feeding each reservoir inlet chamber,
- The two lines leaving each reservoir,
- The main suction header at the inlet of the pumps house,
- Downstream each flow meters and each pipeline leaving the station,
- The suction and delivery sides of each pump
- The branch to each surge vessel and to each surge tank.

On the other hand, manually operated valves have been provided on branches for future connection of a surge vessel. The nozzle type non-return valves have been provided on each delivery pump side. In addition, globe type pressure sustaining valves have been installed at station inlets. These valves are located between the sectioning valves and the flow meters. Their main function is to ensure tight line operations on the upstream twin 1200 mm pipelines from Sweihan to AARS.

The main piping inside the pump house is made of ductile iron with internal lining of fusion-banded epoxy (FBE) for both pipes and fittings. While the external main piping of the pump house is made of ductile iron with internal lining of cement mortar for the pipes and fusion banded epoxy (FBE) for the fittings [8].

3.4 Demands and Pumps Operation and Supply Patterns at AARS

Supply flow to AARS from Fujairah lines was plotted versus outgoing flows to different areas in Figure 3.2. This Figure shows the flow rate which peaks in Markhania to 2,956 l/s from 7:00 to 9:00 hrs, peaks in Khabisi and Power House to 1,702 l/s from 9:00 to 10:00 hrs, peaks in Hili to 1,303 l/s from 7:30 to 20:00 hrs; and peaks in Sarouj to 805 l/s from 7:00 to 13:30 hrs.

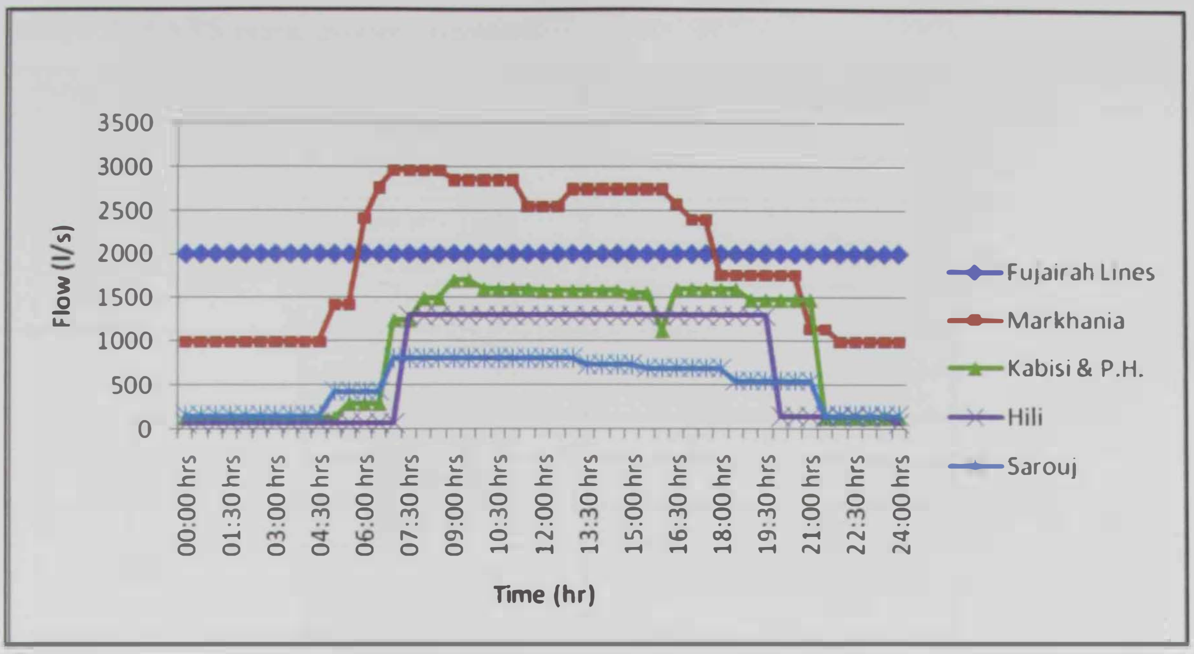


Figure 3.2: Daily water demands for four demand zones (Markhania, Khabisi & Power House, Hili and Sarouj)

As shown in Figure 3.2, Markhania group cannot be supplied directly from Fujairah lines especially during peak hours. Consequently the requirement of bypassing AARS to feed Al Markhania Group cannot be met (refer to Chapter 4 for further discussion).

It is worth mentioning that the current study limits the analysis to the current existing demand patterns. However, it is vital and recommended to search new supply patterns associated with less energy consumption and targeting continuous supply to the customers. This can be further studied upon collecting more data and details in the distribution system and can be achieved using advanced optimization techniques.

3.5 Operations of AARS Pumping Groups

The described variable demands are met by the corresponding supplies via operating different number of pumps in AARS as explained in Table 3.2.

Table 3.2: AARS pump groups operation

Pump Group Name	Time period	No. of Operating Pumps
Markhania	00:00- 05:00	1
	05:00 - 06:00	2
	06:00 - 18:00	3
	18:00 - 21:00	2
	21:00 - 24:00	1
Khabisi	00:00 - 06:30	1
	06:30 - 07:00	2
	07:00 - 21:30	3
	21:30 - 24:00	1
Hili	00:00 - 07:00	1
	07:00 - 20:00	3
	20:00 - 24:00	1
Sarouj	00:00 - 04:00	1
	04:00 - 23:00	2
	23:00 - 24:00	1

3.6 Outflows of AARS

As presented before, AARS is supplying four areas, which are Al Markhania (including Dahma Zakher, and Alwagen), Sarouj, Khabisi and Powerhouse, and Hili. The water supplies to each of these areas are summarized in Table 3.3. Throttle Valves were placed downstream the pumping station at each area in order to maintain the pressure in the distribution system at 3 Bars. However, no real data on the pressure upstream valves were found. Therefore, Pressure Reducing Valves were used in the simulation to achieve this objective. In addition, the incoming pressures to the pumping station reservoirs at each area are being wasted by dumping the water in to the atmospheric pressure (discussed thoroughly in Chapter 1). Certain protection arrangements are being used for the high incoming pressures during the low demand period. Table 3.4 summarizes pumps operation during different periods.

Table 3.3: Water outflow at different areas

Area	Elevation (m)	Incoming water source (mm)	Min. Incoming water Pressure (bar)	No. of Distribution pumps	Distribution network D1 (mm)	Distribution network D1 (bar)	Distributio n network D2 (mm)	Distribution network D2 (bar)	Range (L/Sec)	Average (L/Sec)
At Markhania	250	1000	4.4	8	600	3	600	3	1001.4 to 2956	1962.4
At Dahma	278	800	2.5	8	600	2.3	600	2.3		
At Zakher	250	800	3	8	600	2.3	600	2.3		
At Al wagan	250	800	3	N/A	N/A	N/A	N/A	N/A		
At Khabisi	268	900	3.2	6	600	3	600	3	129 to 1702	969
At Power House	268	600	1.3	5	600	3	600	3		
At Hili	268	900	3.5	10	600	3	600	3	73 to 1303	713
AtSarouj	253	900	4.5	7	600	3	600	3	150 to 805	508

Table 3.4: Pumps operation at each area pumping station during different periods

Area	Time period	No. of Operating Pumps
Khabisi	00:00- 07:00	1
	07:00 - 09:00	3
	09:00 - 16:00	4
	16:00 - 21:30	5
	21:30 - 24:00	1
Power House	00:00 - 07:00	1
	07:00 - 08:00	3
	08:00 - 16:00	4
	21:30 - 24:00	1
Hili	00:00 - 07:30	1
	07:30 - 20:00	9
	20:00 - 24:00	1
Sarouj	00:00 - 05:00	1
	05:00 - 07:00	3
	07:00 - 18:30	6
	18:30 - 21:30	4
	21:30 - 24:00	1

CHAPTER IV

HYDRAULIC SIMULATION MODELS SETUP AND BYPASS PROPOSAL

In this chapter, all related parameters, assumptions, and data used in the hydraulic simulation models setup for the proposed system are addressed.

4.1 Existing System Setup

In this section, main input data required to run the simulation models in both InfoWater and InfoSurge are summarized and addressed. Labels for pipes and nodes are plotted on the network sketches (Figure 4.1 for pipes and Figure 4.2 for nodes). All data for InfoWater are valid for InfoSurge as well.

4.1.1 InfoWater Setup

The data given were used in setting up the InfoWater model. These data include nodes and junctions, pipes, valves, pumps, reservoirs, simulation options, time and reports, etc...

Nodes and Junctions Data

All nodes have an elevation value of 253m with zero demand and no demand pattern except the following nodes in Table 4.1, which represent the areas.

Table 4.1: Junctions input data

ID	DEMAND	PATTERN	Elevation (m)
J52	711.46	KHABISI	268
J54	724	HILI	268
J60	550	DAHMAH	278
J62	628	MAQAM	250
J64	466	ZAKHER	250
J84	516	SAROUJ	253
J92	279.58	POWERHOUSE	268
J66	342		250

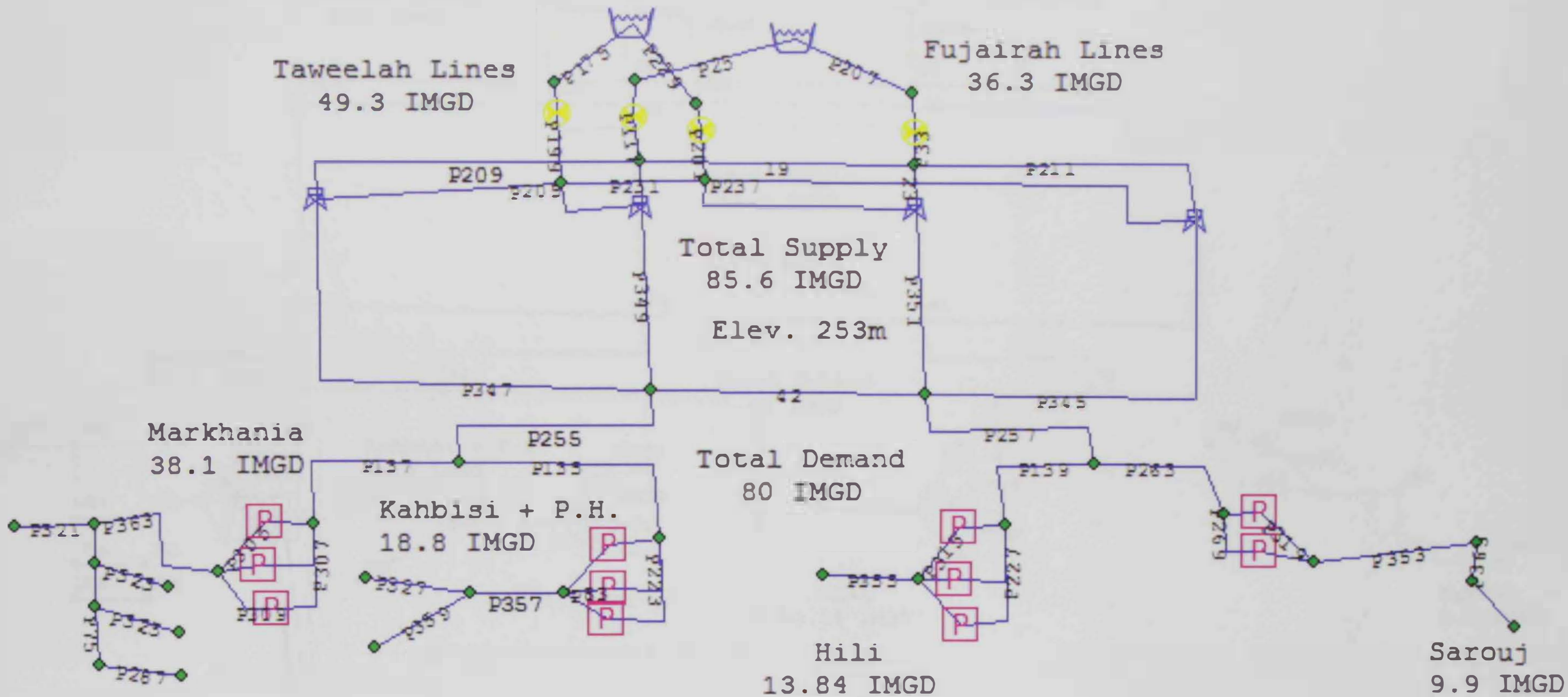


Figure 4.1: Proposed network layout with pipes labels as shown in the simulation model

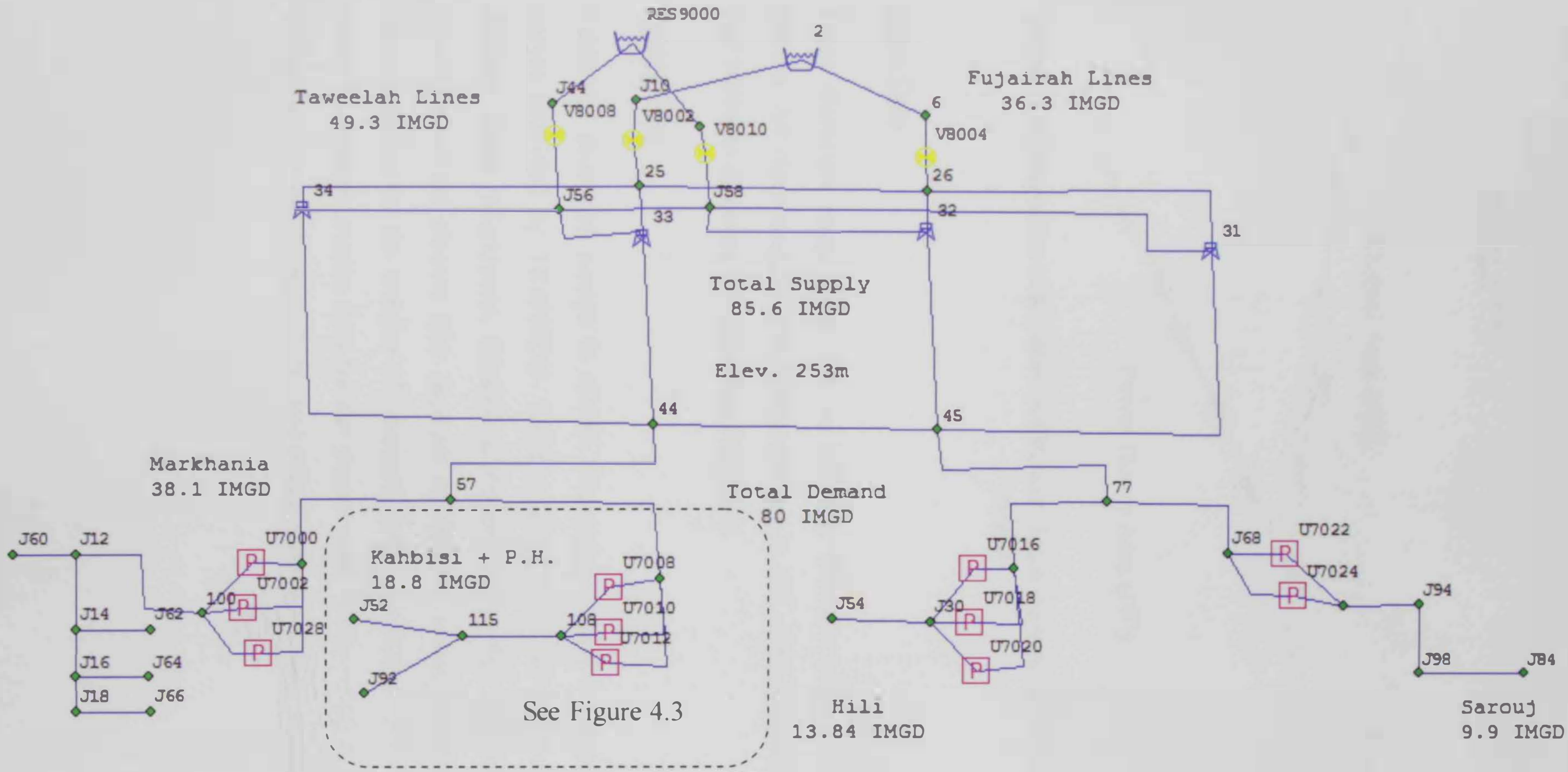


Figure 4.2: Proposed network with junctions labels as shown in the simulation model

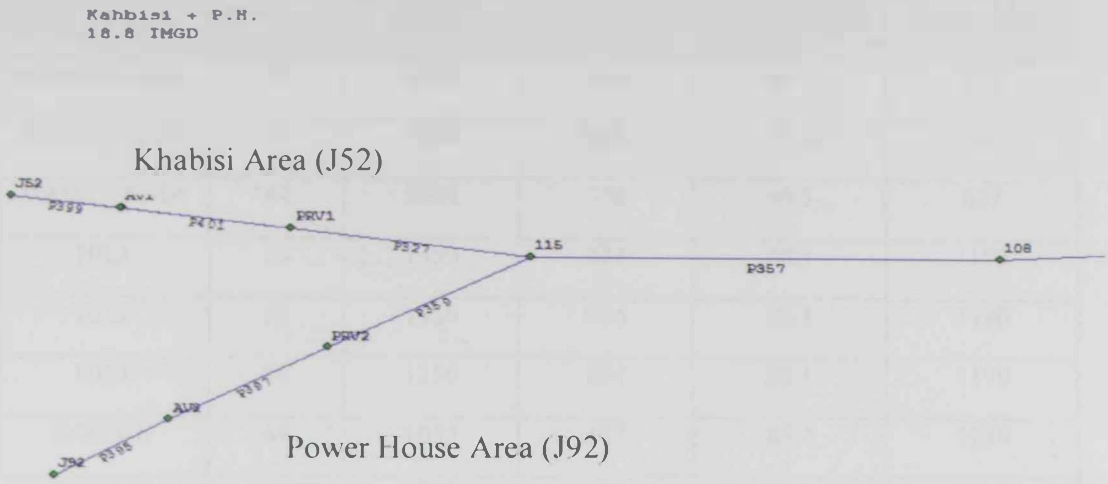


Figure 4.3: Magnifier of Khabisi and Power House connections as shown in the simulation model

Pipes Data

Pipes diameters range from 500 to 1400mm diameter. Pipes lengths ranges from 5 to 7500m. All pipes roughness coefficient value is 150. Check valves were installed on 1200 and 1400mm diameter transmission pipes only.

Pumps Data

Table 4.2 shows all pumps in AARS. The pump curves were defined as multiple point curves provided by TRANSCO. Refer to Figure 5.1 for daily water demands for four demand zones (Markhania, Khabisi & Power House, Hili and Sarouj). Pump Operation Control start and closure time depends on demand change during the day. Pump initial status depends on the number of operated pumps to deliver the required flow. Table 4.3 shows the pumps curves data for the pumps used in the simulation. All pumps in AARS station have an elevation of 253m and 600mm diameter with multiple point curve type.

Table 4.2: Pumps data

ID	Pump Curve	Design Head (m)	Design Flow (m3/hr)	Power (kW)	Efficiency (%)	Speed (RPM)
U7000	MARKHANIA	47	3938	558	90.3	827
U7002	MARKHANIA	47	3938	558	90.3	827
U7028	MARKHANIA	47	3938	558	90.3	827
U7016	HILI	56	1350	234	88.1	1190
U7018	HILI	56	1350	234	88.1	1190
U7020	HILI	56	1350	234	88.1	1190
U7022	SAROUJ	69	1053	187	85.4	1158
U7024	SAROUJ	69	1053	187	85.4	1158
U7008	KHABISI	47	3938	558	90.3	827
U7010	KHABISI	47	3938	558	90.3	827
U7012	KHABISI	47	3938	558	90.3	827

Table 4.3: AARS pumps curves data

Hili		Sarouj	
Flow (l/s)	Head (m)	Flow (l/s)	Head (m)
0	68	0	70
138.89	65	222.22	65
291.67	60	291.67	60
388.89	55	333.33	55
458.33	50	388.89	50
Khabisi		402.78	43
Flow (l/s)	Head (m)	Markhania	
0	73	Flow (l/s)	Head (m)
166.67	70	0	59
333.33	65	462.78	56.67
416.67	60	611.11	55
500	55	944.44	51
527.78	50	1094	47
569.44	45	1333.33	39
		1402.78	35

Table 4.4: Simulation options, simulation time and simulation report data

Simulation Options	
Pressure Unit	meter
Flow Unit	L/s
WQ tolerance	0.001
Viscosity	1×10^{-6} m^2 /s
Extended Run	24 hr
Vapor pressure	-1 bar
Diffusivity	1.21×10^{-9} m^2 /s
Head Loss Equation	Hazen Williams
Un-balances	Stop (√) or Continue
Accuracy	0.001
Number of Trials	40
Specific Gravity	1
Rule Control	√
Simulation Time	
Duration	24 hr
Clock Start Time	0:00 midnight
Pattern Start Time	0:00 midnight
Report Start Time	0:00 midnight
Hydraulic Time Step	0.5 hr
Pattern Time Step	0.5 hr
Quality Time Step	N/A
Report Time Step	0.5 hr
Rule Time Step	0.5 hr
Simulation report	
Hydraulic Status	full
Link Report	all
Node Report	all
Generate Network Summary Table	√
Generate Warning Messages	√

Simulation Rule Control

Currently, AARS delivers their daily water requirements to Al Qidfa Pumping station in Fujairah city. AARS water requirements varies in separate connections (Not shown in this simulation) where VIP farms are supplied with water in addition to their wells when required. Due to non-typical operations throughout the month and even the week, and in order to match the reality as much as possible, some controls were added in the simulation so that it can run properly without simulation errors. The main objective of these controls is to make sure that in case of water demand downstream AARS is reduced (i.e. VIP farms is not taking water from AARS), AARS reservoirs will not waste the water and will send a signal to Qidfa Pumping Station to reduce their flows. Another use of these controls is that the supply from Fujairah lines should not be constant and should match the AARS water demand patterns. This approach is cost effective as it helps in reducing the number of the extra huge reservoirs in AARS. Table 4.5 addresses the rule controls used in the simulation.

Table 4.5: Rule control used in simulation

Type	Keyword	Rule Statement
If: Premise	IF	Tank 34 level >= 4.000
And/Or: Premise	AND	Tank 33 level >= 4.000
And/Or: Premise	AND	Tank 32 level >= 4.000
And/Or: Premise	AND	Tank 31 level >= 4.000
Then: Action	THEN	Setting Valves V8008 Setting Is 600.00
And: Action	AND	Setting Valves V8010 Setting Is 600.00
Else: Action	ELSE	Setting Valves V8008 Setting Is 1400.00
And: Action	AND	Setting Valves V8010 Setting Is 1400.00

Define Elements' Data

This section defines elements data. Both reservoirs are fixed head reservoirs. The reservoir representing Taweelah lines has 262m head while the reservoir representing Fujairah lines has 318m.

4.1.2 InfoSurge Setup

This section addresses all data required to setup the InfoSurge model of the system under study.

- Define Taweelah and Fujairah lines
- Demand Change: refer to Figure 5.1
- Air Valves:
  - Inflow Diameter = 100 mm
  - Outflow diameter = 100 mm
  - Initial air volume = 0 m^3
- Pressure Relief Valves: refer to Tables 5.9 and 5.10.
- Run Manager
  - Select Active Standard (√) for InfoWater steady state and EPS analysis.
  - Select Active Surge (√) for InfoSurge transient analysis with the following
- Global Wave Speed : 1066 m/sec
- Cavitation Head : -10 m (vapour pressure)
- Pressure Sensitive Demand (√)
- Pipe Segment Length Tolerance : 10
- Simulation Run Duration (Seconds) : 2000
- Exit head: Varies depending on the run situation.
- Monitoring Pressure Head Range: Minimum = – 100m & Maximum = + 400m
- Intrusion Calculation Type: No Intrusion Calculation.
- Monitoring Node: J-95 and J 52
- Output Report is generated for all elements either graphically or numerically after a hydraulic run.

<u>Estimation of the Wave Speed (a)</u>

It is important to know, while estimating the wave speed (a), that small quantities of air result in reducing the wave speed and lead to non-conservative estimation of a. In this study the presence of trapped air in the pipe is ignored.

Four main factors should be considered while estimating the wave speed. These factors are:

- Pipe Material (Young's Modulus), Ductile Iron pipes with
- Pipe Restrainment (pipe support and anchors)
- Pipe Properties (Di / Wall Thickness)
- Type of Liquid (Bulk Modulus),

The wave speed (a) was calculated based on the following assumptions:

- Ductile iron pipe
- 900mm diameter (since the long pipes in the system have this diameter)
- Wall thickness = 12 mm
- Young's Modulus = 172 GPa for ductile iron
- Poisson's Ratio (μp) = 0.3.
- KR of 0.91 was calculated from the equation $KR = 1 - \mu p^2$ that considers a pipe anchored against longitudinal movement from both sides (The assumed type of constraints provides a conservative estimation of the wave speed)

As concluded from , and in order to use Figure 2.8, Pipe Properties (Di / Wall Thickness) was determined and found to be 75 (900/12) which corresponded to wave speed 1066 m/sec. This wave speed value (a) was used in the simulation for all pipes and branches in order to avoid any numerical instability as recommended [4].

4.2 Simulation of Current System and Model Verification

Certain arrangements (waterfalls) are used at the reservoirs inlet to eliminate the high-pressure values during the low demand periods. As per TRANSCO, throttle valves are installed downstream each distribution pumping station in different areas to maintain the residual pressure in the distribution networks at 3 bars.

The objective of this section is to verify the model against the real life situation. Even though no field measurements were available or could have been taken during the study, simulation results of the current system were verified verbally with the TRANSCO operating engineers for verification purposes. Figure 4.4 represents the existing pressures upstream the pumping station reservoirs. These values were found reasonable to TRANSCO operators indicating that the developed model is reasonably calibrated.

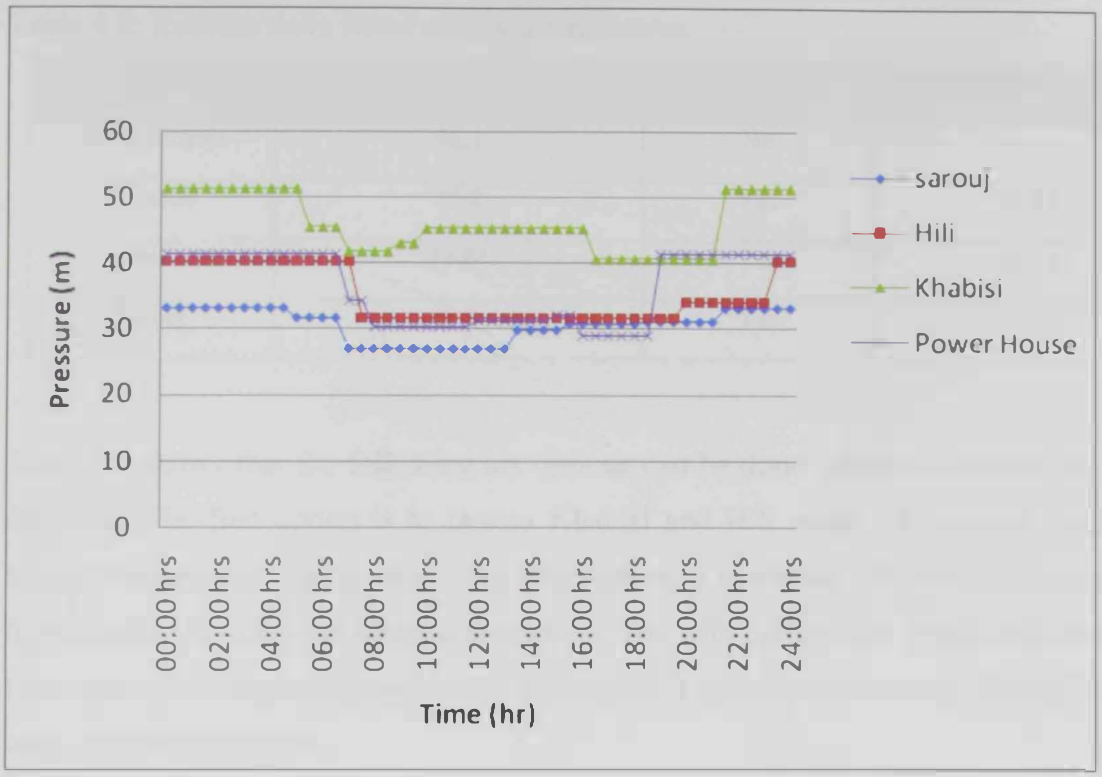


Figure 4.4: Existing outlet pressures of the area pumping stations

4.3 Bypass Proposal

The objective of this research is to study the possibility of bypassing AARS and distribution pumping stations such as Al Markhania, Khabisi, Hili and Sarouj. Two main criteria are considered to determine the area to be bypassed. These two criteria are maximum possible water supply and maximum power saving, which will eventually lead to reduce the operation and maintenance costs drastically.

Criteria One: Maximum bypassed supply

Based on 36.6 IMGD water supply from Fujairha lines, it can be realized that only three areas out of four can be bypassed (as shown in Table4.6), knowing that the average daily water supply from Fujairah lines is 36.6 IMG.

Table 4.6: Average daily water supply for each area

Area	Water Demand (IMGD)	Possibility	Extra Water (IMGD)
Al Markhania	38.1	No	---------
Khabisi	18.8	Yes	17.80
Hili	13.84	Yes	22.76
Sarouj	9.9	Yes	26.70

Table 4.6 shows that the following six options can be done based on average daily water demands. The first option is to bypass Khabisi and Hili areas. The second option is to bypass Khabisi and Sarouj areas. The third option is to bypass Hili and Sarouj areas. The fourth option is to bypass Khabisi area alone. The fifth option is to bypass Hili area alone. Last option is to bypass Sarouj area alone. Figure 4.5 shows a schematic sketch for AARS with water requirements.

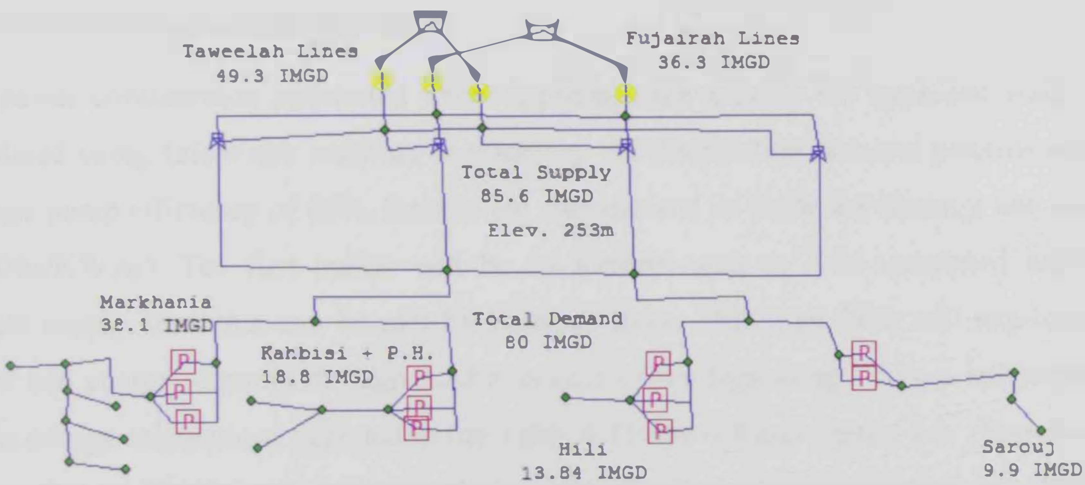


Figure 4.5: Schematic sketch for AARS with water requirements

Considering peak distribution flows based on supply patterns as given in Figure 3.2 in chapter 3 and knowing that the maximum supply flow from each Fujairah line is about 1000 l/s, more findings are concluded as listed in Table 4.7.

Table 4.7: Peak water supply flows for each area and possibility of bypassing

Area	Peek Flow (L/S)	Peak hours	MAX Flow Both Fujairah Lines (L/S)	Bypass Possibility	Extra Flow During P.H. (L/S)
AL Markhania	2,956	7-9	2000	No	N/A
Khabisi	1,702	9-10	2000	Yes	298
Hili	1,303	7:30 - 20	2000	Yes	697
Sarouj	805	7 – 13:30	2000	Yes	1195

Table 4.7 indicates that the flows that can be met by supply from Fujairah lines during peak hours are associated with three options only as follows:

1. Khabisi area alone
2. Hili area alone
3. Sarouj area alone.

Criteria Two: Maximum energy saving

The power consumption associated with supplying each area by the bypassed water was calculated using InfoWater software considering the distribution demand patterns and an average pump efficiency of 80%. Results are summarized in Table 4.8 (Energy unit cost is 0.15Dhs/KW.hr). The first option will be Al Khabisi area as it is associated with the highest supply flow that can be met by Fujairah lines. This high flow will require high power and energy to pump through AARS in case of not bypassing. This is substantiated by the energy calculations depicted in the Table 4.8Table 4.8 and Figure 4.6. Therefore, it is clear that Al Khabisi area will not only have the maximum flow utilization, but also will have the maximum savings in energy and operation costs.

Based on above, the scenario of bypassing Al Khabisi area (which includes Kahbisi and P.H Pumping stations) is selected. The daily energy costs are plotted in Figure 4.6 for further clarifications.

Table 4.8: Power consumption and power cost for different areas

Area	AARS Pumping Station				Zone Pumping Station				Total		
	Pump Power Consumption (KW)	Motor Power Consumption (KW)	Daily Energy r Consumption (KW.hr)	Daily Enrgy r Cost (Dhs)	Pump Power Consumption (KW)	Motor Power Consumption (KW)	Daily Energy Consumption (KW. hr)	Daily Energy Cost (Dhs)	Power (KW)	Daily Energy (KW.hr)	Daily Energy st (Dhs)
Markhania	1164.8	1456.0	34944.0	5241.6	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Khabisi & P.H.	600.2	750.3	18006.9	2701.0	331.1 + 88.2 = 419.3	524.2	12579.9	1887.0	1274.5	30586.8	4588.0
Hili	440.8	551.0	13224.3	1983.6	255.6	319.5	7667.4	1150.1	870.5	20891.7	3133.8
Sarouj	309.6	387.1	9289.2	1393.4	172.1	215.2	5163.6	774.5	602.2	14452.8	2167.9

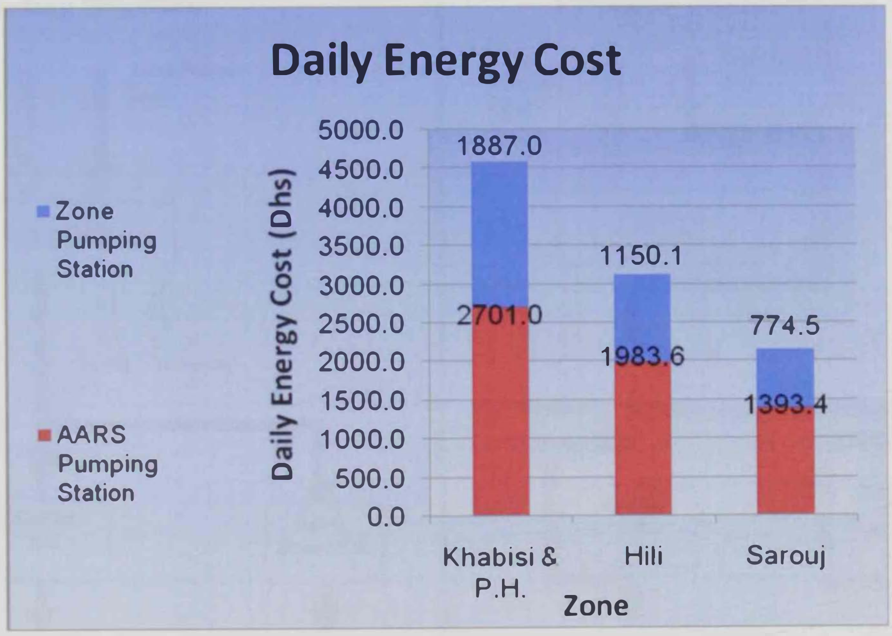


Figure 4.6: Energy cost for pumping stations for concerned areas at AARS

4.3.1 Bypass Scenario

Bypassing complete two booster pumping stations for one area at the same time for 24 hours will be considered. The first booster pumping station is inside AARS and the second booster pumping station is located at the area itself. Excess water will be pumped in AARS reservoirs especially during low demand/supply hours.

The advantage of this scenario is that two pumping stations will not be required and they can be utilized somewhere else. Moreover, the operation and maintenance costs of these pumping stations will be saved as well.

This scenario will be further studied in Chapter 5 to assure that potential undesired transient conditions are avoided. Figure 4.7 represents a sketch for the proposed bypass scenario.

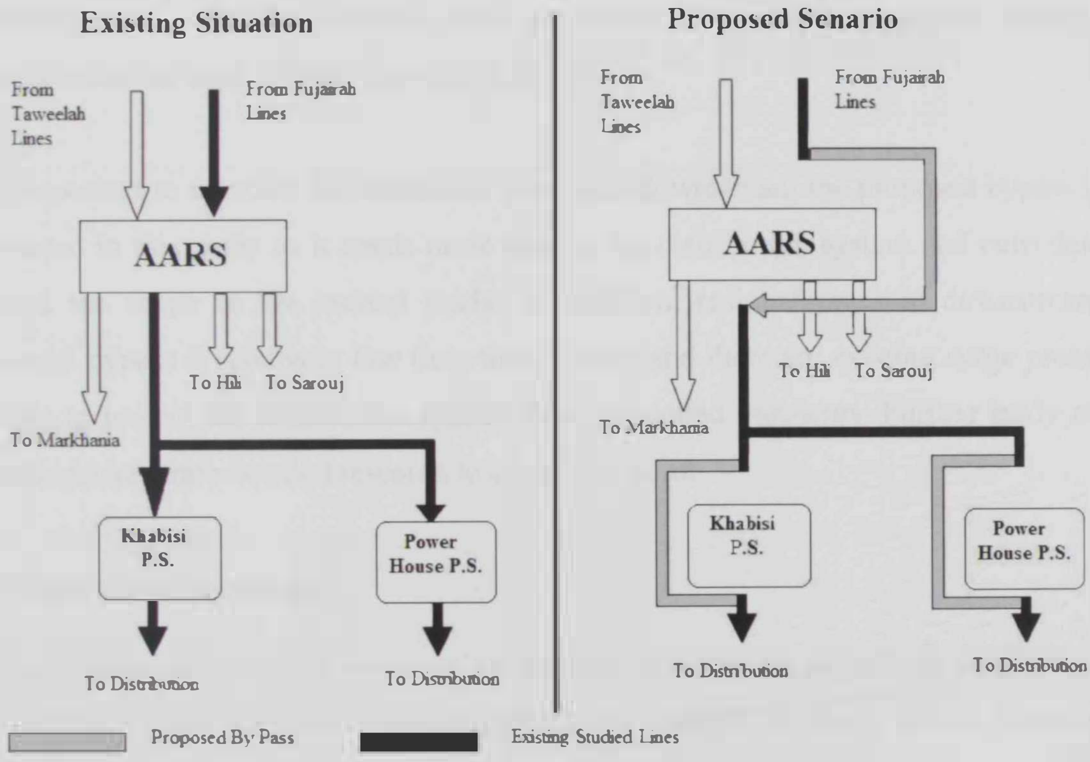


Figure 4.7: Proposed bypass scenario

CHAPTER V

EVALUATION OF THE PROPOSED BYPASS

In chapter 5, the hydraulic transients associated with a number of possible operational scenarios of the proposed bypass are simulated and evaluated. The findings are recorded and analyzed. These scenarios are classified into two classes as follows:

- Normal valve closure/opening scenarios.
- Abnormal, rare and unplanned scenarios such as rapid valve closure/opening, sudden valve closure during peak flows and pipe break.

Variables like safe valve closure/opening times, system head recovery times, and system head variations are evaluated in the considered scenarios given the pressure ratings of the installed pipes. Finally, suitable surge protection devices are proposed, taking into consideration the ones already installed in the system.

It is important to mention that transients generated downstream the proposed bypass is not addressed in this study as it needs more data in the distribution system and considered to beyond the scope of the current study. In addition, residual pressure downstream the proposed bypass is relatively low (less than 3 bars) and there are existing surge protection devices to protect the distribution system from generated transients. Further study can be done under separate detailed research to cover this point.

5.1 Operation Scenarios

Normal operation scenarios representing the typical operating conditions of Khabisi and Power House areas were simulated considering the possible transients events generated. In addition, abnormal operation scenarios representing the rare and unplanned conditions are simulated. Each scenario is explained in detail in the coming sections with its results and findings.

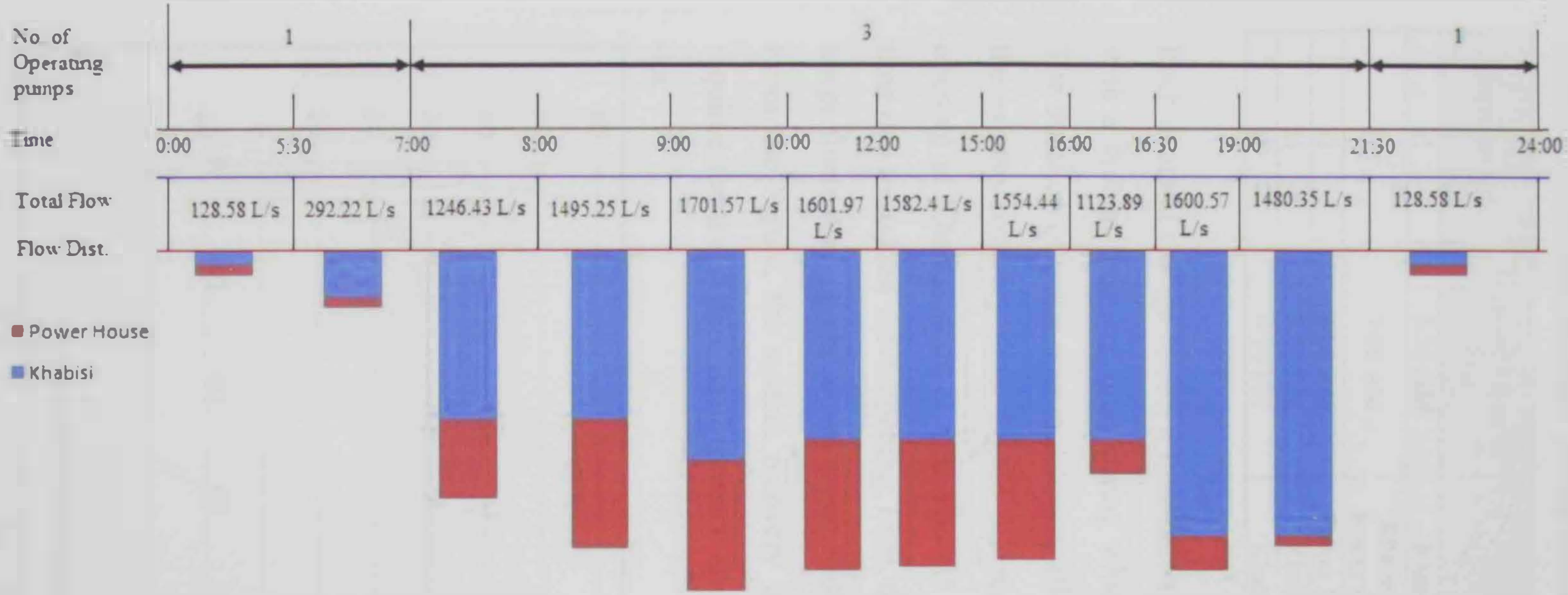
Valve opening or closure poses some hydraulic effects on the system. The increase in nodal demands can be represented by valve opening while the decrease is represented by valve closure. The pressure head rise in the system should not exceed the pipeline rated pressure and it should be maintained within its limits. In general, transients associated with valve closure are more severe than those associated with valve opening. Additionally, in case there were no sufficient surge protective devices available such as air valves and pressure relieve valves, sudden drop in pressure may cause vacuum in the system. Other than that, the water flow rates may exceed the erosion velocity and may cause damage to the pipelines internal linings which may further develop to corrosion that destructs the pipe metals. Figure 5.1 shows the operating pumps at Khabisi pumping group in AARS during different water demand periods.

The high water demand occurs daily from 7 AM until 9:30 PM, whereas peak flows occurs from 9:00 AM until 10:00 AM, which corresponds to 3 pumps operating at AARS. The low water demand starts daily, at 9:30 PM until 7:00 AM, which corresponds to one pump operating at AARS.

In the studied system, four major water demand changes (I2, I5, R4 and R6) out of eleven are selected. Normal operation is associated with valve opening or closing to accommodate these water demand changes. However, abnormal operation is associated with sudden valve opening or closing.

5.2 Normal Operation

In normal operations, gradual valve openings or closings are adjusted in order to accommodate water demand changes during the day by reaching the desired water flow rates. These demand changes occurred at the nodes representing the distribution system where basic demands and the demand patterns are considered. This information represents the daily programs provided by (AADC). In this section, the safe valve opening/closing times will be determined in addition to the system recovery time and system head variation.



Time	Flow (L/s)											
	00:00 hrs - 05:30 hrs	05:30 hrs - 07:00 hrs	07:00 hrs - 08:00 hrs	08:00 hrs - 09:00 hrs	09:00 hrs - 10:00 hrs	10:00 hrs- 12:00 hrs	12:00 hrs - 15:00 hrs	15:00 hrs - 16:00 hrs	16:00 hrs - 16:30 hrs	16:30 hrs - 19:00 hrs	19:00 hrs - 21:30 hrs	21:30 hrs - 24:00 hrs
Khabisi	78.26	241.89	846.63	846.63	1,052.95	953.35	953.35	953.35	953.35	1,430.02	1,430.02	78.26
Power House	50.32	50.32	399.8	648.62	648.62	648.62	629.05	601.09	170.54	170.54	50.32	50.32
Total Flow	**128.58**	**292.22**	**1,246.43**	**1,495.25**	**1,701.57**	**1,601.97**	**1,582.40**	**1,554.44**	**1,123.89**	**1,600.57**	**1,480.35**	**128.58**
	I1	I2	I3	I4	R1	R2	R3	R4	I5	R5	R6	

I : Flow Increase
R : Flow Reduction

Figure 5.1: Operating pumps at different water demand periods

5.2.1 Scenarios of Demand Increases (Normal Valve Opening)

In the studied system and as shown in Figure 5.1, five water demand increases are observed during a typical day of operation exist as listed in Table 5.1.

Table 5.1: Demand increases during the day

Increase No.	Increased flow at time	Area	Flow change from (l/s)	Flow change to (l/s)	Flow change (l/s)
I1	5:30 AM	Khabisi	128.6	292.2	163.5
I2	7:00 AM	Khabisi and Power House	292.2	1246.4	954.2
I3	8:00 AM	Power House	1246.4	1495.3	248.8
I4	9:00 AM	Khabisi	1495.3	1701.6	206.3
I5	4:30 PM	Khabisi	1123.9	1600.6	476.7

Each demand change causes a fluctuation rise in the hydraulic head. This rise is associated with a system recovery time during which the system goes to its equilibrium condition. The duration of valve opening/closing affects the decreased/ increased hydraulic head and the system recovery time. Figure 5.2 represents is an impact example of sudden valve opening at Khabisi node at 4:30 PM (I5 event). It shows pressure head variation versus time due to sudden valve opening. First, the head drops to -10m then increases to 50m and keeps fluctuating up and down around 30m, which is the final head with the new demand. From the Figure, the system recovery time is estimated as 40 seconds. Such time is required for the system head to reach equilibrium and significant head variations dampen out.

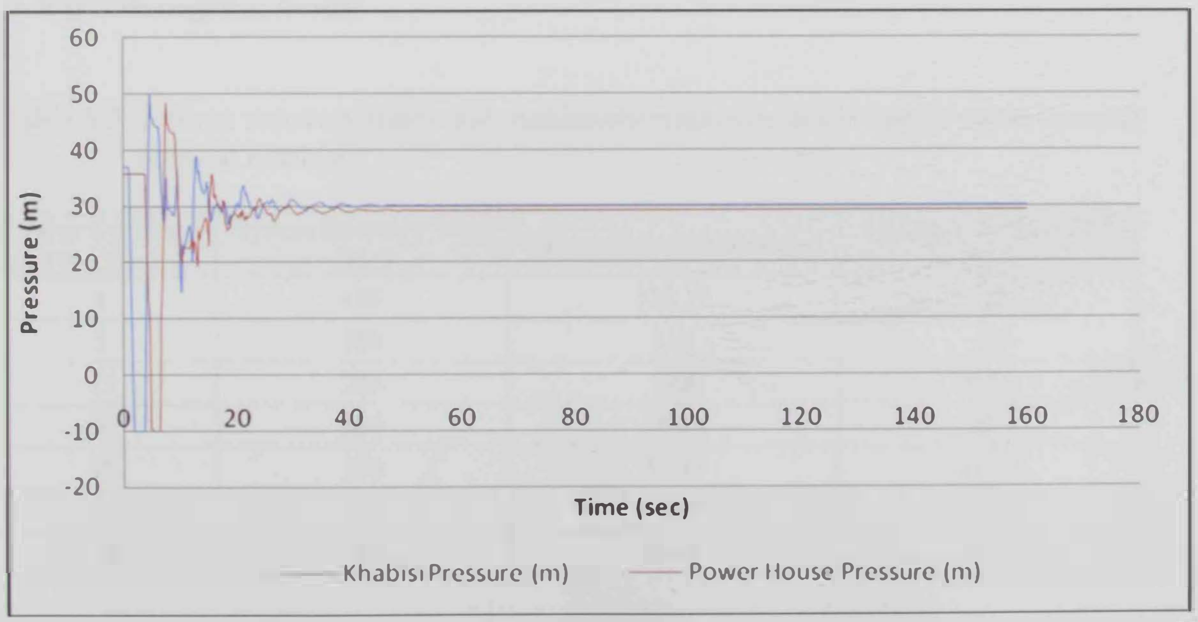


Figure 5.2: Nodal maximum head versus time for sudden valve closure at Khabisi

The most critical scenarios associated with normal valve opening are the largest two increases I2 and I5 where I2 involves 76.6% increase in flow (from 292 l/s to 1246 l/s), while I5 involves 29.8% increase in flow (from 1123.9 l/s to 1600.6 l/s). The two scenarios are simulated and studied.

Flow Increase at Khabisi and Power House at 7:00 AM (I2 Event)

Results for this scenario are summarised in Table 5.2 and Figure 5.3 for Khabisi and in Table 5.3 and Figure 5.4 for Power House.

For Khabisi area, the head right before the I2 event was 39.8m and dropped ultimately to 27.5m after a period of head fluctuations associated with the valve-opening event. Figure 5.3 indicates that the acceptable pressure rating of pipes and fittings of 16 bars is far above the maximum heads of all considered opening events. In addition, the transients generated from 1-second valve opening are within the pipeline pressure range (since zero second of valve opening/closure is not accepted by the software). However, due to long system recovery time (600 seconds), major instability in the system could occur such as gaskets rupture, bolts loose, disconnection of spigots/sockets, etc. Such unstable operation may cause water leakage in the system. Therefore, a minimum of 15 seconds is needed as an acceptable valve opening time at Khabisi after which the time needed to absorb the head fluctuations becomes more or less constant. Opening times less than that will also in undesirable negative heads.

Table 5.2: System recovery times and maximum-minimum heads versus valve opening times at Khabisi

Valve Opening Time (sec)	System Recovery Time (Sec)	Maximum Nodal Head (m)	Minimum Nodal Head (m)
1	600	123.71	-10
5	580	113	-10
15	250	39.8	8.05
30	200	39.81	20.7
60	170	39.81	24.11
90	145	39.81	24.95
120	100	39.81	25.76

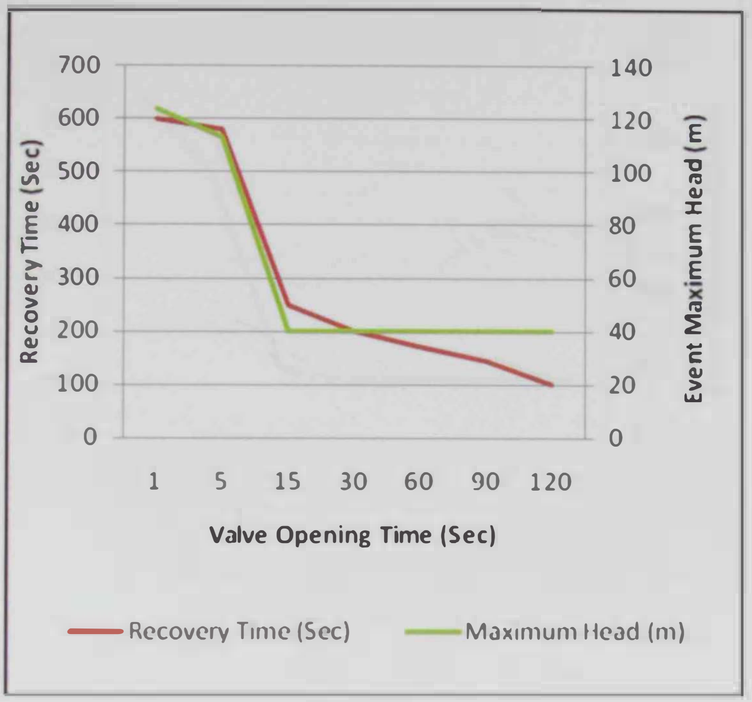


Figure 5.3: System recovery times and maximum head versus valve opening times at Khabisi

Table 5.3 lists the recovery times and maximum/minimum heads associated with different valve opening times at the Power House. The head before I2 event was 39.7m then dropped to 21.0 m after the I2 event at the Power House area. Figure 5.4 indicates that the acceptable valve opening time at Power House is at least 60 seconds within which the system head reaches equilibrium and the highly negative heads are eliminated.

Table 5.3: System recovery times and maximum-minimum heads versus valve opening times at Power House

Valve Opening Time (sec)	System Recovery Time (Sec)	Maximum Nodal Head (m)	Minimum Nodal Head (m)
1	600	230	-10
5	580	178.74	-10
15	250	44.71	-10
30	200	39.67	-3.94
60	170	39.67	0.08
90	145	39.67	13.6
120	100	39.67	16.08

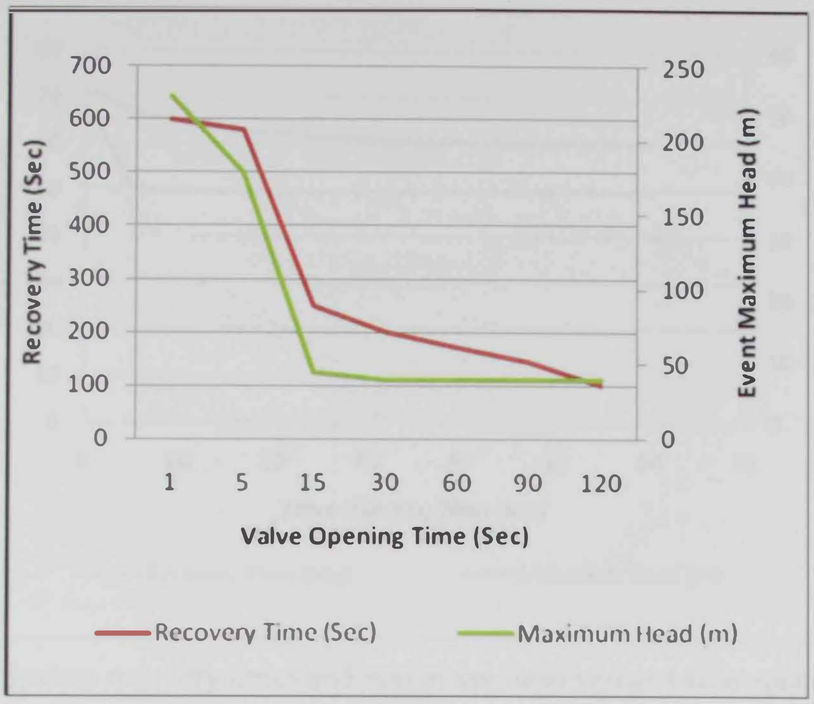


Figure 5.4: System recovery times and maximum head versus valve opening times at Power House

Flow Increase at Khabisi at 4:30 PM (I5 Event)

Results for this scenario are summarised in Figure 5.4 and Figure 5.5. The head before I5 event was 37.1m while the head after the I5 event was 30.2 m. Figure 5.5 shows that the acceptable valve opening time at Khabisi is at least 15 seconds within which the system head reaches settled equilibrium and the high negative heads diminish. As a conclusion of normal valve opening results considering the worst-case scenarios, it is found that pressures the acceptable pipe rating of 160m as well as highly negative pressures (reaching cavitation limit) generate in the system for small opening times. It should be noted the system is usually capable of accommodating some vacuum associated with reasonable negative heads via adequately sized air valves.

Table 5.4: System recovery times and maximum head versus valve opening times at Khabisi

Valve Opening Time (sec)	System Recovery Time (Sec)	Maximum Nodal Head (m)	Minimum Nodal Head (m)
0	71	50.21	-10
5	67	37.96	7.36
15	63	37.11	23.83
30	61	37.11	26.97
60	60	37.11	28.57

Figure 5.5: System recovery times and maximum head versus valve opening times at Khabisi

5.2.2 Scenarios of Demands Drop (Normal Valve Closure)

Figure 5.1 depicts six water demand reductions occurring during the day as listed in the Table 5.5.

Table 5.5: Demand reductions during the day

Reduction No.	Reduced flow at time	Area	Flow change from (l/s)	Flow change to (l/s)	Flow change (l/s)
R1	10:00 AM	Khabisi	1701.6	1602	99.6
R2	12:00 PM	Power House	1602	1582.4	19.6
R3	03:00 PM	Power House	1582	1554.4	28
R4	04:00 PM	Power House	1554.4	1123.9	431
R5	07:00 PM	Power House	1600.6	1480.4	120.2
R6	09:30 PM	Khabisi & Power House	1480.4	128.6	1351.8

The most critical scenarios associated with normal valve closing are the largest two reductions (R4 and R6). R4 event involves 27.7% decrease in flow (from 1554.4 l/s to 1246 l/s) while R6 event involves 91.3% decrease in flow (from 1480.4 l/s to 128.6 l/s). These two scenarios are simulated and evaluated below.

Flow Reduction at Power House at 4:00 PM (R4 Event)

A number of valve closure times at Power House were simulated and evaluated. The results are summarised in Table 5.6 and Figure 5.6. The head right before R4 event was 20.7m and increased to 38.2m as a result of demands increase in the R4 event. Figure 5.6 indicates that the valve closure time at Power House should not be less than 60 seconds to eliminate the high negative heads and to allow the recovery time reaching equilibrium.

Table 5.6: System recovery times and maximum-minimum heads versus valve closure times at Power House

Valve Closure Time (sec)	System Recovery Time (Sec)	Maximum Nodal Head (m)	Minimum Nodal Head (m)
0	150	194.07	-10
5	138	190.10	-10
15	130	85.78	17.71
30	124	58.49	20.74
60	120	46.88	20.74
90	120	44.19	20.74

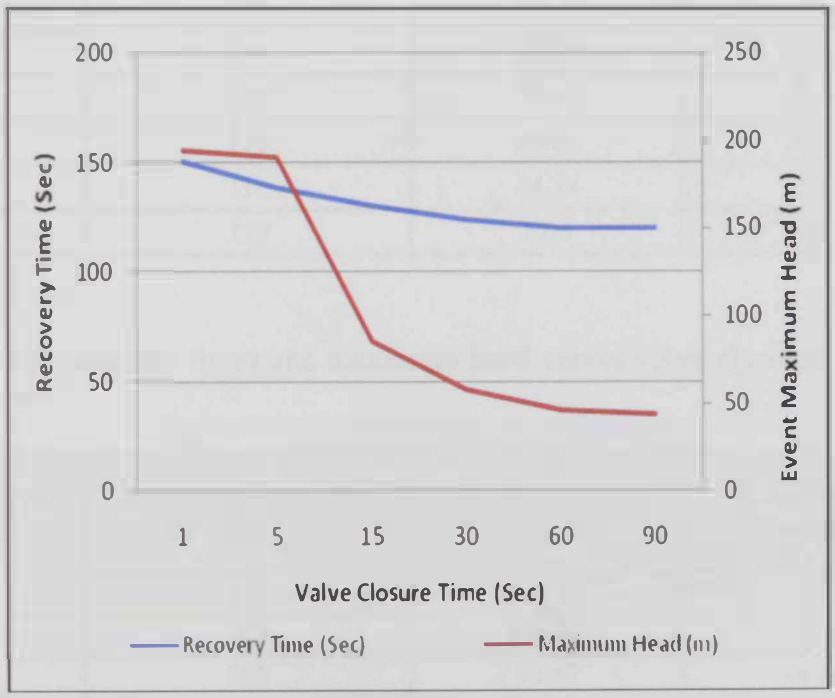


Figure 5.6: System recovery times and maximum head versus valve closure times at Power House

Flow Reduction at Khabisi and Power House at Duration 9:30 PM (R6 Event)

Table 5.7, Table 5.8, Figure 5.7 and Figure 5.8 document the results for this event. For Khabisi area, the head before R6 event was 35.2m and increased to 48.7m due to demands drop during the R6 event. Figure 5.7 shows that the acceptable valve closure time at Khabisi is at least 60 seconds for the same reasons given before.

Table 5.8 and Figure 5.8 report the recovery time of maximum/minimum heads associated with valve closure at Power House for R6 event. The head before R6 event was 39.7m and ultimately rose to 48.5m after the closure event. Figure 5.8 indicates that the acceptable valve closure time at Khabisi should be at least 60 seconds; again for the same reasons discussed before.

Table 5.7: System recovery times and maximum head versus valve closure times at Khabisi

Valve Closure Time (sec)	System Recovery Time (Sec)	Maximum Nodal Head (m)	Minimum Nodal Head (m)
1	155	193.11	-10
5	130	118.06	13.85
15	121	76.15	35.16
30	116	60.63	35.16
60	114	54.39	35.16
90	110	52.5	35.16

Table 5.8: System recovery times and maximum head versus valve closure times at Power House

Valve Closure Time (sec)	System Recovery Time (Sec)	Maximum Nodal Head (m)	Minimum Nodal Head (m)
1	150	296.73	-10
5	140	168.41	-10
15	133	76.97	30.63
30	128	60.58	35.05
60	123	54.32	35.05
90	120	52.39	35.05

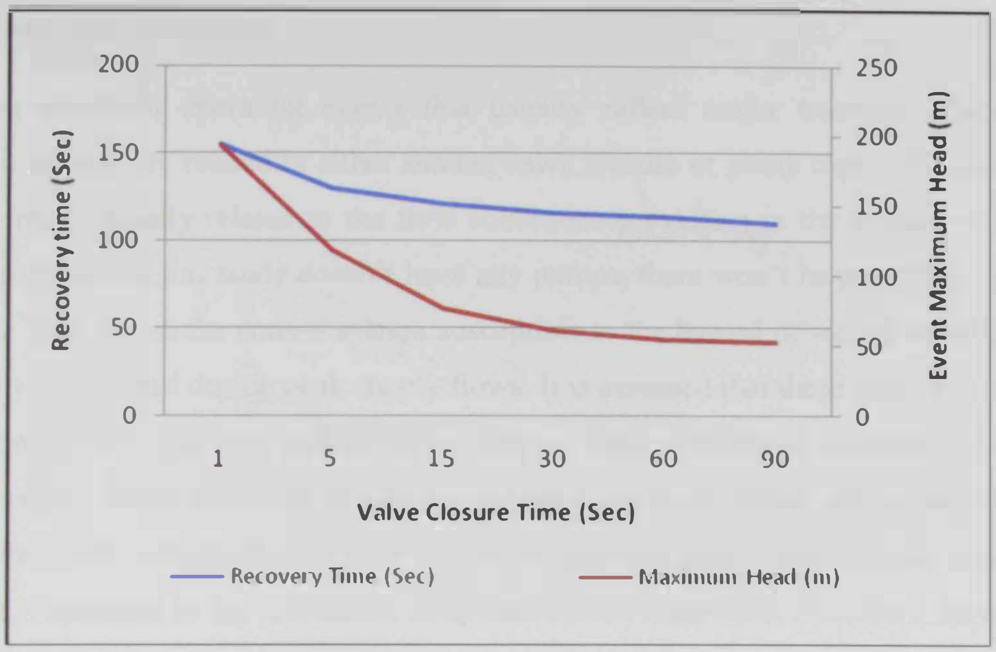


Figure 5.7: System recovery times and maximum head versus valve closure times at Khabisi

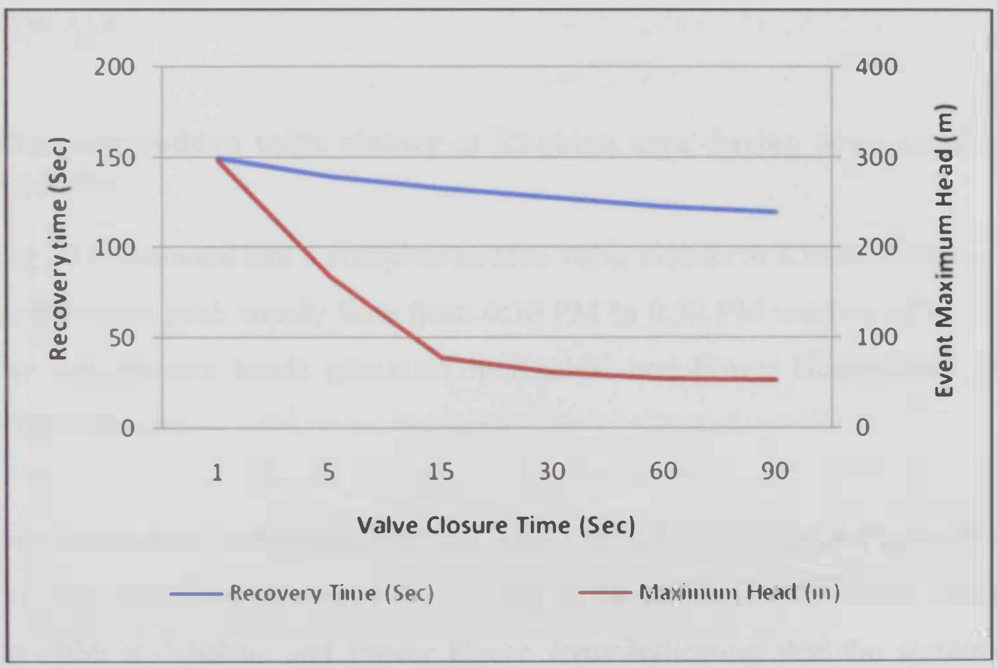


Figure 5.8: System recovery times and maximum head versus valve closure times at Power House

5.3 Abnormal Operation

Common abnormal operating events that usually reflect major transient effects on a hydraulic system are related to either sudden valve closure or pump trips. The severity of such events is usually related to the flow conditions prevailing in the system. Since the bypass proposed in this study doesn't have any pumps, there won't be pump trip events of concern. This leaves the current system susceptible to the hazard of sudden valve closures especially if occurred during peak supply flows. It is assumed that these cases are very rare to happen. In our case, two sudden valve closures were considered, simulated, analyzed, and discussed. These abnormal conditions represent the most critical abnormal conditions posing the most undesirable impacts on the proposed bypass where severe results can occur. It is assumed in the simulation setup that valve closure will occur in 1 second. The considered abnormal operating conditions are as follow:

- Complete sudden valve closure at Khabisis area during peak supply flow at 5:30 PM
- Complete sudden valve closure at Power House area during peak supply flow at 9:30 AM

5.3.1 Complete sudden valve closure at Khabisis area during peak supply flow at 5:30 PM

In this case, it is assumed that a complete sudden valve closure at Khabisi area occurred at 5:30 PM where the peak supply flow from 4:30 PM to 9:30 PM reached 1430 l/s. Figure 5.9 shows the pressure heads generated in Khabisi and Power House areas and their fluctuations with time

The Figure shows that the maximum system head reached to 313.2.m at Power House area that is far the maximum allowable pipe rating of 16 bars. The minimum system head reched to -10m at Khabsis and Power House areas indicating that the system will be subjected to severe cavitation problems.

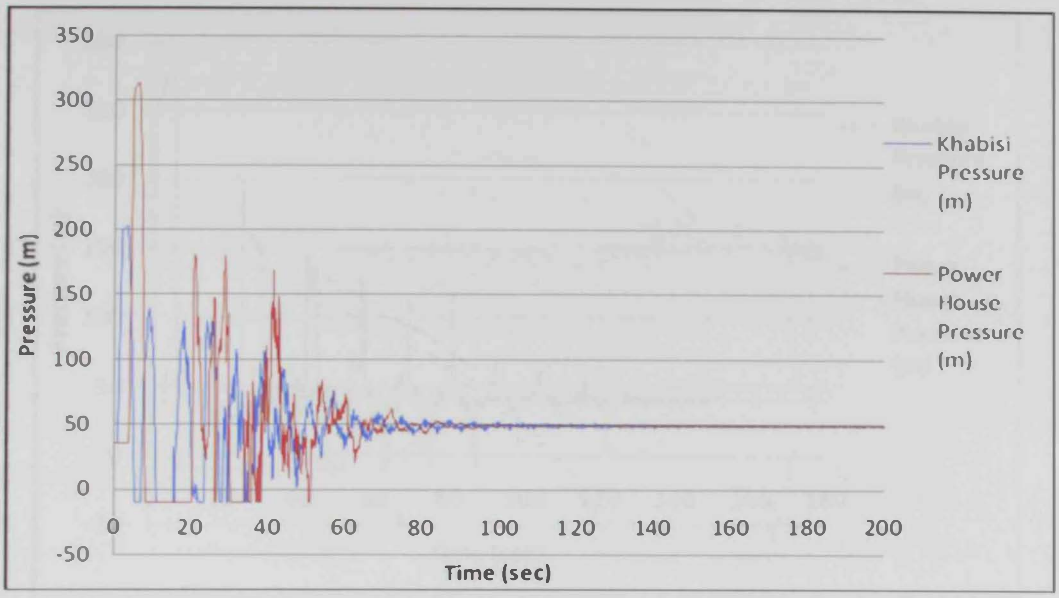


Figure 5.9: Generated pressure in Khabisi and Power House areas due to sudden complete valve closure in Khabisi

5.3.2 Complete sudden valve closure at Power House area during peak supply flow at 9:30 AM

In this case, it is assumed that complete sudden valve closure at Power House area during the peak supply flow from 8:00 AM to 12:00 PM occurred at 9:30 AM where the supply flow reached its peak levels of 648.62 l/s. Figure 5.10 shows the pressure generated in Khabisi and Power House areas associated with this event. The maximum system head reached to 276m at Power House area while the minimum system head reched to -10m at Power House area also.

The results report unacceptable high and low pressures that may cause water leakage in the system other than pipe breaks and erosion. In order to maintain the system pressure within the safe limits with continuous supply to end users, surge protection devices are needed and will be evaluated herein. Section 5.4 discusses in detail the protection scenario for the proposed bypass.

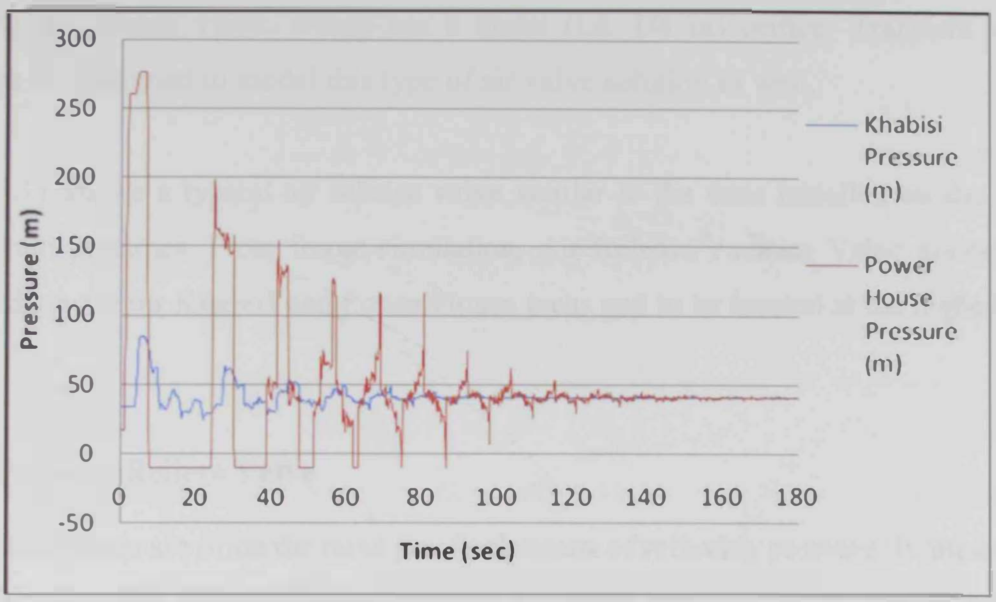


Figure 5.10: Generated pressure in Khabisi and Power House areas due to sudden complete valve closure in Power House

5.4 Proposed Surge Protection Devices

In this section, surge protection devices and their effects on maintaining the system pressure within the desirable ranges will be evaluated. This includes Air Valves, PRVs, and Standpipes or Surge Tanks. In reality, flow control valves were installed on branches that supply water to the consumers in order to manage the water supplies. The operation of these valves generates transients in the system. These transients are sensed from the immediate pulse of the pressure rise at the source branch. Then the pressure wave moves quickly with sound speed to the entire system. At this time, the best protection action to be taken is to reduce the pressure rise at the source by installing protection devices such as PRVs or Surge Vessels. The following is a brief description of common surge protection devices employed in different hydraulic systems.

5.4.1 Vacuum Breaker Valve

The Vacuum Breaker (VB) has similar components to the anti-slam device, except that the VB disc is held closed by a spring while the anti-slam disc is held open. Hence, the vacuum breaker cannot expel air; it only admits air to prevent the formation of a vacuum pocket. This keeps the pipeline at a positive pressure and reduces the surge associated with a column separation. In essence, a large cushion of air is admitted and trapped in the pipeline after a pump trip. The air is then slowly released over a few minutes through the adjoining air release valve, which has a small (i.e. 1/4 in) orifice. Transient analysis programs are designed to model this type of air valve solution as well.

Figure 5.11 shows a typical air release valve similar to the ones installed on the 900mm and 600mm pipelines. From Surge simulation, Air Release/Vacuum Valve diameters are 100mm diameter for Khabisi and Power House areas and to be located at the highest points [13].

5.4.2 Pressure Relieve Valve

Surge relief valves are often the most practical means of relieving pressure. In these valves, a pressure surge lifts a disc allowing the valve to rapidly relieve water to the atmosphere or back to the wet well. Surge relief valves have the limitation that they may not open rapidly enough to dissipate surges in cases where column separation can occur. For the cases where the transient computer model predicts steep or rapid pressure surges, surge relief valves equipped with anticipator controls should be considered. A surge anticipator valve will open rapidly upon the sensing of a high or low-pressure event.

When a pump suddenly stops, the pressure in the header will drop the static pressure and trigger the surge anticipator valve to open. The valve will then be partially or fully open when the return pressure wave occurs. Anticipator valves typically open in less than five seconds, pass high low rates, and re-close slowly at the pump control valve closure rate (60-300 seconds). The sizing of surge relief valves is critical and should be overseen by transient analysis experts. Next section discusses the PRV Sizing.

PRV activates (open) when the pressure in the pipeline at sensing node (not necessary at the valve) exceeds the opening pressure value by ejecting water out through orifice to prevent excessive high-pressure surges. The opening time is the time where the valve goes from the start to fully open position. The PRV closes once the pressure at the sensing node drops lower than the closing pressure. The PRV can remain open for longer periods due to pressure fluctuation at the sensing node. Normally, the PRV ejects liquid into atmospheric pressure (or open surge tank) or into a pressurized region (closed surge tank). Figure 5.12 shows typical PRV [13].

5.4.3 Standpipes and Surge Tanks

Many types of surge relief equipments are used to safeguard pumping systems. For low-pressure systems, a standpipe open to atmosphere will relieve pressure almost instantly by exhausting water. For systems with higher pressure, the height of a standpipe would be impractical therefore a bladder-type accumulator or surge tank, with pressurized air over water, can be used to absorb shocks and prevent column separations as shown in Figure 5.13. For typical pumping systems, however, these tanks tend to be large and expensive and must be supplied with a compressed air system. When used, an additional fast-closing check valve is also needed to prevent surge tank water from reverting back through the pump. This is a common example of when both a pump control valve and a fast-closing check valve are installed. Furthermore, the surge tank creates extremely high deceleration rates (i.e. 25 ft/sec2), hence fast-closing check valves or check valves equipped with bottom-mounted oil dashpots should be used to prevent slamming [13].

Some disadvantages of Surge Vessels are found and can be summarized in the following points:

- More Expensive.
- Large size, which results in installation restrictions such as beautification of the city and unavailability of area.
- Regular maintenance requirements
- Need additional pressurized air system support in case of closed surge vessels type.

The ultimate solution could be PRVs since they are much cheaper, smaller in size and could be hidden in underground chambers, less maintenance and easy to install. As a conclusion, it is recommended to have a PRV at each branch (i.e. Khabisi and Power House) in addition to the AVs.

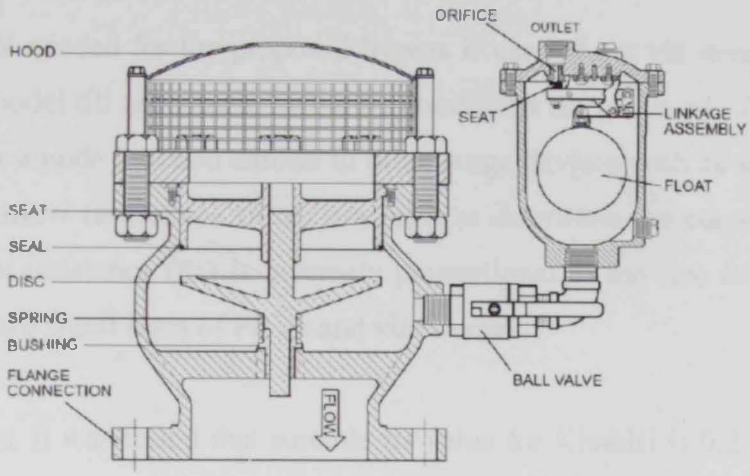


Figure 5.11: Typical air release/vacuum valve [13]

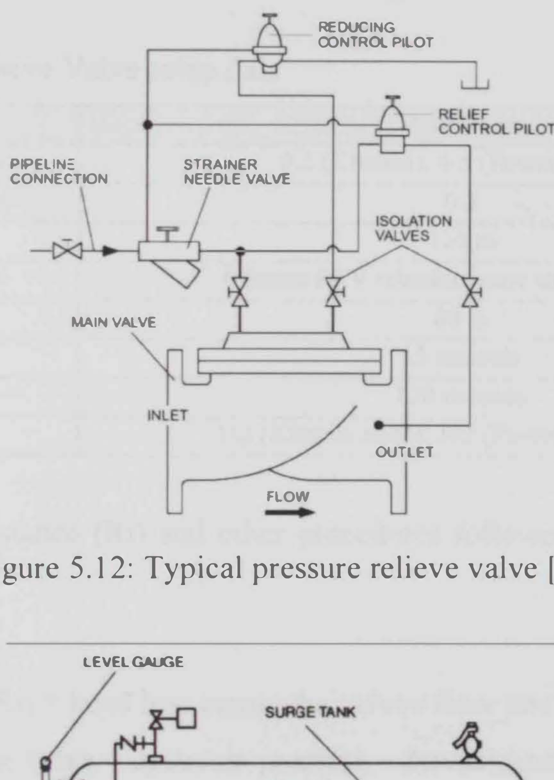


Figure 5.12: Typical pressure relieve valve [13]

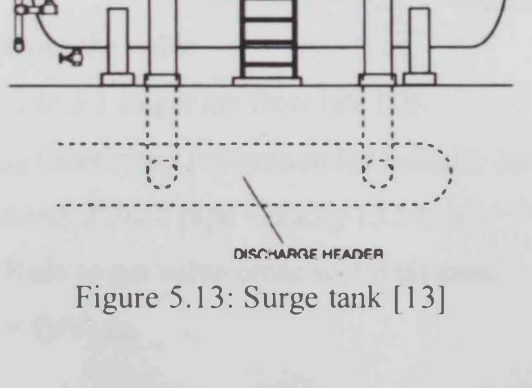


Figure 5.13: Surge tank [13]

5.5 PRV Sizing

Sizing of the PRV needed for the proposed bypass is carried out via iterative simulations using Infosurge model till acceptable hydraulic conditions are achieved. In the model, the PRV is defined as a node junction similar to other surge devices such as surge vessels and air valves. Ri (inflow resistance) values are used to determine the correct PRV sizes as described. Inflow resistance (Ri) is inversely proportional to the size of the PRV where large values produce small sizes of PRVs and vice versa.

After several trials, it was found that suitable Ri value for Khabisi is 0.2 while it was 0.5 for Power House when a set system pressure of 110m is maintained. Table 5.9 lists the data and parameters considered in the final setup of the considered PRVs.

Table 5.9: Pressure Relieve Valve setup data

Required data	Data Value
Inflow Resistance (Ri)	0.2 (Khabisi), 0.5 (Power House)
Outflow Resistance (Ro)	0.2
Opening Pressure	110 m
External Pressure	0 (since PRV releases water to atmosphere)
Closing Pressure	80 m
Opening Time	15 seconds
Closing Time	120 seconds
Sensing Node ID	J52 (Khabisi node), J92 (Power House node)

Formula of inflow resistance (Ri) and other procedures followed in sizing the PRVs are described.

1. Inflow resistance (Ri) = head loss across the valve / flow rate^2 (5.1)
2. Head loss across the valve = upstream pressure – downstream pressure (5.2)

   ➔ Head loss across the valve = 110 (set Pressure) – 0 (atmospheric Pressure)

   = 110 m = 360.89 ft
3. Ignore minor losses across the valve
4. Substitute Equation 5.2 in 5.1 to get the flow rate (Q).
5. Knowing that the V_{valve} (maximum recommended velocity across the valve) = 2 * maximum recommended lined pipe velocity (3.5 m/s) = 7 m/s
6. Applying the Thumb Rule to get valve cross sectional area:

   ➔ Area of valve (A) = Q/V_{valve}

7. Calculate Valve Diameter (D) knowing that Area = $\pi D^2/4$

It was found that PRV size for Khabisi is 468 mm diameter (use 500mm) and for Power House is 372 mm diameter (use 400 mm).

Other valves required for the proposed bypass include Air Valve in each branch with 100 mm diameter (typical diameter used in most applications). Also, a pressure sustaining valve of 1400 mm diameter is required along the Fujaira Line while two Flow Control Valves are needed along the two lines of Khabisi and Power House branches.

5.6 Surge Protection Devices Analysis

This section discusses the effect of the proposed surge protection devices on the system behaviour in case of sudden valve closures in Khabisi and Power House areas.

5.6.1 PRV at Khabisi Branch

Figure 5.14 shows that the maximum pressure head generated in the bypass-proposed system due to sudden and complete valve closure in Khabisi area at 5:30 PM was 110m at Khabisi area while the minimum pressure head was 27.3m at Power House area.

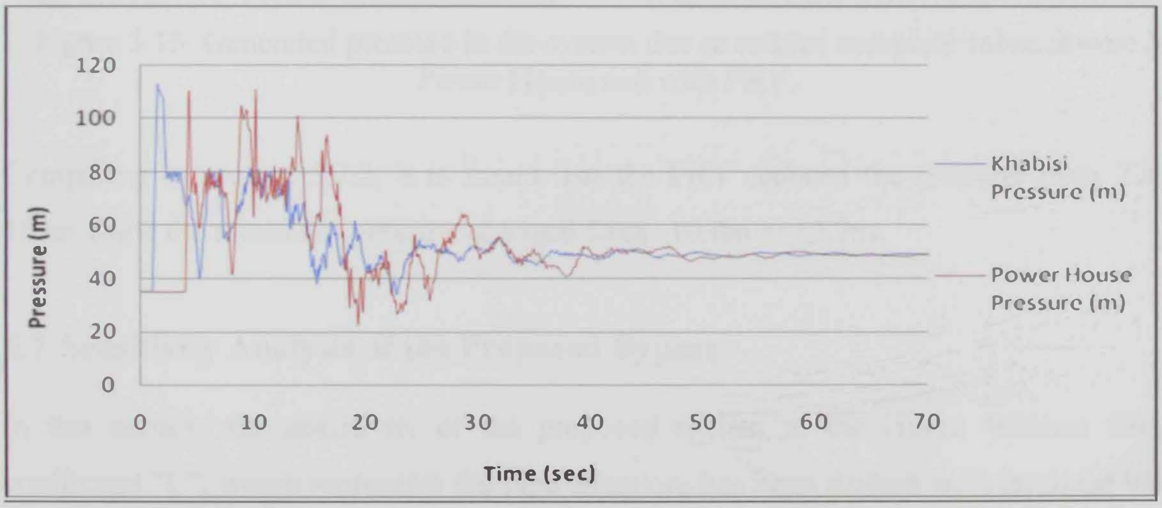


Figure 5.14: Generated pressure in the system due to sudden complete valve closure in Khabisi and with PRV

Comparing to section 5.2.1, it is found that the installed PRV reduced the maximum pressure head in the system from 313.2m to 110m. In addition, it raised the negative pressure head from -10.0m to 27m.

5.6.2 PRV at Power House Branch

Figure 5.15 shows that the maximum pressure head generated in the bypass-proposed system due to sudden and complete valve closure in Power House area at 9:30 AM was 108m while the minimum pressure head was 13.7m; both at the Power House area.

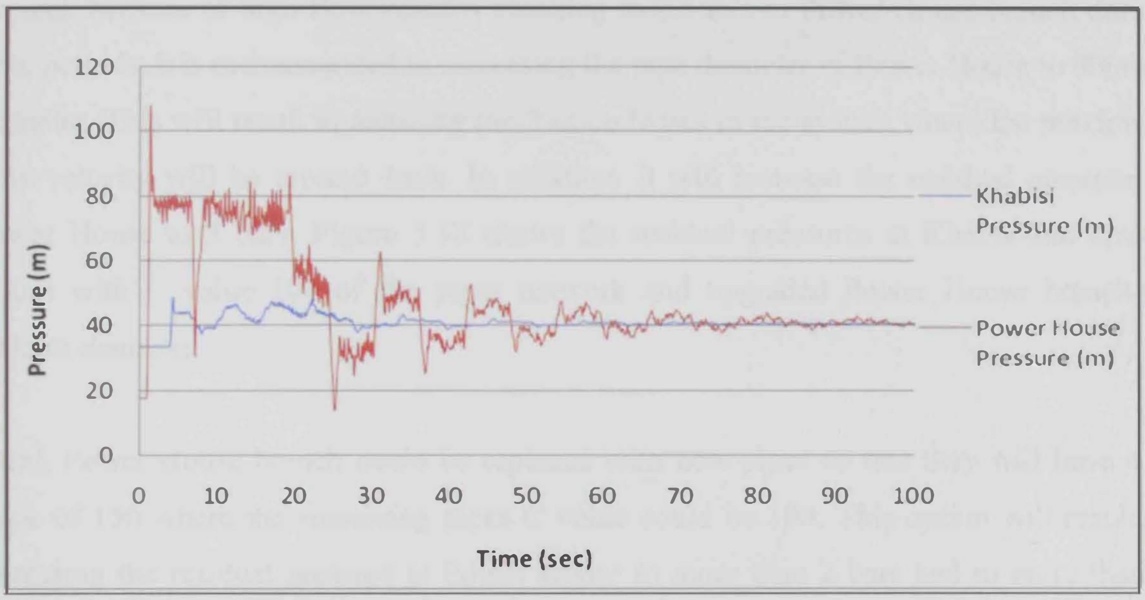


Figure 5.15: Generated pressure in the system due to sudden complete valve closure in Power House and with PRV

Comparing to section 5.2.2, it is found that the PRV reduced the pressure from 276 to 108m while the minimum pressure changed from -10.0m to 13.7m.

5.7 Sensitivity Analysis of the Proposed Bypass

In this section, the sensitivity of the proposed system to the Hazen William friction coefficient "C", which represents the pipe situation, has been studied and simulated where the impact on residual pressure at each area, i.e. Khabisi and Power House, have been concluded in the following Figure 5.16 and Figure 5.17. From these Figures, it could be noticed that even with C value of 100; still the residual pressure at Khabisi area will be more than 3 bars even during peak flows. However, for Power House, it was found that if the value of C reached to 100, the residual pressure that might reach to Power House during peak flows is less than 1 bar which might be very critical and not acceptable.

There are several recommendations to enhance the low residual pressure at Power House in the case of C = 100. First, as stated in section 3.4, it is recommended to search new supply patterns associated with less energy consumption and targeting continuous supply to the customers. This will reduce friction losses in the system especially during peak periods.

Second, because of high flow velocity reaching to 2.3 m/s in Power House branch during peak periods, it is recommended to increasing the pipe diameter of Power House to 900mm diameter. This will result in reducing the friction losses in the system where the maximum flow velocity will be around 1m/s. In addition, it will increase the residual pressure at Power House to 3 bars. Figure 5.18 shows the residual pressures at Khabis and Power House with C value 100 of the pipes network and upgraded Power House branch to 900mm diameter.

Third, Power House branch could be replaced with new pipes so that they will have a C value of 150 where the remaining pipes C value could be 100. This option will result in increasing the residual pressure at Power House to more than 2 bars and to more than 3 bars at Khabisi. Figure 5.19 shows the residual pressure at Khabis and Power House with C value of 100 except Power House (C = 150). As summery, it is recommended to go for the first option since it needs only adjusting the existing operation scenarios based on further detailed study and will utilize the existing system. In addition, option one does not need any capital investments in upgrading the networks nor enhancement.

Figure 5.16: Effect of pipe roughness on the performance of proposed bypass at Khabisi area

Figure 5.17: Effect of pipe roughness on the performance of proposed bypass at Power House area

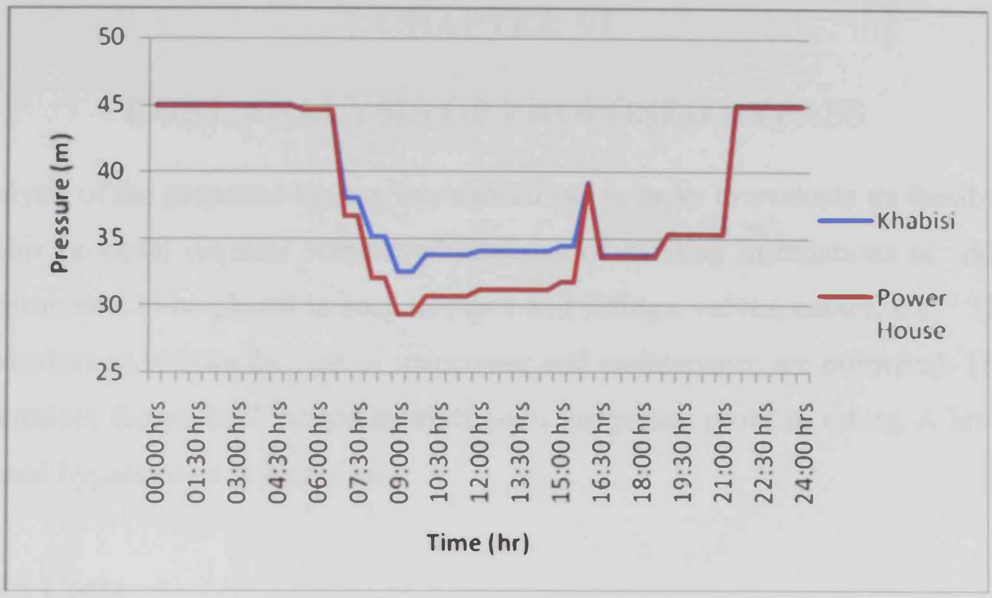


Figure 5.18: Residual pressures at Khabis and Power House with C value of 100 and 900mm diameter Power House branch

Figure 5.19: Residual pressure at Khabisi and Power House with C value of 100 except Power House branch (C=150)

CHAPTER VI

COST ANALYSIS OF PROPOSED BYPASS

Cost analysis of the proposed bypass was carried out in order to evaluate its feasibility and value. This proposal requires some modifications of existing installations in addition to some instruments to be placed in such as pipes and fittings, valves, cables, etc. The costs of such devices as well as the cost of manpower and maintenance are estimated. The study also determines the payback period in addition to the gained profit in future. A breakdown of proposed bypass costs is presented.

6.1 Pipes Costs

General rates were collected from different local contractors and the average rates are given in Table 6.1 with main items breakdown such as supply, excavation and installation costs. Table 6.2 lists the main pipes required for the proposed bypass and their costs.

Table 6.1: Rate for per meter pipeline construction including supply, excavation and installation

SR No	Description	Rate AED/m					Total AED/m
		Supply			Excavation	Installation	
		Standard	Restrained	Average			
1	DI pipes DN 200mm		281	281	180	80	541
2	DI pipes DN 300mm	385		385	180	80	645
3	DI pipes DN 500mm	722		722	220	90	1,032
4	DI pipes DN 600mm	918	1,276	1,097	220	90	1,407
5	DI pipes DN 800mm	2,401	2,401	2,401	275	125	2,801
6	DI pipes DN 900mm	1,722	2,838	2,280	275	125	2,680
7	DI pipes DN 1000mm	1,954	3,132	2,543	325	150	3,018
8	DI pipes DN 1200mm	2,970	5,748	4,359	350	160	4,869

Table 6.2: Main pipes costs

Pipe dia required (mm)	length (m)	Total Cost (AED)
1200	500	2,434,500
900	100	268,000
600	100	140,700
Total Price Cost (AED)		**2,843,200**

From Table 6.1 and Table 6.2, pipes cost was determined to be around 2.84 Million AED (around 0.78 Million US $).

6.2 Valves and Fitting Costs

Table 6.3 lists average costs for the valves needed for the proposed bypass system.

Table 6.3: Valves costs

Area	Valves Type	Valves Dia (mm)	Construction Cost (AED)	Material Cost (AED)	Total Cost (AED)
AARS	PSV	1400	70,000	700,000	770,000
Khabisi	FCV	900	40,000	400,000	440,000
Power House	FCV	600	22,000	220,000	242,000
Khabisi	PRV	500	20,000	220,000	240,000
Power House	PRV	400	20,000	200,000	220,000
Khabisi	AV	100	5,000	7,000	13,000
Power House	AV	100	5,000	7,000	13,000
Total Valve Cost AED					**1,938,000**

6.3 Electric Cables and Control Systems

Electric Cables and Control Systems costs are estimated as 15% of pipe materials, valves and fittings cost. The total estimated cost is around AED 717,180 (US $ 196,487). This cost includes all cable materials and construction costs.

- ⇨ Total Capital Cost AED = 2.843 Million + 1.938 Million + 0.717 Million
- ⇨ = 5.5 Million (About US $ 1.506 Million)
- ⇨ Considering Contingency fees 20%, the total bypass project value = AED 6.6 Million (about US $ 1.808)

6.4 Energy Consumption Costs

The proposed bypass was evaluated using EPS runs by InfoWater software considering typical demand patterns. Power consumptions for each area was studied and summarized in the Table 6.4. Power costs were plotted in Figure 6.1 for more illustration.

Table 6.4: Power consumption and power cost for Al Khabisi and Power House areas

	AARS Pumping Station			Zone Pumping Station			Total		
Area	Average Pump Power Consumption (KW)	Average Power Consumption (Pump + Motor)(KW)	Daily Energy Costs (Dhs)/ day	Average Pump Power Consumption (KW)	Average Power Consumption (Pump + Motor)(KW)	Daily Energy Cost (Dhs/ day)	Overall Power (KW)	Overall Power (AARS + AREA) (KW/Day)	Daily Energy Cost (AARS + AREA) (Dhs)
Khabisi and Power House	600.23	= 600.23/0.8 =750.3	= 750.3 * 24 X 0.15 AED/Kw = 2701	419.33	524.16	1886.98	=750.3+ 524.16= 1274.5	=1274.5* 24 = 30588	=30588 *0.15 AED/ kW = 4588.1

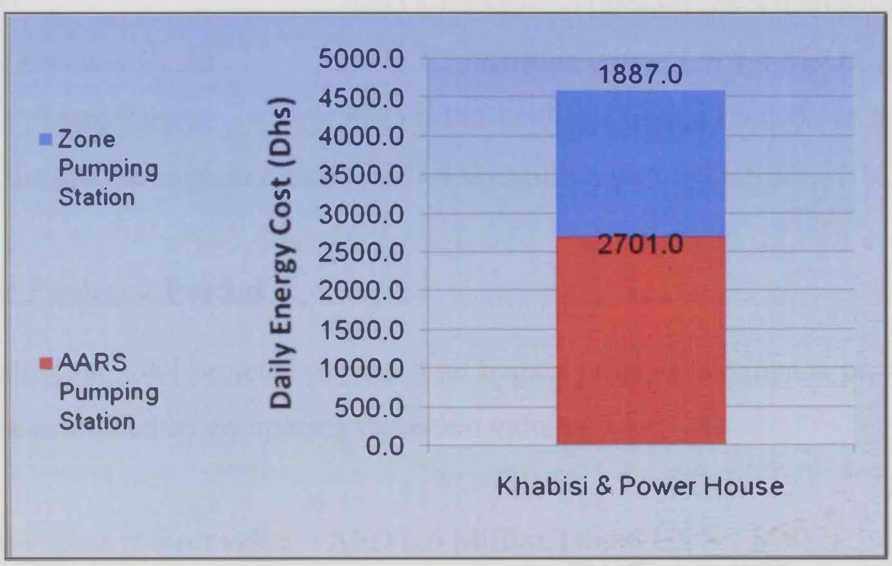


Figure 6.1: Power consumptions cost for pumping stations for concerned areas and at AARS

From Table 6.4 and Figure 6.1, it is expected to have about 4,588 AED per day as power cost saving, which is equivalent to annual savings of AED 1.67 Million (about US $ 0.46 Million). This value is excluding the operational cost such as manpower cost, maintenance, spare parts, etc.... Moreover, existing pumps and surge vessels can be utilized in other areas and purposes and possibly save from purchasing new pumps and serge vessels.

6.5 Operation and Maintenance Costs

Based on annual cost of man-hours from TRANSCO, Personnel Policy and spare parts from TRANSCO stores, the Operation and Maintenance (O&M) costs is estimated and summarised in Table 6.5.

Table 6.5: Operation and maintenance costs

Pumping Station	Manpower Cost (AED/Year)	Spare Parts Costs (AED/Year)
Khabisi Pumping Station	750,100	47,000
Power House pumping Station	750,100	47,000
Khabisi and Power House Pumping Station groups at AARS	750,100	47,000
Sub Total	2,250,300	141,000

From Table 6.4 and Table 6.5, the annual savings are:

Energy Cost:	AED 1.67 Million (around US \$ 0.46 Million)
Operation/Manpower Costs:	AED 2.25 Million (about US \$ 0.617 Million)
Maintenance (Spare Parts):	AED 0.141 Million (around US \$ 0.386 Million),

This makes the total savings to around AED 4.06 Million (around US \$ 1.11 Million).

6.6 Project Payback Period

After estimating the total project cost and total bypass proposal savings, a project payback period can be estimated by comparing these two values.

- ⇨ Total bypass project value = AED 6.6 Million (about US \$ 1.808)
- ⇨ Total annual Energy, Operation and Maintenance costs AED 4.06 Million (about US \$ 1.11 Million).
- ⇨ Project Payback Period = AED 6.6 Million / AED 4.06 Million = 1.63 years which is about 1 year and 8 months.
- ⇨ This concludes that after 1 year and 8 months, the annual savings will be around AED 4.06 Million (around US \$ 1.11 Million). These results demonstrate the feasibility and probity of this proposal.

CHAPTER VII

SUMMARY & CONCLUSIONS

Chapter 7 summarizes the work undertaken in this study and provides a concise list of conclusions.

7.1 Summary of Tasks

The tasks that have been followed in order to achieve the objective of this study, utilizing the available residual pressure at Fujairah Transmission lines to deliver water to the end users of the distribution network, are as follow:

1. Collecting data for the existing settings and operation. These data include available residual pressure and flows of transmission water lines feeding AARS (Fujairah and Taweelah lines), AARS pumping stations and individual pumping station for each area flows patterns, pressure, pumps capacities, number of pumps, pumps operation, controls, water demands and demand patterns for each area.
2. Analysing the collected data and determining the source of possible bypass.
3. Different possible scenarios of bypassing AARS pumping stations and individual pumping stations of each area have been studied considering the optimization of the available residual pressure and flows, which reflects the maximum possible cost saving. The optimum scenario has been considered. InfoWater software has been used to achieve this task.
4. The considered bypass scenario has been studied and analysed against the potential transient effects by determining the possible sources of transient generation. These sources have been classified as Normal and Abnormal operating conditions. Each operating condition has been simulated using InfoWater and InfoSurge software where their effects have been identified and evaluated.
5. A number of surge protection devices were proposed, analysed, designed and simulated such as Air Valves, PRVs etc.... The optimum protection devices have been selected and their impacts on the observed generated transients have been simulated using InfoWater and InfoSurge software.

6. The capital cost of the proposed bypass was estimated and compared against the running operation costs. The results of this cost comparison were used to estimate the Pay Back Period.
7. Based on the results and findings, this study can be used during the feasibility study and concept design of a detailed engineering design.

7.2 Summary of Findings

7.2.1 Bypass proposal

The results showed that Khabisi and Power House area achieves the most benefit of bypassing the Fujaira Line in order to utilize the residual pressure available in this line. This has been accomplished considering two criteria; mass balance during the peak supply periods, and maximum operation savings associated with maximum operational cost.

7.2.2 Normal operating conditions (Typical demand changes)

The normal operating conditions included two cases; demand increase represented by valve opening and demand decrease represented by valve closure. The results of the worst-case scenario (1 second valve opening/closing), it was found that maximum pressures generated in case of demand increase (230m) and in case of demand decrease (297) exceed the pipe rating pressure of 160m. In addition, it was found that the generated negative pressures reach the cavitation limit of -10m in both situations as well. The vacuum in the system is taken care of by adequately sized air valves at high elevations. The safe valve opening/closing times during demand change were determined and found to be 60 seconds in both cases.

7.2.3 Abnormal operating conditions

The abnormal operation herein was represnted by sudden and complete valve closure during the peak supply periods. From the results considering the worst-case scenario (1-second valve closure) where the flow drops to zero, it was found that the maximum pressure generated (313m) exceeds the pipe rating pressure (160m). In addition, it was found that a negative pressure reaching to (-10m) is generated as well. Such impacts can pose hazardous situation and therefore required some remediation measures.

7.2.4 Proposed remediation measures for sudden valve closures

The results concluded that installing a 500-mm PRV at Khabisi branch and a 400-mm PRV at Power House branch is sufficient to maintain the residual pressure in the network within the pipes pressure rating. Moreover, Air relief / vacuum valves are to be installed at the highest points of these branches in order to eliminate the negative pressure waves generated during transient events. The performance of the bypass proposed system with the selected protection valves was simulated using Infosurge Model and found to remain within the acceptable operating ranges. This result proves the feasibility of supplying the water demands to Khabisi and Power House without any interruption in addition to ensuring that the system fittings will not to be damaged.

7.3 Cost analysis of proposed bypass

After estimating the total project cost and total bypass proposal savings in chapter 6, a project payback period can be estimated as follows:

- ⇨ Total bypass project value = AED 6.6 Million (about US $ 1.808)
- ⇨ Total annual Energy, Operation and Maintenance costs AED 4.06 Million (about US $ 1.11 Million).
- ⇨ Project Payback Period = AED 6.6 Million / AED 4.06 Million = 1.63 years which is about 1 year and 8 months.

This concludes that after 1 year and 8 months, the annual saving will be around AED 4.06 Million (around US $ 1.11 Million). These results demonstrate the feasibility and probity of this proposal.

7.4 Conclusions and recommendations

This study can be used as a useful reference for future studies of water transmission and distribution systems in Al Ain region since it has determined the residual pressures and flows under different operation conditions and throughout the day. Beside that, the study should assist in reducing the potable water supply costs by minimizing the operation costs of the proposed bypassed pumping stations, saving spare parts requisition, maintenance, man-hours, etc...with better water quality by eliminating the stagnant time of high quality water in the reservoirs and supply the consumers directly from the water transmission mains via proposed bypass system. i.e., potable water consumers in Al Ain in general and in Khabisi and Power House in particular will receive un-interrupted water supply with better water quality and with lower costs through the proposed bypass system. However, the existing reservoirs and pumps at the proposed bypassed booster stations should be still maintained, water circulated and frequently operated in order to be used for emergencies and during the upstream facility planned shutdowns.

The project payback was found upon implementing the proposals in this study to be around 1 year and 8 months. This study indicates highly profitable proposal knowing that the estimated saving is $ 1.11 Million/ year which makes this alternative very attractive from economical and technical points of view.

After conducting this study and due to the -mentioned reasons (cost effective, better water quality, and safer water supply), it is strongly recommended to implement booster pumping stations bypasses at AARS pumping stations, Khabisi and Power House immediately and without the need for consultancy fees to do the feasibility study or concept design.

In addition, it is recommended to study transient's generation downstream proposed bypass in detailed study covering all required data in order to ensure that the distribution system will be safe.

It is worth mentioning that the current study limits the analysis to the current existing demand patterns. However, it is vital and recommended to search new supply patterns associated with less energy consumption and targeting continuous supply to the customers. This can be further studied upon collecting more data and details in the distribution system and can be achieved using advanced optimization techniques.

REFERENCES

1. ADWEA Statistical Report 2006.
2. Tullis J P, Hydraulics of Pipelines: Pumps, Valves, Cavitation, Transients, Canada, John Wiley & Sons, Inc., 1989.
3. Batterton S. 2006. Water Hammer: An Analysis of Plumbing Systems, Intrusion, and Pump Operation [Thesis]. The faculty of the Virginia Polytechnic Institute and State University.
4. Boulos P F, Karney B W, Wood D J and Lingireddy S, Numerical Methods for Modelling Transients Flow in Distribution Systems, Journal AWWA, 2005, 97:5, 104:115.
5. Robert Fitzgerald and Vicki L. Van Blaricum. Water Hammer and Mass Oscillation (WHAMO) 3.0 User's Manual. 1998.
6. Prof van Vuuren SJ. THEORETICAL OVERVIEW OF SURGE ANALYSES. 2001.
7. Hydraulic Transient Modeling Software.1986-2003 Haestad Methods, Inc. *HAMMER User's Guide.*
8. Al Mamari M S, 2007, PRESSURE UTILIZATION IN WATER TRANSMISSION PIPELINES IN ALAIN REGION CONSIDERING TRANSIENT CONDITIONS [Thesis]. Department of Civil & Environmental Engineering Faculty of Engineering, UAE University
9. Fleming K K, Gullick R W, Dugandzic J P, LeChevallier M W. Susceptibility of Potable Water Distribution Systems to Negative Pressure Transients. 2006.
10. Lingireddy S, Wood D J and Zloczower N, Pressure Surges in Pipeline Systems Resulting from Air Releases, Journal AWWA, 2004, 96:7, 88:94.
11. MWH Soft, Inc.a: InfoWater 3.2 Users Guide 2006.
12. MWH Soft, Inc.b: InfoSurge 5.0 Users Guide / 1996-2004.
13. VAL-MATIC VALVE AND MANUFACTURING CORP, Surge Control in Pumping Systems, 905 RIVERSIDE DRIVE, ELMHURST, IL 60126, 2005.
14. Bruce R. M., Donald F. Y., Theodore H. O.: Fundamentals of Fluid Mechanics, Third Edition, Canada, Jon Wiley & Sons, Inc., 1998.

توطئه

إن الهدف الرئيسي من هذه الدراسة هو التوظيف الجيد لطاقة ضغط المياه المتوفرة في خطوط النقل عن طريق التوصيل المباشر بين شبكات النقل وشبكات التوزيع بدون الحاجة إلى المرور بمحطات استقبال وإعادة ضخ المياه كما هو الوضع الحالي، مع الإبقاء على القدرة في توصيل كميات المياه المطلوبة للمستخدمين في شبكات التوزيع، آخذين بالإعتبار التغيرات العابرة في الضغط والتي يمكن أن تحدث نتيجة حالات التشغيل الاعتيادية وغير الإعتيادية. إن التوظيف الجيد لطاقة الضغط المتوفرة في خطوط النقل تساعد في توفير طاقة إعادة الضخ المستهلكة حالياً بالإضافة لتوفير تكاليف التشغيل والصيانة لتلك المحطات.

تركز هذه الدراسة على محطة العين لاستقبال وضخ المياه والمحطات التابعة لها والمنتشرة في المناطق المختلفة، وخصوصا محطتي ضخ مياه الخبيصي ومركز الطاقة، والتي تستمد مصادرها المائية بشكل رئيسي عن طريق خطين للأنابيب سعة كل منهما 1400 مم قادمين من مدينة الفجيرة. كما تقترح هذه الدراسة التعديلات اللازمة بعد دراسة الاحتياجات المائية للمناطق المختلفة، وتحديد المناطق التي يمكن أن يتم التوصيل المباشر فيها بين خطوط نقل الفجيرة وشبكات التوزيع، مع مراعاة الجوانب التصميمية للأنابيب.

لقد تم إجراء دراسة مستفيضة لتقييم التغيرات العابرة في الضغط باستخدام برنامج حاسب آلي متطور يقوم بمحاكاة هيدروليكية دقيقة، ويظهر الضغوط العابرة لحالات التشغيل الإعتيادية وغير الإعتيادية المختلفة، حيث أظهرت هذه الدراسة مدى إمكانية تحمل شبكات المياه الموجودة للضغوط العابرة وفي أسوأ الحالات إذا ما أجريت بعض التعديلات في الشبكة الحالية والمتمثلة في تركيب بعض المحابس الطاردة للضغط العالي، والتي تستطيع امتصاص أقصى الضغوط الناتجة عن حالات التشغيل المختلفة، وإبقاء الشبكة في وضع آمن تحت أي ظرف كان.

كما تناولت هذه الدراسة حسابات تقديرية للتكاليف الحالية والمتمثلة في تشغيل وصيانة محطات إعادة الضخ المختلفة والتي يمكن توفيرها بمجرد تنفيذ هذا المقترح، بالإضافة لحسابات تقديرية لتكاليف رأس المال اللازم لتطبيق هذا المقترح. لقد أظهرت هذه الدراسة إمكانية الاستغناء عن عدد من محطات إعادة الضخ بما فيها من مضخات وخزانات المياه وهي مجموعة مضخات الخبيصي الواقعة داخل محطة العين لاستقبال المياه بالإضافة لمحطتي ضخ مياه الخبيصي ومصدر الطاقة. حيث تبين الدراسة أنه يمكن توفير حوالي أربعة ملايين وستين ألف درهم سنويا بعد مرور سنة وثمانية أشهر، وهي الفترة الكافية لدفع تكاليف تنفيذ هذا المقترح.

كما أشارت هذه الدراسة إلى بعض الجوانب الإيجابية لها والتي تضم تحسين جودة المياه عن طريق تقليل الوقت اللازم لوصول المياه المطلوبة من المصدر إلى المستخدم، هذا بالإضافة إلى تقليل كمية غاز ثاني أكسيد الكربون المنبعث في الهواء نتيجة توليد الطاقة الكهربائية المستخدمة في تشغيل تلك المحطات والتي نهدف إلى تخفيضها. لقد أوصت هذه الدراسة بتنفيذ المقترح فوراً لما له من دور فعال في مجالات مختلفة كالتوفير الإقتصادي الكبير، والحفاظ على البيئة، وتحسين جودة المياه الواصلة للمستخدم، وغيرها.

جامعة الإمارات العربية المتحدة
عمادة الدراسات العليا
برنامج ماجستير العلوم في موارد المياه

عنوان الرسالة:

توظيف ضغط المياه المتوفر في الخطوط الواردة من الفجيرة لتجنب المرور في محطة العين للاستقبال مع الأخذ بالاعتبار التغيرات المفاجئة في الضغط

تقديم:

حازم بكري الناصر

إشراف الدكتور / وليد الشوربجي
أستاذ مشارك
قسم الهندسة المدنية والبيئية
كلية الهندسة - جامعة الإمارات العربية المتحدة

يناير 2009

جامعة الإمارات العربية المتحدة
عمادة الدراسات العليا
برنامج ماجستير العلوم في موارد المياه

توظيف ضغط المياه المتوفر في الخطوط الواردة من الفجيرة لتجنب المرور في محطة العين للاستقبال مع الأخذ بالاعتبار التغيرات المفاجئة في الضغط

حازم بكري الناصر

رسالة مقدمة إلى عمادة الدراسات العليا
استكمالاً لمتطلبات الحصول على درجة الماجستير في علوم موارد المياه

جامعة الإمارات العربية المتحدة

يناير 2009