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Pressure Utilization in Water Transmission Pipelines in Alain Region Considering Transient Conditions

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

PRESSURE UTILIZATION IN WATER TRANSMISSION PIPELINES IN ALAIN REGION CONSIDERING TRANSIENT CONDITIONS

By
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ABSTRACT

The residual pressure energy available in water transmission pipelines can be easily utilized via direct connection to the distribution system and bypassing the storage reservoirs and booster pumping stations. This thesis investigates 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. The system also has three surge vessls and air valves. The objective of the study has been achieved by conducting a comprehensive transient analysis to evaluate the potential of pressure rises associated with various operating scenarios. Such scenarios were devided into normal and abnormal operating conditions. The normal operating conditions are related to the typical water supply changes occurring during any typical day. The abnormal operating conditions represent rare and odd operating scenarios such as sudden valve closures/openings and pump trips. Limiting pressure heads of 160 m for the pipe pressure rating and 60 m for the pump shutoff head were considered to identify safe performance of the proposed bypass setting. British Standards (BS), International Standard Organization (ISO) and Abu Dhabi Water and Electricity Authority (ADWEA) Standards are adpated for the design of pipes and fittings. Simulation results associated with the normal operating conditions indicated that the proposed setting is safely performing in such conditions and has yielded safe closure times for different valves. Results of the abnormal operating conditions indicated unsafe performance in the case of sudden valve closures. To protect the system against severe transient pressures under these conditions, remedial measures are proposed. Pressure relief valves should be installed on the branches at the upstream side of the distribution network. The proposed valves are sized and evaluated when the system pressure is maintained below 100 m. The savings in energy cost associated with the proposed bypass setting is estimated about $ 850,000 per year. Such savings are found to be available after a pay back period of one year and one month to cover the capital cost of the needed bypasses and pressure releif valves.

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List of Symbols

A : is the cross sectional area

AARS : AlAin Reception Pumping Station

AV : Air Valve

c : is the sonic wave speed in the pipe

D : inside diameter of the pipe

D/e : diameter/ wall thickness

dZ/Z : incremental change in liquid volume with respect to initial volume

dp : static pressure rise

dρ/ρ : incremental change in liquid density with respect to initial density

E_c : Young's Modulus

EGL : Energy Grade Line

EPS : Extended Period Simulation

E_v : bulk modulus of elasticity

f (Q) : is a pipe resistance (nonlinear) term that is a function of flow rate

f : Darcy-Weisbach friction coefficient

FCV : Flow Control Valve

g : Gravitational acceleration constant (m/s^2, ft/sec^2)

HGL : Hydraulic Grade Line (i.e., elevation plus pressure head)

H : Pressure head (pressure/density)

H_a and H_b :Total energy at nodes a and b

h_L : Combined headloss (m, ft)

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

K_1 : Loss equation coefficient

K_R : Constant coefficients that account for the type of support for the pipeline such as the restraint against longitudinal pipe movement.

L : Length

MED : Multi Effect Destillation

MSF : Multi Stage Flash

O&M : Operation and Maintenance

P : Pressure (N/m^2, lb/ft^2)

PRV : Pressure Relief Valve

Q : Water Flow Rate (m^3/s, ft^3/s)

Q_{in} : inlet flow rate

Q_{out} : outlet flow rate

R.O : reverse osmosis

SPD : Surge Protective Devices

t : time

T_M : time of maneuver

V : velocity of fluid inside the pipe, parallel to x-axix.

WHO : World Health Organization

WL : Tank water Level

x : distance along the pipe centerline

z : Elevation at the centroid (m, ft)

γ : Specific weight (N/m^3, lb/ft^3)

ΔH : change in head (m)

ΔT : change in time (seconds, s)

μ : Poisson's ratio

ρ : mass density of water

CHAPTER I

INTRODUCTION

1.1 Background

Desalinated water has become the main source of freshwater in the U.A.E. In the last decade, consecutive expansion projects of desalination plants of different types (such as Multi Stage Flash (MSF), Multi Effect Destillation (MED) & Reverse Osmosis (R.O.) have been successfully executed to cope with the increasing demands for high water quality as a result of social growth and industrial and agricultural activities. The low selling cost (close to zero) of desalinated water in the Emirate of Abu Dhabi has encouraged the people to use the costly and high quality desalinated water, in cleaning, gardening, irrigation, car wash and other general water uses that may tolorate lower water quality. Because of that, the water consumption per capita (115 gallon/person/day) in Abu Dhabi is considered to be one of the highest in the world.

To promote freshwater sustainability in Abu Dhabi Emirate, The local government has defined a number of strategic plans to supply water to all regions that are threatened by water shortage. In order to reduce the load on existing water supply schemes (water from Taweelah and Umm Alnnar), new alternatives for Al Ain region (Showaihat and AlFujairah-AlAin Water Projects) were commissioned and put in operation few years back. Fujairah Water Scheme aims to deliver water from AlFujairah to AlAin via a twin-1600 mm transmission pipelines. The pipe system is designed to include strategic water storage tanks and to serve as an emergency water back-up system to AlAin and Abu Dhabi. Moreover, a large volume of this high quality-water will be mixed with brackish ground water and used for farm irrigation to preserve the natural underground water storage. Considerable volumes of desalinated water are already used in aquifer recharge pilot studies.

To supply water to areas distant to desalinated water production facilities in the coastal areas, long transmission pipelines have been constructed with booster water pumping stations along these pipelines. Cities like Al-Ain, Liwa, Al Wagen, Al Quoe and Ghayathy in the heart of Abu Dhabi desert are receiving desalinated water via transmission pipelines reaching distances as long as 300 km from the water production facilities. Long distance water transmission pipelines and boosting pumping stations have contributed significantly to the high cost in terms of capital and operation expenditures of water production. Further expansions on water production, transmission and distribution are planned to cater to the forcasted supply until year 2020.

Abu Dhabi Transmission Company (TRANSCO), a member of Abu Dhabi Water & Electricity Authority (ADWEA) group, was established (founded) 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 the Emirate.

1.2 Problem Statement

The booster stations in these transmission systems are associated with large storage facilities. At each station, a signifciant amount of energy may be wasted when the residual pressure is lost by ending the lines in the tanks that are open to the atmospheric pressure. Re-boosting is then required to send water to consumers.

Figure1.1 demonstrates the inefficiencies of the present tank storage–pump station configuration with elevations (in meters) shown to demonstrate the loss of energy as flow enters the tank and the addition by pumps.

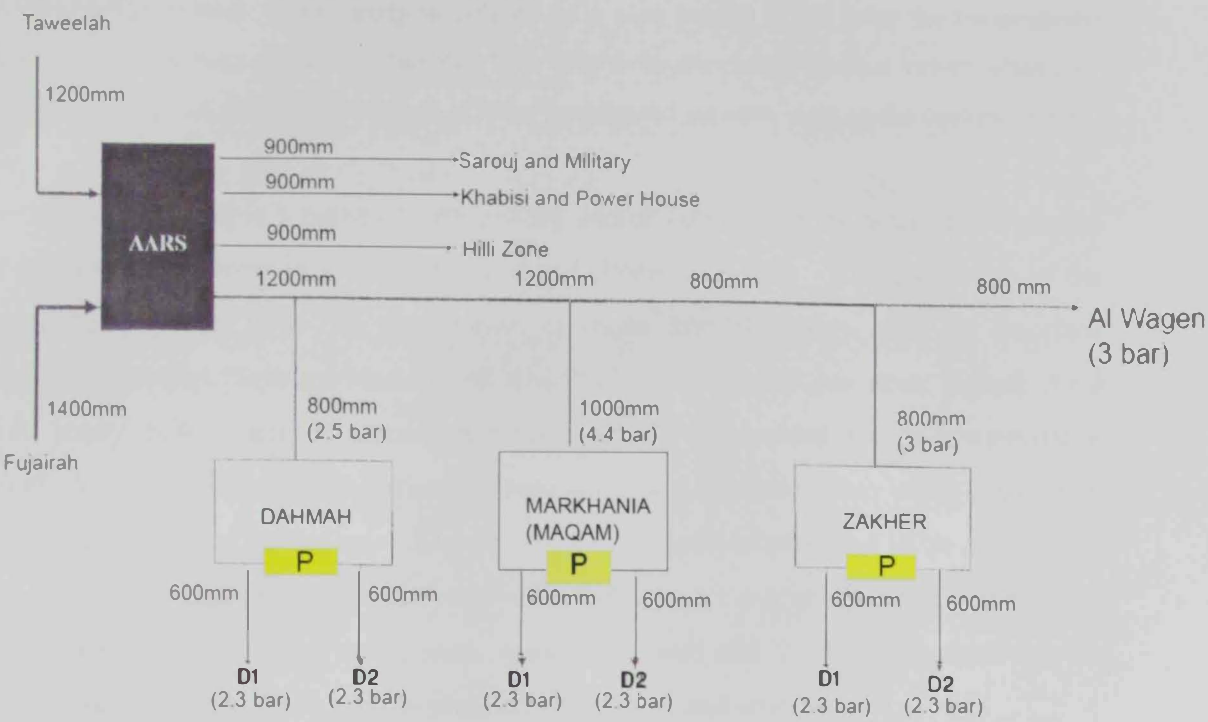

Figure 1.1 shows the elevations for AARS, Dahma, Maqam, Zakher and Al Wagen

This study focuses on the utilization of an existing pressure energy available in transmission pipelines via direct connection to the distribution system bypassing the storage reservoirs and pumping stations. The propsed bypass lines are evaluated by conducting a comprehensive transient simulation study to assess the effect of operating conditions of the distribution system upon the upstream transmission system. The pressure profile along the transmission pipeline is illustrated, taking into account a number of water supply scenarios in each zone of the city upon which the available energy that is usually wasted under the current operating conditions is estimated. Means of utilizing such pressure energy are evaluated. Special valves to sustain and/or regulate or even reduce the pressure in the branches upstream of the distribution stations has been studied, sized and evaluated. The effect of this design on transmission pumps and the potential transients created by the proposed scheme in this study are carefully examined, including sizing surge vessel, examining the hydraulics of the transmission pipelines and assessing the transmission pump operation control philosophy. Savings in energy and total cost are estimated in this research.

The distribution system in this study is defined by a pipe branch taken from the transmission system to fill customers domestic tanks that their feed lines are closed by float valves when they are full. This type of distribution system is not a pressurized network right to the customers tabs.

The proposed bypass is a means of integrating and/or connecting the water transmission and distribution systems in a more efficient and economical way. The integration of the transmission system with the distribution is more feasible today after an interface agreement between the transmission and distribution companies has been signed. As a result, many bulk water consumers are now directly connected to the transmission system. Administrativly and logistically, there are fewer obstacles than in the past when the interface between transmission and distribution is more emphasized to be after a tank with a boosting station. The water reservoirs and booster pumps that are available in booster stations can still be maintained, water circulated and frequently operated to be kept for use during upstream facility planned shutdowns and emergencies.

Nowadays, the interface has become acceptable and some times encouraged to directly tap off to the main transmission line to save capital money by avoiding upgrading distribution pump and tank capacities. The interface agreement between transmission and distribution mandates certain criteria be met for direct consumers' connections and shall be agreed upon by both parties before the start of the connection. Special installations are needed at the interface point in this case such as double isolation valves, a flow control valve, a certified and regularly calibrated flow meter, pressure elements, water quality measuring elements for chlorine residual, PH, and conductivity.
The road of integration between transmission and distribution in Al Ain region started with existing Fujairah transmission system. The original design of Fujairah Water Transmission System (FWTS) at each branch was to feed into a tank with a booster pump station where needed, before delivering to consumers. A major deviation from the original design took place after the FWTS commissioning. The bulk of water customers find it very convenient to be directly connected to the transmission line closest to their properties (farms, horse stables, camel barns, and villas). Another example of integration is the design of huge float tanks (3 x 20 MIG) in Um Ghafa (elevation 370 m) with a

common inlet and oulet feed line from the bottom of the tank. These tanks, located at a high elevation (370 m), are connected by a 1600 mm transmission line to a booster station at low elevation (225 m) that feeds a large number of consumer. During the daily peak supply, the tanks and the booster station simultaneously feed the consumers. During low supply, the tanks are filled by the booster station while consumers are supplied from the same transmission line. In the case described: tanks, line and station are considered part of the transmission system while consumers' branches are considred part of the distribution system.

It shoud be noted that the delivery of drinking water in correct quantities and qualities with a sufficient pressure reaching to consumers is the main concern of any utlity. This is the case with the the Regulation and Supervision Bureau (RSB), a governmental body, assigned by the Executive Council to supervise the water and electricty sector, monitor and control capital expenditures, operation and maintenace cost, system failures, shortage of supply and water quality. RSB is not concerned about the locations of the interface points as long as these locations have been agreed upon by the transmission and distribution companies.

1.3 Objectives

The main objective of this study is to reduce, and attempt to minimize, the water conveyance cost by reducing the pumping time, pumping flow rate, spare parts requirements, maintenance frequency, manpower and future expansion projects. This goal will be accomplished by utilizing the available pressure energy in the transsmission pipelines before it is lost within the storage tanks (energy dissipation). It is aimed to achieve the above objective while considering potential transient problems that may arise during operations due to the direct connection of the transmission and the downstream distribution systems.

1.4 Methodology

The utilization of the available pressure energy and its feasibilty to implement a proposal on AARS 's Markhania Pump Group has been done by undertaking the following steps:

1. System data inclding operating and design data for AARS' pumps, the characteristics of the main water transmission pipelines (1200 mm D.I and associated installations), and supply at the final destination point (demand node) on each consumption branch.
2. Set up hydraulic models for each study scenario based on gathered data using simulation programs such as InfoWater and InfoSurge.
3. Determine the available pressure in each consumption branch and at the final destination point (water demand node) under various flow conditions.
4. Investigae incorporating various new regulating flow control valves and surge protection devices in all consumption branches.
5. Select the best setting of valve installations in association with identified operating conditiions for different demand scenarios.
6. Study the system hydraulic performance and potential of transient conditions caused by the new proposal under different flow scenarios. Transient simulations will be completed using a professional software that addresses hydraulic transients in pipe networks. Such simulations allow the moduler to evaluate the existing surge vessels'size and water source pumps' operation philosophy and control, confirm their suitability for the new mode of operation and provid a basis for recommendation of possible modifications where needed.
7. Estimate the capital and operational costs associated with potential modifications.
8. Summarize research findings, give recommendations and suggestions as a conceptual basic design scheme (feasibility study) for further detailed engineering development and implementation.

It should be noted that EPS simulation results under normal operating conditions are verified by field readings at various locations such as the inlet of reservoirs and the outlet of distribution pump discharge.

1.5 Expected Impact of This Work

This study shall identify the major problems for the system rating pressure during normal and abnormal operating conditions such as the safe closure times for all FCVs installed at different nodes in the water transmission sys [illegible] Al A [illegible] s examine sudden closure of valves which may produce unsafe pressures and shall suggest some remediation measures such as the sizing and the installation of PRVs on the proposed station bypasses. Criteria of operating scenarios will be established throughout the subjected transmission loops as a useful reference for future modifications and studies. In addition to providing un-interrupted water supply from the water transmission mains directly to the consumers via the proposed by-pass systems, the stagnant time of the high quality water in the storage tanks will be minimized or even elimimated and positively improve the quality of drinking water delivered to AlAin. The water reservoirs and pumps that are available in booster stations can still be maintained, water circulated and frequently operated to be kept for use during upstream facility planned shutdowns and emergencies. The mecahmnism for reducing the cost of drinking water conveyance to AlAin will be outlined and identified. The minimization in operating hours of the by-passed booster pumping stations shall reflect positively into savings of spare parts acquisition, maintenance and man-power (number of shift staff working for the Operation and Maintenance of the booster Pumping stations) Upon implementing the proposal in this study, the project cost and pay back to be calculated. Conclusion and recommendations shall be drawn to implement the proposed schemes.

1.6 Thesis Contents

Following this introduction, theoretical background of hydraulic and transient simulations and relevant case studies are presented in Chapter 2. System description: demand fluctuation, valves and pump operating philosophy, distribution reservoir bypass, and sizings are provided in Chapter 3. Description of the simulation models (InfoWater and InfoSurge) used in this study, model setups for all inputs are demonstrated in Chapter 4.

In Chapter 5, a number of scenarios are planned and simulated such as the affect of valve closures in normal operating conditions in section 5.2.1, valve openings in section 5.2.2, sudden valve closures in abnormal operating conditions in section 5.3.1 and pump trips in section 5.3.2. Remediation measures are suggested and discussed in section 5.4. The economy (cost analysis) of the remedial measures and pay back period are discussed in detail in section 5.5. Water quality implications are discussed is in section 5.6. Summary of the thesis and the main conclusions and recommendations are presented in Chapter 6.

Chapter II

THEORETICAL BACKGROUND

Hydraulic modeling is an important tool to maximize the use of the existing facilities, reduce the operating cost, improve the system performance, maintain the water quality and optimize the capital investment. This chapter provides the theoretical aspects of hydraulic simulations in pipe networks. It elaborates the aspects related to the simulation of steady state conditions, gradual unsteady flow conditions, and rapid or transient flow v [illegible] networks are also addressed.

2.1 Simulation of Steady State Conditions

Steady flow hydraulic modeling provides a snapshot of the conditions in a water network assuming that the hydraulic conditions have reached equilibrium. Flow is considered steady when pressure and flow do not vary with time, or when fluctuations are small with respect to mean flow values. The main equations used in simulating steady state flow in pipe networks are:

<u>Conservat[illegible] (Continuity)</u>

$$\sum Q_{in} = \sum Q_{out} \qquad \text{at a node} \qquad (2.1)$$

where Q_{in} is the inlet flow rate and Q_{out} is the oulet flow rate.

Conservation of Energy

$H_a - H_b = K_l \, (Q_l)^n$ in a pipe (2.2)

Where H_a and H_b are the total energy at nodes connected by the pipe. K_l is the pipe specific loss equation coefficient. Q is the flow rate in pipe and ^{n} is the exponent from the head loss equation.

Conservation of Energy In Pumps

In addition to pressure head, elevation head, and velocity head, a pump adds head (energy) to the system, and the friction head losses are removed from the system. These changes in the system head are either head gains or head losses. Balancing energy between two points in the system, the energy can be obtained in a form of energy equation for steady-state flow:

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

Where: P = Pressure (N/m^2, lb/ft^2)

γ = Specific weight (N/m^3, lb/ft^3)

z = Elevation at the centroid of the pipe (m, ft)

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

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

h_p = Head added by a pump (m, ft)

h_L = Combined headloss of friction and minor losses (m, ft).

Components of the energy equation can be combined to express two useful quantities: the hydraulic grade and the energy grade:

- **Hydraulic Grade**-The hydraulic grade is the sum of the pressure head (p/γ) and elevation head (z). The hydraulic head represents the height to which a water column would rise in a piezometer. The plot of the hydraulic grade in a profile is often referred to as the hydraulic grade line or HGL.
- **Energy Grade**-The energy grade is the sum of the hydraulic grade and the velocity head ($V^2/2g$). This is the height to which a column of water would rise in a pitot tube. The plot of the hydraulic grade in a profile is often referred to as the energy grade line or EGL. At a lake or reservoir, where the velocity is essentially zero, the EGL is equal to the HGL, as can be shown in Figure 2.1.

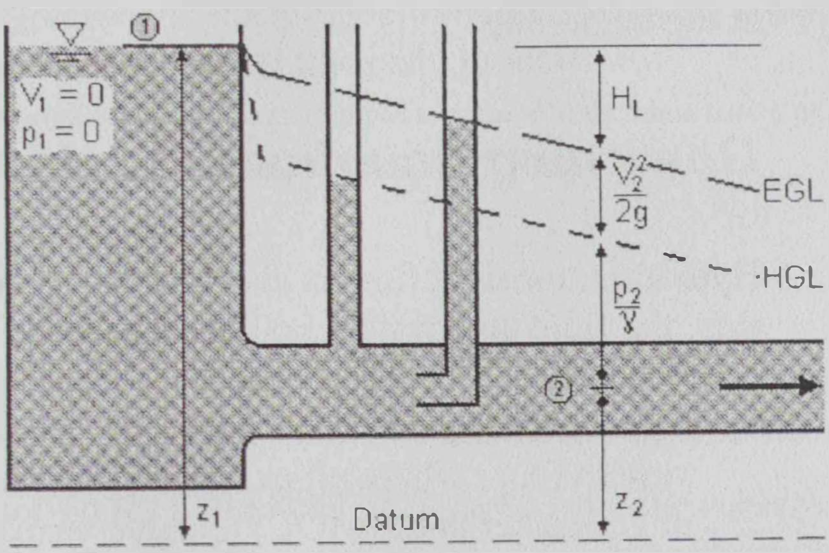

Figure 2.1 HGL and EGL (*HAMMER User's Guide*, 2003).

Pipes in series can provide insight into the variation of a total head.
This is represented (MWH Soft, 2nd edition 2006) by the head loss equation:

$$h_L = \sum K_m Q^n_m \quad (2.4)$$

where, m is the pipe identifier, K is the pipe coefficient for pipes in series (influenced by diameter, length and roughness), Q is the flow rate inside the pipes in series and n is the exponent from the headloss equation.

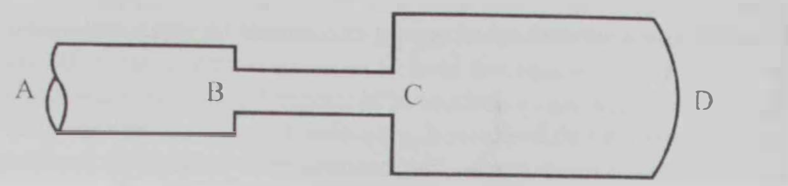

Figure 2.2 Pipe in series (Boulos / lansey / Karney, 2006).

Pipes in parallel provide the first application of mass conservation at a junction ($q_a = q_b$ between two nodes), and energy conservation around a loop (H_a - $H_b = H_L$ between two nodes

The fundamental of connecting one or more pipes in parallel to the same junctions lead to the relationship of full network modeling(MWH Soft, 2nd edition 2006):

$$h_L = K_{m.} (Q^n)_{total} \quad (2.5)$$

where, K is the equivalent pipe coefficient for parallel pipes (influenced by diameter, length and roughness), Q is the flow rate inside the parallel pipes and n is the exponent from the head loss equation (Boulos et al., 2006).

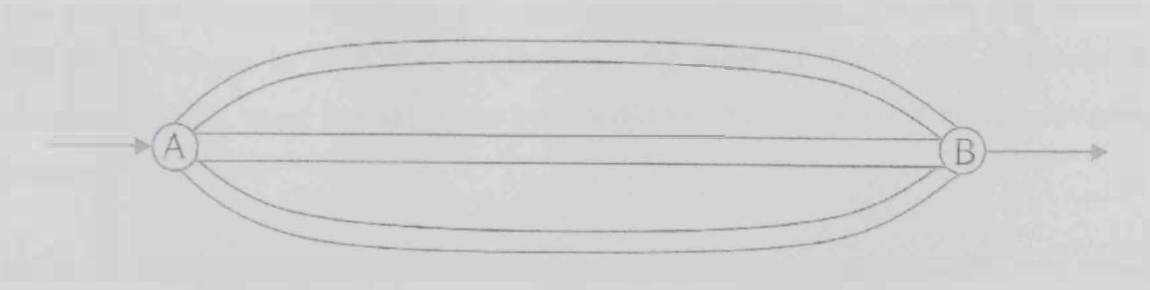

Figure 2.3 Pipe in parallel (Boulos / lansey / Karney, 2006).

<u>System of Equations</u>

After applying the continuity at all nodes, the energy in all pipes and around closed loops, a system of equations is developed either for unknown pipe flows or unknown nodal heads. The developed system of equations needs to be solved using a suitable numerical technique.

2.2 Simulation of Unsteady Gradual Flow Variation

The temporal variations in demands over time cause the pressure and flow distributions to change. This unsteady state flow conditions can be modeled using the Extended Period Simulation (EPS) approach. EPS is called step-wise (quasi-steady state) dynamic simulation that involves a series of steady state simulations with tank levels being updated between steady state analyses. The equation describing the change of tank water level (WL) at any time t: $\mathbf{WL_{t+\Delta t} = WL_t \pm (Q_p^t\ \Delta t)/A_T}$ (2.6)

Where Q_p^t is the flow in pipe p at time t where the pipe p is connected to the tank and A_T is the tank area.

The data requirements for an EPS are the physical description of the tank, nodal demands as a function of time, and operational controls. Simultaneous quasi-linear equations can be solved for pipe flows and nodal heads for steady state and EPS; although an additional relationship describing changes in tank levels is required for EPS. The time step between tank level changes in an EPS is typically on the order of hours. This increment is acceptable under most normal operating conditions, particularly for a well-designed system.

2.3 Simulation of Hydraulic Transient Flows

Hydraulic transients are a form of unsteady state fluid flow that can cause large pressure forces and rapid fluid accelerations. Any rapid disturbance in the water, generated during a change in the mean flow conditions, will initiate a sequence of transient pressures (waves) in the water distribution system. Terms like "water hammer", "transient flow", and "surge" describe the rapid unsteady flow of fluids in pipes.

The dynamic simulation of transient flows involves the acceleration and deceleration of a fluid mass. It clearly accounts for the inertia in the water column with limitation in the rigid column approach in which it is assumed that the water in each pipe, throughout its length, acts as single mass with a uniform velocity in each conduit.

When a column of fluid is rapidly accelerated, it causes a small amount of mass to accumulate within the pipe, mobilizes inertia terms, and affects the fluid compressibility. It is this compressibility that sends a pressure signal through the pipe and informs the whole column of the changes taking place. To explicitly permit the propagation of a pressure signal, the pipe is discretized and broke up into pieces. This approach permits a greater degree of pressure and velocity variation within the pipe, but also has greater computation demands. These compressible or water hammer models often result in more realistic unsteady flow calculations than those associated with the rigid model (MWH Soft, 2nd edition 2006).

2.3.1 Causes of Transient Initiation

Hydraulic transient may occur due to any sudden change in the fluid itself or due to any sudden change at the pressurized system's boundaries including:

Changes in fluid properties such as sudden opening of a relief valve and air entrainment to the system.

Changes at system boundaries such as rapidly opening or closing a valve and pipe burst. System boundaries are nodes at which transient pressure waves are typically reflected back to the system and where inflows or outflows may occur. Such sudden changes create a transient pressure pulse that rapidly propagates away from the source of disturbance, in all possible directions, and throughout the entire pressurized system boundaries. Unsteady-flow conditions continue until the transient energy is completely damped and dissipated by friction, unless another transient event is triggered by the pressure wave fronts.

Disturbances in water distribution systems can be initiated by the following situations and devices (*HAMMER User's Guide*, 2003):

a) Pump startup or shutdown
b) Valve opening or closing (variation in flow area)
c) Changes in boundary pressures such as a change in reservoir level or pressure tank.
d) Rapid changes in demand conditions such as hydrant flushing.
e) Changes in transmission conditions such as pipe burst.

2.3.2 Governing Equations

The fundamental equations describing hydraulic transients in water distribution systems are developed from the basic conservation relationships of physics or fluid mechanics. They can be fully described by Newton's second law (equation of motion) and conservation of mass (kinematic relation). These two equations can incorporate typical hydraulic devices and their interactions with the wave conditions in the pipes.

Applying these basic laws to an elementary control volume, a set of nonlinear hyperbolic partial differential equations can be derived. If x is the distance along the pipe centerline, t is the time and partial derivatives are represented as subscripts, then the governing equations for transient flow (MWH Soft, 2nd edition 2006) can be written as:

Continuity Equation:

$$H_t + (c^2 Q_x) / (g A) = 0 \qquad (2.7)$$

Momentum (Dynamic):

$$H_t + Q_t/(g A) - f(Q) = 0 \tag{2.8}$$

where H is the pressure head (pressure/density), Q is the volumetric flow rate, c is the sonic wave speed in the pipe. A is the cross sectional area, g is the gravitational acceleration, and f (Q) is a pipe resistance (nonlinear) term that is a function of flow rate.

The continuity equation (*HAMMER User's Guide*, 2003) may also be written as:

$$\Delta V/\Delta T + V(\Delta V/\Delta X) + g(\Delta H/\Delta X) + f.V|V|/2D = 0 \tag{2.9}$$

Where: f = Darcy-Weisbach friction coefficient, D = inside diameter of the pipe, V = velocity of fluid, and

$$\Delta H/\Delta T + V(\Delta H/\Delta X) + c^2/g(\Delta V/\Delta X) = 0 \tag{2.10}$$

Where: c = pressure wave speed, H = HGL, V = velocity of fluid inside the pipe, parallel to x-axix and ΔT = change in time.

Analytical solutions do not exist for these equations except for a few simple applications that neglect or greatly simplify the boundary conditions and the pipe resistance term. When pipe junctions, pumps, surge tanks, air vessels and other hydraulic components are included, the basic equations even become more complicated. As a result, numerical methods are used to integrate and solve the transient flow equations.

2.3.3 Numerical Solution Methods

A transient flow solution can be obtained numerically by solving Eqs. (2.9) and (2.10) (along with the appropriate initial and boundary conditions) in which pressure and flow are variables and dependent upon position and time. Five different numerical procedures are commonly used to approximate the solution of the governing equations (MWH Soft, 2nd edition 2006). Three Eulerian methods update the hydraulic state of the system in fixed grid points as time is advanced in uniform increments. The two Lagrangian methods update the hydraulic state of the system only at times when a change actually occurs. Each method assumes that a steady-state hydraulic solution is available and gives initial flow and pressure distributions throughout the system. A comparison of the various methods can be found in Boulous et al (1990 and 2005) and Wood et al (2005).

2.3.4 Elastic Theory

Pressure waves created by velocity changes depend on the elastic properties of the pipe and liquid, and they propagate throughout the water network system at speeds that depend directly on these elastic properties. Elastic theory describes unsteady flow of a compressible liquid in an elastic system (e.g., where pipes can expand and contract). It is used to describe a water hammer state in which changing the momentum of a liquid requires pressure changes that result in expansion or compression of the pipe and liquid. Abrupt reduction in velocity converts the kinetic energy carried by the moving fluid into strain energy in the pipe walls, causing a "pulse wave" that may be of abnormal pressure to travel from the disturbance into the pipe system (Boulos et al., 2004 and 2005; Streeter and Wylie, 1967). The hammering sound that is sometimes heard when the flow is abruptly stopped indicates that the fluid's original kinetic energy has been converted into a high pressure, which creates an impact force. Energy losses such as fluid friction, as well as the reflection and transmission of waves at pipe junctions, cause the transient pressure waves to gradually decay until new steady pressures and velocities are established.

The elastic model assumes that changing the momentum of the liquid causes expansion or compression of the pipeline and liquid - both assumed to be linear-elastic. Since the liquid is not completely incompressible, its density can change slightly during the propagation of a transient pressure wave (MWH Soft, 2nd edition 2006).

The transient pressure wave (*HAMMER User's Guide*, 2003) has a finite velocity that depends on the elasticity of the pipeline and of the liquid. In 1898, Joukowski established a theoretical relationship between pressure and velocity change during transient flow condition. *In 1902, Allievi* independently developed a similar elastic relation and applied it to a uniform valve closure. The elastic theory developed by these two pioneers is fundamental to the field of hydraulic transients enclosed system. The combined elasticity of both the water and the pipe walls is characterized by the pressure wave speed, c , can be expressed by the following two pairs of equations (*HAMMER User's Guide*, 2003):

$$+ g / c (\Delta H/\Delta T) + \Delta V/\Delta T + f.V |V| / 2D = 0 \qquad \Delta X/\Delta T = + c \qquad (2.11)$$

$$- g / c (\Delta H/\Delta T) + \Delta V/\Delta T + f.V |V| / 2D = 0 \qquad \Delta X/\Delta T = - c \qquad (2.12)$$

In the elastic theory, the following equation is applicable to an instantaneous stoppage of velocity:

$$(H - Ho) = c / g (V - Vo) \qquad (2.13) a$$

Where: o = denotes initial conditions.

For an instantaneous valve closure or stoppage of flow, the upsurge pressure ($H-H_o$) is known as the "Joukowsky head" is the same as in Joukowsky equation *(Thorley, 2004)* that provides an estimate of the maximum change in head (ΔH) created when water with velocity V is brought to a sudden stop:

$$\Delta H = \pm c/g \, \Delta V \qquad (2.13) b$$

where c is the acoustic wave speed and g is acceleration due to gravity. The negative sign represents a propagation traveling upstream and the positive sign represents a propagation traveling downstream.

Given that '**c**' is roughly 100 times as large as **g**, a 1.0 ft/sec (0.3 m/s) change in velocity can result in a 100-ft (30 m) change in head. Because changes in velocity of several feet or meters per second can occur when a pump shuts off or a hydrant or valve closes, it is expected to cause large transients in water systems.

The mass of fluid that enters the part of the system located upstream of the valve immediately after its sudden closure is accommodated through the expansion of the pipeline due to its elasticity and through slight changes in fluid density due to its compressibility. The Joukowsky equation provides a worst case estimate of surge magnitude, since the flow change is considered to occur instantaneously. This equation does not strictly apply to the drop in pressure downstream of the valve, if the valve discharges flow to the atmosphere (*HAMMER User's Guide*, 2003).

2.3.5 Rigid Column Theory

Rigid column theory describes unsteady flow of an incompressible fluid in a rigid system. It is only applicable to slower transient phenomena.

The rigid model assumes that the pipeline is not deformable and the liquid is incompressible; therefore, system flow control operations only affect the inertial and frictional aspects of transient flow. Given these considerations, it can be demonstrated using the continuity equation that any system flow control operations will result in instantaneous flow changes throughout the system, and that the liquid travels as a single mass inside the pipeline, causing a mass oscillation. If liquid density and pipe cross-section are considered constant, the instantaneous velocity will be the same in all sections.

These rigidity assumptions result in easy-to-solve ordinary differential equations; however, its application is limited to the analysis of surge. Newton's Second Law of Motion is sufficient to determine the dynamic hydraulic of a rigid water body during the mass oscillation (*HAMMER User's Guide*, 2003):

$$\Delta H = f\,(L/D)(V|V|\,/\,2g) + (L/g)\,(\Delta V/\,\Delta t) \qquad (2.14)$$

where ΔH = change in head (m)

Using the fundamental rigid model equation, the hydraulic grade line can be established for each instant in time. The slope of this line indicates 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. For the case of flow reduction caused by a valve closure ($\Delta Q/\Delta t < 0$), the slope is reduced. If a valve is opened, the slope increases, potentially allowing vacuum conditions to occur. The change in slope is directly proportional to the flow change. Generally, the maximum transient head envelope calculated by rigid water column theory (RWCT) is a straight line as shown in Figure 2.2.

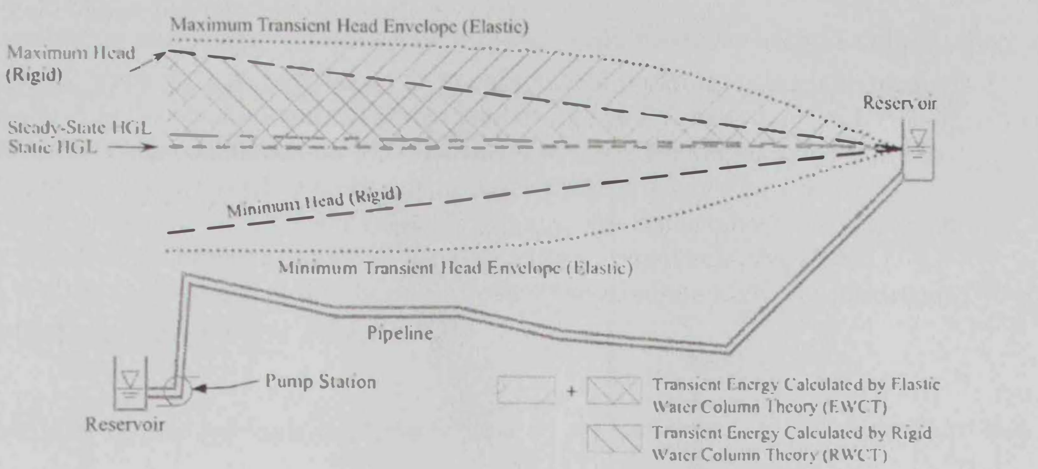

Figure 2.4 Static and Steady HGL versus Rigid and Elastic Transient Head Envelopes (*HAMMER User's Guide*, 2003)

The rigid column model has limited applications in hydraulic transient analysis because the developed equations do not accurately model pressure waves caused by rapid flow control operations. 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 (*HAMMER User's Guide*, 2003).

Rigid-column theory is suitable for simulating changes in hydraulic transient flow or head which are **gradual** in terms of the system's **characteristic time**, T = 2 L/c. This type of hydraulic transient is often referred to as a mass-oscillation phenomenon, where gradual changes in momentum occur without significant or sharp pressure wave fronts propagating through the system. For example, mass oscillations can occur when a vacuum-breaker or combination air valve lets air into the system at a local high point to limit sub-atmospheric pressures. The water columns separate and move away from the high point as air rushes in to fill the space between them. Eventually, flow reverses towards the high point where the air may be compressed as it is expelled. This back-and-forth motion of the water columns may repeat many times until friction dissipates the transient energy. The rigid model will track the extent of the air pocket and the resulting mass-oscillation and water column accelerations. Some models use rigid-column theory only for the pipes nearest to the high point. This results in more accurate solutions without increasing execution times (*HAMMER User's Guide*, 2003).

2.3.6 Celerity and Pipe Elasticity

The elasticity of any medium is characterized by the deformation of the medium due to the application of a force. If the medium is a liquid, the force will be a pressure force. The elasticity coefficient (also called the elasticity index, constant, or modulus) is a physical property of the medium that describes the relationship between force and deformation. This describes the water system characteristic.

Thus, if a given liquid mass in a given volume (Z) is submitted to a static pressure rise (dp), a corresponding reduction ($dZ < 0$) in the fluid volume occurs. The relationship between cause (pressure increase) and effect (volume reduction) is expressed as the bulk modulus of elasticity (E_Z) of the fluid, as shown below (*HAMMER User's Guide*, 2003):

$$E_Z = dp / (dZ/Z) = dp / (d\rho/\rho) \qquad (2.15)$$

Where: E_Z = bulk modulus of elasticity, dp = static pressure rise, dZ/Z = incremental change in liquid volume with respect to initial volume and $d\rho/\rho$ = incremental change in liquid density with respect to initial density.

A (*HAMMER User's Guide*, 2003) relationship between a liquid's modulus of elasticity and density yields its characteristic wave celerity, is:

$$c = \sqrt{(E_l/\rho)} = \sqrt{(dp/d\rho)} \qquad (2.16)$$

where ,c, is the characteristic wave celerity of the liquid. The characteristic wave celerity (c) is the speed with which a disturbance moves through a fluid. Its value is approximately equal to 4,716 ft./sec (1,438 m/s) for water and approximately 1,115 ft./sec (340 m/s) for air.

In 1848, Helmholtz demonstrated that wave celerity in a pipeline varies with the elasticity of the pipeline walls. Thirty years later, Korteweg developed an equation to determine wave celerity as a function of pipeline elasticity and liquid compressibility. Some times an elastic model formulation that requires the wave celerity to be corrected to account for pipeline elasticity is used (*HAMMER User's Guide*, 2003). This equation is given by:

$$c = \sqrt{((E_Q / \rho) / (1 + (E_Q \Delta A / A \Delta p))} \qquad (2.17)$$

The preceding equation is valid for thin walled pipelines (D/e > 40).

For thick-walled pipelines, there are theoretical equations proposed to compute the wave celerity; however, field investigations are needed to verify these equations. Table 2.1 lists the physical properties of some common pipe materials that are used to estimate the pressure wave velocity during transient analysis. Table 2.2 lists the physical properties of some common liquids that are useful to calculate celerity during transient analysis. Figure 2.3 provides a graphical solution for celerity given pipe wall elasticity and various diameter/thickness ratios (*HAMMER User's Guide*, 2003).

Table 2.1: Physical properties of common pipe materials (MWH Soft, 2nd edition 2006).

Material	Young's Modulus, E_c Pa x 10^9	Young's Modulus, E_c PSI x 10^6	Poisson's Ratio, μ_p
Asbestos Cement	23-24	3.3 – 3.5	-
Cast Iron	80-170	11.6 – 24.7	0.25 – 0.27
Concrete	14-30	2.0 – 4.4	0.1 – 0.15
Reinforced Concrete	30-60	4.4 – 8.7	-
Ductile Iron	172	24.9	0.3
PVC	2.4 – 3.5	0.3 – 0.5	0.46
Steel	200 - 207	29 – 30	0.3

Table 2.2: Physical Properties of Some Common Liquids that is useful to calculate celerity during transient analysis (*HAMMER User's Guide*, 2003).

Liquid	Temperature (°C)	Bulk Modulus of Elasticity		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	31.0 to 40.0	1.5 to 1.9	1.67 to 1.73	860 to 890
Kerosene	20	27.0	1.3	1.55	800
Methanol	20	21.0	1.0	1.53	790

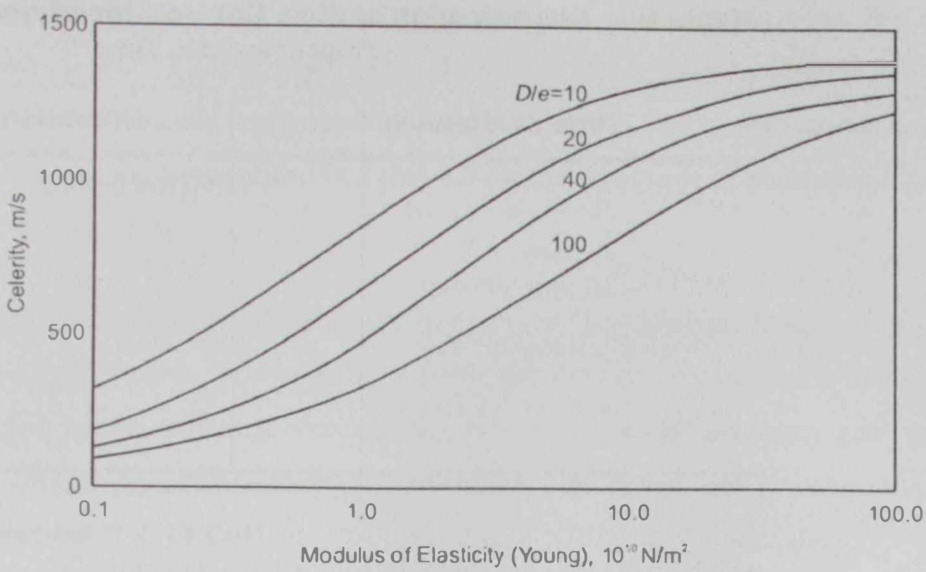

Figure 2.5 Celerity versus Pipe Wall Elasticity for Various D/e Ratios (*HAMMER User's Guide*, 2003).

Covas et al. (2004) showed that plastic pipes which exhibits significant viscoelastic effects, including creep, can affect wave speed in pipes and must be accounted for if accurate results are desired. They proposed several methods that account for viscoelastic effects in both the continuity and momentum equations.

2.3.7 Wave Propagation and Characteristic Time

The representative system length, L, can be approximated for a network by taking the longest path connecting a pump to a storage element, such as a tank or reservoir. L can be a pipe branch from the main network system as well.

For instance, the pressure wave generated by a flow control operation in the longest pipe branch propagates with speed; c, reaching the other end of the pipeline in a time interval equal to L/c seconds. The same time interval is necessary for the reflected wave to travel back to its origin, for a total of 2L/c seconds. The quantity '2L/c' is termed the characteristic time for the pipeline. It is used to classify the relative speed of a maneuver that causes a hydraulic transient. If a flow control operation produces a velocity change in a time interval less than or equal to a pipeline's characteristic time, the operation is considered "rapid." Flow control operations that occur over an interval longer than the characteristic time are designated "gradual" or "slow." The classifications and associated nomenclature are summarized in Table 2.3 for different time of maneuver, T_M (*HAMMER User's Guide*, 2003).

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 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.

Table 2.3: Classification of Flow Control Operations Based on System Characteristic Time (*HAMMER User's Guide*, 2003).

Time of Maneuver	**Operation Classification**
$T_M = 0$	Instantaneous
$T_M \leq 2 L / c$	Rapid
$T_M > 2 L / c$	Gradual
$T_M >> 2 L / c$	Slow

2.3.8 Wave Propagation and Reflection in Water Pipelines

Hydraulic Transient; describe the disturbances in a fluid caused during a change from one steady-state to another. The principle components of the disturbances are pressure changes caused by the propagation of pressure waves throughout the distribution system. These pressure waves continue to propagate with the velocity of sound until they are dissipated down to the level of the new steady-state by the action of some form of damping or friction. Only if the flow is regulated extremely slowly is it possible to undertake a smooth transition from one steady-state to another without large fluctuations in pressure head or pipe velocity.

To estimate the pressure wave velocity in different pipe materials, the ratio of pipeline diameter to its thickness needs to be obtained. Figure 2.6 shows the pressure wave velocity for water in round pipes versus the ratios of different diameters to thicknesses. These relations are defined for K_R 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. There are different pipe support cases:

- Anchored at the upstream end only, $\mathbf{K_R = 1\text{-}\mu_p / 2}$
- Anchored throughout against axial (longitudinal) movement, $\mathbf{K_R = 1} - \mu^2_p$
- Equipped with functioning expansion joints throughout, $\mathbf{K_R = 1}$
- Supported only at one end and allowed to undergo stress and strain both laterally and longitudinally, $\mathbf{K_{R.} = 5/4 - \mu_p}$ where μ_p is Poisson's ratio for pipe materials.

The number to the right of curves indicates E_c value x 10^{-6} (Table 2.1), in PSI that was used to construct the curve.

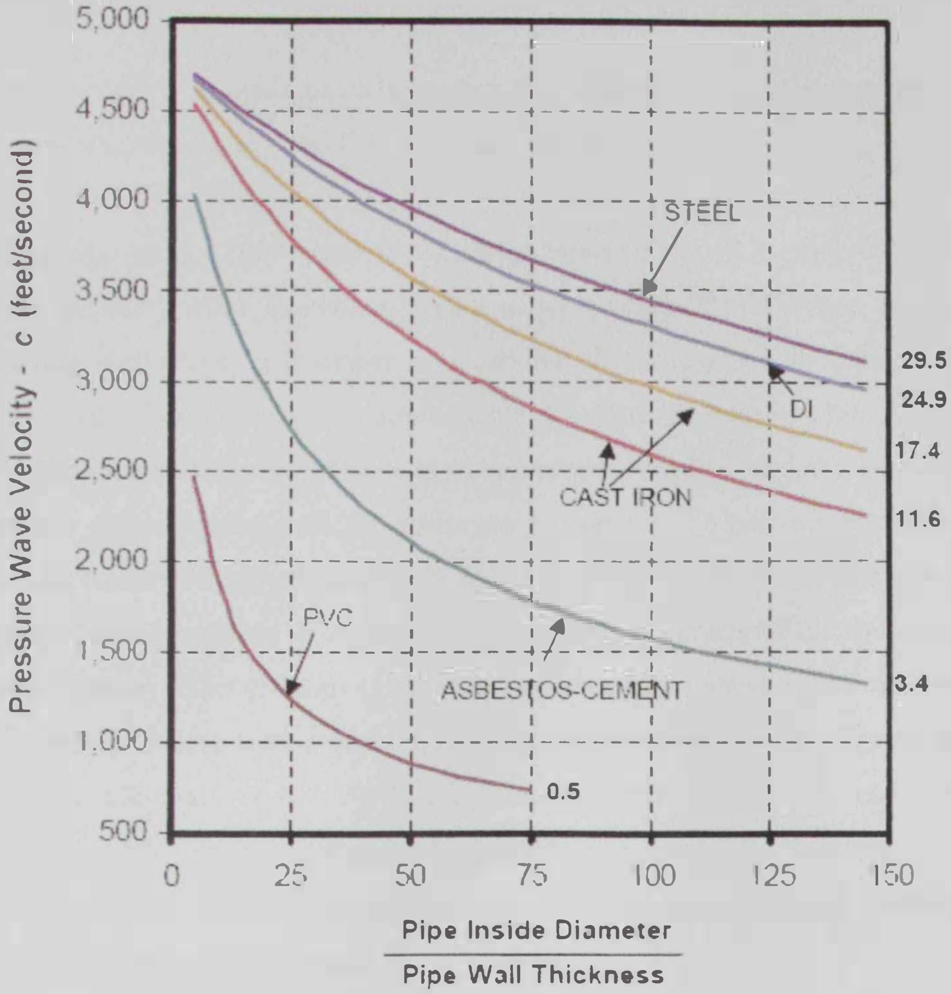

Figure 2.6 Pressure Wave Velocity for water in round pipes with different diameters and thicknesses and K_R of 0.91 where pipe is anchored against axial (longitudinal) movement and $\mu_p = 0.3$ for ductile iron (Wood et al. 2005; Fleming et al. 2006).

2.4 Applications of Hydraulic and Transient Modeling (Case Studies)

In this section, a number of case studies that utilized transient simulations to validate pilot studies (proto-types and field tests) are discussed in this section.

Bülent Selek et al. (2004) studied a Çatalan Power Plant in Turkey. It was observed a "water hammer" when ever there was a major change of water flow rate in a pipeline supplying water from a reservoir to a turbine plant (penstock in Çatalan Power Plant, Turkey), significant transient pressures were developed. To resolve the shock behavior of the penstock pipeline, the numerical schemes were applied and the computed transient pressures are compared with the prototype test results. To perform transient simulation, the basic fixed-grid numerical scheme had been modified in various ways to improve the computational procedure in variable series of pipes. The use of interpolation techniques such as Karney and Ghidaoui (1997) who developed a hybrid model of interpolation in both space and time with a wave speed adjustment technique by neglecting the friction effect and Ghidaoui et al. (1998) who computed the total energy associated with the fixed-grid MOC in a pipeline had resulted in second-order accurate explicit finite schemes to solve the water hammer equations. An algorithm for the variable-grid MOC for pipes in series was presented.

It was concluded that the second-order methods were superior to resolve the shock behavior of the penstock pipeline. The numerical schemes were applied and the computed transient pressures were compared with the prototype test results. From the comparison of theoretical and experimental results, it was found that the water hammer model with variable-grid method of characteristics produces the nearest computational values to the experimental findings of the maximum transient pressure during valve closure, water rejection and pump emergency shut down. Also, the Fixed Grid MOC gave results almost as good and confirmed the usefulness of this approach.

David H. Axworthy and Normand Chabot (2004) considered a Canadian Sewage Force Main where a severe water hammer following pump shutdown was suspected to be the cause of pipe displacement at a pump station in Saguenay, Quebec. It was initially suspected that insufficient surge protection devices were installed on the force main. Data logging of a field test of the sewage pump station undergoing power failure was conducted in combination with the development of a water-hammer computer model for the design of additional surge protection devices. A hammer model based on the method of characteristic was developed and calibrated in conjunction with field test (high-frequency, pressure-sensing devices). This was done to predict the effect of the pressure drop wave on the entire water main in a pump station at certain locations following pump startup and shutdown. Preliminary results from a field test showed that a large pressure drop wave was introduced to the water main after a pump lost power. The results of the pressure surge analysis perfectly agreed with the field tests that the pressure was predicted to drop to vapor pressure over a significant length of the water main and that the vapor cavities that were formed would collapse upon re-pressurization of the water main by a reflected water hammer wave. Furthermore, the water-hammer model was used to develop surge protection alternatives for the water main to eliminate the formation of vapor cavities following shutdown of the pump station. The model result were used to add surge protective devices such as air/vacuum valves at locations suggested by the hammer model.

Kala Fleming et al. (2006) developed surge models for sixteen distribution systems (selected from ten states, including California, New Jersey, and West Virginia in USA). These surge models were used to identify the locations where low or negative pressures, and by extension backflow, were most likely to occur. The distribution systems with low flow rates ranged in maximum day system delivery capacity from 0.3 to 73 mgd and covered a range of operating pressures and configurations. Pressure monitoring (using high-speed, electronic pressure monitors) was conducted in three distribution systems to confirm the vulnerability of the areas determined to be susceptible to low and negative pressure transients via modeling. Vulnerable areas were identified from the surge modeling and were prioritized for a disinfectant residual, optimization of surge control, leak detection, and backflow prevention. However, the surge modeling results from this study indicated that systems with the following characteristics are more susceptible to low and negative pressure transients:

- Less than 10 MGD system delivery; may be due to their location at the end of the distribution system that is associated with high pressure losses and possible leaks out of the system.
- Ground-water source; this is associated with a lift up pressure losses and leaks.
- Few floating storage facilities; due to their location at higher elevations and at the end of the network.

In addition, the following characteristics of various locations within the distribution system were found to increase susceptibility to low and negative pressure transients:

- Near a pump station with downstream velocity greater than 3ft/s
- Greater than one mile away from elevated storage
- Elevations greater than 40 ft to 50 ft above surroundings
- Near a dead end
- Located near a hydrant on a major main

From the simulation results and field verifications, it was concluded that the final choice for surge protection devices should be based on the initial cause and location of the transient disturbance(s), the specific system, and the cost of the protection measure(s).

The following recommendations were made by Fleming et al. (2006):

• Slowing the flow rate at which a flow control operation occurs will reduce the magnitude of the surge produced.

• Increasing pump inertia, slowing the opening and closing of fire hydrants, prolonging valve opening and valve closing times, and avoiding complete pumping failure by putting a major pump on a universal power supply.

• Installing hydro-pneumatic tanks or air vacuum valves to control the magnitude of the low / negative pressure transient (surge) once it has been created.

Nicole S. Arbon et al. (2006) studied an Australian composite water main of Acrylonitrile-Butadiene-Styrene (ABS) and Ductile Iron Cement Lined (DICL) pipe was fed by untreated and high nutrient water. The water main was designed to deliver 105 L/s and at the time of testing was capable of delivering only 87 L/s. It was suspected that this pipe was blocked due to the low flow during testing.
The pipe was analyzed using transient analysis verified by field data collected during transient testing. This online and offline testing was undertaken with transients generated by the fast closure of a valve connected to the system via a standpipe. Both openings and closures of the valve were recorded, as were varying nozzle sizes. Online transients were also initiated by pump failures.

A combination of transient and steady state analysis was used to determine the presence and location of anomalies within the pipeline with the aim to discover possible causes of the reduced flow capacity and assess the condition of the pipeline. An emphasis on the analysis of transient reflections was made including investigation of the consistency of results between openings and closures and varying nozzle sizes. Transient analysis of the recorded responses indicated that no significant leak or blockage was present in the test pipeline. An air pocket was found to be present, however, the impact of this fault did not account for the reduction in capacity of the pipeline.

Transient analysis indicated that the pipeline had a significantly high roughness, most probably due to the presence of a biological film. The untreated and possibly high nutrient water and the low typical pumping rates encourage the growth of a biological film in the system. The presence of a 4 mm biological film layer was shown to account for the reduction in capacity experienced. Hence, the undertaken transient analysis had a significant contribution to investigate the condition of pipelines in the field.

CHAPTER III
SYSTEM DESCRIPTION

This chapter describes the hydraulic system addressed in this study. It includes a general description about Al Ain Water Reception Station (AARS), design/operating data, water demands fluctuation (EPS flow plots), and a bypass proposals and sizing.

3.1 General Layout

The system in the considered study is composed of a large 2 x 3 km Water Reception Station that receives water from two huge desalinated water sources ,i.e., Fujairah and Taweelah. Fujairah is located on the eastern coast of UAE on the Arabian Sea some 250 km from Al Ain. Taweelah is located on the western coast of UAE on the Arabian Gulf some 160 km from Al Ain. The case study of this thesis targets the largest of the pumping groups ,i.e., Markhania Group to utilize the available pressure energy at water consumption nodes to directly supply water to distribution networks without the need for booster stations. It is located at elevation of 253 m and comprised four variable speed pumps (that receive water from the common four water reservoirs in AARS), three pressurized surge vessels , a single 1200 mm ductile iron pipeline , flow control valves, check valves, air valves and shut-off valves. The first tap off connection (node) to the outlet of this group is Dahma Pumping Station some 3.6 km from AARS at elevation of 278 m above sea level. The second tap off connection (node) to the outlet of this group is Markhania (Maqam) Pumping Station some 12.1 km from AARS at elevation of 250 m above sea level. The third tap off connection (node) to the outlet of this group is Zakher Pumping Station some 20.7 km from AARS at elevation of 250 m above sea level. The fourth tap off connection (node) to the outlet of this group is Al Wagan Node some 21.2 km from AARS at elevation of 250 m above sea level (Figure 3.1).

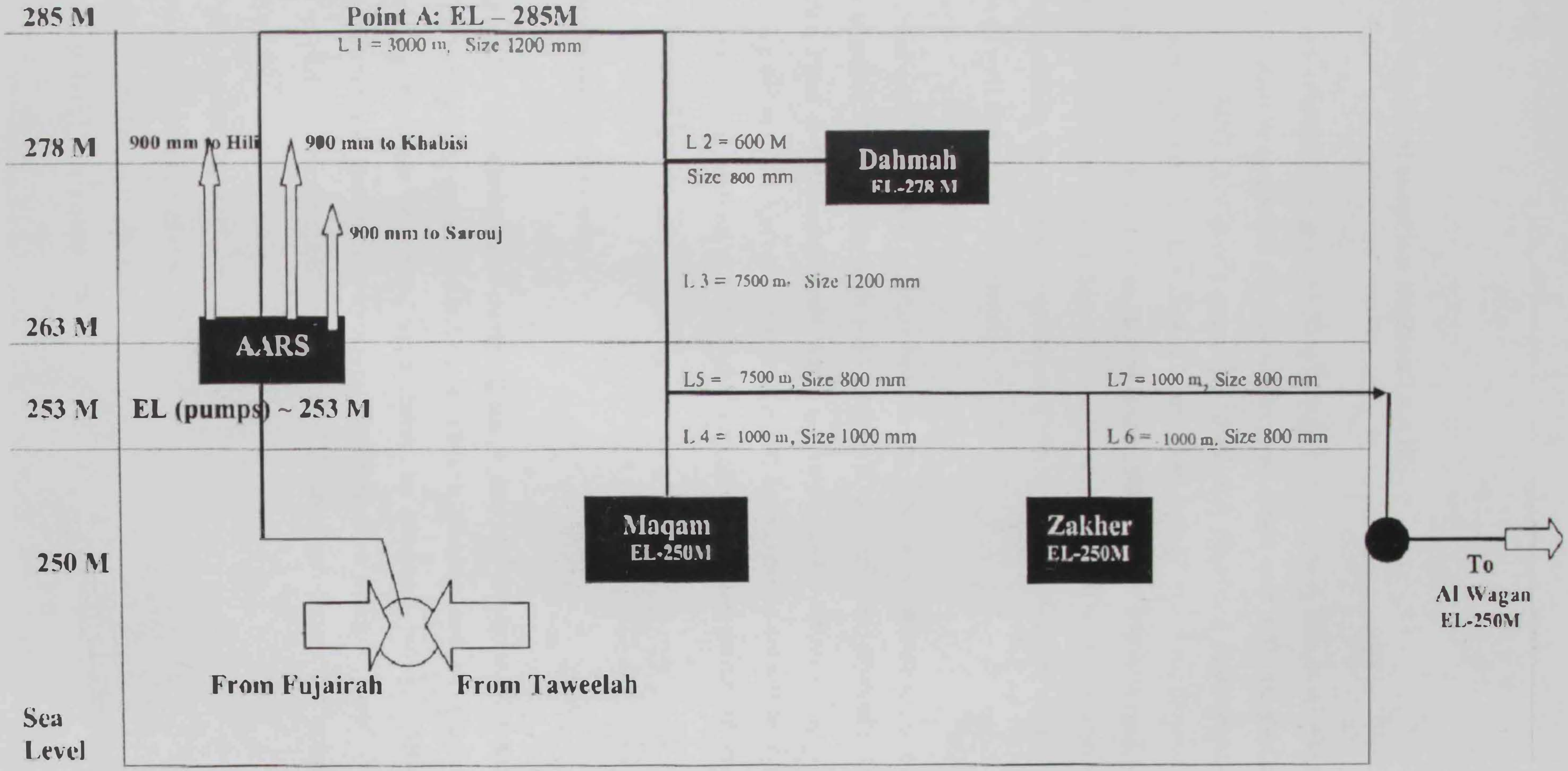

Figure 3.1 Elevation view of considered system (Markhania Pumping Group)

3.2 Al Ain Water Reception Station (AARS)

Al Ain Water Reception Station (AARS) is located in the northern side of Al Ain near the airport. The station is composed of a boundary wall (1900 m x 800 m), four concrete reservoirs with 5 MIG (22730 m^3) capacity and 80 m x 60 m x 6 m-dimensions, four pumping groups contained inside pumping house (146 m x 27 m x 8 m) dimensions and four outlet pipelines (1200mm D.I to Dahma, Markahnia, Zakher and AlWagan, 900 mm DI to Hili, 900 mm D.I to Khabisi and 900 mm D.I to Sarouj). The station also houses a chlorination building for chlorine disinfection. Chlorine is injected at the inflow mains to the station as well as at the outlet mains.

Fujairah Desalination Plant has recently become the main water source to Al Ain water reception complex (AARS), through twin 1400 mm transmission pipelines. Taweelah Desalination Plant is the second main water source to AARS through twin 1200 mm transmission pipelines. 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.

3.2.1 Markhania Pump Group

It is composed of four transmission pumps, three on duty and one on standby arranged in parallel to supply the water to Markhania, Dahma, Zakher and Al Wagan. These transmission pumps are centrifugal type driven by electrical motors. Between the electrical motors and the pumps hydraulic coupling type variable speed drives are installed to perform steeple speed regulation. All pumps and drivers are housed in the pump house.

- Pump Design Capacity: 3938 m^3 /h
- Pump Differential Head: 47 m
- Pump Rated Power: 558 KW
- Motor Rated Power: 980 KW

3.2.2 Anti-Surge Equipment

The anti-surge equipment in ARS includes the following:

- Surge tank on the incoming main suction header to Markhania pump group at AARS. Each surge tank is provided on the incoming main to the pump house (pump house main suction header) with a nominal capacity of 37.5 m^3.
- A set of surge vessels on the water pipeline to Markhania is composed of 3 units that are arranged in parallel at the pumps'outlet 1200 mm ductile iron transmission pipeline. Each surge vessel has a nominal capacity of 73.55 m^3.

3.2.3 Piping and Fittings

The pump house piping is made of ductile iron with internal lining of fusion banded epoxy (FBE) for both pipes and fittings. Main piping (outside pump house) is made of ductile iron with internal lining of cement mortar for the pipes and fusion banded epoxy (FBE) for the fittings.

3.2.4 Pipe Pressure Rating

The pipe pressure rating varies relative to the pipe location as follows:

- Inlet pipe work to reservoirs 25 bar
- Suction pipe work from reservoirs to pumps 10 bar
- Delivery pipe work from transmission pumps 16 bar
- Delivery pipe work from distribution pumps 16 bar

3.2.5 Valves

Butterfly type valves are provided on all pipes with nominal diameter greater than 300 mm. Gate type valves are provided on all pipes with nominal diameter less than or equal to 300 mm. All the valves are capable of being operated, electrically or manually, under the maximum unbalanced pressure that may occur across the valve.

Electrically actuated section valves are provided at the following sections:

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

Manually operated valves are provided on branches for future connection of a surge vessel, the nozzle type non-return valves have been provided on each delivery pump side. Also, globe type pressure sustaining valves have been installed at station inlet. They are located between the sectioning valves and the flow meters. They ensure tight-line operations on the upstream twin 1200 mm pipelines from Sweihan to AARS.

3.2.6 Provision for Future Installation

The following provisions have been provided for future installations:

- 1 transmission pump to supply the water pipeline to Markhania;
- 1 surge vessel on the water pipeline to Markhania;- Pipe connection provision and space for 4 future reservoirs.
- Magnetic flow meters on station incoming and outgoing pipelines.

3.2.7 AARS Water Outflow

Four transmission outlet pipelines are installed to supply water to:

- Dahma, Markhania, Zakher, and Alwagen via.1200 mm pipeline.
- Sarouj and Military via.900 mm pipeline.
- Khabisi and Powerhouse via.900 mm pipeline.
- Hili and Zoo via.900 pipeline.

Total Water OutFlows from AARS to Markhania Group is explained below:

At Dahma Node:

The incoming water source is 800 mm dia from AARS at 2.5 bars that is feeding a 5 MG water reservoir. The outlet of the reservoir feeds 8 distribution pumps that in turn feed two distribution networks.

- Distribution to AADC – D1 is 600 mm dia. at 2.3 bars.
- Distribution to AADC – D2 is 600 mm dia. at 2.3 bars.

At Markhania (Maqam) Node:

The incoming water source is 1000 mm dia from AARS at 4.4 bars that is feeding 3 x 5 MG water reservoirs. The out let of these reservoirs feeds 8 distribution pumps that in turn feed two distribution networks.

- Distribution to AADC – D1 is 600 mm dia. at 2.3 bars.
- Distribution to AADC – D2 is 600 mm dia. at 2.3 bars.

At Zakher Node:

The incoming water source is 800 mm dia from AARS at 3 bars that is feeding 5 MG water reservoir. The out let of the reservoir feeds 8 distribution pumps that in turn feed two distribution networks.

- Distribution to AADC – D1 is 600 mm dia. at 2.3 bars.
- Distribution to AADC – D2 is 600 mm dia. at 2.3 bars.

At Al wagan Node:

It is a pipe extension from the main 800 mm D.I pipeline to Zakher. The size of this extension is 800 mm D.I that is fitted with flow control valve, flow meter, and isolation valves.

The flow patterns to the various nodes (costumer tap off) are given below:

	Range (L/sec)	Average (L/sec)
Feed to Dahma P.S	60 to 1050	550
Feed to Markhania P.S	340 to 1025	628
Feed to Zakher P.S	230 to 740	466
Feed to Al Wagan	342 (fixed)	342

3.2.8 Water Supply and Operation of Pumps at AARS

Figure 3.2 shows water supply peak to Dahma node (J49) of 1050 L/s from 6 00 to 1130 hrs, peak to Maqam node (J39) of 1025 L/s from 1300 to 1800 hrs, supply fluctuation to Zakher node (J43) between 230 to 740 L/s and constant supply to Al Wagen node (J45) at 342 L/s.

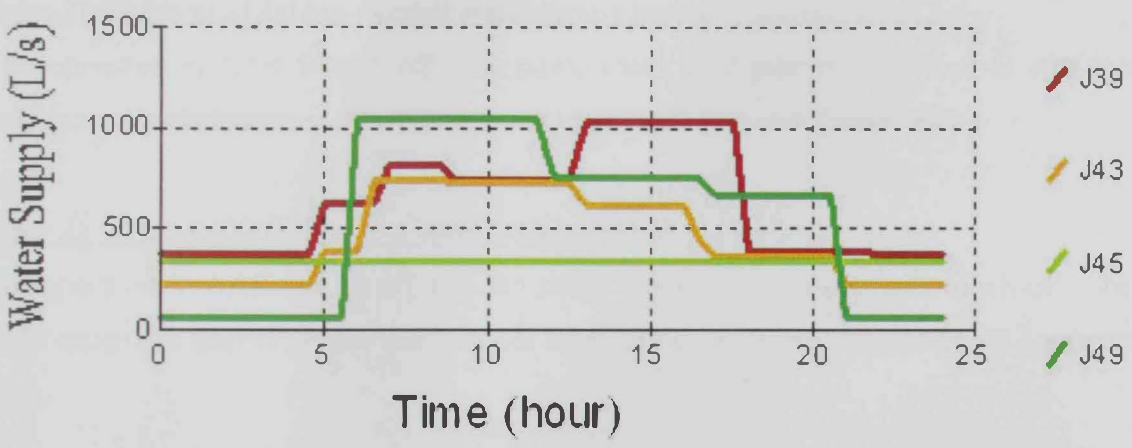

Figure 3.2 Daily water supply for four zones (Dahma node J-49, Maqam node J-39, Zakher node J-43 and Al Wagen node J-45).

The described variable supplies are met by operating different number of pumps in AARS as explained below:

From 00:00 hrs to 05:00 hrs-low water supply period

Presently the pump operation in Al Ain Reception Pumping Station starts with one pump operation at mid night (00:00 hrs) to feed the water to Dahma, Maqam, Zakher Water Reservoirs and Al Wagan tap off.

From 05:00 hrs to 06:00 hrs- normal water supply period

The operators in AARS start the 2^{nd} pump (2 pumps operation) to feed the same zones with increased water supply in Maqam and Zakher nodes.

From 06:00 hrs 18:00 hrs-peak water supply period

This is the daily water peak supply period and 3 pumps operation to feed the same zones with additional increased water supply in Dahma, Maqam, and Zakher nodes reaching peak water demands at different times within this period.

From 18:00 hrs to 21:00 hrs- normal water supply period

The operators in AAR Switch off one pump (back to 2 pumps operation) to feed the same zones with decreased water supply in Dahma, Maqam and Zakher nodes.

From 21:00 hrs to 00:00 hrs- low water supply period

The operators in AAR Switch off the 2nd pump (back to 1 pump operation) which is the daily water low supply period that extends from 21:00 hrs (every day) to 05:00 hrs (next day).

The above operating conditions will be further explained in Chapter-V with the aid of Figure 5.1.

Figure 3.3 shows the hydraulic conditions of the existing system. This section describes the main hydraulic conditioins prevailing in the four supply zones and then variation during a typical operating day. Such conditions are obtained from 24-hour EPS using InfoWater Model and verified against the actual field conditions. The presented conditions include pressure head, hydraulic head, total flow, head loss and velocity in the main 1200 mm line.

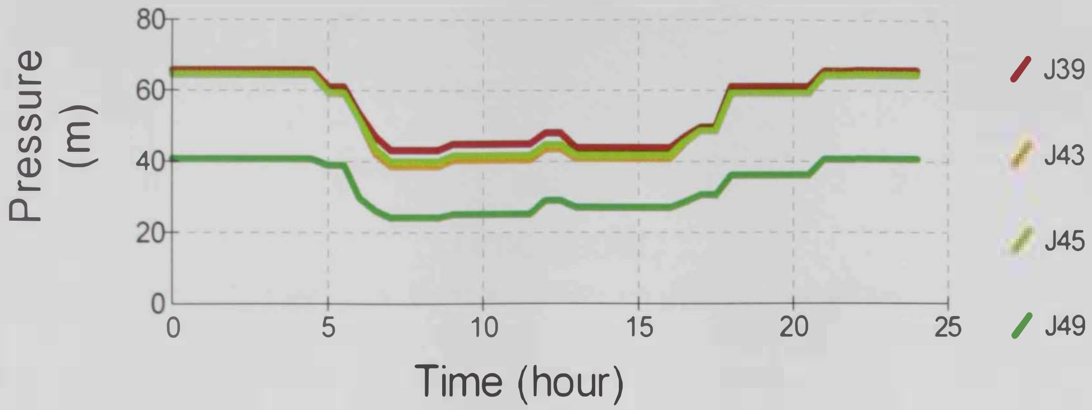

Figure 3.3 Pressure head (m) for the four supply nodes (Dahma node J-49, Maqam node J-39, Zakher node J-43 and Al Wagen node J-45)

Figure 3.4 shows that the head profiles for all supply nodes is hardly fluctuating given the current water supply changes during the day and are almost constant.

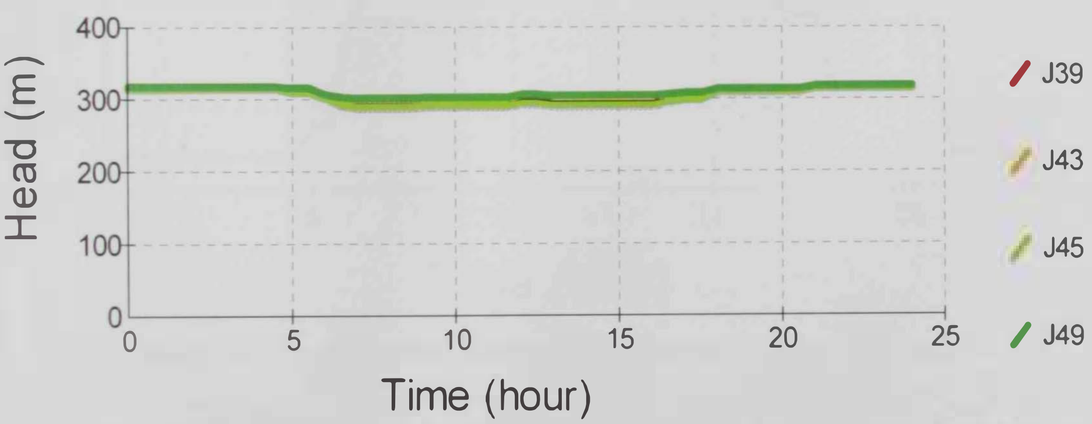

Figure 3.4 Hydraulic head (m) for the four supply nodes (Dahma node J-49, Maqam node J-39, Zakher node J-43 and Al Wagen node J-45)

Figure 3.5 shows the main system (1200 mm pipeline) total flow rate peaks at 2957 L/s from 7 00 to 800 hrs and the system flowrate fluctuates down to 1000 L/s at minimum.

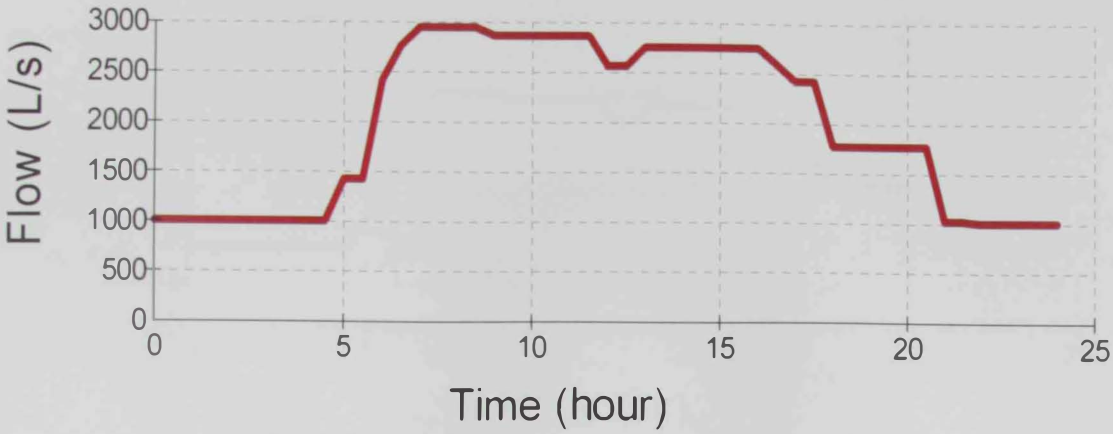

Figure 3.5 Total flow (L/s) at location on pipe 1200 mm before J49.

Figure 3.6 shows that the main system (1200 mm pipeline) headloss/100 m of pipe length at peak flow rate is 0.31 m from 7 00 to 800 hrs and the system headloss fluctuates down to 0.05 m. This is the headloss slope (m/100 m).

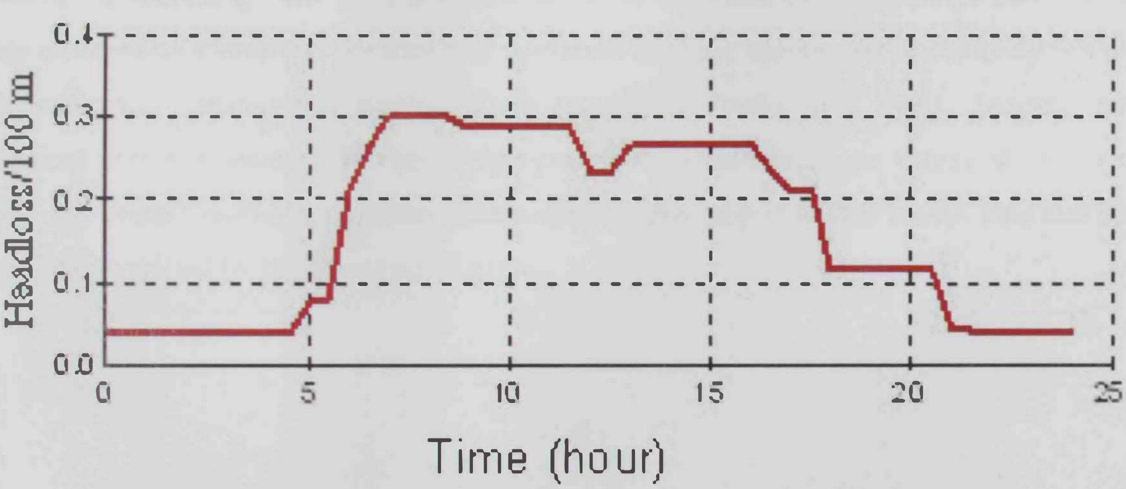

Figure 3.6 Headloss/100 m at portion of pipe 1200 mm before J49.

Figure 3.7 shows the main system (1200 mm pipeline) velocity peaks at 2.6 m/s from 7 00 to 800 hrs and the system velocity fluctuates down to 0.9 m/s at the minimum water supply.

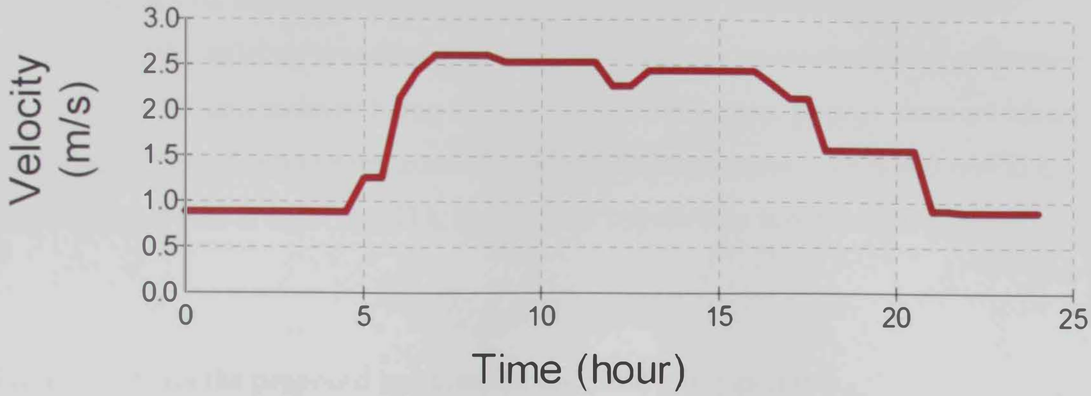

Figure 3.7 Velocity (m/s) at location on pipe 1200 mm before J49.

3.3 Transient Conditions for the Existing System

Hybrid flow-level control valves (FCV/LCV) are installed upstream the existing tanks to control the incoming flow rates and tank levels at the same time. Transient conditions may develop as a result of the mode of operation of these valves and apotentially affect the upstream transmission system. Even though the tanks will likely dampen the transient pressure waves in the short pipe portions downstream those valves, they won't have any impact on the main transmission system; the subject of this study. This can be ultimately verified by InfoSurge simulations.

3.4 Bypass Proposal

As was explained earlier, due to the availability of the incoming flow and pressure requirements from AARS to the demand nodes (Dahma, Maqam and Zakher), it is proposed to bypass the reservoirs and booster pumps. The proposed bypasses are extensions of the existing branched-pipelines to be directly connected to the consumers without being let into tanks or being further boosted by pumps. Bypass sizes are identical to the size of pipes feeding each zone. Therefore, Dahma bypass size is 800 mm D.I, Maqam bypass size is 1000 mm D.I, and Zakher bypass size is 800 mm D.I.

Figure 3.8 shows the proposed bypasses on the 1200 mm pipeline.

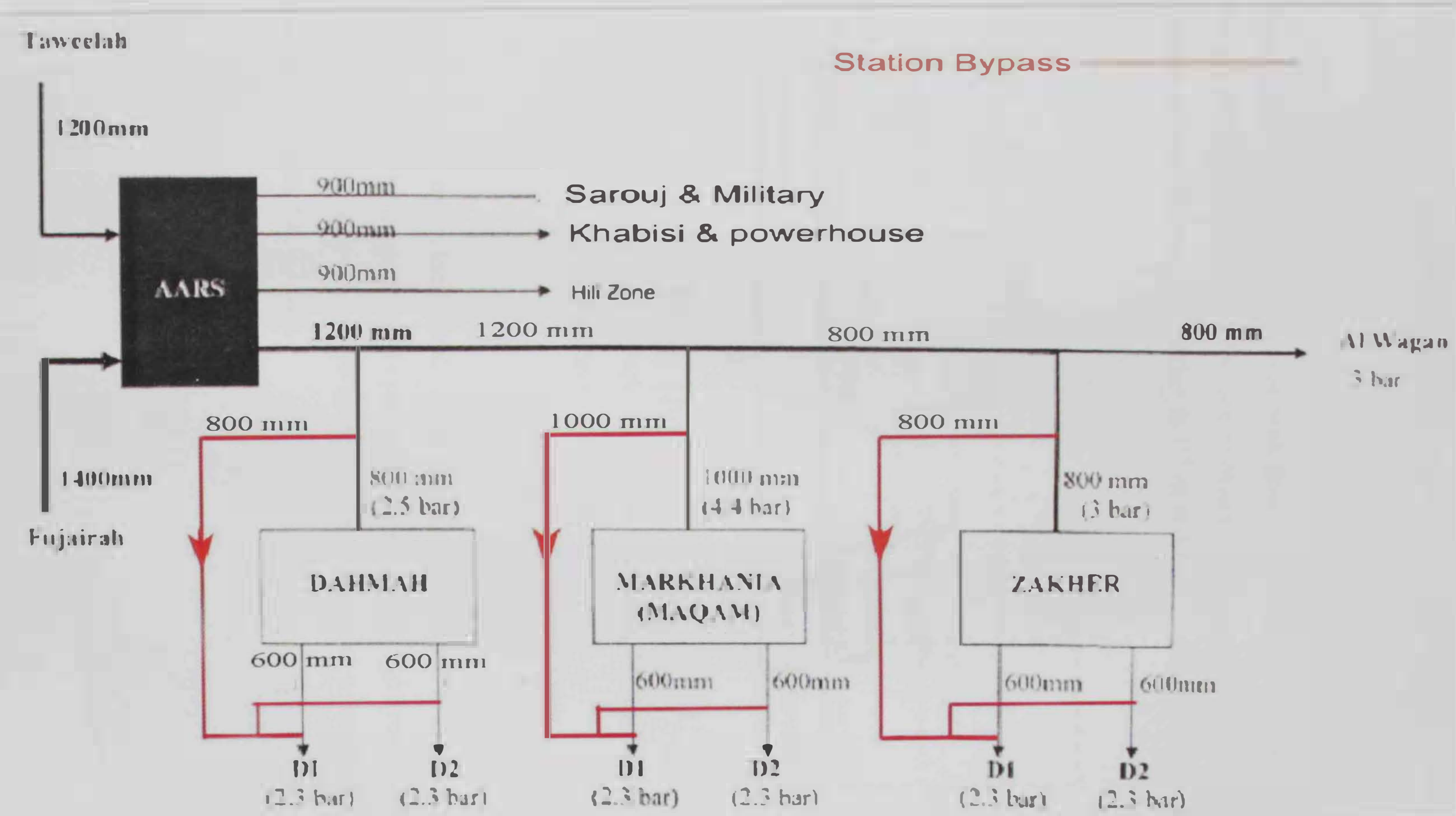

Figure 3.8 The proposed bypasses on the 1200 mm pipeline

Such sizes accommodate the given water supply (Figure 3.3) and the corresponding pressure heads available in the current system (Figure 3.4). This has been verified using EPS as explained earlier.

Figure 3.9 shows that the headloss/600 m at peak flow rate of Dahma is 1.68 m from 6:00 to 1130 hrs, the headloss/km at peak flow rate of Maqam is 1.1 m from 1300 to 1800 hrs, the headloss/km at peak flow rate of Zakher is 1.7 m and the headloss/km at Al Wagen is constant at 0.4 m.

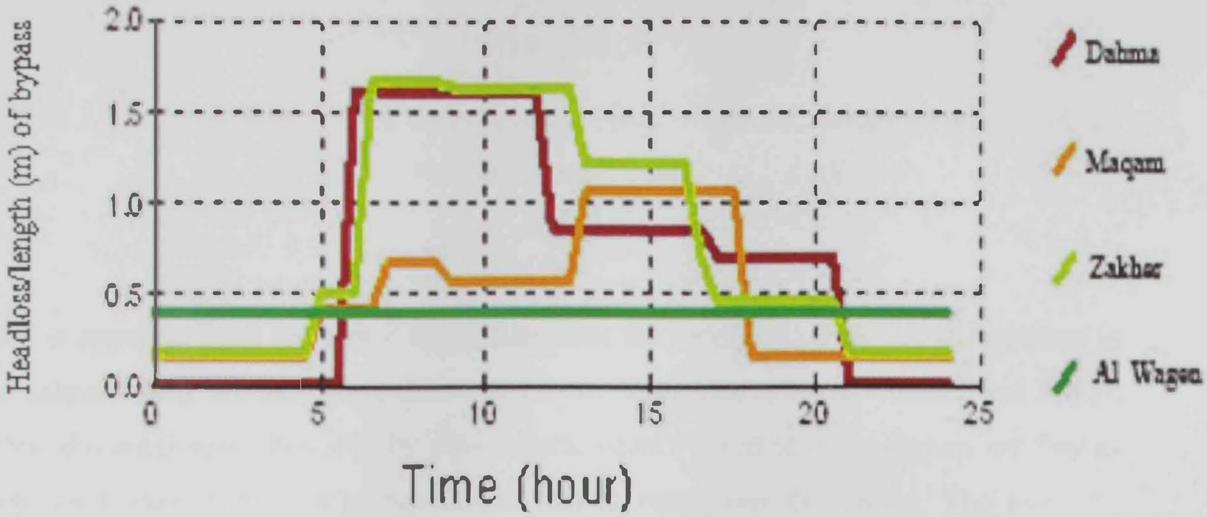

Figure 3.9 Headloss / length (m) for proposed bypass of Dahma, Maqam, Zakher and Al Wagen existing pipe

Figure 3.10 shows that the velocity peak in the Dahma pipe branch is 2.1 m/s from 600 to 1130 hrs, and the velocity peak in Maqam pipe branch is 1.35 m/s from 1300 to 1800 hrs. The Zakher water velocity in the pipe branch is 1.5 m/s and the watwer velocity in Al Wagen pipeline is constant at 0.7 m/s.

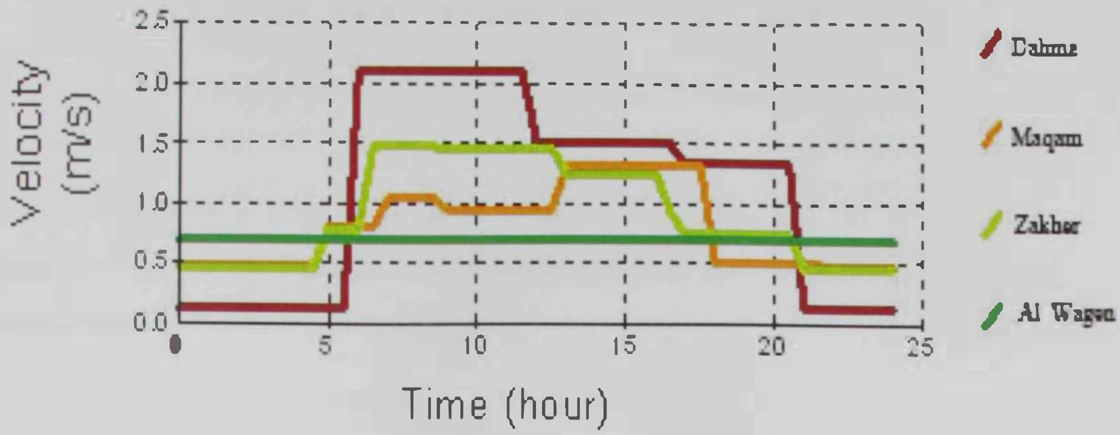

Figure 3.10 Velocity profile for the proposed bypass of Dahma, Maqam, Zakher and Al Wagen existing pipe

As it is apparent from the above simulations that the proposed sizings of all bypasses in the subject study are satisfying their objectives. Velocities are not violated that are all below the maximum allowable by pipe mnufacturers (~ 4 m/sec), headlosses are fine as they don't exceed 1.75 m/branch length (m) at maximum flow rates. The available pressure at all nodes is moderately higher than the distribution requirements but still within the desirable distribution limits. In Chapter V, the evaluation of the bypass proposals during normal and abnormal operations shall be defined.

Chapter IV
MODELS DESCRIPTION AND SETUP

This chapter provides a brief description of the two models utilized in this study; InfoWater and InfoSurge. The main information and data used in setting up a typical simulation using the two models are also presented

4.1 Description of InfoWater Software

InfoWater is a fully GIS integrated water distribution modeling and management software. It integrates advanced water network modeling and optimization functionality with the ArcGIS to work simultaneously on the same integrated platform. It allows commanding GIS analysis and hydraulic modeling in a single environment using a single dataset. InfoWater delivers good levels of geospatial analysis, infrastructure management and business planning. It allows creating, editing, modifying, running, mapping, analyzing, designing and optimizing network models and reviewing query and display simulation results within ArcGIS. It also extends the core features of ArcGIS, providing a comprehensive geospatial environment for complete network model construction, graphical editing, network simulation, results presentation, map generation, and data sharing and exchange. It adds discipline-specific functionality to ArcGIS designed to streamline the water distribution modeling workflow aspects. It helps delivery of informed GIS solutions to meet drinking water quality standards, optimize system performance and capital improvements, and enhance operations. More information about the software can be found in MWH website (MWH, 2006a).

The following is a list of applications of water distribution systems that can be resolved by InfoWater Model:

- Real-time Hydraulic and Water Quality Simulation

- New System Design and Facility Sizing
- Fire Flow Assessment and Leakage Control
- Pump Scheduling and Operator Training
- Operational Study and Emergency Response
- Master Planning and Infrastructure Rehabilitation
- Sampling Program Design and Satellite Treatment Identification
- Energy Consumption Minimization
- Capital Budgeting and Conservation Studies
- System Expansion and Improvement

4.2 Description of InfoSurge Software

InfoSurge is a modeling tool that assists water utilities in analyzing water systems under transient conditions. The program provides information, which, if properly obtained and properly interpreted, aids the utilities in making the right decisions. Continuous updates in research and development effort in transient flow analysis are at a rapid pace to refine InfoSurge. InfoSurge enables to report the water distribution system's response to pump station power failures, valve closures, and pump speed changes, and identify special protection measures to reduce pipe leaks, avoid breaks, investigate control actions and strategies, and improve water quality in distribution systems. A number of devices which are employed to control pressure and flow transients can be modeled by InfoSurge (MWH, 2006b). These include:

- Open and Closed Surge Tanks
- Bladder and Hybrid (vented to admit air) Tanks
- Bypass Lines and Check Valves
- Feed Tanks (provide inflow to prevent cavitation)
- Air Release/Vacuum Valves (2 and 3 stage valves)
- Pressure Relief Valves and Surge Anticipation Valves

InfoSurge is capable of assessing the response of systems to changes in surge protection devices and their characteristics. It can be used to evaluate the relative merits and shortcomings of each protection device when usually the best solution is a combination of several protection devices. It solves the basic conservation equations of fluid mechanics for the transient flow of an incompressible fluid in pipe networks. It can also solve a broader range of problems when coupled with InfoWater's capabilities such as:

- Rapidly varying or transient flow
- Slightly compressible, two-phase fluids (vapor and liquid) and two-fluid systems (air and liquid)
- Closed-conduit pressurized systems with air intake and release at discrete points.
- Network Risk Reduction by performing a hydraulic transient analysis using advanced surge protection
- Reduce the risk of transient-related damage to maximize operator safety and reduce the frequency of service interruptions to customers.
- Reduce the risk of water contamination during sub atmospheric transient pressures, during which groundwater and pollutants could be sucked into the pipe.
- Predict overflows at outfalls or spills to the environment more accurately
- Manage the risk 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. (MWH, 2006b).

4.2.1 InfoSurge Theory

InfoSurge solves the basic conservation equations of fluid mechanics for the transient flow of an incompressible fluid in pipe networks. The wave characteristic method is used, which combines the convergence properties of Lagrangian-type methods (e.g., wave plan method) and Eulerian-type methods (e.g., method of characteristics). The method produces essentially the same results as the method of characteristics, which compares to analytical solutions. The wave characteristic method is Lagrangian in nature and is based on tracking the movement and magnitude of pressure waves as they move along pipes and get transmitted and reflected at the junctions between fixed-length time steps.

Pressure and flow time histories are computed for any point in the network by summing the contributions of incremental waves with time. Friction effects are considered in a distributed manner. The wave characteristic method is conceptually simple and provides valuable insights into the transient behavior of their systems. In addition, for larger systems, the wave characteristic method requires far fewer computational efforts than Eulerian-type methods which require extensive calculations to be carried out in very small time steps and at numerous locations.

It seems that InfoSurge uses a combination of steady state and unsteady state friction methods. The modeling approach converts the partial differential equations (PDEs) of continuity and momentum (e.g., Navier-Stokes) into ordinary differential equations that are solved algebraically along lines called characteristics. An MOC solution is exact along characteristics, but unsteady state friction method, vaporous cavitation, and some boundary representations introduce errors in the results (Gray, 1953; Streeter and Lai, 1962; Elansary, Silva, and Chaudhry, 1994). The wave characteristic method uses common steady-state friction methods, such as Hazen-Williams or Darcy- Weisbach-or more accurate quasi-steady or unsteady (transient) friction methods. It considers simple pump representations or multi-point head-discharge curves, complete with four quadrant characteristics automatically selected based on specific speed. It also uses simple valve representations or multi-point head-discharge (Cv) curves. Finally, the wave characteristic method is less sensitive to the structure of the network and to the length of the simulation than Eulerian-type methods, and allows large systems to be accurately analyzed because of its Lagrangian nature (MWH, 2006b).

4.2.2 Transient Analysis

Usually, hydraulic systems operate at a steady state or dynamic equilibrium and changes in flow take minutes to hours. "Normal" hydraulic transients may occur several times a day as pumps start or stop. "Emergency" transients may only occur once every month, year, or decade when power fails or pipes break. Hydraulic transients and surge-protection needs must be considered in the context of a water utility's risk management and environmental protection plan.

Steady-state hydraulic models simulate systems in which a dynamic equilibrium has been achieved and where changes in head (pressure) or flow (velocity) take minutes to hours in a slow motion. In contrast, InfoSurge simulates hydraulic systems whose balance has been upset by rapid control valve operation, pump trips or other emergencies-all occurring in a matter of seconds or fractions of a second.

Hydraulic transients describe the disturbances in a fluid caused during a change from one steady-state to another. The principle components of the disturbances are pressure changes caused by the propagation of pressure waves throughout the distribution system. These pressure waves continue to propagate with the velocity of sound until they are dissipated down to the level of the new steady-state by the action of some form of damping or friction. Only if the flow is regulated extremely slowly, it is possible to undertake a smooth transition from one steady-state to another without large fluctuations in pressure head or pipe velocity (MWH, 2006b).

4.3 Model Set Up

This section summarizes the main inputs used in defining typical simulations of the system resolved in the study. The hydraulic data needed by InfoWater is first listed then followed by the data needed by InfoSurge. The labels of pipes and nodes are based on the network sketch shown in Figure 4.1.

It should be noted however, that all InfoWater data are also active and available for InfoSurge Model.

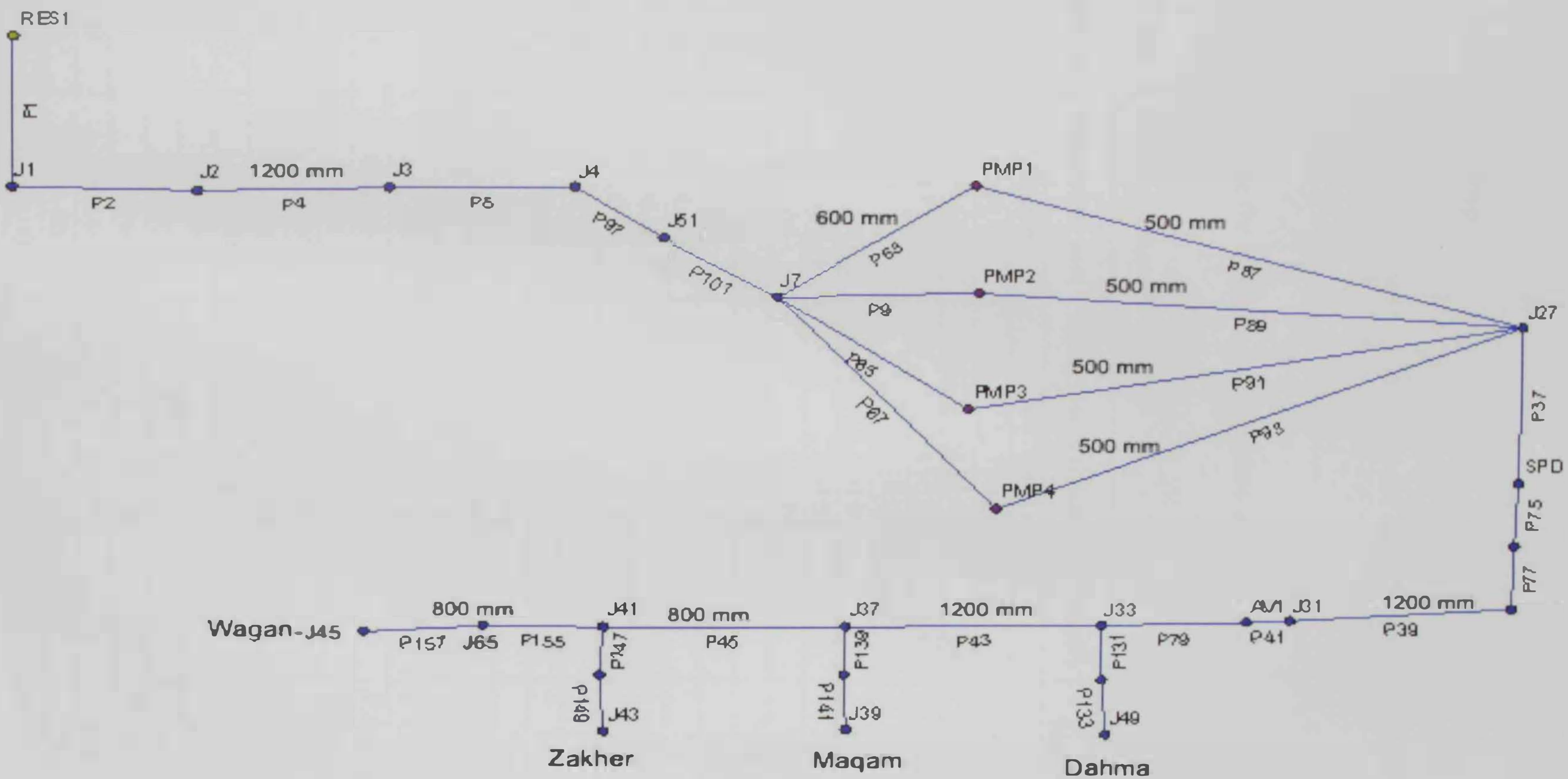

Figure 4.1 Network lay-out as depicted in the simulation models

4.3.1 Data for InfoWater Setup

The main data used in setting up InfoWater Model for the current system is listed below.

Pipes

Table 4.1 shows input data file for all pipes in the system under study.

Table 4.1 Pipes data in InfoWater Setup

Pipe ID	Length, m	Diameter, mm	Roughness (Hazel-Williams)	Check valve
P1	10.00	1,200.00	150.00	No
P101	10.00	1,200.00	150.00	No
P131	500.00	800.00	150.00	No
P133	10.00	800.00	150.00	No
P139	990.00	1,000.00	150.00	No
P141	10.00	1,000.00	150.00	No
P147	990.00	800.00	150.00	No
P149	10.00	800.00	150.00	No
P155	990.00	800.00	150.00	No
P157	10.00	800.00	150.00	No
P2	400.00	1,200.00	150.00	No
P37	100.00	1,200.00	150.00	No
P39	3,000.00	1,200.00	150.00	No
P4	400.00	1,200.00	150.00	No
P41	5.00	1,200.00	150.00	No
P43	7,500.00	1,200.00	150.00	No
P45	7,500.00	1,200.00	150.00	No
P5	100.00	1,200.00	150.00	No
P63	0.70	600.00	150.00	P.Suction
P65	0.70	600.00	150.00	P.Suction
P67	0.70	600.00	150.00	P.Suction
P75	5.00	1,200.00	150.00	No
P77	5.00	1,200.00	150.00	No
P79	100.00	1,200.00	150.00	No
P87	0.60	500.00	150.00	**Yes***
P89	0.60	500.00	150.00	**Yes***
P9	0.70	600.00	150.00	P.Suction
P91	0.60	500.00	150.00	**Yes***
P93	0.60	500.00	150.00	**Yes***
P97	50.00	1,200.00	150.00	No

*Pump Discharge Lines at AARS.

Junctions and Nodes

Table 4.2 shows input data file for all junctions in the system under study.

Table 4.2: Junction Data in InfoWater Setup

ID	Base Flow-1	Flow Pattern 1	Base Flow-2	Flow Pattern 2	Base Flow-3	Flow Pattrn 3	Base Flow-4	Description	Elevation
AV1								Air valve	285.00
J1								New Junction	258.00
J2								New Junction	258.00
J27								New Junction	253.00
J29								New Junction	258.00
J3								New Junction	258.00
J31								Air valve	285.00
J33								New Junction	278.00
J37								New Junction	251.00
J39	0.00		370.00	Flow Pattern 2	0.00		0.00	Maqam Junction	250.00
J4								New Junction	258.00
J41								New Junction	251.00
J43	0.00		0.00		230.00	Flow Pattrn 3	0.00	Zakher Junction	250.00
J45	0.00		0.00		0.00		342.00	Al Wagan	250.00
J49	60.00	Flow Pattern 1	0.00		0.00		0.00	Dahma Junction	278.00
J51								New Junction	258.00
J53								New Junction	278.00
J57								New Junction	250.00
J61								New Junction	250.00
J65								New Junction	250.00
J7								New Junction	253.00
SPD								Closed Surge Vessel	258.00
SPD1								Closed Surge Vessel	258.00

Active Valves

Valves, such as flow control valves, are modeled as a motorized throttle valve (Throttle Control Valve) which is used to describe a variety of valves with potential changes in their stem positions during a transient analysis.

Pumps

InfoWater and InfoSurge automatically generate a three point (quadratic) head-flow characteristic curve for all pumps described by constant power input (horsepower or kilowatts) or by design point curve. For pumps described by a multiple point curve (more than three data points), only three data points will be utilized by InfoSurge (the data points will automatically include the steady-state operating head-flow data point). These pumps should be used for transient analysis only if they always operate in the normal zone of operation (positive head-positive flow). All pumps can include a check valve (prevents reverse flow through the pump), a non-reopening check valve (does not open once it has closed), and/or bypass line (allows flow to bypass the pump when the suction head exceeds the discharge head). Table 4.3 shows input data file for pump set of Markhania Group in AARS.

Table 4.3: Pump data in InfoWater

ID	Type	Elev. (m)	Diameter (mm)	Shut_ Off Head	Design _ Head (m)	Design_ Flow (m)	High _ Head (m)	High_ Flow (m)
PMP1	Multiple Point Curve	253	600	59	47	1094	39	1,333.3
PMP2	Multiple Point Curve	253	600	59	47	1094	39	1,333.3
PMP3	Multiple Point Curve	253	600	59	47	1094	39	1,333.3
PMP4	Multiple Point Curve	253	600	59	47	1094	39	1,333.3

Simulation Options

- Flow Unit = L/s
- Head Loss Equation: Hazen Williams
- Number of Trials = 40
- Accuracy = 0.001
- Specific Gravity = 1
- Un-balances : Stop (√) or Continue
- WQ tolerance = 0.001
- Pressure Unit: meter
- Viscosity = 1×10^{-6} m^2/s
- Diffusivity = 1.21×10^{-9} m^2/s
- Vapor pressure = 0.25536 bar
- Extended Run = 24 hr

Simulation time

- Duration = 24 hr
- 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
- Pattern Start Time = 000 midnight.
- Report Start Time = 000 midnight.
- Clock Start Time = 000 midnight.

Simulation report

- Hydraulic Status (full)
- Node Report (all)
- Link Report (all)
- Generate Network Summary Table (√)
- Generate Warning Messages (√)

Define Elements' Data

- Define reservoir at AARS:
  - Type : Fixed Head Reservoir
  - Head: 263 m
- Define pattern curve for each demand junction (see Figure 3.2)
- Define pump curve type: Multiple Point Curve for Markhania Pump Set at AARS:
  - Pump Elevation = 253 m
  - Shut off Head = 59 m
  - Design Head = 47 m
  - Design Flow = 1094 L/s
  - High Head = 46

- High Flow = 1333.33 L/s
- Pump initial status: open (√) or closed
- Pump Operation Control: start time and closure time; depends on demand change during the day.

- Define Nodes: water supply scenarios, pattern scenarios and elevation data (see Figures 3.1 and 5.1)
- Define Pipes: length, diameter and roughness
- Define Valves: type, elevation, diameter, setting and minor losses.

4.3.2 Data for InfoSurge Setup

The main data used in setting up the InfoSurge Model for the current system is listed below.

1. Change of Water Supply
2. Pump Surge data
    - Check valve installed (√)
    - Cv resistance = 0.001
    - Cv opening/closing time = 0.01
    - Non re-open (√)
    - No pump bypass installed ( )

3. Open Surge tank:
    - Inflow Resistance = 0.1 where, R = (Head Loss: ΔH) / Q^2
    - Outflow Resistance = 0.1
    - Diameter = 1200 mm
    - Maximum tank Level = 5 m

4. Closed Surge tank:
    - Inflow Resistance = 0.45
    - Outflow Resistance = 0.45
    - Tank volume = 70 m^3
    - Initial Gas Volume = 35 m^3

- Exponent Constant for air = 1 to 1.4

5. Air Valves:
    - Inflow Diameter = 100 mm
    - Outflow diameter = 100 mm
    - Initial air volume = 0 m^3

6. Pressure Relief Valves:
    - Inflow resistance = 0.5
    - Outflow resistance = 0.5
    - Opening pressure =100 m
    - External pressure = 0 m
    - Closing pressure = 80 m
    - Opening time = 20 s
    - Closing time = 120 s

Run Manager

(a) Select Active Standard (√) for InfoWater steady state and EPS analysis.

(b) Select Active Surge (√) for InfoSurge with the following parameters to be filled:

- Global Wave Speed : 1037.4 m/sec
- Cavitation Head : -10 m
- Pipe Segment Length Tolerance : 3
- Simulation Run Duration (Seconds) : 43200 or 86400
- Pressure Sensitive Demand (√)
- Exit head:= o
- Intrusion Calculation Type: No Intrusion Calculation.
- Monitoring Node: J-31 (the highest elevation 285 m in the entire transmission system; Point–A in Figure 3.1).
- Monitoring Pressure Head Range: Minimum = – 20 m & Maximum = + 200 m

Output Report: After a hydraulic run, out put report is generated for all elements either graphically or numerically.

4.3.2.1 Estimation of the Wave Speed (c)

The wave speed depends on four major factors; Type of Liquid (Bulk Modulus), Pipe Material (Young's Modulus), Pipe Properties (D_i / Wall Thickness) and Pipe Restrainment (pipe support and anchors). However, small quantities of air result in reducing the wave speed and leads to non-conservative estimate of c. The presence of trapped air in the pipe has been ignored in this study. The wave speed (c) is determined based on a 1200 mm ductile iron pipe since the long pipes in the system have this diameter. Such pipes have a wall thickness = 15.3 mm, Young's Modulus = 172 GPa for ductile iron and Poisson's Ratio (μ_p) = 0.3.

The K_R of 0.91 was calculated from the equation $\mathbf{K_R = 1 - \mu_p^2}$ that considers a pipe anchored against longitudinal movement from both sides. The assumed type of constraints provides a conservative estimate of the wave speed. Using Figure 2.6 *[where pressure wave velocity for water in round pipes with different diameters and thicknesses and K_R of 0.91 if pipe is anchored against axial-longitudinal-movement and μ_p = 0.3 for ductile iron (Wood et al. 2005; Fleming et al. 2006)]*, the internal pipe diameter / pipe thickness is 78 which corresponds to a wave speed of 1037.4 m/s. This value is used for all pipes and branches in further simulations where it is recommended to consider a constant wave speed to avoid numerical instability.

CHAPTER V

EVALUATION OF THE PROPOSED BYPASS

This chapter addresses water supply changes during the day, pump operation, normal and abnormal valve operation in terms of the effect of valve closure times, system head recovery times and system head variations. The latter three parameters are analyzed in consideration of pipeline pressure rating, pump shutoff head at Al Ain Water Reception Station (AARS) and cavitation limits. The chapter also examines remediation measures for abnormal valve operation, pump trips, various sectional discussions, evaluation of the bypass economics for booster stations and the pay back period.

Refer to figure 3.1 that shows the supply nodes on the water transmission 1200 mm pipeline from AARS with elevations reference to sea level.

5.1 Simulation Operating Modes

The results discussed in this chapter are obtained from two modes of operation; the Normal Operation Mode and the Abnormal Operation Mode. The normal operation mode considers the transient events possibly developed during typical operating conditions while the abnormal operation mode considers the transient events associated with rare and unplanned conditions. Further explanation of the two considered models of operation is presented below.

The opening and closing of the flow control valve at each node has its own hydraulic implication and effect on the system. Normally the transients associated with valve opening are less severe than the ones associated with valve closures. The opening of flow control valve at each node is associated with increased supply. Regardless of how fast the valve opening is, the head rise in the system should not exceed the pipeline rated pressure. The sudden drop in pressure though may cause vacuum in the system if it doesn't have sufficient surge protective devices such as air valves and

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The peak water supply in the considered system starts daily, at 6 AM until 6 PM which corresponds to 3 pump operation at AARS to feed the tap off connections which in turn feed the customers of Al Ain Distribution Company (AADC). The normal water supply starts, daily, at 5 AM until 6 AM and 6 PM until 9 PM which corresponds to 2 pump operation at AARS. The low water supply starts daily, at 9 PM until 5 AM which corresponds to 1 pump operation at AARS. Figure 5.1 shows the operating pumps at different water supply periods.

There are seven daily water supply changes on the system; four of which are major changes. During the valve opening and closing to accommodate normal water supply changes (increase or decrease), the condition is described by "Normal Operation".

The abnormal operation is enccountered during the sudden valve opening and closing, due to poor operation, pipe ruptures, power failures, gate valve sudden drop and improper design of fast-operating flow control valves.

5.2 Normal Operating Conditions

This section discusses the change of water supply at different nodes associated with either gradual or sudden partial flow control valve closures or openings to reach the desired water flow rates according to the distribution daily programs at different nodes in the system.

5.2.1 Partial Valve Closing Periods

Here, the water supply reduction is associated with gradual or sudden partial flow control valve closures. Figure 5.1 shows that a typical day involves seven instants of flow reductions described as follows:

- R1: Minor flow reduction at 09:00 AM due to drop in water supply requirements at Zakher and Maqam (Markhania) nodes.

- R2: Major flow reduction at 11:30 AM due to drop in water supply requirements at Dahma node.
- R3: Major flow reduction at 04:30 PM due to supply drop at Zakher node.
- R4: Major flow reduction at 5:00 PM due to supply drop at Dahma and Zakher nodes.
- R5: Major flow reduction at 6:00 PM due to water supply drop at Maqam node.
- R6: Major flow reduction at 9:00 PM due to supply drop at Dahma and Zakher nodes.
- R7: Minor flow reduction at 10:00 PM due to supply drop at Maqam node.

w Distribution	Flow (L/Sec)												
	0:00 – 05:00	05:00 – 6:00	06:00 – 6:30	06:30 – 7:00	07:00 – 9:00	09:00 – 11:30	11:30 – 13:00	13:00 – 16:30	16:30 – 17:00	17:00 – 18:00	18:00 – 21:00	21:00 – 22:00	22:00 – 22:00
Dahmna Flow	60	60	1050	1050	1050	1050	750	750	750	670	670	60	60
Maqam Flow	370	625	625	625	825	725	725	1025	1025	1025	390	390	370
Zakher Flow	230	390	390	740	740	730	730	630	460	360	360	230	230
Al Wagon Flow	342	342	342	342	342	342	342	342	342	342	342	342	342
Total Flow	**1002**	**1417**	**2407**	**2757**	**2957**	**2847**	**2547**	**2747**	**2577**	**2397**	**1762**	**1022**	**1002**

Base I1 I2 I3 I4 Maximum R1 R2 I5 R3 R4 R5 R6 R7

I: water supply increase

R: water supply reduction

Figure 5.1 Hourly supply changes and associated operating pump

Each of the above reduction cases involves a rise of the hydraulic head at the source of supply reduction and other points in the entire system particularily the nodes at the low elevation (Maqam, Zakher and Al Wagen). The rise of the hydraulic head is associated with a recovery time after which the system gets back to its equilibrium condition as the effect of increased head on the system diminishes. Both, the increased head and the recovery time are affected by the period of valve closures. A sample result of such impact is depicted in Figure 5.2 that shows the change of head with time for the cases of flow reduction at Dahma node at 11:30 AM considering a sudden valve closure. The closure time was set in the simulation model, in this case, at one second to avoid numerical instability. The recovery time is estimated from figure 5.2 as 1321-599 = 722 seconds. The system head after the recovery time is higher than its original value before the valve closure occurance due to water supply reduction at Dahma node that occurs simultaneously with the valve closure.

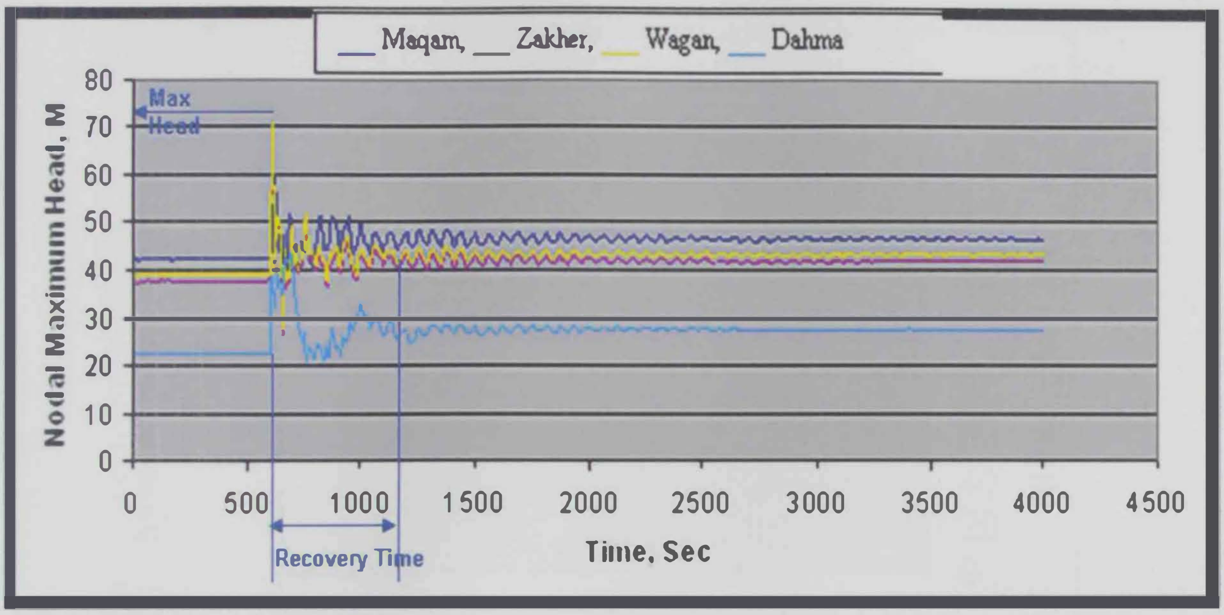


Figure 5.2 Nodal maximum head versus time for the sudden valve closure at Dahma

Four major reduction cases; R2, R3, R5 and R6 were simulated and the simulated head and recovery time are pllotted versus closure times for each case. All flow reduction simulations were carried out for 11 hours (39600 seconds) starting at 11:20 AM to 10:20 PM. This means the zero time for these simulations represents 11:20 AM in Figure 5.1.

5.2.1.1 Flow reduction (R2) at Dahma at 11:30 AM

Figure 5.3 shows the maximum nodal head and system head recovery times versus valve closure times at Dahma PS. The considered valve closure times are: 1, 30,60 ,90 ,120 and 150 seconds. The simulated system head recovery times are: 722, 478, 233, 180, 127 and 75 while the simulated maximum nodal heads are: 71, 50, 48, 47, 47 and 47; respectively. Such results indicate that it is acceptable and safe for Dahma PS valve to close in at least 60 seconds which is sufficient time for the system head (pressure) to settle back within the acceptable range of pipe and fittings pressure ratings. Although the pipeline itself can stand the transients resulted from 1 sec and 30 sec valve closures, the recovery time for the system head is long enough to cause major instability. This may cause the bolts to get loose, the gaskets to get ruptured, the spigots/sockets to get disconnected and hence water leaks may develop. Therefore, it is unsafe for Dahma valve to close in less than 60 seconds.

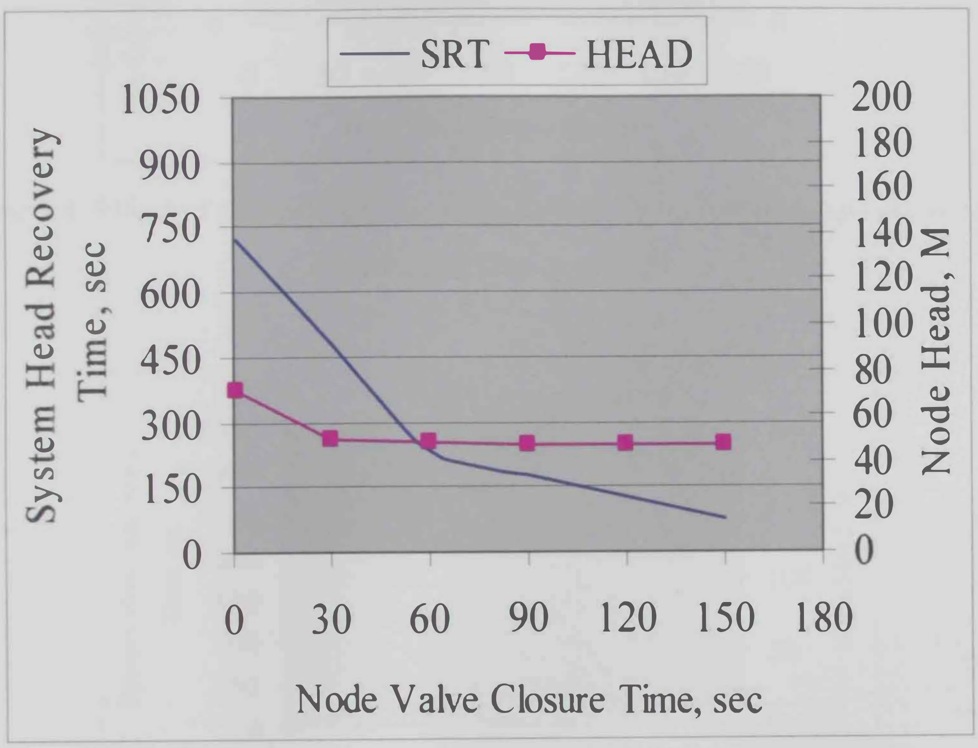


Figure 5.3 Effect of valve closure times at Dahma PS on maximum nodal head and system head recovery times

5.2.1.2 Flow reduction at Other Supply Nodes

Figures 5.4, 5.5, and 5.6 show the maximum nodal head and system head recovery times versus valve closure times for flow reductions applied at Zakher PS (@ 04:30 PM), at Maqam (@ 6.0 PM), and at Dahma and Zakher (@ 09:00 PM), respectively. As before, the considered valve closure times are: 1, 30, 60, 90, 120 and 150 Seconds. Similar results and observations found with Dahma case (Sectn 5.2.1.1) are noticed here.

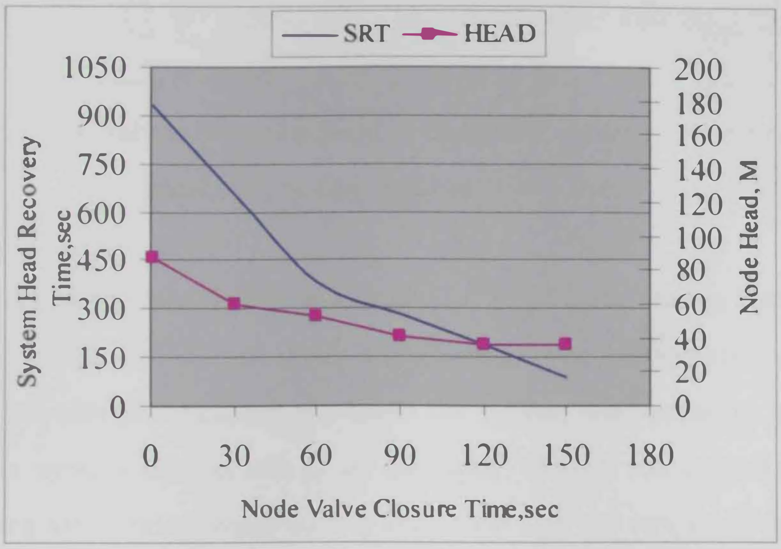


Figure 5.4 Effect of valve closure times at Zakher PS on maximum nodal head and system head recovery times

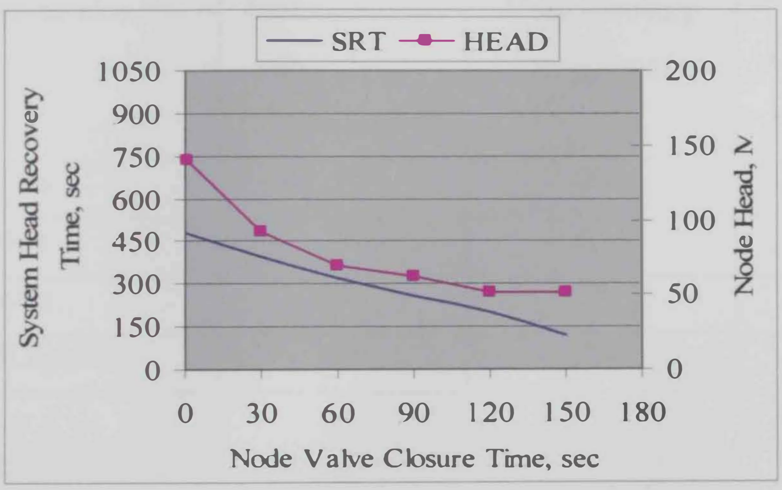


Figure 5.5 Effect of valve closure times at Maqam PS on maximum nodal head and system head recovery times

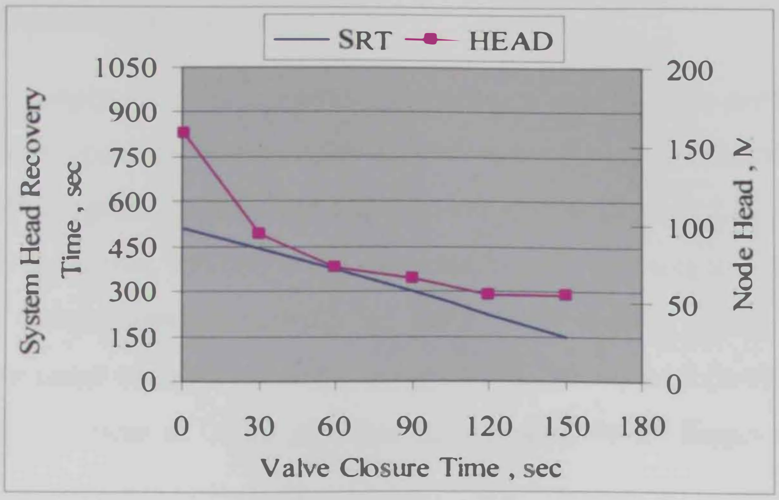


Figure 5.6 Effect of valve closure times at at Dahma & Zakher PS on maximum nodal head and system head recovery times

As a summary, the above simulation results and table 5.1 have shown how slow and how fast the control valves at different water supply nodes can be operated. The slower the water flow control valves are closed, the faster the system head/pressure is recovered and the safer for the system fittings not to be damaged. Hence, the overall system control valve safe closing time during water supply reduction is in the range of 90 to 150 seconds for all nodes.

Table 5.1 Summary of valve safe closure time (sec), head recovery time and nodal head

Location of valve closure	Minimum Safe valve closure time (sec)	Head recovery time (sec)	Maximum Nodal Head (m)
Dahma	60	200	47
Zakher	90	253	40
Maqam	90	247	53
Dahma & Zakher	90	280	55

5.2.2 Valve Opening Periods

Here, the water supply increase is associated with gradual, sudden, partial or complete flow control valve openings to reach the desired water flow rates described as per the distribution daily programs at different nodes in the system (Figure 5.1). The daily valve opening periods in Dahma, Maqam and Zakher starts early morning at 5 AM until 1 PM. This period is always associated with the peak water supply to the system nodes. Specifically, five cases of increased flows occur during the day as follows:

- I1: Increased flow at 05:00 AM due to increased water supply to Maqam and Zakher.
- I2: Increased flow at 06:00 AM due to increased supply to Dahma.
- I3: Increased flow at 06:30 AM due to increased supply to Zakher.
- I4: Increased flow at 07:00 AM due to increased supply to Maqam.
- I5: Increased flow at 01:00 PM due to increased supply to Maqam.

The above cases were all simulated considering the worst scenarios of valve opening in which the valve opening was set at 1 second. The results (not shown here) indicated the pump shut-off heads and the pipe rating pressure are never reached. The vacuum in the system is taken care of by air-pressurized surge vessels at the discharge of the pumps of AARS and adequately sized air valves at high elevations. The water supply/demands are instantaneously changed with the valve opening because the distribution system in this study is defined by a pipe branch taken from the transmission system to fill customers domestic tanks that are closed by float valves when they are full. This type of distribution system is not a pressurized network right to the customers tabs; it is composed of domestic tanks that holds water just enough for one day consumption.

5.3 Abnormal Operating Conditions

In this section, sudden complete flow control valve closures at different nodes and pump trips at AARS during peak water supply are simulated to represent the most severe conditions for the system analysis. Even though rare and to some extent hypothetical; these conditions can result in undesirable impacts on the proposed bypass setting that implies specific remediation measures.

5.3.1 Scenarios of Sudden Complete Valve Closures

Here, the sudden complete flow control valve closures at different nodes during peak water supply are considered. The valve closures in these cases are assumed to happen aprubtly, i.e 1-second time in the model setup.

5.3.1.1 Complete valve closure at Dahma

The flow control valve installed at Dahma bypass is suddenly closed during the peak supply in the period of 6:00 AM to 11:30 AM when the flow reaches 1050 L/s.

Figure 5.7 shows the simulated heads at four nodes representing the four supply zones. The simulation started at 6:00 AM and ended at 6:00 PM. The closure was set at 07:45 AM that corresponds to 6300 seconds in Figure 5.7. It is noticed that the maximum system head reaches 120 m at Wagan Node while the minimum system head reaches -10 m (cavitation) at Dahma Node itself. The existing surge protective devices (air valves and surge vessels) can not cope up with such severe transient conditions; additional surge devices shall be examined in later stages as part of this study.

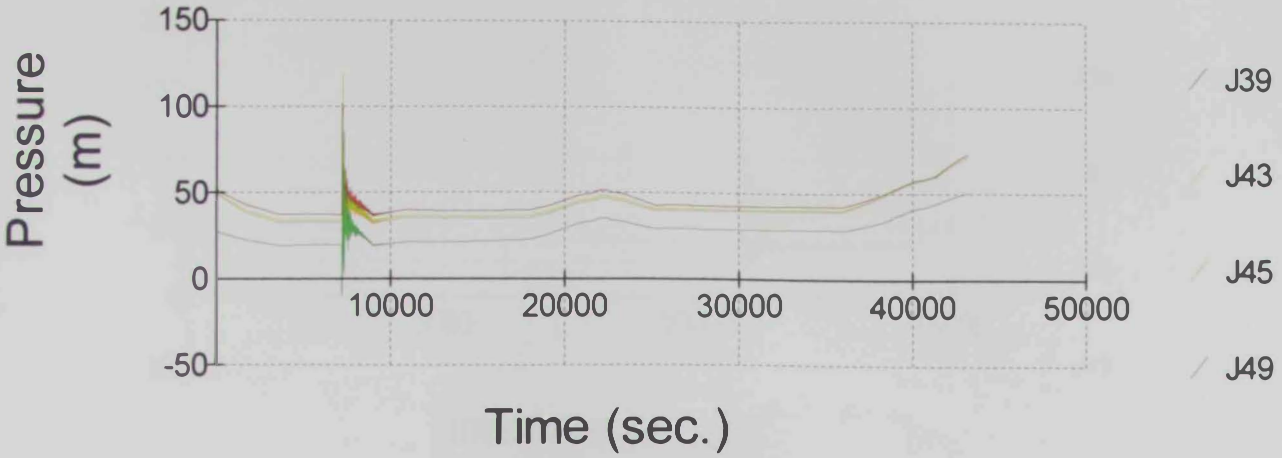


Figure 5.7 Hydraulic head versus time at different nodes for the case of valve closure at Dahma Node (J-39: Maqam, J-43: Zakher, J-45: Wagan, and J-49: Dahma)

5.3.1.2 Complete valve closure at Maqam

The flow control valve installed at Maqam bypass is suddenly closed during peak demand in the period of 1:00 PM to 6:00 PM when the flow reaches 1025 L/s. Figure 5.8 shows the simulated heads at four nodes representing the four demand zones. The simulation started at 1:00 PM and ended at 5:00 PM. The closure was set at 4:00 PM that corresponds to 10833 seconds in Figure 5.8. It is noticed that the maximum hydraulic head reaches 197 m at Maqam Node itself while the minimum system head reaches -10 m at four nodes (J-39: Maqam, J-43: Zakher, J-45: Wagan, and J-49: Dahma). The transient conditions are very severe in this case that exceed the pipeline design ratings from both prospectives (high and low system heads).

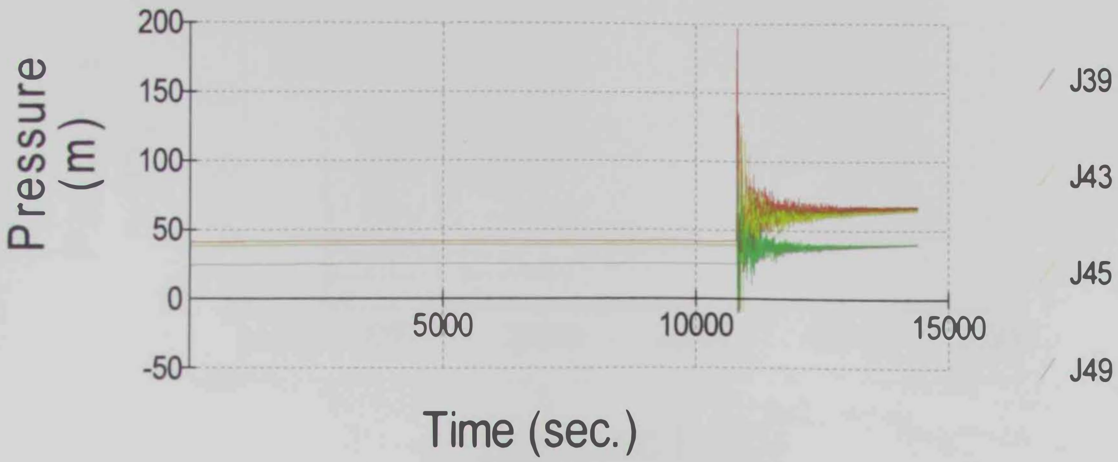


Figure 5.8 hydraulic head at different nodes versus time during Maqam node valve closure (J-39: Maqam, J-43: Zakher, J-45: Wagan and J-49: Dahma)

5.3.1.3 Sudden complete valve closure at Zakher

The flow control valve installed at Zakher bypass is suddenly closed during peak supply in the period of 1:00 PM to 6:00 PM when the flow reaches 740 L/s. Figure 5.9 shows the simulated heads at four nodes representing the four supply zones. The simulation started at 6:00 AM and ended at 6:00 PM. The closure was set at 07:45 AM that corresponds to 6300 seconds in Figure 5.9. It is noticed that the maximum hydraulic head reaches 196 m at Maqam Node itself while the minimum system head reaches -10 m at four nodes (J-39: Maqam, J-43: Zakher, J-45: Wagan and J-49: Dahma). The transient conditions are very severe here similar to the previous case of Maqam.

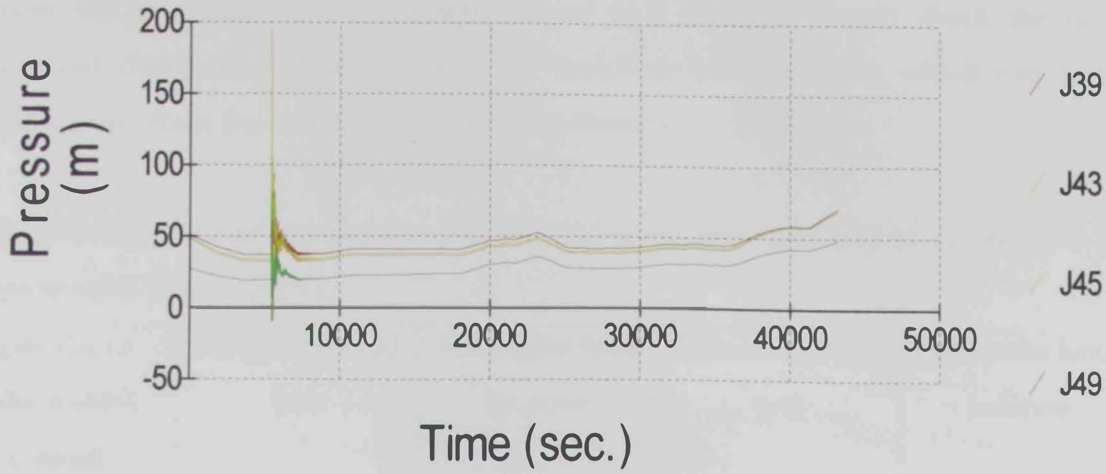


Figure 5.9 Hydraulic head at different nodes vrsus time during Zakher node valve closure (J-39: Maqam, J-43: Zakher, J-45: Wagan and J-49: Dahma)

5.3.1.4 Discussion of the above simulation results

The classified pressure rating of the pipes is reached and even exceeded during fast or sudden complete closure of control valves especially at nodes of low elevations like Maqam and Zakher. Although the pipeline itself can withstand 20% higher pressure than its rating to account for major transient events, the bolts normally get loose, gaskets get ruptured, spigots/sockets get disconnected and hence leaks may develop. As a summary, the above simulation results and table 5.2 have shown how serious is the effect of fast/sudden valve closures of control valves at different demand nodes. Hence, to cater for sudden valves closures during high demands at the lowest nodes of the system (Maqam, Zakher and Wagen), pressure relief valves at the pipe branches of all nodes are examined. The goal is to maintain the system head (pressure) within safe limit for the system fittings not to be damaged and the water supply to customers not to be interrupted. More discussion on the use of pressure relief valves is presented in Section 5.4 after discussing the pump trip scenarios.

The distribution systems are designed to withstand the same pressure rating similar to the transmission system and they have their own surge protective devices as well. However, the distribution systems need to be further checked through new surge simulations to confirm the adequacy of their design. There is a concern though about the non-pressurized distribution systems about the backflow (reverse flow) which can bring contamination from the surrounding into the system.

Table 5.2 Summary of maximum head (m) and minimum head (m) in the absence of pressure relief valves (PRV).

Location of sudden valve closure	Maximum head (m)	Maximum head location	Minimum head (m)	Minimum head location
Dahma	120	Al Wagen	-10	Dahma
Maqam	197	Maqam	-10	All nodes
Zakher	196	Zakher	-10	All nodes

5.3.2 Scenarios of Pump Trips

Pumps are an integral part of many pressure systems. Pumps add energy; or head gains, to the flow to overcome the head losses and hydraulic grade differences within the system. From a hydraulic transient perspective, certain dynamic variables must be considered, including: power input; rotating speed; and the moment of inertia of the pump, motor and shaft (including couplings). In uncontrolled valve opening, the system head curve can suddenly drop far below its usual head requirement, and the pump no longer needs to add much (if any) energy to supply the required flow. In such cases, the pump's run-out head becomes higher than the required static lift. Under such conditions, very large losses in the suction system may result in cavitation and overspeed conditions, both of which can cause pump wear and even damage. This can be avoided by proper pump controls to shut the pump down and reduce or stop flow during such transients. Low-inertia pumps maintain forward flow for a shorter length of time and stop sooner after a power failure. This result in more sudden changes in flow and pressures than would occur with heavier pumps, and consequently in more severe water hammer in most cases.

Al Ain Water Reception Station (AARS) is the main source of water supply to the four nodes considered in the current study. For the available four medium size pumps; the number of pumps in operation depends on the hourly nodal water supply changes in 24-hour cycle. A maximum of three pumps can be in operation during peak hours. The fourth one is on stand bye. Specifically, three cases of pump operation scenarios occur during the day, as the following:

- One pump operation from 9:00 PM to 5:00 AM (the next day).
- Two pump operation from 5:00 AM to 6:00 AM and at 6:00 PM to 9:00 PM on daily basis.
- Three pump operation at 6:00 AM to 06:00 PM on daily basis.

This section discusses the impacts of trip of one and two pumps upon the system performance in the presence of the proposed bypass.

5.3.2.1 <u>Trip of one pump</u>

The current set of scenarios simulates the trip of one pump (sudden trip) and leaving two pumps in operation. One pump trip was set at time 6300 sec (07:45 AM) to capture the extreme conditions of peak flow.

Figure 5.10 shows the pressure profile for Dahma Node(J49), Maqam Node (J39), Zakher Node (J43) and Wagan Node (J45). It is clearly observed that the available pressure at all nodes drops about 13 Meters below the normal operation conditions. This causes system instability and some transient surge but all stayed above the cavitation pressure in the whole system. The minimum pressure is found to be 7 m above zero indicating the adequacy of the existing surge protective devices (SPD) installed in the system.

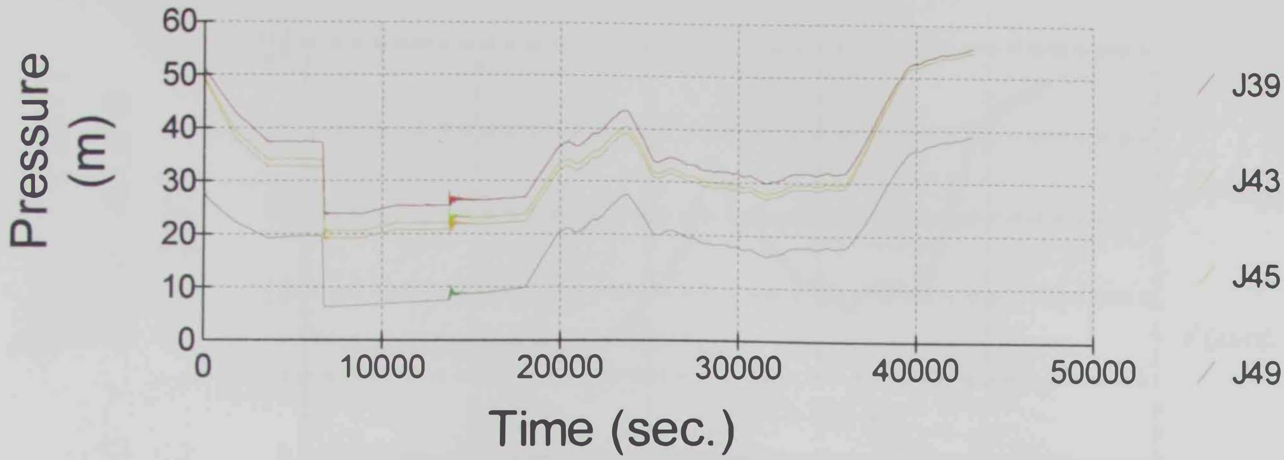


Figure 5.10 Nodal pressures for 1 pump trip scenario (J-39: Maqam, J-43: Zakher, J-45: Wagan and J-49: Dahma)

Figure 5.11 shows the available suction pressure of two pumps at AARS when 1 pump trips out of 3 pump operation at 6300 seconds. During peak supply 6:00 AM to 6:00 PM, the suction pressure decreases slightly below 10 m (to 7.5 m). At 19800 seconds (11:30 AM) when water supply at Dahma drops by 300 L/s, the suction pressure increases to 18 m. At 25000 seconds (1:00 PM), the suction pressure starts to decrease till reaching 10 m when supply increases by 300 L/s at Maqam. At 37800 seconds (4:30 PM), the suction pressure starts to increase until it reaches to 23 m when supply starts to decrease at Zakher followed immediately by three other water reductions at Zakher and Dahma at 39600 seconds (5 PM) and Maqam at 43200 seconds (6 PM). As seen in the figure, the conclusion is that suction pressure of pumps is very sensitive to supply changes, valaves opening and closures. The explanation for this phenomenon is that during one pump trip; the two remaining pumps jump to full speed (full flow) to compensate the loss of 1 pump resulting in more transient events sensed in the suction side.

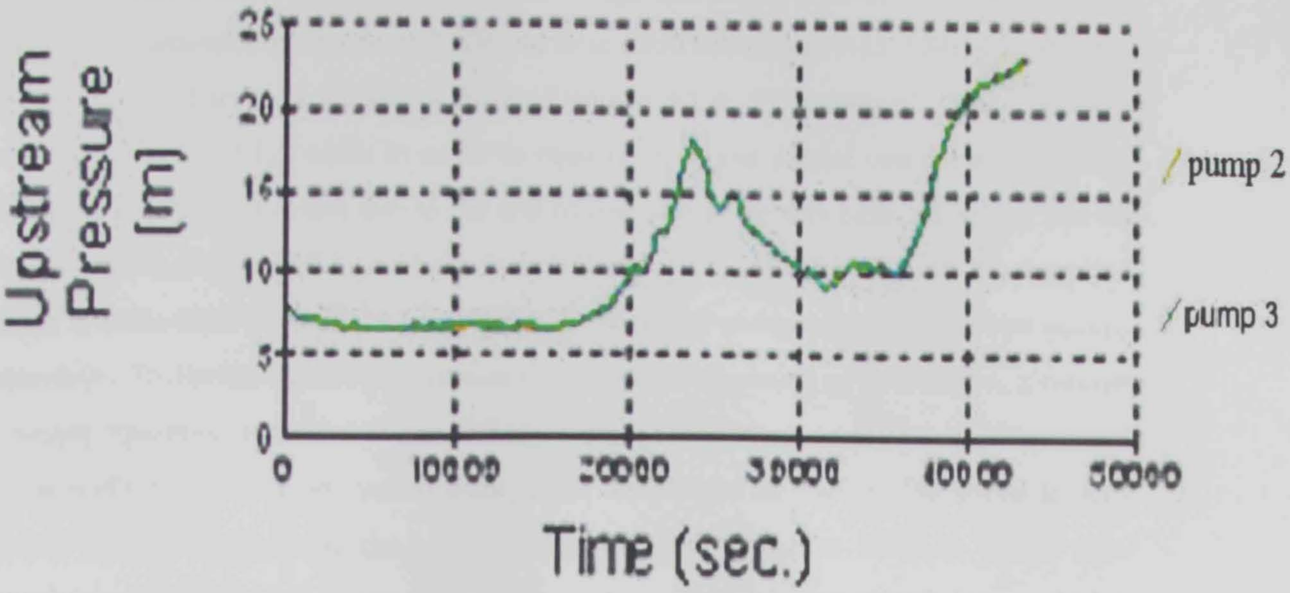


Figure 5.11 Upstream pump pressures at AARS for a one pump trip scenario.

Figure 5.12 shows the available discharge pressure of two pumps at AARS when 1 pump trips out of 3 pump operation at 6300 seconds. The discharge pressure decreases to a minimum of 42 m and stays steady (slightly erratic) all the way until the end of the period. The shape of the graph for downstream pressures mimics almost the pressure changes in the upstream pressures.

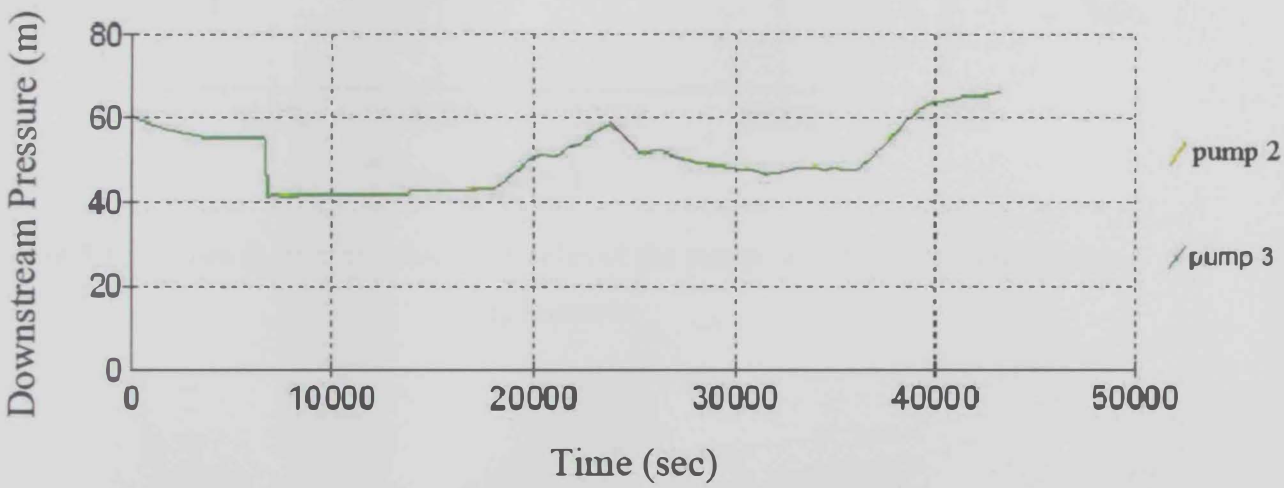


Figure 5.12 Downstream pump pressures (m) at AARS for a one pump trip scenario.

Figure 5.13 shows the water flows, at the discharge side, of two pumps at AARS during the trip of transmission pump #1 at AARS at time 6300 seconds (07:45 AM). The flow of pump #1 dropped to zero while the flow of pumps #2 & #3 increased instanteneously from 983 L/s to 1350 L/s each; in order to equally share the partial compensation of the water shortage of 983 L/s lost due to the trip of one pump. In this case, pump # 2 and #3 flow rates increase by 367 L/s. There is a difference of 249 L/s between the supplied quantity and the flow rates of the two pumps that can't be compensated by the two pumps in operation. So for the model to continue through the full period of simulation, although the supply quantity is not met, an external flow equivalent to 249 L/s is assumed hypothetically to get into the system from surge vessels and air valves. The surge devices normally supply water into the system during rapid drops in head to avoid sub-atmospheric pressures and water column separation. This alleviates both low and high pressures in the system.

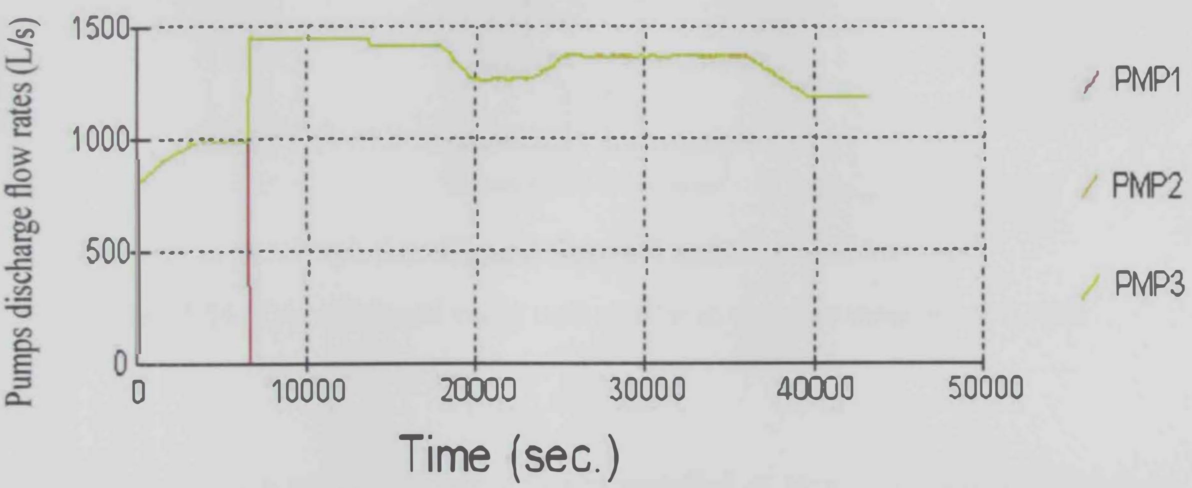
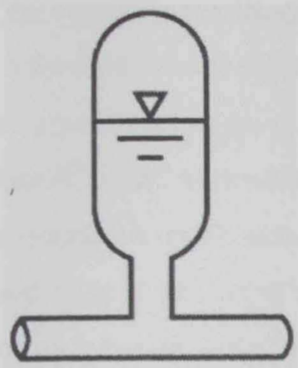


Figure 5.13 Flows (L/s) at the discharge sides of the pumps at AARS for a one pump trip scenario

A gas vessel (also known as an air chamber or hydro-pneumatic tank) is a pressure vessel that contains water and a volume of air that is maintained by an air compressor. When pumps are shut down and the flow and pressure decrease the air in the chamber expands as a result of the pressure drop and water enters the system from the chamber. Redundant compressors are required to inject air at the correct pressure into the gas vessel because the pressurized air will dissolve into the water as time passes. It controls pressure surges generated by rapid changes in flow at a pump station. The surge tank supplies water into the system during rapid drops in head to avoid sub-atmospheric pressures and water column separation. This alleviates both low and high pressures in the system. Its size should be sufficient to prevent it from draining completely (to prevent air intrusion into the system) and to prevent it from overflowing when pressures increase again and the tank refills during the transient (see Figure 5.14).



Closed Surge Tank

Figure 5.14 Typical closed surge tank similar to the ones installed in AARS

Figure 5.15 shows a typical air valve that is installed at local high points (Point-A in Figure 4.1) to allow air enter the system during periods when the head drops below the pipe elevation and expels air from the system when water columns begin to rejoin. The presence of air in the line will limit sub-atmospheric pressures in the vicinity of the valve and for some distance to either side.

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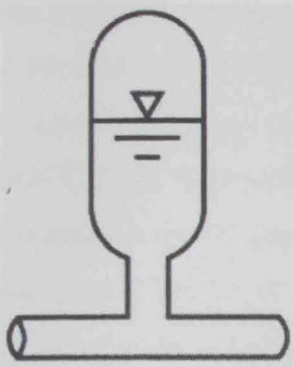

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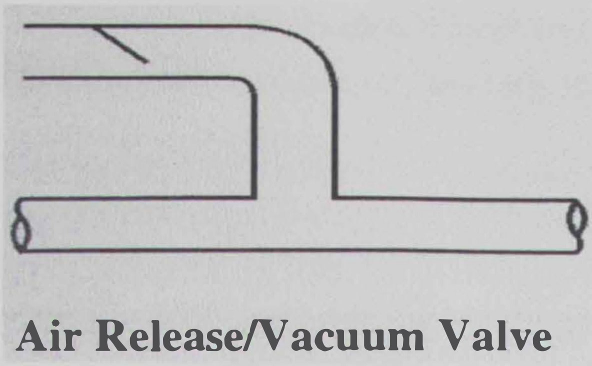

Figure 5.15 Typical air release valve similar to the ones installed on the main 1200 mm pipeline

Figure 5.16 shows the available inlet pressure of one surge vessel and one air valve at pumps' main discharge 1200 mm pipeline. The inlet pressure to the surge vessel is supposed to be slightly less than the available discharge pressure of the pumps at AARS and follow the same erratic trend after a one pump trip. The inlet pressure to the air valve is supposed to be equivalent to the lowest system pressure (21 m) at the discharge of the pumps at the highest elevation point (EL=285 Meters in figure 3.1). During the case of a one pump trip out of three pump operation (peak water supply) the inlet pressure head decreases to a minimum of zero and stays at this value for 2 hours then increases slightly to 18 m, after wards the inlet pressure to the air valve approaches 30 m.

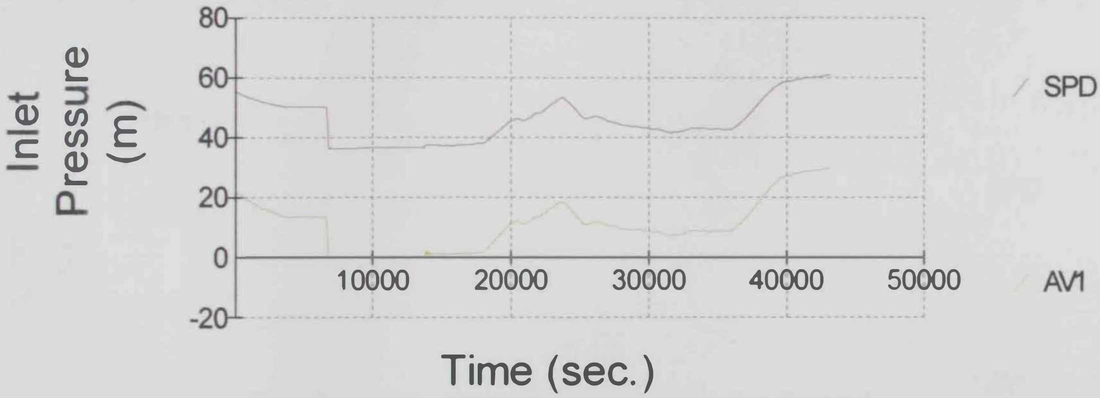

Figure 5.16 Inlet pressure of one surge vessel and one air valve for a one pump trip scenario.

Figure 5.17 shows the external water inflow/outflow through air valves (-70 / + 70 L/s). In this scenario, +35/-110 L/s of water are drawn off / into each individual surge vessel to damp the surge waves resulted from the trip.

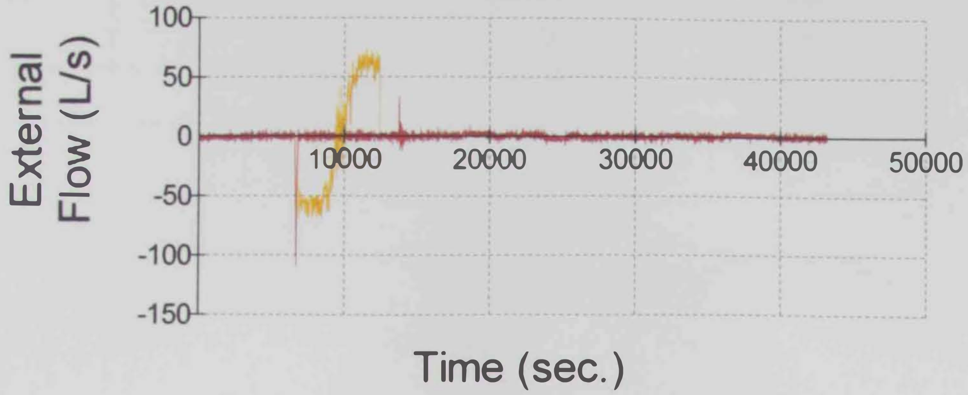

Figure 5.17 External water inflow/outflow of the surge vessels for a one pump trip scenario.

Figure 5.18 shows the external compressed air volume to pressurize the surge vessels (35 m^3 in normal operation). About 46 m^3 of compressed air are drawn into each individual surge vessel to damp the surge waves resulted from one pump trip and continued steady (slightly erratic) until the end. About 65 m^3 of atmospheric air are drawn through each air valve into 1200 mm pipeline.

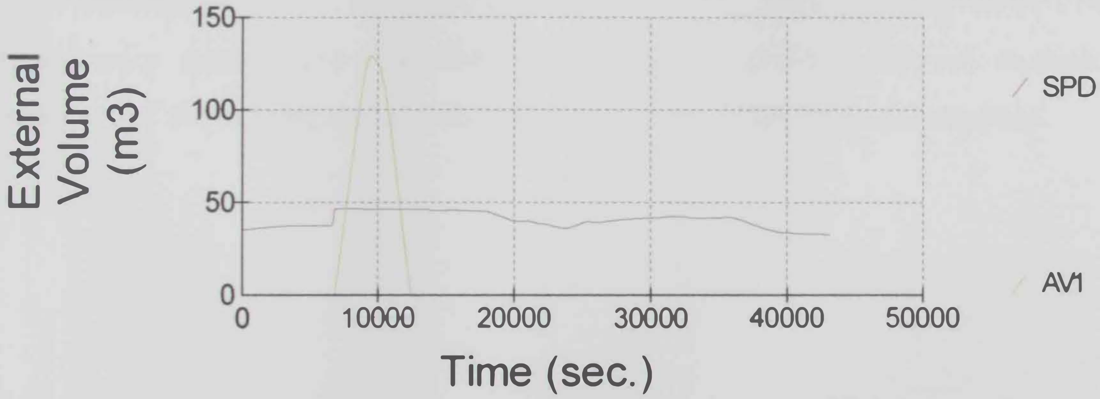

Figure 5.18 External compressed air volume to pressurize the surge vessels and external atmospheric air volume to get into the system through air valves for a one pump trip scenario.

Table 5.3 Summary of various parameters as a result of one pump trip

Maximum head drop (m)	Minimum nodal head (m)	Minimum nodal head location	Minimum head at highest elevation point (m)	Volume of compressed air into surge vessels (m^3)	Volume of atmospheric air into the 1200 mm pipeline (m^3)
13	7	Dahma	0	138	195

5.3.2.2 Trip of two pumps

The current set of scenarios simulates the trip of two pumps (sudden trip) and leaving one pump only in operation. Two pump trip was set at time 6300 sec (07:45 AM), similar to the one pump trip scenario, to capture the extreme conditions of peak flow.

Nodes

Figure 5.19 shows the pressure profile for Dahma Node(J49), Maqam Node (J39), Zakher Node (J43) and Wagan Node (J45). It is clearly observed that the available pressure at all nodes drops about 14 m below normal operation. This causes system instability with some transient events but all stayed above the cavitation pressure in the entire 1200 mm transmission system. The minimum pressure is 5 m above cavitiation indicating the adequacy of the existing surge protective devices (SPD) installed in the system.

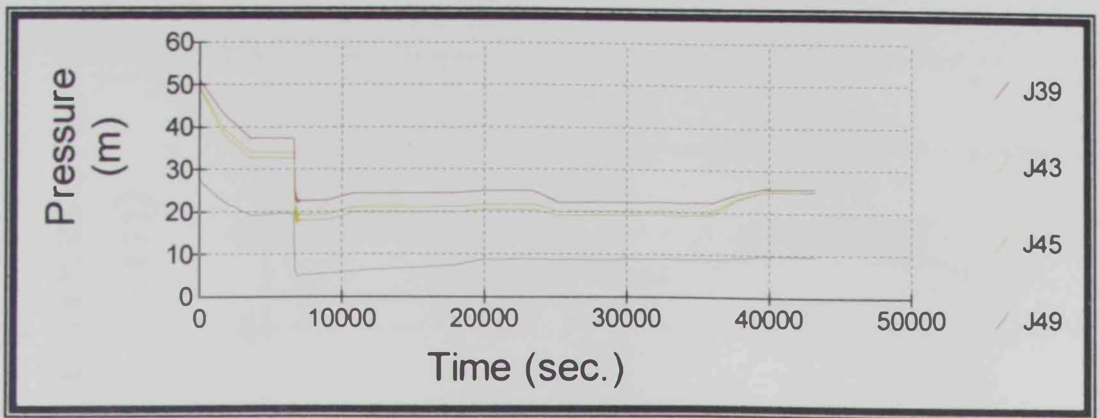

Figure 5.19 Nodal pressures for two pump trip scenario (J-39: Maqam, J-43: Zakher, J-45: Wagan and J-49: Dahma)

Pumps

Figure 5.20 shows the available suction pressure of one pump at AARS after two pump trip. The suction pressure gradually started to drop from 7.9 m until reaching 5.5 m and after that it increases to 6 m. The suction pressure is slightly eratic with less peaks (better stability) during two pump trip than in the case of a one pump trip. The explanation for this phenomenum is that during two pump trip; only one pump jumps to full speed (full flow) which results in less transient events opposite to the case of a one pump trip.

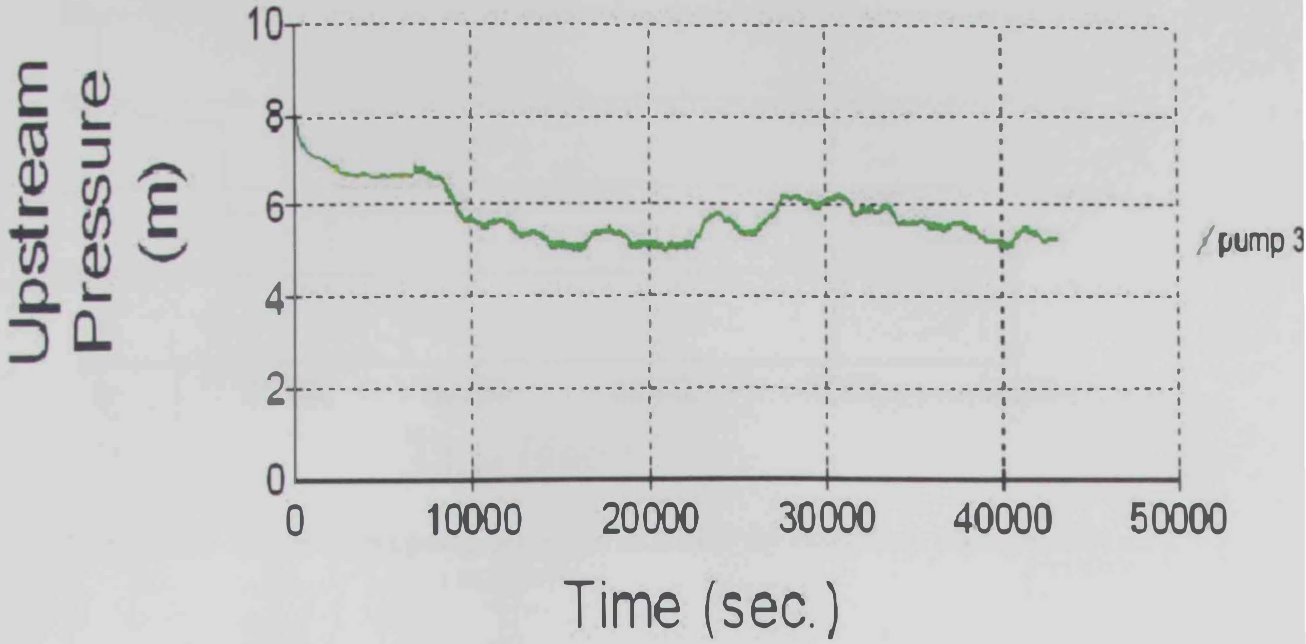

Figure 5.20 Upstream pump pressures at AARS for two- pump trip scenario

Figure 5.21 shows the available discharge pressure of one pump at AARS following the trip of two pumps. The discharge pressure, is initially at 60 m, drops to 35 m and stays at this pressure value with a one pump operation until the end of this cycle. This indicates that the only operating pump is forced to supply its maximum flow and operate at its lowest head (35 m) regardless of the downstream variable water supply.

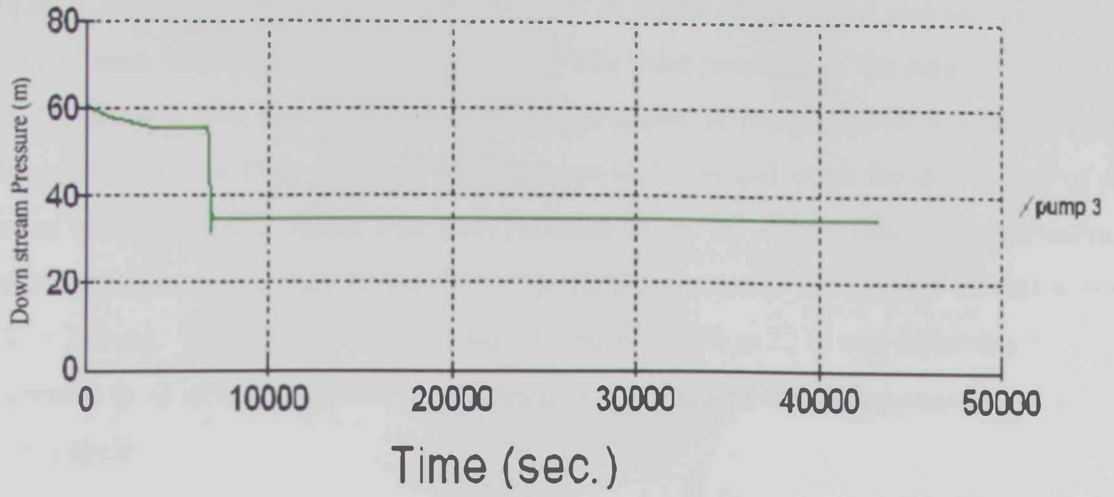

Figure 5.21 Downstream pump pressures at AARS for two-pump trip scenario

Figure 5.22 shows the flow, at the discharge side, of one pump at AARS during the trip of two transmission pumps (#1 and #2) at AARS out of three pump operation at time 6300 seconds (07:45 AM). The flow of pumps #1 and #2 went to zero while the flow of pump #3 went up immediately from 983 l/s to 1700 L/s; in order to, helplessly, cater for some of the water shortage as flow rate jumps to 1700 L/s along the remaining time of this cycle. The surge tanks, surge vessels and air valves compensate the shortage in water supply into the system during rapid drops in head to avoid sub-atmospheric pressures and water column separation. This alleviates both low and high pressures in the system.

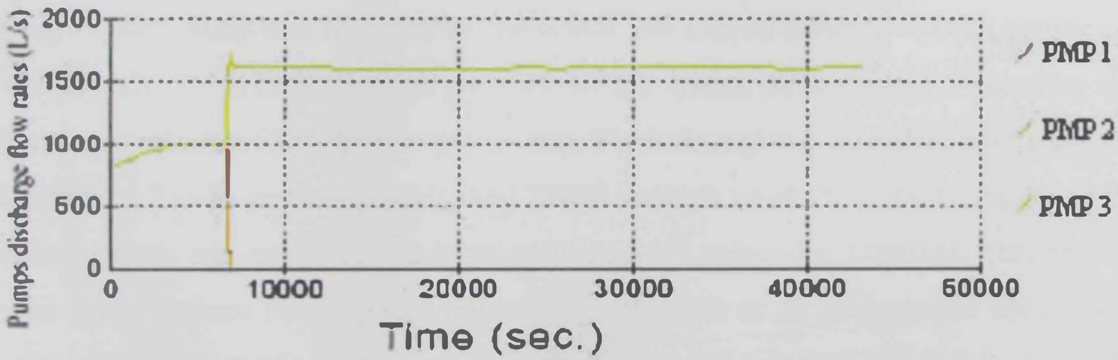

Figure 5.22 Flows at the discharge sides of the pumps at AARS for two-pump trip scenario

Figure 5.23 shows the available inlet pressure of a one surge vessel and one air valve at pumps' main discharge 1200 mm pipeline. The inlet pressure to the surge vessel is (55 m) slightly less than the available discharge pressure of the pumps at AARS. After the two pump trip, the PSD pressure comes down to 29 m and stays for the course of this period at this pressure value. The inlet pressure to the air valve should be equivalent to the lowest system pressure at the discharge of the pumps at the highest elevation point (EL = 285 m). Initially, the inlet pressure to the air valve is 22 m and following the trip it decreases to -2 m and after that increases later to zero and stays unchanged until the end of this cycle.

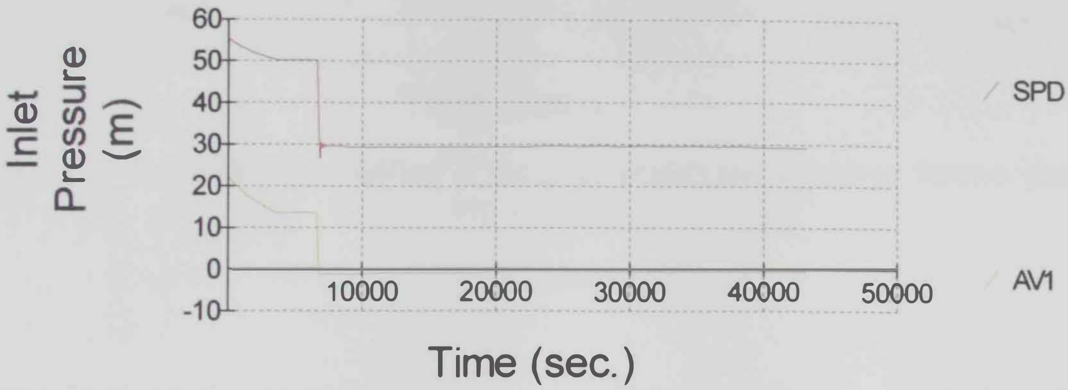

Figure 5.23 Inlet pressure of a one surge vessel and one air valve for two pump trip scenario

Figure 5.24 shows the external water inflow/outflow of the surge vessels and air valves. During the two-pump trip, some eratic (-900 to + 300 L/s) of water have been getting out of the air valves to release the high pressure off the system during water hammering and surge waves resulted from the two pumps trip. Flow through air valve becomes steady at 480 L/s from 25200 seconds (1 PM) until 37800 seconds (4:30 PM). As a conclusion the transient events are maximized at locations where air valves are installed. This may be related to the 2-phase flow regimes (air mixed with water or air pockests) in atmospheric condition. Some water (+ 90 L/s) gets into the surge vessel and some (-150 L/s) gets out of the surge vessel to the system. Transient events at surge vessels are minimized and system condition is stable due to having clear single phase flow. Under pressurized conditions, air works as a blanket on the top of water surface to damp transient wave inside the vessel; in turn water transfers damp-effect to the system and hence minimize transients. Normally, air doesn't intrude into the system through surge vessels. If air intrudes into the system through surge vessels, the surge vessels become like big air valves that can cause sever damage to the water facilities.

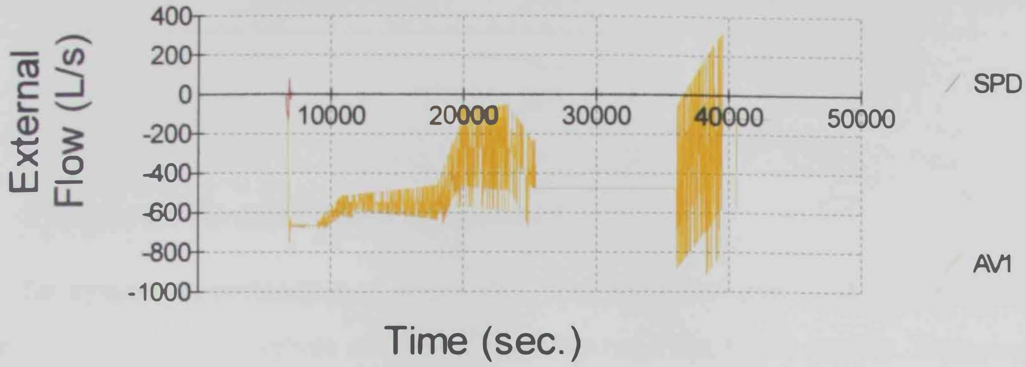

Figure 5.24 External inflow/outflow of the surge vessels and air valves for two pump trip scenario

Figure 5.25 shows the external atmospheric air volumes entering the system through air valves to prevent vacuum pressure at high elevation point. During the two pump trip, continuous and gradually increasing atmospheric air volumes (up to 17000 m^3) are drawn into each individual air valve to prevent absolute vacuum pressure inside the pipelines. An average air volime of 10000 m^3 is drawn through each air valve into the 1200 mm pipeline.

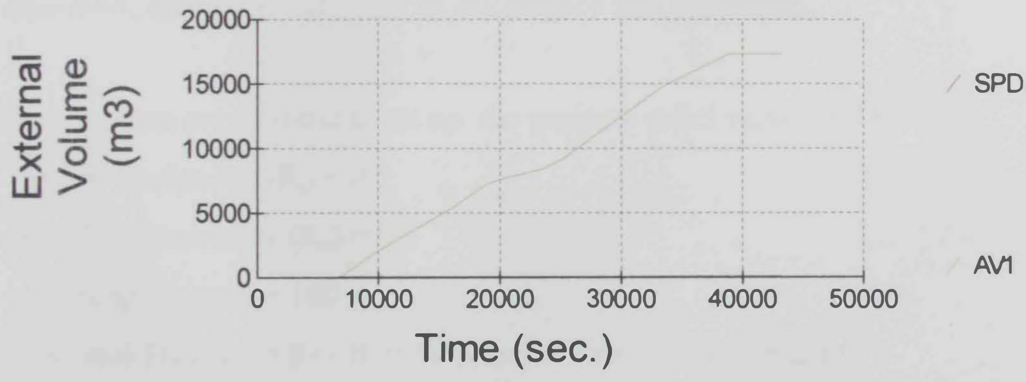

Figure 5.25 External atmospheric air volume to get into the system through air valves for two pump trip scenario

Table 5.4 Summary of various parameters as a result of two-pump trip

Maximum head drop (m)	Minimum nodal head (m)	Minimum nodal head location	Minimum head at highest elevation point (m)	Volume of compressed air into surge vessels (m^3)	Volume of atmospheric air into the 1200 mm pipeline (m^3)
14	5	Dahma	-2	-	30000

5.4 Proposed Remediation Measures

Since the system boundaries are nodes on pipe branches that feed water directly to costumers, flow control valves are installed to manage the water supply. Transients are associated with these valves during operations. The immediate pulse of the pressure rise is first sensed at the source branch. The pressure wave then quickly travells with the speed of sound to the rest of the system. The best remediation action is to damp the pressure wave at source where it intially starts. Some of the remediation measures include installing either surge vessels or pressure relief valves. Surge vessels are larger, needs support of external pressurized air system, needs frequent maintenace and they are more expensive. Also, municipal restriction on their installation in the city that is part of region beautification. They are therefore common downstream the pump station in which major pressure drop may occure. The arbitrary choice is installing pressure relief valves on all four branches. They are simple, smaller in size, normally contained or hidden in valve chambers, easy to install , easy in maintenace and less expensive.

The following data are required to set up the pressure relief valves (PRV):

- Inflow Resistance (R_i) = 0.5
- Outflow Resistance (R_o) = 0.5
- Opening Pressure = 100 m
- External Pressure = 0 → if PRV releases water to atmosphere.
- Closing Pressure = 80 m
- Opeining Time = 20 Seconds

- Closing Time = 120 Seconds
- Sensing Node ID = Specific junction of respective station i.e Dahma Node-J49, Maqam Node-J39, Zakher Node-J43 and Wagan Node-J45.

PRV begins to open when the pressure at the sensing node exceeds the opening pressure. The opening time is the response time for the valve to go from the start to the fully open position. The valve closure is initiated when the pressure (head) drops below the closing pressure. Because of pressure fluctuation at the sensing node, this valve can remain open for longer periods. If the valve exits to a pressurized region (e.g., a tank) the external pressure should be specified. The pressure to activate the valve is generally sensed at the valve but any node can be used for this purpose. PRV in the model is not considered like an active normal valve that is supposed to be defined by type, size and certain characteristics. The principle of PRV in InfoSurge Model is considered a typical surge device that is defined as node junction similar to other surge devices such as surge vessels and air valves with the following requirements:

Inflow resistance (R_i) = head loss across the valve / flow rate^2. R is inversely proportional to the size of the PRV. A higher R value produces smaller sizes of PRVs and vice versa. Initially, R_i is assumed = 1 which is resulted in PRV that can't hold the set 100 m pressure in the system (upstream of PRV). After several trial and errors when R_i = 0.5 is used, the set system pressure of 100 m is achieved. Hence,

  - R_i = 0.5 is used to predict the correct PRV size.
  - Head loss = upstream pressure – down stream pressure
  - Upstream pressure = 100 m which is the set pressure of the system.
  - Downstream pressure = 0 which is the atmospheric pressure.
    - Head loss =100 m – 0 = 100 m = 328.1 ft.

- Minor losses across the valve are ignored.
- Solve the equation of R (in fps system) to obtain the flow rate through the PRV: $\rightarrow$ R = head loss / $Q^2 \rightarrow 0.5\ Q^2 = 328.1$ ft $\rightarrow Q = 25.62\ ft^3/s = 0.7253\ m^3/s$.
- Maximum Recommended Velocity across the valve = V (valve) = 2 x V_{max} (of lined pipes) = 2 x 3.5 m/sec = 7 m/sec.
- By rule of thump $\longrightarrow$ A (area of valve) = Q/V
- Q/V = (0.7253 m^3/sec) / 7 m/sec) = 0.1036 m^2 $\rightarrow$ D = 363.31 mm $\rightarrow$ 400 mm (standard valve).

In conclusion, four pressure relief valves with a size of 400 mm are proposed to maintain the system head (pressure) within safe limit and to be installed on the following:

- Dahma pipe branch size 800 mm
- Maqam pipe branch size 1000 mm
- Zakher pipe branch size 800 mm
- Wagan pipe size is 800 mm

5.4.1 Pressure Relief Valve at Dahma Node Pipe Branch

Figure 5.26 shows that the maximum system head reached 89 m at Maqam Node, while the minimum system head reached 20 m at Dahma Node. It is clear that the presence of pressure relief valve (PRV) at Dahma bypass line has reduced the maximum head from 120 m to 89 m. Also, the minimum pressure increases from -10 m to 20 m.

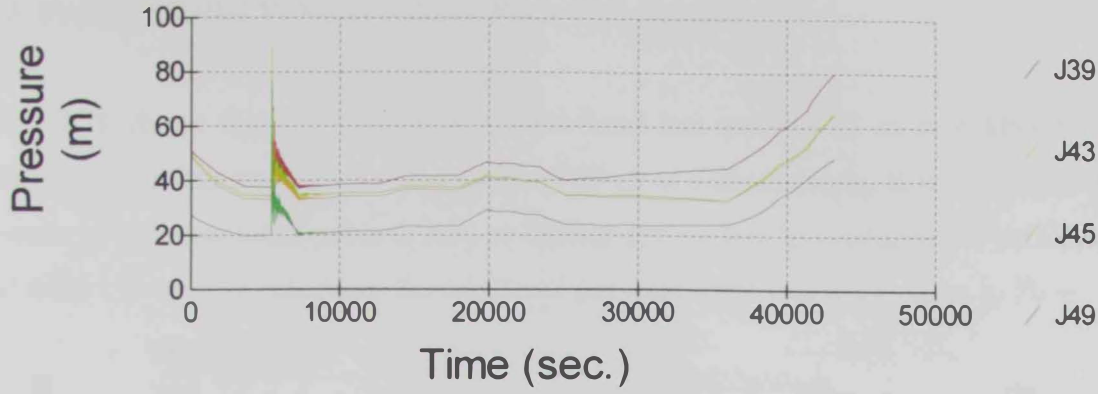

Figure 5.26 System head at different nodes versus time during Dahma Node valve closure

5.4.2 Pressure Relief Valve at Maqam Node Pipe Branch

Figure 5.27 shows that the maximum system head reached 90 m at Maqam Node while the minimum system head has reached 20 m at Dahma Node. The simulation covered the period from 6 AM to 6 PM. The valve closure occurred at 32833 seconds which correponds to 3:07 PM. In comparison to figure 5.8 where the vlve closure occurred at 4 PM, no discrepancy between ther two simulated periods because there is no water supply changes in these times. It is clear that the presence of pressure relief valve (PRV) at Maqam bypass line has reduced the maximum head from 197 m to 90 m. Also, the minimum pressure has increased from -10 m to 20 m.

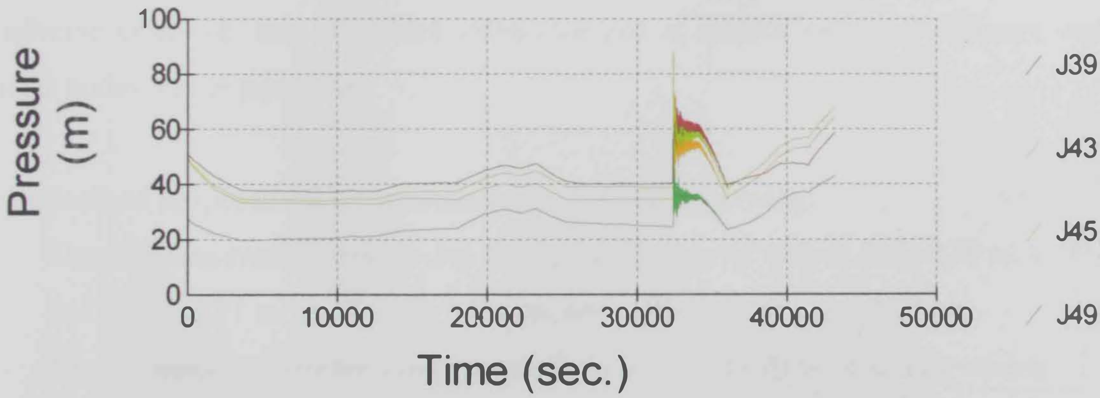

Figure 5.27 System head at different nodes versus time during Maqam node valve closure

5.4.3 Pressure Relief Valve at Zakher Node Pipe Branch

Figure 5.28 shows that the maximum system head has reached 91 m at Zakher Node while the minimum system head has reached 20 m at Dahma Node. It is clear that the presence of pressure relief valve (PRV) at Zakher bypass line has reduced the maximum head from 196 m to 91 m. Also, the minimum pressure increases from -10 m to 20 m.

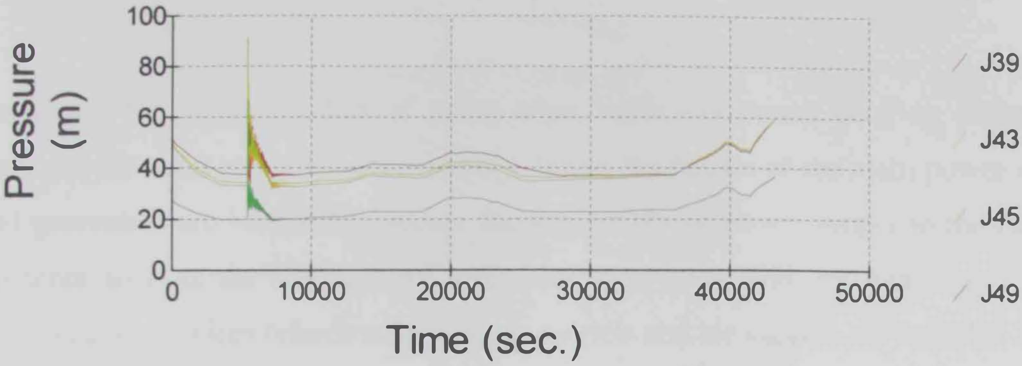

Figure 5.28 System head at different nodes versus time during Zakher Node valve closure

5.4.4 Discussion of Simulation Results

The classified pressure rating of the pipes has not been reached during sudden complete closure of control valves in the presence of PRVs. As a summary, the above simulation results and table 5.5 have shown how useful is the pressure relief valves in minimizing the adverse effect of fast or sudden valve closures of control valves at different water demand nodes on the pipelines.

The impacts of proposed PRVs are summarized in the following:

- Drop of maximum pressure due to sudden closure of valves from 120 m to 89 m in Dahma, 197 m to 90 m in Maqam, and 196 m to 91 m in Zakher.
- Rise of associated minimum pressure from – 10 m to 20 m at all four nodes.

Table 5.5 Summary of maximum head (m), minimum head (m) with PRV installed

Location of sudden valve closure	Maximum head (m)	Maximum head location	Minimum head (m)	Minimum head location
Dahma	89	Al Wagen	20	All nodes
Maqam	90	Maqam	20	All nodes
Zakher	91	Zakher	20	All nodes

To minimize the adverse effect of pump trips, sufficient power back up (redundant) systems are installed to cut-in automatically during the failure of the main power source. Diesel generators are installed to secure the availability of power supply to the essential equipments such as the fire, control and chlorine systems and the lightings. Adequte surge protection devices (check valves, surge vessels and air valves) are installed as well to protect the pumps and the entire system from severe transient conditions during pump trips. Gradual reduction of flow rate by small increments using pump variable speed control is applied before the pump reaches to a complete shutdown. The emergency pump shutdowns are rarely practiced. The gradual increase of speed is controlled by the speed control when pump starts up and during the increase of water supply to avoid undesirable transient conditions in the transmission system.

5.5 Cost Analysis

In order to show the merit of the proposed bypass, a cost analysis is conducted to compare the expenditures of existing system and those expected when the bypass is in place. The expendures include installation costs of pipes, valves and fitttings, and the operational energy cost. The cost also considers the labor and maintenance cost associated with the current pump operation. The cost calculation is needed in this study to determine the pay back period and the profit gain afterwards.

The cost estimations are based on the following:

- Consultation fees are not applicable in this study.

- The proposed bypass setting is based on cost of pipes, valves, fittings, and construction outlined in Tables 5.6 and 5.7.
- Electrical cables, instrumentations and control systems are estimated at about 15 % of pipe materials ,valves and fittings cost.
- Contengency fees is added (normally 20 % of the total capital cost for contengency such as variation orders).
- The energy cost estimation is based on actual charges of $0.0411 per KWH for the bypassed booster stations i.e Dahma, Maqam, and Zakher and 85 % pump effeciency is assumed as an average value that corresponds to the best effecincy point on pump curves.
- The operation and maintenace (O&M) cost estimation is based on annual cost of manhours from Transco Personnel Policy and spare parts are from Transco stores.

5.5.1 Cost of Pipes

The method of pipe cost estimation is abstracted from Transco Power & Water Projects Budget Estimate for the Year 2004. Inflation rate of 7 % per year is applied to adjust the cost for year 2007.

Table 5.6 Cost estimate of needed pipes

Label	Size (mm) & Length (M)	Construction Costs ($)	Material Costs ($)
Bypass/Maqam	1000mm – Length – 100M	45,000	50,500
Bypass/Zakher	800mm – Length – 100M	39,200	43,600
Bypass/Dahma	800mm – Length – 100M	39,200	43,600
Total (Pipes) Cost		123,400	137,700

From the above table, the cost estimate of needed pipes is $ 261,100.

5.5.2 Cost of Valves & Fittings

The method of valve cost estimation is obtained from Transco Store Value. Inflation rate of 7 % per year is applied to adjust the cost for year 2007.

Table 5.7 Cost estimate of needed valves and fittings

Label	Size (mm)	Construction Costs (AED)	Material Costs (AED)	Total Cost (AED) / 3.65 = $
Bypass/Maqam	FCV 1000	20,000	200,000	220,000 / 3.65 = $60,274
Bypass/Zakher	FCV 800	17,000	160,000	177,000 ~ $ 48,493
Bypass/Dahma	FCV800	17,000	160,000	217,000 ~ $ 48,493
Bypass/Maqam	BV1000	20,000 x 2	89,936 x 2	219,872 ~ $ 60,239
Bypass/Zakher	BV800	17,000 x 2	87,412 x 2	208,824 ~ $ 57,212
Bypass/Dahma	BV800	17,000 x 2	87,412 x 2	208,824 ~ $ 57,212
Bypass/Maqam	PRV400	8,500	11,000	19,500 ~ $ 5,342
Bypass/Zakher	PRV400	8,500	11,000	19,500 ~ $ 5,342
Bypass/Dahma	PRV400	8,500	11,000	19,500 ~ $ 5,342
Bypass/Maqam	BV400	8,500 x 2	28,300 x 2	73,600 ~ $ 20,164
Bypass/Zakher	BV400	8,500 x 2	28,300 x 2	73,600 ~ $ 20,164
Bypass/Dahma	BV400	8,500 x 2	28,300 x 2	73,600 ~ $ 20,164
Total (Valves) Cost				$ 408,441

From the above tables, the total capital cost (Pipes + Valves) = $261,100 + $408,441 = $ 669,541

5.5.3 Cost of Electric Cables, Instrumentation and Control Systems

This cost is estimated 15 % of pipe materials,valves and fittings cost. The total estimated cost is about $100,431 that include all cable materials and construction costs.

- Total CAPEX cost = $ 669,541+ $ 100,431 = $ 769,972

- Contengency fees is added = 1.2 ($ 769,972) = $ 923,967
- Chlorine boosting is not required.
- The method of this cost estimation is abstracted from Transco Power & Water Projects Budget Estimate for the Year 2004.

• Hence, the total bypass project value = $ 923,967

5.5.4 Energy Use Cost ($/Year)

The energy cost is estimated for three booster pumping stations that are proposed to be bypassed in the current study ,i.e, Dahma, Maqam, and Zakher. The energy cost estimation is based on actual charges of $ 0.0411 per KWH and 85 % pump effeciency is assumed as an average value that corresponds to the best effecincy point on pump curve. The estimated daily power cost is shown in table 5.8.

Table 5.8 Estimated daily energy cost

Label	Daily Volume Pumped (m^3)	Average Pump Efficiency (%)	Daily Energy Use (KWH)	Daily Energy Cost ($)
Dahma pump set	47,234.89	85	4,274.45	173
Mqam pump set	54,273.67	85	5,065.84	205
Zakher pump set	39,881.45	85	3,665.53	149
Sub total	141,390	-	13,006	527

From the above table, the annual energy cost is estimated as $ 192,355.

5.5.5 Operation and Maintenence Cost ($)/ Year

The Operation and Maintenace (O&M) cost estimation is based on annual cost of manhours from Transco Personnel Policy, chlorine cost of 6 Dh/kg as consumables and spare parts are from Transco stores. The operation and maintenace annual cost is estimated in Table 5.9.

Table 5.9 Manpower and spare parts annual cost estimate

Label	Manpower Cost ($)/ year	Spare Parts and Consumables (including chlorine use) Cost ($)/ year
Pump Set/Dahma	205	12500 inclusive of chlorine $ 2500
Pump Set/Maqam	205	15000 inclusive of chlorine $ 3000
Pump Set/Zakher	205	1250012500 inclusive of chlorine $ 2500
Sub total	616,500	40,000

From the above table, the annual manpower and spare parts cost is $ 656,500.

5.5.6 Project Pay Back

The bypass project pay back is calculated by the comparison of total project value to energy, O&M and spare parts annual expenditures.

- Total project value = $ 923,967
- Annual expenditures (operation and maintenance + energy) on the current bypassed pumping stations = (Operation Cost ($)/ year) + (Spare Parts Cost ($)/ year) + (Daily Energy Use Cost ($)/year) = 615,000 + 40,000 + 192,355 = $ 847,355

Project Pay Back = $ 923,967/ 847,355 = 1.09 years is about 1 year, 1 month.

• After this period , it is anticipated to save $ 847,355 per year. This indicates a feasible and highly profitable proposal.

5.6 Water Quality

The water quality is one of the most important issues in the code of water transmission and distribution systems. Based on the water quality regulation in Abu Dhabi, the drinking water should comply with World Health Organization (WHO) Standards.

The chlorine concentration in the distribution networks should be in the range of 0.2 – 0.5 part per million (ppm) at the point of connection to the customer. This is viewed as an adequate concentration level to deactivate pathogenic microorganism. The chlorine concentration in the transmission networks should be in the range of 0.6 – 1 part per million (ppm) which is may be a problem of acceptability to the customers who may sense the chlorine through smell.

From a health prospective (as per WHO Guidelines), the maximum concentration of chlorine that the body can tolerate is 5 ppm, if the chlorine level exceeds this limit; it will be harmful and some consequences may result such as eyes and skin irritations and stomach discomfort. The chlorine injection, also, produces side products in the water such as halomethanes (Ch_3Cl and Ch_3Br) which are harmful to health. Chlorine also is accused of causing environmental problems such as global warming and damaging the ozone layer. Bromate,as well, recently has become an issue from regulatory point of view. The recent drinking water standards mandate the bromate level to be less than 10 part per billion (ppb) due to the risk of causing cancer to human.

The high cost of chlorine gas of 6 Dirham/Kg and the hazards of chlorine,besides the safety issues of storaging and handling, the government of Abu Dhabi decided to shut down facilities and stop the production of chlorine gas. Instead, as alternative to chlorine gas, the government has decided to use ultra pure (99.9% sodium chloride) salt instead of sea water. The brine solution is fed to a generator to produce sodium hypochlorite liquid of 4% solution concentration which is considred friendly with the environment and free of bromides and bromate. This is the new direction that is sought by the government now days.

CHAPTER VI

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This chapter summarizes the undertaken study and the findings obtained from the hydraulic simulations. Conclusions and recommendations are also presented.

6.1 Summary Of Tasks

The feasibility study to utilize the valuable pressure energy from AARS 's Markhania Pump Group has been done via implementing the following tasks:

- Actual operating and design data have been collected for AARS' pumps, the main water transmission pipelines 1200 mm D.I, the associated installations, and the varying water supply of final destination points (supply nodes) at each consumption branch.
- Scenarios of potential transient effects on the proposed bypass system have been determined and classified to normal and abnormal operating conditions.
- The gathered data were used to setup two hydraulic models (InfoWater and InfoSurge Software) for each considered scenario.
- The available pressures in each scenario have been determined at different consumption branches and at the water demand nodes.
- Various new regulating flow and pressure control valves (and required surge protection devices such as pressure relief valves and air valves) have been incorporated at different consumption branches for evaluation purposes.
- The best settings of valve installations have been selected in association with identified operating conditions for different demand scenarios.
- The system hydraulic performance and potential of transient conditions caused by the new proposal under different flow scenarios have been studied. This has been done by conducting transient simulations using a specialized software that handles the hydraulic transients in pipenetworks. Such simulations have allowed to

evaluate the existing surge vessels' size and water source pumps' operating schemes, confirmed their suitability for the new mode of operation and helped in providing recommendations for possible modification if needed.

- The capital and operational costs associated with potential modifications have been estimated.
- The findings and recommendations are presented and can be utilized as a conceptual basic design scheme (feasibility study) for further detailed engineering development and implementation.

6.2 Summary Of Findings

6.2.1 Normal operating conditions (Typical Supply Changes)

6.2.1.1 Valve closing periods

During normal water supply reduction in which gradual and sudden partial closure of control valves occur, the pipeline classified pressure rating (160 m) has not been exceeded. Although the pipeline itself can take high pressure changes during major transient events, the flange bolts normally get loose, gaskets get ruptured, spigots/sockets get disconnected and hence water leaks may develop.

The simulation results have indicated how gradual (slow) and fast the control valves at different water supply nodes can be operated. The slower water flow control valves are closed, the faster the system head (pressure) is recovered and safer for the system fittings not to be damaged. Overall, the safe closing time during water demand reduction for all system control valves is in the range from 90 to 150 seconds for all nodes.

6.2.1.2 Valve opening periods

Increases of water demands during typical operation are associated with gradual or sudden, partial or complete flow control valve openings to reach the desired flow rates prescribed by the distribution daily programs at different demand nodes. The daily valve opening periods in Dahma, Maqam and Zakher starts early morning at 5 AM until 1 PM. This period is always associated with the peak water demands at the system nodes. The effect of flow control valves openings during this period is less severe than the period of valve closing periods. As a result the pump shut off heads and the pipe rating pressure have never been reached. The vacuum in the system is taken care of by air-pressurized surge vessels at the discharge of the pumps of AARS and adequately sized air valves at high elevations.

6.2.2 Abnormal operating conditions

6.2.2.1 Scenarios of sudden complete valve closures

The pipeline classified pressure rating has been reached and exceeded during sudden complete closure of control valves. Although the pipes can usually withstand 20% higher pressures than their rating to account for major transient events, the flange bolts normally get loose, gaskets get ruptured, spigots/sockets get disconnected, and eventually water leaks may develop. Hence, the above simulation results have indicated how serious is the effect of fast/sudden and complete-valve closures of control valves at different water demand nodes on the pipelines. Therefore, to cater for sudden valves closures during high water demands at nodes at the lowest elevations in the system, the effect of *pressure relief valves* has been studied at the pipe branches of all nodes on maintaining the system head (pressure) within safe limit for the system fittings not to be damaged and for the water supply not to be interrupted.

6.2.2.2 Scenarios of pumps trips

It is clearly observed that during the trip of one transmission pump out of the three pump operating at AARS to feed these nodes at time 6300 seconds (07:45 AM), the available pressure at these nodes drops by 12 m below normal operation to potentially causes system instability and some transient surges. However, the simulated minimum pressure is found above the cavitation pressure at any node or point in the system indicating the adequacy of the existing surge protective devices (SPD) installed in the system such as surge vessels and air valves.

The trip of two transmission pumps at AARS out of the three operating pumps at time 6300 seconds (7:45 AM) has been simulated. The results showed that the minimum available pressure drops by about of 16 m below normal operation that may cause system instability and some transient surges. The pressure at Dahma Node (and other high-elevation points) was found to be 5 m above cavitation pressure index reflecting the adequacy of the existing surge protective devices (SPD) installed in the system including surge vessels and air valves to confront potential vacuum problems.

6.2.3 Proposed remediation measures for sudden valve closures

Sizing and installation of pressure relief valve at each water consumption pipe branch (node) was needed to maintain the system pressure below the pipe rating. Incorporating such pressure relief valves in the model setup, the pipeline classified pressure rating has not been reached during fast or sudden complete closure of control valves. As a conclusion, the simulation results indicated how useful is the pressure relief valves to minimize the adverse effect of fast/sudden valve closures of control valves at different water supply nodes. Therefore, to cater for sudden valves closures during high water supply at the nodes in the lowest elevation points in the system (Maqam, Zakher and Al Wagen), it is recommended to install *pressure relief valves* at the pipe branches of all nodes in order to maintain the system head within the rating pressure of the pipeline which is safe for the system fittings not to be damaged and water supply to customers not to be interrupted.

6.2.4 Proposed remediation to pump trips scenarios

To minimize the adverse effect of pump trips, sufficient power back up (redundant) systems are installed to cut-in automatically during the failure of the main power source. Diesel generators are installed to secure the availability of power supply to the essential equipments such as the fire, control and chlorine systems and the lightings. Adequate surge protection devices (check valves, surge vessels and air valves) are installed as well to protect the pumps and the entire system from severe transient conditions during pump trips. Gradual reduction of flow rate by small increments using pump variable speed control is applied before the pump reaches to a complete shutdown. The emergency pump shutdowns are rarely practiced. The gradual speed increase by the speed control is applied also when pump starts up and during the increase of water supply to avoid undesirable transient conditions in the transmission system.

6.3 Cost Estimate and Project Pay Back

- The total project cost for station bypasses = $ 923,967
- Current annual expenditures (operation and maintenance + energy) on the bypassed pumping stations = (Operation Cost ($)/ year) + (Spare Parts Cost ($)/ year) + (Daily Energy Use Cost ($)/year) = 615,000 + 40,000 + 192,355 = $ 847,355
- The pay back period = $ 923,967 / 847,355 = 1.09 years = 1 year and 1 month.

Summary of Conclusions

- Normal operating conditions haven't shown major problems for the system rating pressure
- Safe closure time of 120 seconds was found adequate for all FCVs in agreement with current practice.
- Sudden closure of valves produced unsafe pressures and required some remediation measures.
- Four PRVs were sized and proposed to be installed on the proposed bypasses.
- The elimination of the operating by-passed booster pumping stations has reflected positively into savings of spare parts requisition, maintenance, man-hours (operating hours) and number of shift staff working for the Operation & Maintenance of the booster Pumping stations.
- Providing un-interrupted water supply right from the water transmission mains directly to the consumers via the proposed by-pass system was found possible.
- The stagnant time of the high quality water in the storage tanks will be eliminated and will eventually improve the quality of drinking water delivered to consumers.
- The water reservoirs and booster pumps that are available in booster stations can still be maintained, water circulated and frequently operated to be kept for use during upstream facility planned shutdowns and emergencies.
- Upon implementing the proposal in this study, the project pay back was found to be around one year and one month (roughly 13 months) is highly profitable by saving of $847,355/year.

6.5 Recommendations

It is feasible, very attractive economically and safe to implement the station bypasses for Dahma, Markhania and Zakher pumping stations for the reasons mentioned previously. Only minor civil works, pipe materials, control valves, pressure reducing valves, isolation valves, short electrical cabling, instrumentation and simple control systems are required to be purchased. Installation fees are included in the cost estimates. Therefore, it is recommended to implement these bypass proposals.

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تـوطـئـة

ان اسـتغـلال طاقـة الـضـغط المتـوفـرة في انابـيب نقل المياه يمكن ان يستغل استغلالا حسنا عن طريق التوصيل المباشر لشبكات توزيـع المياه ومنها مباشرة الى المستهلك. جرت العادة أن تهدر هذه الطاقة الثمينة في الخزانات ومن ثم يتم اعادة ضخها الى المستهلك فيما لو استغلت بالطريقة السليمة سيوفر على الدولة مبالغ طائلة. يتم في هذه الاطروحة اجراء دراسة محددة لتحديد المحطات بمنطقة العين التي يمكن ان يستغنى عنها نظرا لتوفر طاقة الضغط الضرورية عند مداخلها. في هذه الدراسة يتم التركيز على محطات الدهماء ، المرخانية (المقام) و زاخر التي تستمد مصادرها المائية من محطة استقبال المياه با لقرب من مطار العين الدولي عن طريق خط انابيب سعة 1200 مم. على طول هذا الخط يتم تحديد الضغوط المتوفرة وامكانية استغلالها عن طريق استخدام برامج هيدروليكية في الحاسب الآلي تبين الحالات العابرة (غير المستقرة) للضغط في الحالات الاعتيادية للتشغيل والحالات الغير إعتيادية. يتم مراعات الجوانب التصميمية للأ نابيب مثل السعة والتحملة القصوى للضغط . لقد بينت هذه الدراسة أن هذه الانابيب يمكنها تحمل الضغوط العابرة (غير المستقرة) وعلى اسوأ الحالات مع إفتراض عمل بعض التعديلات الطفيفة مثل تركيب محابس طاردة للضغط العالي ومنفسة فعالة للانابيب وكذلك المحابس الطاردة و المدخلة للهواء في الحالات الاعتيادية وغير الإعتيادية للتشغيل. هذه الدراسة تأخذ في الإ عتبار تكاليف راس المال ، التشغيل والصيانة و إستهلاك الكهرباء ،المطلوبة لتدوير المضخات، التي هي من فوائد هذه الدراسة. لقد بينت نتائج هذه الدراسة إنه يمكن الاستغناء عن عدد كبير من الخزانات و المضخات التي تستخدم لإعادة الضغط الضروري لإيصال الماء للمستهلكين. من هذه النتائج يتبين إنه يمكن توفير ثلاثة ملا يين درهم سنويا بعد السنة الاولى التي تكفي لدفع رأس المال الخاص بتركيب المحابس الطاردة للضغوط العالية. أوصت الدراسة بالبدأ فورا بالتنفيذ لما له من مردود إقتصادي كبير.

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محمد سلطان المعمري

رسالة مقدمة إلى عمادة الدراسات العليا

إستكمالاً لمتطلبات الحصول على درجة الماجستير في علوم موارد المياه

يونيو 2007

بسم الله الرحمن الرحيم