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Quantity and Quality Simulation in Al Ain Water Distribution System

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QUANTITY AND QUALITY SIMULATION IN AL AIN WATER DISTRIBUTION SYSTEM

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A Thesis Submitted to the Deanship of Graduate Studies in Partial Fulfillment of the Requirements for the Degree of Master of Science in Water Resources

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Thanks to my family members, who has been incredible source of love and support through all my life. Their encouragement has been invaluable in helping me though the most difficult parts of my study. Without those people, neither the desire nor the opportunity to pursue advanced degrees would have been possible.

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Above all I give praise to God the most merciful, the most gracious who makes this work, these dear human relationships and everything else good possible.

ABSTRACT

A hydraulic and chlorine simulation is conducted for three selected areas in Al Ain region. WaterCad 7.0, a water distribution system model, was chosen to conduct the simulation of the hydraulic behavior within the water distribution system. The fate of residual chlorine flowing through these systems over time is tracked by this software. The conducted hydraulic model predicts the performance of Al Yahar, Al Dhaher and Sweihan networks. Skeletonaization approach is employed in setting up the model for each system. In this study, the customer storage tanks (CSTs) are taken into consideration in performing the extended period simulation. Each group of (CSTs) is represented by a so-called Internal Storage Tank (IST). Al Yahar and Sweihan areas were selected since they exist at first locations supplied from the main desalination plants located in Abu Dhabi costal boundary called Um Al Nar and Tawillah via Al Saad and Sweihan pumping stations. Al Dhaher area, the third simulated system, is also selected as it is located at the last eastern site of Al Ain region that is supplied from Tawillah and Fujairah desalination plants via UmGhafa reservoir. The three selected networks are located somewhat on the edges of Al Ain region. They are considered district metered areas (DMAs), where each network has an adjusted flow control valve at its feeding sources. Therefore, the flow and the pressure for each system is known. Moreover, a data logger was installed by Al Ain Distribution Company (AADC) at a specific location in each network to record the residual pressure for a duration of one year. The simulation is done utilizing the water demand provided by the company. Water demands vary and consideration of the probabilistic nature of variations leads to more instructive assessment of the performance of the three selected distribution systems. Calibration of the hydraulic model was conducted by tuning the ISTs' capacities and water levels (initial and maximum). Hazen-william coefficients were maintained constant (C=120), as all networks pipes and connections were installed five years ago only in addition to the very good quality of desalinated water passing through each system. Chlorine simulation is conducted and calibrated utilizing the residual chlorine data collected for one year from the same data logger location in each area. With regard to the Chlorination By-Products, trihalomethanes (THMs) compounds are sampled and analyzed. All obtained results were less than the international limits of 80 ppb. Laboratory experiments were conducted to analyze the formation of TTHM species in flowing pipelines. The results show

that bromoform lead the THM species in all samples and at all times. On the other hand, the bromoform was found to undergo major hydrolysis after sometime approaching zero levels. All simulated and calibrated results, i.e. pressures, velocities and residual chlorine are compared to that in the water distribution code issued by Al Ain Distribution Company and with the international standards. From economic point of view, the energy cost for Al Yahar and Sweihan networks is calculated based on 0.05 Dhs/kwh for one and 20 years. Some alternative systems are studied to enhance the performance and efficiency of each network. One of the most important issues discussed is the possibility of removing the customer storage tanks, where cost savings can be achieved by controlling the delivered quantities to each customer tank and eventually enforcing the demand management concept. Installing an elevated tank or more is also discussed and scrutinized. The energy cost is also determined for each option and compared with the original configuration for each network.

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CHAPTER 1

INTRODUCTION

In this Chapter, a general description is introduced for Al Ain water system with an overview of its implemented Asset Management (AM) regime. Furthermore, the amount of water distributed and its break down are mentioned. The pumping stations are also described with their capacities to deliver the required water quantities to different areas in Al Ain region. The water demand forecasted for 2006 is also presented. Finally, the thesis objectives and the followed methodologies are introduced.

1.1 Overview of Al Ain Water System

Al Ain is a peripheral City with 170 and 120 Kms length and width respectively with an estimated 400,000 inhabitants. Al Ain is principally a residential city with fewer industries on south-west outskirt and many Palaces & Farms belonging to ruling families and VIPs. There are detached concentrated pockets of population, few Kms distant apart from each other, along the four major leading roads to the City. Al Ain Distribution Company is responsible to supply the Water & Electricity to Al Ain City and its elongated and stretched rural outskirts.

The Region has a water supply network of over 3000 Kms, laid properly as per the international standards, by qualified Contractors under the supervision of competent Consultants. The initial Project (WE-2), in which AC transmission line and distribution lines for few oldest city areas were laid, was completed in 1977. Right after that, another Project (WE-4) started in which city main transfer and distribution mains were laid. This was overlapped by a reservoir Project (WE-5) in which nine zonal reservoirs were built. These twos had, a complementary distribution network Project (WE-8) in which city's sectors, sub-sectors and other supply and distribution components were completed by 1985. Water Network Master plan for Al Ain region was issued on 1999. Its recommendations for improving the transmission and the distribution systems were initiated. Moreover, the needed rehabilitation and replacement works are currently carried out as per the asset management plans.

The city has an interconnected zone-based water supply system with pressure water distribution zones. Key Plan of Al Ain City distribution network highlighting the nine pumping zones is shown in Figure 1.1. Each zone has its own reservoir, transmission mains (owned and controlled by a transmission and dispatch company) as described in Table 1.1 where the storage and the pumps capacities are illustrated with their dedicated upgradeable capacities and the spaces for additional pumps (Transco report, 2005). The zone usually has two pumped distribution mains which are fulfilling the demands of the districts and the sectors belonging to that zone. Concentrated rural communities, scattered along four main roads, as much as 70-110 km away from city center, are supplied independently by their own individual system and are being integrated with the main supply system.

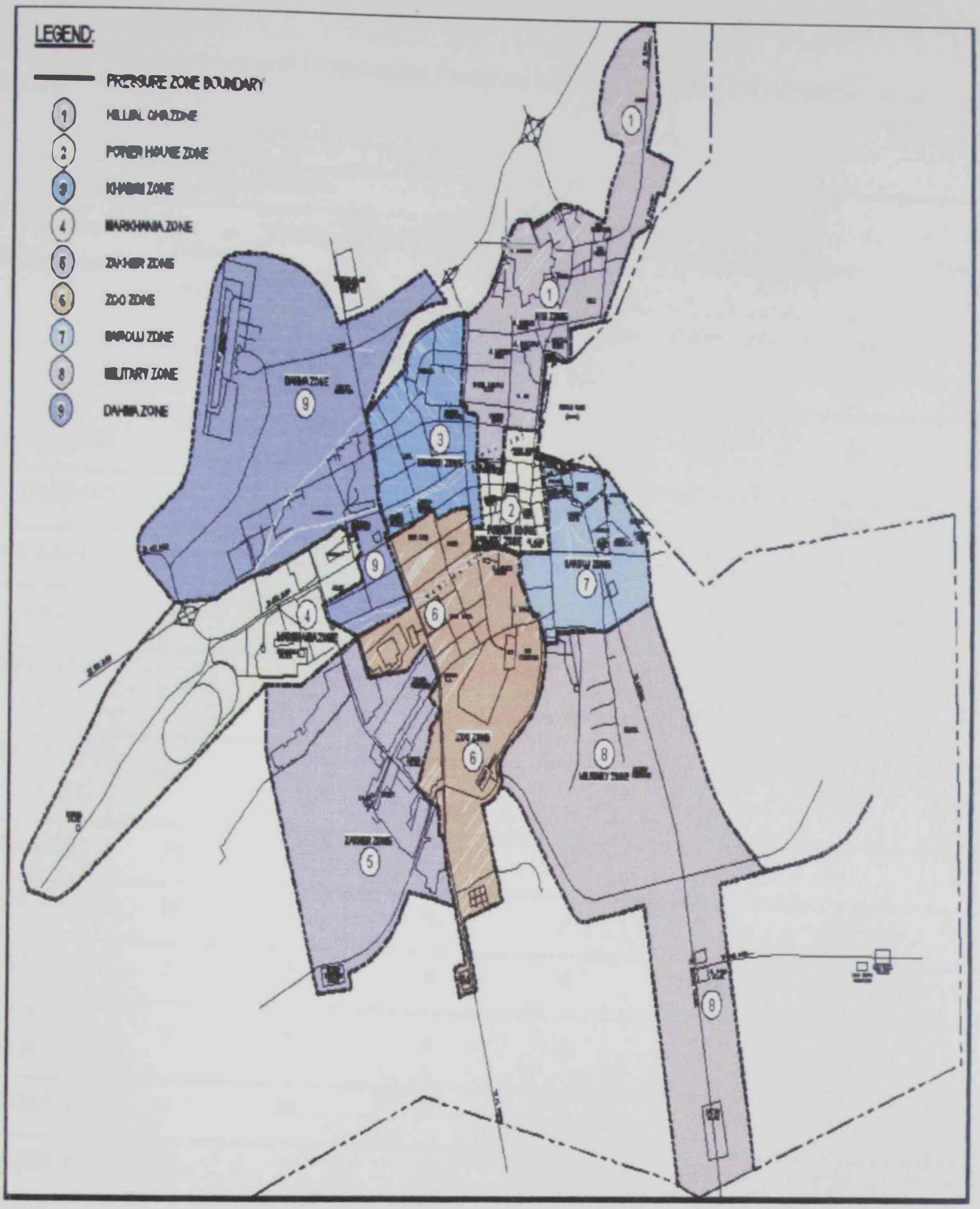


Figure 1.1 Pressure zones For Al Ain City

Table 1.1 Transmission and Distribution Pumping stations in Al Ain City (Transco Report, 2005)

Pumping Station Name	Storage Details		Pump Details				
	Existing Capacity	Dedicated Upgradeable Capacity	Number of Pumps		Installed Capacity		Space for Additional Pumps
			Transmission	Distribution	Transmission	Distribution	
	MG	MG	No.	No.	MGD	MGD	No.
AL AIN RECEPTION	20	20	16	0	167.5	0	2
AL DAHMAH	5	14	0	8	0	28.5	2
AL SAAD	1	1	0	4	0	8	1
HILI	10	5	3	11	12	33	0
JABEL HAFIT BASE	0.56	0.5	4	0	1.9	0	0
JABEL HAFIT SUMMIT	0.5	0	0	3	0	0.5	0
KHABISI	10	10	2	6	7.6	32.7	2
MARKHANIA	10	5	0	8	0	44.6	0
MILITARY	5	5	2	6	4.2	12.1	0
POWER HOUSE	5	5	0	8	0	18.5	0
REMAH	20	20	5	3	25	3	0
SAROUJ	10	5	2	7	3	17.8	1
SWEIHAN	10	5	6	3	72.1	6	4
ZAKHER	5	5	0	6	0	10.5	2
ZOO	5	5	0	8	0	17.6	0

The present city demand is 160 MGD (AADC water demand forecast report, 2005). However, only 133 MGD is available for supply. The water consumption consists of mainly domestic consumption and supply to Palaces, Agricultural Farms and Forests, with a minimal industrial consumption (4 %) where Al Ain is not a major business or industrial centre. Palaces and Farms supply constitutes 40% of the total supply, which is an important feature of the city supply. The consumption for Palaces and Farms is expected to increase tremendously in near future for which water has to be made available without failure as and when it is required to be supplied. Accordingly necessary arrangements/projects have been made to enhance Al Ain Water system efficiency. These projects are currently under design stage to achieve the main objective of 24 - hour continuous supply system for the whole city.

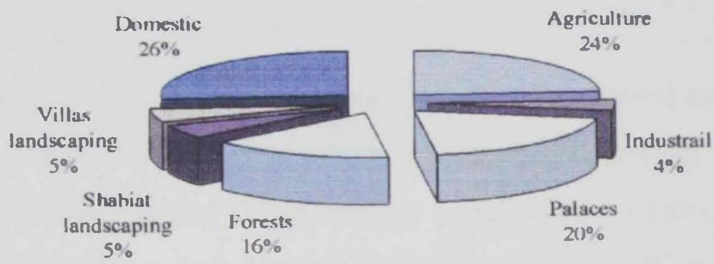


Figure 1.2 Water supply percentages for different categories for 2006 (AADC report, 2005)

The sources of water supply to Al Ain city are three different desalination plants located around 200 Kms away in addition to a scattered well filed system. The water enters the city, through two differently located reception reservoirs; Shobaisi and Al Ain Reception Pumping Stations. The well water is amalgamated in one of the reservoirs to improve its quality. The well field is scheduled to be capped in 2006/07 and to be kept in reserve, as alternative source to be used in the case of outage of any of the desalination plant.

1.2 Operation and Management Regimes

Nowadays, Al Ain Distribution Company (AADC), is developing an asset management regime. Its businesses goal is to develop and put in place, a new AM regime based on well proven methodologies as well as practices in addition to make sure that, the right amount of money is ultimately spent, on the right projects and at the right time. This is referred to as sound AM planning. Accordingly, the main goal was to develop a sound asset specific AM framework, which will deliver the following objectives:

- Ensure competent investment in new infrastructure, for all assets to efficiently meet the proposed levels of service.
- Develop of sound investment rules for the business.
- Develop of sound financial, commercial and management systems to enable a best practice infrastructure business.
- Develop a clear differentiation between mandatory and discretional capital expenditure in the business planning process to ensure proper evaluation between risk and return.
- Conduct a review of the existing O&M strategy to refocus on a more reliability centered strategy in line with industry best practice (maintenance optimisation), to optimise whole life costing.
- Ensure short medium and long term asset investment plans, contracts, procurement agreements and outsourcing plans are in place at the proper time.
- Conduct a review of Asset information and cost information held by the Company, on an ongoing basis, and what is required for the future AM regime.
- Develop an asset hierarchy information tree as a requirement for AM data.
- Develop new asset registers that will hold key information on the condition, criticality, capability and capacity of all assets, but only that what is needed for management purposes
- Conduct ongoing monitoring and regulating of assets including tight controls on whole life costs. This will include trade off's between short and long life plant and materials.
- Conduct a review of progress and performance particularly in respect to capital investment programs and standards of service to customers.
- Conduct a review of specification levels and design standards in line with whole life cycle costing
- Develop of a first class relationship from the outset with the staff.

- Develop reporting Protocol for AM
- Develop a transparent Investment Planning process
- Develop a safe and secure asset base
- Review relevant organisational structures and training

A Best Practice AM Regime, documented and computerised has been developed and termed AMP1. Further AMP's (AMP2, AMP3 etc.), will be developed over time as deemed necessary by ADWEA or the asset manager. The AMP's consist of a framework of Directives, Strategies, Methodologies, and Investment rules, and aligned to optimise the spending on the department's assets. This ensures that the business achieves a correct level of service to its customers, and one that outputs short medium and long term Asset Investment Plans on a rolling basis.

The whole AM process has been developed as an asset management plan - i.e. each and every asset will be assessed and be allocated a 20 year investment profile for capital expenditure and operational and maintenance expenditure. Output profiles are tabulated and displayed graphically, at the discretion of the Asset Manager. Such 20 year profiles are built up by identifying whole life costing analysis for each and every asset.

1.3 Problem Statement

Water distribution systems are considered to be one of the most important components in any water resources system. After careful design, they usually need continuous and wise operation, maintenance, and rehabilitation. Poor distribution systems can potentially result in significant losses of drinking water, a precious resource especially in arid areas like the United Arab Emirates.

It is therefore necessary to conduct various studies in Al Ain distribution system to manage the supply, demands, and operation so that conservation of water losses and energy is achieved while quality considerations are observed and maintained to acceptable regulated limits

This thesis addresses quantity and quality issues related to a portion of Al Ain water distribution system. Hydraulic simulation is conducted to reveal several features about the system adequacy, shortcomings, and alternative solutions to possible problems.

In addition to the hydraulic simulation, the thesis deals with quality simulation in the considered portion. Non-conservative substances are modeled including the Chlorine residual and chlorination byproducts, specifically Trihalomethanes 'THMs'.

This work is expected to provide valuable documented data to Al Ain water Distribution Company. Such data will allow an efficient management tool for the water distribution system in Al Ain which could be used in conserving the supplied water and minimizing the operation costs. The simulation setup represents a design tool for potential future expansion. The work also contributes to the knowledge of reaction kinetics of chlorination byproducts under dynamic effects.

1.4 Thesis Objectives

- To conduct a hydraulic simulation of a portion of Al Ain water distribution system.
- To evaluate the hydraulic performance, operation, and potential problems of the simulated system.
- To conduct water quality simulation for selected non-conservative constituents in the considered system.
- To evaluate the chlorine distribution in the system under different operating scenarios and the associated byproducts; in particular THMs.
- To assess the hydraulic dynamic effects on the speciation of chlorination byproducts and make relevant recommendations.

1.5 Methodology

- Collect data relevant to Al Ain water distribution system such as, topographic maps, zone maps, demands, supplies, main transmission lines, distribution networks, pumping stations, reservoirs ...etc.
- Determine the system to be considered in simulation based on the gathered data.
- Feed all the data into the water distribution system software (WaterCad)
- Identify operating scenarios based on the gathered data and conduct simulation for each scenario.
- Carry out a sound calibration to the considered system based on field measurements.
- Evaluate the hydraulic performance and identify potential problems and their possible solutions
- Carry out a field-sampling program to identify the constituents to be considered in the quality simulation work and to also verify the model results.
- Carry out laboratory experiments to identify the reaction rates of chlorine, and THM (based on the field sampling program) in pipelines under static and dynamic conditions as well.
- Carry out quality simulation considering the reaction rates determined earlier.
- Evaluate the system water quality for different operating scenarios.

CHAPTER 2

LITERATURE REVIEW

A number of studies have been devoted to the hydraulic analysis and water quality simulation. A number of literature review papers have been published in many journals by different researchers covering almost all aspects of hydraulic and water quality modeling. This chapter provides a brief overview about the historical development and advancement in hydraulic and quality modeling in water distribution systems. Detailed coverage of each type of modeling is then discussed. Finally, a brief coverage of the computational tool (software) used in this study and called WaterCad by Haestad Co. is presented at the end of this chapter.

2.1 Historical Background

Mathematical modeling has been used over more than 60 years to analyze flow in water-distribution system networks since the concept was proposed by Cross (1936). Using computers for conducting analyses of flow in pipe networks originated in the early 1960s and was greatly expanded during the ensuing decade of the 1970s with the advent of enhanced solution algorithms (Epp and Fowler 1970, Wood and Charles 1972) and the implementation of modeling techniques for devices such as pumps and valves (Jeppson and Davis 1976). In the late 1970s, single-time-period simulations were advanced to extended period simulations with techniques developed by Rao and Bree (1977). Hydraulic models can be used to analyze systems where demand and operating conditions are static or are time varying. The former type of model is a 'steady-state' model, and the latter is referred to as an 'extended period simulation' or EPS model.

Modeling the spatial distribution of water quality in pipelines first began with a steady-state modeling approach as suggested by Wood (1980) who studied slurry flow. Other researchers developing steady-state water-quality models in the 1980s and early 1990s include Chun and Selznick (1985), Metzger (1985), Males et al. (1985), Clark et al. (1988), Grayman et al. (1988a), Wood and Ormsbee (1989), and Clark (1993). The representation of temporally varying conditions for contaminant movement in a distribution system or 'dynamic' water-quality models began to be used in the mid-1980s. Investigators developing such models include Clark et al. (1986), Liou and Kroon (1986), Grayman et al. (1988b), and Hart (1991) With the widespread use and relatively low cost of personal computers and desktop workstations during the mid-1980s and 1990s, many models, both proprietary and public domain, can now be used to conduct hydraulic and water-quality analyses. Two such models in use today are the proprietary model Piccolo (SAFEGE Consulting Engineers 1994) and the public domain model, EPANET (Rossman 1994, Rossman et al. 1994) developed by the U.S. Environmental Protection Agency. The reader is referred to Rossman (1999) and Clark (1999) for a thorough discussion on the evolution and development of hydraulic and water-quality models.

2.2 Hydraulic modeling concepts

2.2.1 Network Hydraulics

The interconnected hydraulic elements are defined by two concepts; conservation of mass and energy. The principle of conservation of mass dictates that the fluid mass entering any pipe will be equal to the mass leaving the pipe (since fluid is typically neither created nor destroyed in hydraulic systems). In network modeling, all outflows are lumped at the nodes or junctions as shows in equation (1)

$$\sum_{PIPES} Q_i - U = 0 \qquad (1)$$

where, Q_i = Net inflow to node in i-th pipe (L^3/T)

U = Water used at node (L^3/T)

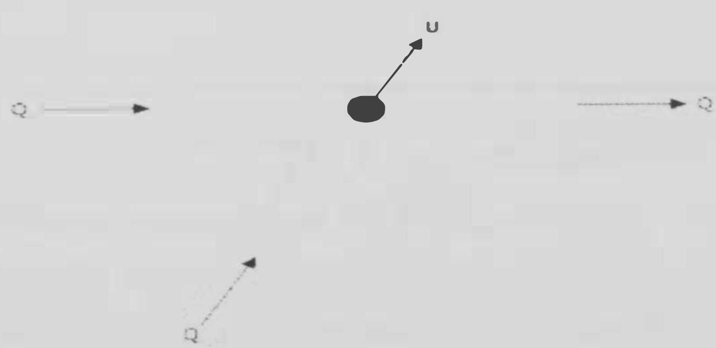


Figure 2.1 Mass conservation principle

In view of the fact that the existence of the storage tanks in the distribution system creates an accumulation of water, this accumulation can be described as following in equation (2)

$$\sum_{pipes} Q_1 - U - \frac{dS}{dt} = 0 \qquad (2)$$

where $\frac{dS}{dt}$ = change in storage (L^3/T)

One equation is written for each junction, node and tank in a network representing the conservation of mass. Bernoulli, (1738) presented that the difference in energy between two points must be the same regardless of the path that is taken. For convenience within a hydraulic analysis, equation (3) is written in terms of head to identify the principle of conservation of energy

$$Z_1 + \frac{p_1}{y} + \frac{V_1^2}{2g} + \sum h_p = Z_2 + \frac{p_2}{y} + \frac{V_2^2}{2g} + \sum h_L + \sum h \qquad (3)$$

Where Z = elevation (L)

P = pressure ($M/L/T^2$)

γ = fluid specific weight ($M/L^2/T^2$)

V = velocity (L/T)

g = gravitational acceleration constant (L/T^2)

h_P = head added at pumps (L)

h_L = head loss in pipes (L)

h = head loss due to minor losses (L)

Thus the difference in energy at any two points connected in a network is equal to the energy gains from pumps and energy losses in pipes and fittings that occur in the path between them. Figure 2.2 illustrate the difference between the hydraulic and the energy grade lines.

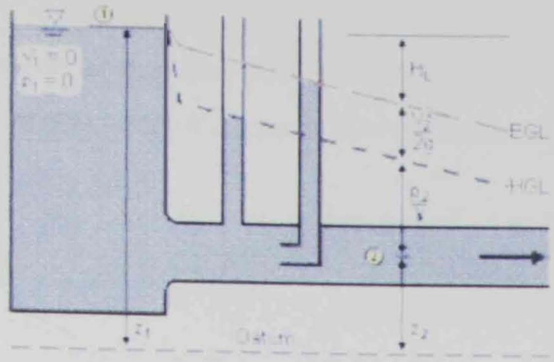


Figure 2.2 HGL & EGL Differences.

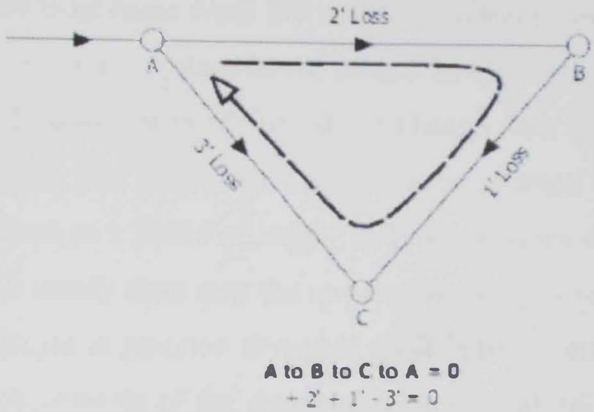


Figure 2.3 Conservation energy concept.

One continuity equation must be developed for each node in the system, and one energy equation must be developed for each pipe (or loop) Taking into consideration that real water distribution systems do not consist of a single pipe and cannot be described by a single set of continuity and energy equations. For real systems, these equations can number in the thousands.

Hardy Cross (1936) developed the first systematic approach for solving these equations. Many numerical techniques set up and solve the system of equations describing the hydraulics of the network in matrix form. Because the energy equations are nonlinear in terms of flow and head, they cannot be solved directly. Instead, these techniques estimate a solution then iteratively improve it until the difference between solutions falls within a specified tolerance At this point, the hydraulic equations are considered solved

Some of the methods used in network analysis are described in Bhave (1991); Lansey and Mays (2000); Larock, Jeppson, and Watters (1999), and Todini and Pilati (1987).

2.2.2 Types of Simulation

Many types of simulation can be performed depending on what is needed to be predicted or observed. The most basic types are steady state and extended period simulation (EPS), where in steady state one the system flows, pressures..etc. are computed assuming that the hydraulic demands and boundary conditions don't change with respect to time. For the EPS, the hydraulic demands and boundary conditions do change with respect to time. Moreover, the EPS is defined as a series of steady state simulations that are run together in sequence, where after each steady state step the system boundary conditions are reevaluated and updated to reflect changes in junction demands, tank levels ...etc. Most of the testing results in steady state measurements of the distribution system are taken at a single point in time under a single set of conditions. This information is useful for estimating various parameters used in steady-state and EPS models. When an extended-period simulation model is developed, it is necessary to supplement the static field testing with field measurements taken over a period of several days. This information can be used for calibrating an EPS model and verifying that such model adequately represents the behavior of the distribution system over time. Two types of data that are useful for calibrating and validating an extended-period simulation model; time-varying measurements of flow, pressure, and tank water levels in the distribution system and concentrations of a conservative tracer over time throughout the system

2.2.3 Calibrating Hydraulic Network Models

Flows, pressures, tank water levels, and other characteristics vary throughout the distribution system both temporally and spatially. Seasonal variations, variations by day of the week, diurnal variations, and small time scale stochastic variations typically occur. If an extended-period model of the distribution system has been properly constructed and calibrated, the model results should approximately mimic the behavior of the system over a period of time. The hydraulic simulation software simply solves the equations of continuity and energy using the supplied data, thus, the quality of the data will dictate the quality of the results. The accuracy of a hydraulic model depends on how well it has been calibrated, so a calibration analysis should always be performed before a model is used for decision-making purposes.

Calibration is the process of comparing the model results to field observations and, if necessary, adjusting the data describing the system until model-predicted performance reasonably agrees with measured system performance over a wide range of operating conditions (Walski, 1984). The process of calibration may include changing system demands, fine-tuning the roughness of pipes, changing the tank water level, and adjusting other model attributes that affect simulation results.

The calibration process is necessary for the subsequent reasons (R Burrows, Sep 2000). Calibration demonstrates the model's capability to reproduce existing conditions; thereby increasing the confidence the engineer will have in the model to predict system behavior. Results provided by a computer model are frequently used to aid in making decisions regarding the operation or improvement of a hydraulic system. The process of calibrating a hydraulic model provides excellent insight into the behavior and performance of the hydraulic system. In particular, it can show which input values the model is most sensitive to, so the modeler becomes more careful in determining those values. With a better understanding of the system, the modeler will have an idea of the possible impact of various capital improvements or operational changes. One area of calibration that is often overlooked is the capability to uncover missing or incorrect data describing the system, such as incorrect pipediameters, missing pipes, or closed valves. Thus, another benefit of calibration is that it will help in identifying errors caused by mistakes made during the model-building process.

Chapter 5 in this thesis begins with a discussion of data requirements and the reasons for discrepancies between computer-predicted behavior and actual field performance of a water distribution system. In making comparisons between model results and field observations, the modeler must ensure that the field data are correct and useful. This section focuses on identifying data useful for calibration (Dusan Obradovic, 2000).

There should be at least one flow test conducted in each pressure zone, and the number of flow tests should be roughly proportional to the size of the pressure zone. In general, more tests will increase the confidence the user will have in the model. One approach to selecting sampling locations uses a special procedure to select locations that minimize the uncertainty of the model's parameters (Bush and Uber, 1998). Another approach uses genetic algorithms to determine the best locations to conduct fire hydrant flow tests to maximize the coverage of the pipe network (Meier and Barkdoll, 2000).

Errors in roughness coefficients and demands affect the slope of the hydraulic grade line. If data are collected near the boundary nodes, the differences between the model and the field data may appear to be small because of the short distance even though the slope of the HGLs (and hence the roughness coefficient and demand) are significantly in error. Head data for model calibration should generally be collected at a significant distance from known boundary heads. Data should also be collected for pipes that have not been removed from the model during skeletonization (T.T. Tanyimboh, 2005).

Data collection can be classified as either point reading (grab samples) or continuous monitoring. Point reading involves collecting data for a single location at a specific point in time, and continuous monitoring involves collecting data at a single location over time. For point readings, samples should be collected at locations where the parameter being measured is steady so that the sample measurement is representative of the location over a fairly long period of time. To get the most out of continuous monitoring, the data should be collected from locations where the parameter being measured is dynamic. In situations where a point reading must be made at a dynamic location, it is critical to carefully note the time and boundary conditions corresponding to the data point.

When comparisons are made between field and model results, there is no mathematical reason to use pressures instead of hydraulic grades, or vice versa. Because pressure is just a converted representation of the height of the HGL relative to the pipeline elevation, the two are essentially equivalent for comparison purposes. For calibration purposes, however, there are several convincing arguments for working with hydraulic grades rather than pressures (Herrin, 1997), one of these point of views is that the hydraulic grades provide the modeler with a sense of the accuracy and reliability of the data. If computed and measured hydraulic grade values are significantly different from one another, it should immediately signal the modeler that a particular value may be in error. For example, an elevation may have been entered incorrectly. Another opinion says working with hydraulic grades makes it easier to work with pressure measurements not taken exactly at node locations within the model, because it is not the node that is used to convert measured pressure into HGL but, the elevation of the pressure gauge.

Although both HGL and pressure comparisons will lead to the same results if all other factors are equal, pressure comparisons make it much easier to overlook errors and much harder to track down inconsistencies between real-world observations and the model results. Accordingly, the first step the modeler should complete upon collection of field data is to convert pressure and tank water level data into the equivalent HGLs. Subsequent comparisons should be made between observed and modeled HGLs.

Before beginning the calibration of an EPS model, the modeler needs to be confident that the steady-state model is calibrated correctly in terms of elevation, spatial demand distribution, and pipe roughness. Once calibration on that level is achieved, the EPS calibration procedure can begin and will consist primarily of the temporal adjustment of demands. Depending on the intended use of the model, the focus of the EPS calibration may vary. For example, for hydraulic studies, the comparison between field and model conditions will be centered on the prediction of tank water levels and flows at system meters. On the other hand, for an energy analysis, the capability of the model to predict pump station cycling and energy consumption will be the focus (Cesario, L 1995).

The magnitude of the demand adjustment required can be approximated by the difference in tank storage volumes between modeled and observed conditions. For example, if the modeled tank and the observed tank both contain 1.3 MG , but at the end of a one-hour time step the modeled tank contains 1.7 MG, while the real tank contains 1.6 MG, then the demands in the model may need to be increased by 0.1 MG during that hour. As always with calibration, such adjustments need to be logical and justifiable, and the calibration should result in a fairly smooth curve in agreement with the observed data (Obradovic, 1998).

Most EPS calibration deals with the examination of plots of observed versus modeled tank water levels. As a general rule of thumb, if the observed and modeled water levels are both heading in the same direction but at slightly different rates, then the water use in that pressure zone needs to be corrected. However, if the water levels are going in opposite directions, then the on/off status at pumps or valves is usually the culprit. Data from chart recorders placed at key locations in the system can provide insights into what to adjust.

In fact, no guidelines exist for the acceptable level of calibration, however, many modelers agree that the level of effort required to calibrate a hydraulic network model and the desired level of calibration accuracy will depend upon the intended use of the model (Ormsbee and Lingireddy, 1997; Cesario, Kroon, Grayman, and Wright, 1996; and Walski, 1995). Regardless of which approach to calibration is adopted, a realistic model should achieve some level of performance criteria. For an EPS, in addition to pressures and flows, the volumetric difference between measured and predicted tank storage between two consecutive time steps should be ±5 percent of the total tank turnover for significantly large tanks (tank turnover is taken to be total volume in plus total volume out between two time intervals) (Nguyen, B 1994).

The true test of model calibration is that the end user (for example, the pipe design engineer or chief system operator) of the model results feels comfortable using the model to assist in decision-making. To that end, calibration should be continued until the cost of performing additional calibration exceeds the value of the extra calibration work.

2.2.4 System Skeletonization

Skeletonization is the process of selecting for inclusion in a water distribution model only the parts of the hydraulic network that have a significant impact on the behavior of the system (Eggener and Polkowski, 1976). The portions of the network that are not modeled are not ignored; rather, the effects that these elements contribute to the system are accounted for within the parts of the system that are included in the model.

When creating a water distribution model, including each individual service connection, valve, and every one of the numerous other elements that make up the actual network would be a huge undertaking for larger systems.

A fully realized water distribution model can be an enormously complex network consisting of thousands of discrete elements, and not all of these elements are necessary for every application of the model. When elements that are extraneous to the desired purpose are present, the efficiency, usability, and focus of the model can be substantially impacted, and calculation and display refresh times can be seriously impaired

In addition to the logistics of creating and maintaining a model that employs little or no skeletonization, there is another reason that this level of detail is unnecessary. Depending on the application to which the model is being applied, incorporating all of these elements in the model has no significant impact on the accuracy of the results that are generated

In fact, multiple models are required for different applications. since different levels of skeletonization are appropriate depending on the intended use of the model. For an energy cost analysis, a higher degree of skeletonization is preferable. For fire flow and water quality analysis, minimal skeletonization is called for. Because of this necessity, various automated skeletonization techniques have been developed to assist with the skeletonization process (Fujiwara O, Tung H, 1991).

2.3 Water Quality Modeling in Distribution System

It is well known that the quality of drinking water can change within a distribution system. The movement or lack of movement of water within the distribution system may have deleterious effects on a once acceptable supply. These quality changes may be associated with complex physical, chemical and biological activities that take place during the transport process. Such activities can occur either in the bulk water column, the hydraulic infrastructure, or both, and may be internally or externally generated. The ability to understand these reactions and model their impact throughout a distribution system will assist water suppliers in selecting improved operational strategies and capital investments to ensure delivery of safe drinking water .

Basically water quality modeling is simulated in a steady or a dynamic environment. In steady-state modeling, the external conditions of a distribution network are constant in time and the nodal concentrations of the constituents that will occur if the system is allowed to reach equilibrium are determined. These methods can provide general information on the spatial distribution of water quality. In dynamic models the external conditions are temporally varied and the time varying nodal concentrations of the constituents are determined. The algorithms developed include steady state and dynamic models.

Rossman and Boulos have given a comprehensive description of dynamic modeling and the existing numerical solution methods hence a review will not be repeated here. Instead the treatment given to the reactions by these methods are presented and also the advantages and limitations of existing Lagrangian methods are discussed.

As described by (Munavalli, Kumar 2004) in the Time-Driven-Method (TDM), the constituent concentration of a segment is subjected to reaction at every water quality time step (Qstep). The Qstep is a computational time step at which the quality conditions of the entire network are updated.

In the Event-Driven-Method (EDM) procedure the constituent concentration in all the pipe segments are subjected to reaction with respect to the length of the subhydraulic time step. In both methods the kinetic reaction mechanism continues with time under the conditions of zero flow or flow reversal in pipes. Both the TDM and EDM are free from numerical dispersion and phase shift errors when compared with Eulerian methods. Basically the TDM simulation procedure is carried out in steps of pre-specified Qstep. Hence it is possible that during any step more than one segment may be consumed at the downstream node of a pipe. If the segments consumed have different concentrations then this leads to an artificial mixing whose effect is more pronounced in tracing sharp concentration fronts. In addition, the TDM solutions are affected by a loss of resolution in concentration and accuracy is dependent on both Qstep and concentration tolerance used. Even though the EDM is supposed to be accurate irrespective of the Qstep used, the concentration tolerance used and the tolerance dependent subsegmentation process at changing hydraulic conditions may affect the accuracy of the method. In the EDM procedure the concentration conditions at a node are updated only when an event occurs at that node. Also all the segments and nodes are updated at the end of a hydraulic time step or output reporting time whichever occurs first

At the start of the simulation the event occurrences are dictated by the travel time in the pipes. Rossman and Boulos tested and compared the Eulerian (FDM and DVEM) and Lagrangian (TDM and EDM) methods. They concluded that the Lagrangian methods are more efficient for simulating the chemical transport in a water distribution system. The testing of the methods was done for analytical solutions, actual field studies and variable sized networks. The models are contrasted with respect to analytical solutions for validation at zero concentration tolerance and a particular Qstep. It is useful to study the differences exhibited by the Lagrangian methods for a normally used hydraulic time step of 1 h as reporting time under varying tolerance and Qstep values with no restriction on the number of segments generated. It is also interesting to study how the analytical solutions are contrasted with respect to the solutions obtained by these methods under varying concentration tolerance and Qstep values. It is obvious that the solution given by TDM and EDM may perform better against the analytical solution for zero concentration tolerance and reasonably small quality time step. The relative comparison of the methods with analytical solutions considering the variations in concentration tolerance and Qstep brings out the degree of variability exhibited by the methods with respect to the true solution for the system. Application of the methods to real life networks will generate a large number of new segments and this segmentation can be controlled by imposing a concentration tolerance.

Also there is a need to develop a methodology which can nearly eliminate the limitations discussed earlier in the Lagrangian models for the transport of chemical species. A hybrid methodology (Keedwell, 2004) developed utilizes the better features of existing Lagrangian methods. The performance of all the methods is tested against available analytical solutions under varying conditions of concentration tolerance and Qstep for both reactive and non-reactive constituents. The methods are also applied to network problems of varying size and a set of solutions is obtained for a range of concentration tolerance and Qstep values. An attempt is made to compare the representative solutions given by existing methods and the proposed hybrid method at selected nodes of a network problem. The results are interpreted in terms of the maximum number of segments generated (maximum segmentation of the network) at any time during the simulation and the solution time.

Water quality simulations use the network hydraulic solution as part of their computations. Flow rates in pipes and the flow paths that define how water travels through the network are used to determine mixing, residence times, and other hydraulic characteristics affecting disinfectant transport and decay. The results of an extended period hydraulic simulation can be used as a starting point in performing a water quality analysis Grayman, Rossman, and Geldreich (2000).

Most water quality models make use of one-dimensional advective-reactive transport to predict the changes in constituent concentrations due to transport through a pipe, and to account for formation and decay reactions. Equation (4) shows concentration within a pipe i as a function of distance along its length (x) and time (t).

$$\frac{\partial C_i}{\partial t} = \frac{Q_i \partial Ci}{A \partial x} + \theta(C_i), i = 1 \ldots p \qquad (4)$$

where C_i = concentration in pipe I (M/L^3)

Q_i = flow rate in pipe I (L^3/T)

A_i = cross-sectional area of pipe i (L^2)

$\theta(C_i)$ = reaction term ($M/L^3/T$)

Equation above must be combined with two boundary condition equations (concentration at x = 0 and t = 0) to obtain a solution. The equation for advective transport is a function of the mean velocity of the fluid. Thus, the bulk fluid is transported down the length of the pipe with a velocity that is directly proportional to the average flow rate. The equation is based on the assumption that longitudinal dispersion in pipes is negligible and that the bulk fluid is completely mixed (a valid assumption under turbulent conditions).

Water quality simulation uses a nodal mixing equation to combine concentrations from individual pipes described by the advective transport equation, the boundary conditions for each pipe. The equation shows a mass balance on concentrations entering a junction node.

$$\left[c_{OUT_i} = \frac{\sum_{i \in JN_j} Q_i C_{i,n_j} + U_j}{\sum_{i \in out_j} Q_i} \right] \qquad (5)$$

where c_{OUT_i} = concentration leaving the junction node j (M/L^3)

OUT_j = set of pipes leaving node j

IN_j = set of pipes entering node j

Q_i = flow rate entering the junction node from pipe i (L^3/T)

C_{i,n_j} = concentration entering junction node from pipe i (M/L^3)

U_j = concentration source at junction node j (M/T)

The nodal mixing equation describes the concentration leaving a network node as a function of the concentrations that enter it and describes the flow-weighted average of the incoming concentrations which are mixed according to Equation showed above, and the resulting concentration is transported through the outgoing pipes modeled as demand leaving the system. The nodal mixing equation assumes that incoming flows are completely and instantaneously mixed. The basis for the assumption is that turbulence occurs at the junction node, which is usually sufficient for good mixing. If a source is located at a junction, constituent mass can also be added and combined in the mixing equation with the incoming concentrations.

Pipes are sometimes connected to reservoirs and tanks as opposed to junction nodes. The following equation explains the mass balance of concentration entering or leaving the tank

$$\frac{dC_k}{dt} = \frac{Q_i}{V_K}\left(C_{I,NP}(T) - C_K\right) + \theta(C_K) \tag{6}$$

where C_k = concentration within tank or reservoir k (M/L^3)

Q_i = Flow entering the tank or reservoir from pipe i (L^3/T)

V_k = volume in tank or reservoir k (L^3)

$\theta(C_K)$ = Reaction term (M/L^3/T)

As a result of water entering from upstream pipes mixes with water that is already in storage. The above equation applies when a tank is filling if the concentrations are different, blending occurs. The tank mixing equation accounts for blending and any reactions that occur within the tank volume during the hydraulic step. During a hydraulic step in which draining occurs, terms can be dropped and the equation (3) is simplified as.

$$\frac{dC_K}{dt} = \theta(C_K) \qquad (7)$$

Specifically, the dilution term can be dropped because it does not occur. Thus, the concentration within the volume is subject only to chemical reactions. Furthermore, the concentration draining from the tank becomes a boundary condition for the advective transport equation written for the pipe connected to it.

Equations (6) and (7) assume that concentrations within the tank or reservoir are completely and instantaneously mixed. This assumption is frequently applied in water quality models. There are, however, other useful mixing models for simulating flow processes in tanks and reservoirs (Grayman et al., 1996). For example, contact basins or clearwells designed to provide sufficient contact time for disinfectants are frequently represented as simple plug-flow reactors using a "first in first out" (FIFO) model. In a FIFO model, the first volume of water to enter the tank as inflow is the first to leave as outflow.

If severe short-circuiting is occurring within the tank, a "last in first out" (LIFO) model may be applied, in which the first volume entering the tank during filling is the last to leave while draining. More complex tank mixing behavior can be captured using more generalized "compartment" models. Compartment models have the ability to represent mixing processes and time delays within tanks more accurately. All the models mentioned in this section can be used to simulate a non-reactive (conservative) constituent, as well as decay or formation reactions for substances that react over time. The models can also be used to represent tanks that either operate in fill and draw mode or operate with simultaneous inflow and outflow.

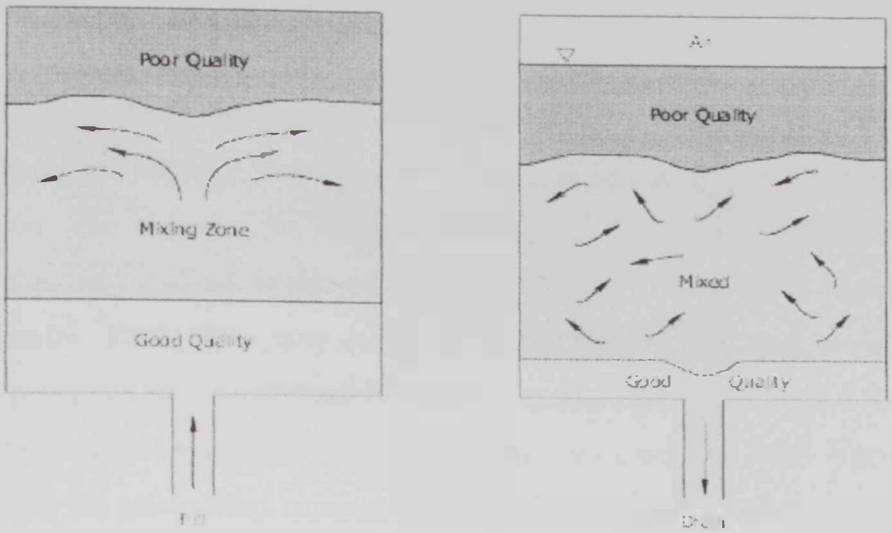


Figure 2.4 Tank mixing model with Three compartments.

Chemical reaction terms are present in concerned equations showed above. Concentrations within pipes, storage tanks, and reservoirs are a function of these reaction terms. After water enters the distribution system, it is subject to many complex physical and chemical processes, some of which are poorly understood, and most of which are not modeled. Three chemical processes that are frequently modeled, however, are bulk fluid reactions, reactions that occur on a surface (typically the pipe wall), and formation reactions involving a limiting reactant. First, an expression for bulk fluid reactions is presented, and then a reaction expression that incorporates both bulk and pipe wall reactions is developed.

Bulk fluid reactions occur within the fluid volume and are a function of constituent concentrations, reaction rate and order, and concentrations of the formation products. A generalized expression for n^{th} order bulk fluid reactions is developed in the below equation (Rossman, 2000).

$$\theta(C) = \pm K C^n \qquad (8)$$

where $\theta(C)$ = reaction term ($M/L^3/T$)

K = reaction rate coefficient [$(L^3/M)^{n-1}/T$]

C= concentration (M/L^3)

n = reaction rate order constant

Equation (8) is the generalized bulk reaction term most frequently used in water quality simulation models. The units of the reaction rate coefficient depend on the order of the reaction. The order of the reaction depends on the composition of the reactants and products that are involved in the reaction. The reaction rate order is frequently determined experimentally. Zero-, first-, and second-order decay reactions are commonly used to model chemical processes that occur in distribution systems. Figure 2.4 is a conceptual illustration showing the change in concentration versus time for these three most common reaction rate orders. Using the generalized expression in the last Equation, these reactions can be modeled by allowing n to equal 0, 1, or 2 and then performing a regression analysis to experimentally determine the rate coefficient. The rate expression accounts for only a single reactant concentration, tacitly assuming that any other reactants (if they participate in the reaction) are available in excess of the concentration necessary to sustain the reaction. The sign of the reaction rate coefficient, k, signifies that a formation reaction (positive) or a decay reaction (negative) is occurring.

The most commonly used reaction model is the first order decay model, for first order reactions, the units of k are (1/T) with values generally expressed in 1/days or 1/hours. This has been applied to chlorine decay, radon decay, and other decay processes. A first order decay is equivalent to an exponential decay, represented by the Equation (9).

$$C_T = C_O C^{-kt} \qquad (9)$$

where C_t = concentration at time t (M/L^3)

C_o = initial concentration (at time zero)

k = reaction rate (1/T)

Disinfectants are the most frequently modeled constituents in water distribution systems. Chlorine (the most common disinfectant) is shown reacting in the bulk fluid with natural organic matter (NOM), and at the pipe wall, where oxidation reactions with biofilms and the pipe material (a cause of corrosion) can occur.

Many disinfectant decay models have been developed to account for these reactions The first-order decay model has been shown to be sufficiently accurate for most distribution system modeling applications and is well established Grayman (1994).

Figure 2.5 Disinfectant reactions occurring within a typical distribution system.

When n = 1, then equation (5) is considered a simple first-order reaction. One shortcoming of the first-order reaction model is that it accounts for the concentration of only one reactant. This model is sufficient if only one reactant is being considered. For example, when chlorine residual concentrations are modeled, chlorine is assumed to be the limiting reactant and the other reactants-material at the pipe walls and natural organic matter (NOM)-are assumed to be present in excess. The behavior of some disinfection by-product (DBP) formation reactions, however, differs from this assumption. NOM, not chlorine, is frequently the limiting reactant. DBP formation is just one example of a generalized class of reactions that can be modeled using a limiting reactant. The reaction term for this class of formation and decay reactions as proposed by Rossman (2000) and shown in the following Equation

$$\theta(c) = \pm k(c_{\lim} - c)c^{n-1} \tag{10}$$

where C_{lim} = limiting concentration of the reaction (M/L^3)

The reaction rate coefficient K, however, is now a function of the bulk reaction coefficient and the wall reaction coefficient, as indicated in the following equation.

$$K = k_b + \frac{k_w k_f}{R_H(k_W + k_f)} \tag{11}$$

where k_b = bulk reaction coefficient (1/T)

k_w = wall reaction coefficient (L/T)

k_f = mass transfer coefficient, bulk fluid to pipe wall (L/T)

R_H = hydraulic radius of pipeline (L)

The rate that disinfectant decays at the pipe wall depends on how quickly disinfectant is transported to the pipe wall and the speed of the reaction once it is there. the dimensionless Sherwood number, along with the molecular diffusivity coefficient (of the constituent in water) and the pipeline diameter is used to determine the rate at which disinfectant is transported.

$$k_f = \frac{S_H d}{D} \tag{12}$$

where S_H = Sherwood number

d = molecular diffusivity of constituent in bulk fluid (L^2/T)

D = pipeline diameter (L)

For stagnant flow conditions (Re < 1), the Sherwood number, S_H, is equal to 2.0. For laminar flow conditions (1 < Re < 2,300), the average Sherwood number along the length of the pipe can be used. To have laminar flow in a 6-in. (150-mm) pipe, the flow would need to be less than 5 gpm (0.3 l/s) with a velocity of 0.056 ft/s (0.017 m/s). At such flows, head loss would be negligible.

$$S_H = 3.65 + \frac{0.0668\left(\dfrac{D}{L}\right)(RE)(\dfrac{V}{d})}{1 + 0.0 + \left[\left(\dfrac{D}{L}\right)\mathrm{Re}\left(\dfrac{v}{d}\right)\right]^{-2/3}} \tag{13}$$

where L = pipe length (L)

While, for turbulent flow (Re > 2,300), the Sherwood number is computed using Equation below

$$S_H = 0.023\,\mathrm{Re}^{0.83}\left(\frac{V}{d}\right)^{0.333} \quad (14)$$

where Re = Reynolds number

ν = kinematic viscosity of fluid (L^2/T)

Using the first-order reaction framework developed immediately above, both bulk fluid and pipe wall disinfectant decay reactions can be accounted for. Bulk decay coefficients can be determined experimentally. Wall decay coefficients, however, are more difficult to measure and are frequently estimated using disinfectant concentration field measurements and water quality simulation results.

A first-order growth rate to a limiting value has been used to represent the formation of trihalomethanes, a common form of DBP, in distribution systems (Vasconcelos et al., 1996). Mathematically this is represented by the below Equation and shown graphically in the Figure 2.6

$$THM(t) = C_o + [FP - C_o] \quad (15)$$

where THM(t) = THM concentration at time t

C_o = initial THM concentration

FP = formation potential (concentration)

k = reaction rate (a positive value)

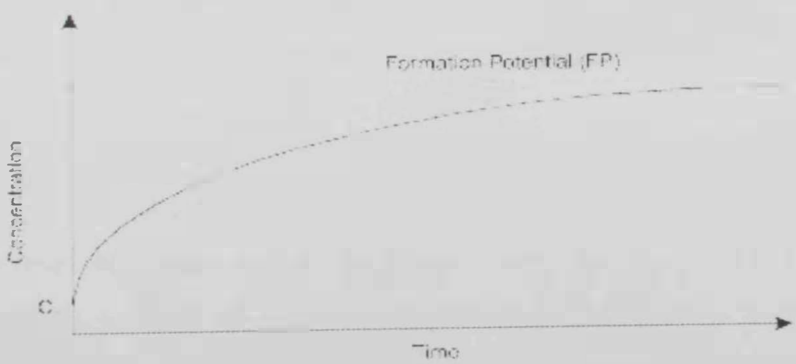


Figure 2.6 First order growth for THM.

Many researches were done aiming to best allocation and managing of disinfection stations along water distribution systems, are presented. With the main objective of proposing a criterion to limit chlorine and disinfection by-product concentrations in the drinking water supplied to users without compromising its agreeableness and safety characteristics. The procedures are based on the evaluation of two different indexes obtained through the analysis of several operating conditions related to changes of water demands in both space and time and to the random behaviour of users: the Presence Index and the Efficiency of Allocation Index. The proposed procedures have been applied to a case study in order to determine, respectively, (1) the number and the allocation of chlorine booster stations according to minimum and maximum values of chlorine concentration indicated in drinking water standards and to minimum chlorine dosage allowed for each station; and (2) the optimal dosage of chlorine according to the number and allocation of chlorine disinfection stations and to the changes of water demand nom users.

2.4 Simulation Tool

Hydraulic modeling of water-distribution systems can be conducted by solving mathematical equations that characterize the pipe network of the distribution system. The WaterCad 7.0 water-distribution system model was chosen to conduct an extended period simulation of the hydraulic behavior within the water-distribution system. WaterCad 7.0 solves the mentioned equations in section 2.1 for each storage nodes tanks or reservoir.

The fate of a dissolved substance flowing through a distribution network over time is tracked by WaterCad 7.0 dynamic water-quality simulator. To model water quality of a distribution system, WaterCad 7.0 uses flow information computed from the hydraulic simulation as input to the water-quality model. The water-quality model uses the computed flows to solve the equation for conservation of mass for a substance within each link connecting nodes.

Identifying the source of delivered water in a distribution system has become a necessity when trying to determine the location of a source that may supply water that exceeds a given level of a chemical or biologic constituent. Wood and Ormsbee (1989) developed an explicit method to calculate the percentage of flow, under steady flow conditions, originating at various source points at a specific location in a distribution system.

WaterCad 7.0 also has the ability to track the percentage of water reaching any point in the distribution network over time from a specified location (source) in the network (i.e., the "proportionate contribution" of water from a specified source).

The primary component of a WaterCAD project is the network model. The element types that are used to form a network models are:

- **Pressure Pipes**, where pipes are link elements that connect junction nodes, pumps, valves, tanks, and reservoirs to each other. The only way for water to travel from one node to another is by following a path through one or more pipes. Data pertaining to the pipeline characteristics constituting the distribution system network were retrieved from GIS and as built drawings supplied by the water utility. Parameters required by WATERCAD 7.0 to describe pipes include (Table 2A): a pipe identification label, starting and ending node labels, length, diameter, roughness coefficient, and the status of the pipe (open or closed).
- **Pressure Junctions**, where junctions are non-storage nodes where water can leave the network to satisfy consumer demands, water can enter the network as an inflow, or chemical constituents can enter the network. WATERCAD 7.0 identifies junctions (or nodes) as the beginning and ending points associated with each pipe or pipe segment in the model network. Each junction is assigned an alpha-numeric identification label, an elevation, a demand (or consumption) value, and a demand pattern number (Table 2A). Because the goal of the study is to conduct a water distribution performacnce assessment, geo-spatial location information for pipe junctions, pipelines, and network facilities is required. These known coordinates were used to geo-reference all model nodes (and links) in the distribution-system network. The last parameter associated with junction data is the demand pattern. With this parameter, WATERCAD 7.0 has the ability to modify the nodal demand data based on the demand pattern. For example, if the water utility serviced residential, commercial, and industrial users, each group of water users have a different diurnal demand pattern and therefore, nodal demand data would need to be modified depending on the type of use. This is accomplished in WATERCAD 7.0 by assigning a demand pattern number to each tank. All model nodes that were assigned a positive demand value (indicating outflow from the system) used the same diurnal demand pattern, the same demand pattern number was assigned to each model junction identified as having a positive consumption value.

Table 2A Set of input data properties required by WATERCAD 7.0 to model the water-distribution systems

Component	Properties
Junction	Identification label Elevation Demand Demand pattern
Tanks	Identification label Bottom elevation Initial water level Minimum allowable water level Maximum allowable water level Tank diameter
Pipes	Identification label Start node label End node label Length Diameter Roughness coefficient
Patterns	Identification label Multiplication factors
Time Parameters	Duration Hydraulic Time Step Pattern Time Step

- **Tanks**, where tanks are a type of Storage Node. The water surface elevation of a tank will change as water flows into or out of it during an extended period simulation. Tanks can have either a circular or irregular cross section. Internal storage tanks (IST's) are associated with model junctions in WATERCAD 7.0. The parameters used to describe storage tanks in WATERCAD 7.0 are listed in 2A; specific features for each storage tank are listed in appendix D. For this study, all storage tanks were modeled as having cylindrical geometries. The initial, minimum and maximum water level for each tank was assumed as mentioned in chapter 4

- **Reservoir** which are a type of storage node. The water surface elevation of a reservoir does not change as water flows into or out of it during an extended period simulation, unless an HGL Pattern has been applied to the reservoir. Reservoirs can be used to model external water sources such as lakes, streams, and wells. When an HGL pattern is applied, reservoirs can also be used to represent tidal activity and connections to other systems where the pressure varies over time.
- **Valve**, where it is an element that opens, throttles, or closes to satisfy a condition you specify. It is represented in WaterCAD as a node.
- **Pump** is an element that adds head to the system as water passes through. It is typically defined by a pump curve and control elevations, which turn the pump on or off. It is represented in WaterCAD as a node.

Requirements for Model Input

The WATERCAD 7.0 water-distribution system model was used in conjunction with the collected field-test data to develop and calibrate a model of the present-day (2006) water-distribution systems serving the selected areas. Information required to conduct a simulation using WATERCAD 7.0 include data describing pipeline characteristics, consumption and diurnal demand patterns, tank geometries and initial water levels, and simulation time parameters. Table 2A describes the set of input data properties needed to model components of the water-distribution system serving the selected areas using WATERCAD 7.0.

Time Parameter Data

WATERCAD 7.0 assumes that consumption values, supply rates, and concentrations at source nodes remain constant over a fixed period of time. However, these parameter values can change from one time period to another. To conduct an extended period hydraulic simulation, WATERCAD 7.0 requires three time parameters: (1) the duration of the simulation, (2) the hydraulic time-step size, and (3) the pattern time-step size. For the selected areas simulations, the duration of the simulation was set equal to the duration of the tests 8760 hours. The hydraulic and pattern time-step sizes were set equal to 60 hour. In figure 2.5 WaterCAD allows defining tanks with either fixed or variable sections. For steady-state simulations, a tank is considered to have a constant water surface elevation, similar to a reservoir. Tank simulation organizes the related input data and calculated results. The following are needed as inputs

- Assignment of demands or inflows to tank elements in order to simulate water leaving or entering the network
- Data defining the geometric characteristics of the tank and its operating level range.
- Input parameters used when performing a Water Quality Analysis

The designer can define a hydraulic load consisting of multiple demands and inflows for each tank node in the network. Each individual hydraulic demand or inflow consists of a baseline flow rate and a pattern that is applied when performing an Extended Period Simulation (EPS). This software provides a table for editing hydraulic loads. Each row represents an individual hydraulic demand or inflow.

The following can be determined:

- Demand represents a withdrawal of water quantity from the network system (if the value entered is negative, then the liquid is entering network). Inflow represents the addition of water quantity to the system (negative inflow represents flow leaving the system)
- Enter the baseline flow rate for the load. The entered figure will always be positive. The units are volume per unit time (typically l/s or gpm). Choosing the EPS pattern that will apply to this load. Each load in the table can have a different EPS pattern. The multipliers defined in the pattern will be applied against the baseline load.

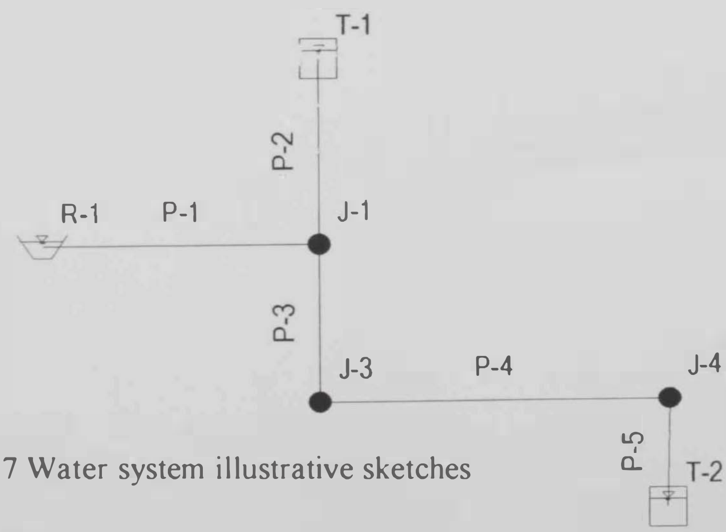


Figure 2.7 Water system illustrative sketches

As shown in the above sketch, which represent a small system where we can apply the methodology described above. The elevations of all nodes will be known and the pressure at the source will be presented as a HGL or elevation according to the field measurements i.e. the pressure reading at the feed main to each water network system. The levels i.e. initial, minimum and maximum, in the tanks can be entered according to a certain assumptions or field measurements. The pipes length can be also defined.

CHAPTER 3

DESCRIPTION OF AL AIN WATER NETWORK

In this Chapter, a brief description is presented for each source supplying Al Ain region with the requested water quantities. These sources are desalination plants located along the costal boundary of Abu Dhabi Emirate in addition to some well fields expected to be closed soon due to their water quality deterioration reported lately. Moreover, a brief explanation for the water distribution code represents the analysis guidelines in addition to the water demand forecast for Al Ain city and its factors are presented followed by the water distribution system material and accessories. Intermittent water supply pattern is explained with its advantages and disadvantages. The chapter concludes with selecting three zones of Al Ain distribution system to focus on the current study.

3.1 Water Supplied to Al Ain Region

3.1.1 Current Production Capacity

Water is scarce in the Al Ain region and a majority of the supplies are pumped from desalination plants over 120 km away in Abu Dhabi, in the west of the country. Figure 3.1 shows the three main sources of water supplying Al Ain region and defined as follows:

- Taweelah Desalination plant through Sweihan Pumping station to Al Ain Reception Pumping station in the north-west;
- Unit IV of Um Al Nar Desalination plant through Shobaisi Pumping station to Remah Pumping station in the south-west;
- Al Fujairah Desalination plant through 1600 mm Carbon Steel line to Al Ain Reception Pumping station.

An additional source of water from the neighboring state of Al Fujairah has become available since late 2003. This supply takes spare capacity of 40 mgd from the Al Fujairah desalination plant, 140 km away on the coast of the Gulf of Oman.

Two methods to calculate the total inflow into the Al Ain region were considered as follows

1. Total water arriving at the boundary of the region as shown in Figure 3.1 and,
2. Calculation of water distributed across the region.

The first method was to calculate the total inflow to Al Ain region by summing up the average daily flows through Sweihan, Remah and Al Khazna Pumping Stations, flows to Al Khazna Palace, in addition to the quantities delivered through fujairah line. The production from the Al Ain Well-fields is consumed by the nearby VIP's properties.

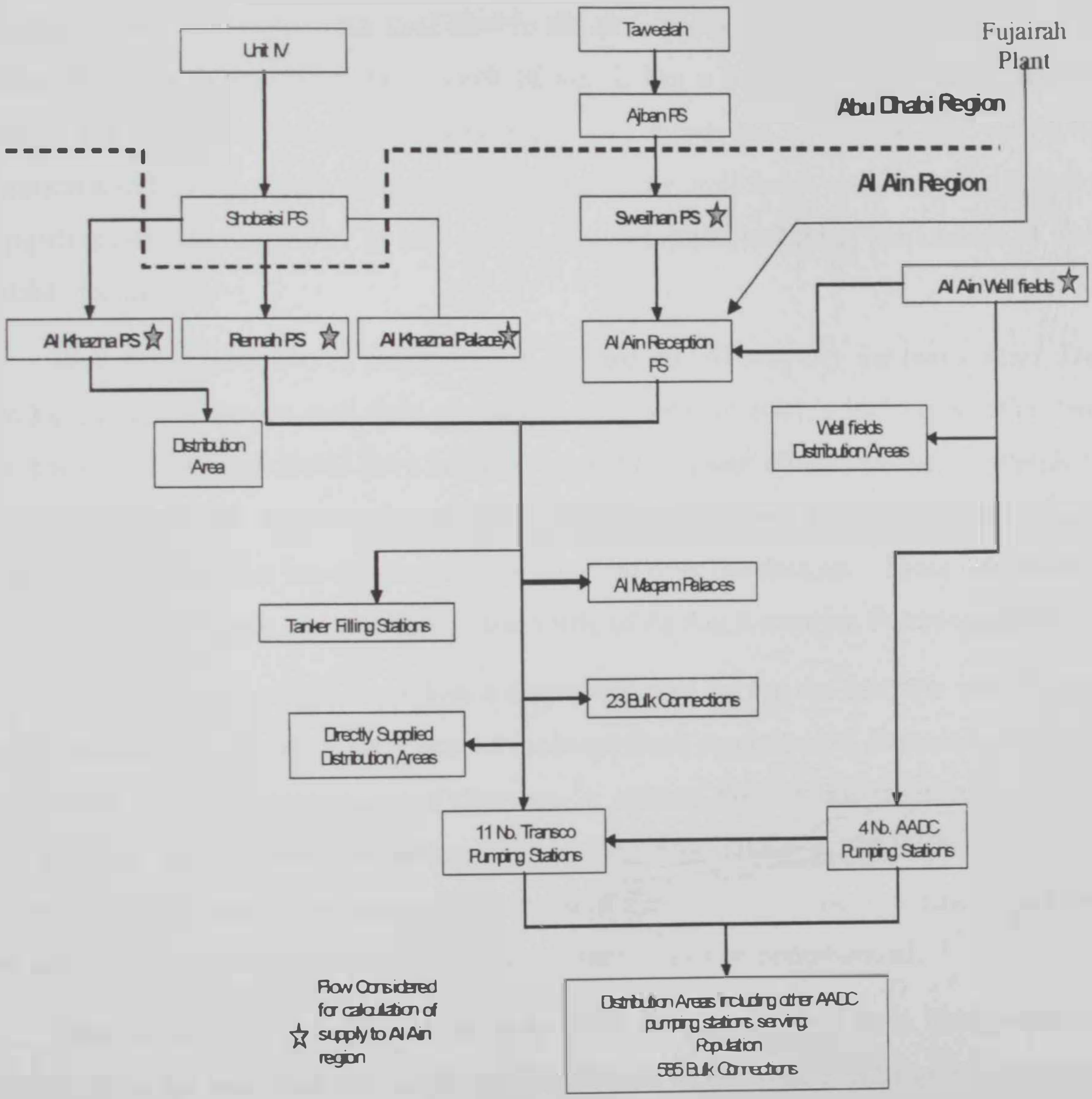


Figure 3.1 Water Inflows to Al Ain Region

The second method was to calculate the total inflow to Al Ain region by summing up the total daily flows through each of the Transco pumping stations, the areas directly connected to the Transco mains upstream of the Transco Pumping stations. The first method was chosen as the most appropriate, as all measuring points are at the boundary to the region and this ensures that all water entering the Al Ain region is considered.

3.1.2 Well Field Production

Well fields have been developed across Al Ain but are mainly concentrated in the northern region of the city. The wells in the south and east of Al Ain region supply discrete areas and do not contribute to the total flow to the city. Previously, the well fields supplied a number of AADC pumping stations, each of which has a dedicated distribution network. Much of the flow from the northern well fields was forwarded to Al Ain city via Al Ain Reception and Hili PS. Some consumption occurs in the well fields and along the length of the pipelines transferring water to Hili and Al Ain Reception, reducing the amount of water available for the city.

Well fields have been a major source of water for Al Ain city for many years. Data provided by AADC covers well field production from 1990 to 2001, which reveals the trends over this time. New well fields have been developed to replace others ceasing to provide the quantity they once did. However, as new well fields come on line to replace failing sources, average daily production has remained fairly constant over the data set. These well fields in Al Ain region are mainly concentrated in the north, of Al Ain Reception Pumping station.

Well field production has taken a downward turn during the last few years, largely because abstracted volumes greatly exceed recharge from precipitation. Examining the last 4 years of the available data shows a decrease in output. Should this trend continue it is envisaged that groundwater abstraction will no longer be viable by 2009 (see Table 3.3). Furthermore, if the rate of development of new well fields slows or the production from them does not match that of the cancelled wells, the situation will be compounded.

The AADC Monthly Report for July 2005 holds details of well field production volumes. Data for well field production are considered as the Well Field Input. The average daily flow for the well fields during the peak month was 5 MIGD calculated by summing up the daily production and dividing by the number of days. Currently, this quantity supplies the VIP's properties such as the farms

3.1.3 Additional Supply to Al Ain Region

There are proposals to increase water supply to Al Ain region by providing additional production capacity at Fujairah and through the construction of a new transmission main to bring water from Shuweihat. Details of the additional flows are provided in Table 3.1.

Table 3.1 Additional Water Supply to Al Ain region (ADWEC Report, 2003)

Scheme	Production Available for Al Ain		Receiving Pumping Station
Additional production capacity at Fujairah	Phase 1: 2004	40mgd	Al Ain Reception
	Phase 2: 2007	60mgd	Moyhayer
Shweihat Pipeline	Phase 1: 2006	51mgd	Shweihat to Shobaisi
	Phase 2: 2008		Shobaisi to Military via South West (new PS)
Total of additional quantities from Fujairah and Shwiehat desalination plant	2004	40mgd	
	2006	91mgd	
	2007	151mgd	

Current proposals are for a total of 180mgd additional production capacity at Fujairah (90mgd under each phase), of which 100mgd will be made available to Al Ain region.

Additional supply from Shweihat will be limited by the capacity of the pipeline which will be capable of transferring 51mgd initially to Shobaisi by 2008 and subsequently to a new pumping station to the South west of the city (South West PS) and on to Military by 2008 The available supply is shown in Table 3 2 and Figure 3 2

Table 3 2 Available Supply to Al Ain in MIGD

	2003	2004	2005	2006	2007	2008	2009	2010
Swiehan PS	57.2	57 2	57 2	75	84	84	84	84
Remah PS	14.3	14.3	14.3	14 3	14.3	14.3	15.4	15 4
Well Fields	19 8	5	5	2	1.1	1 1	0	0
Al Khazna	1 7	1.7	1.7	1.7	1.7	1.7	1 7	1.7
From Fujairah		40	40	40	100	100	100	100
From Shweihat						51	51	51
Total	**93**	**118.2**	**118.2**	**133**	**201.1**	**252.1**	**252.1**	**252.1**

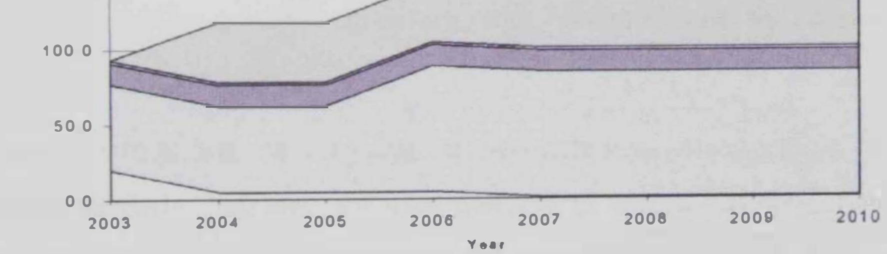


Figure 3.2 Components of Water Supplied into Al Ain Region

The supply into Al Ain Region was calculated using data of average day in July 2005 as shown in Table 3.3. The difference between the demand and the supply is around 26.8 Mgd This is illustrated in Table 3.4 and 3.5 respectively; if 100% demand management is applied.

Table 3.3 Inflow Calculation for Al Ain region

Component	Mgd
Sweihan PS	57.2
Remah PS + Al Khazna PS	16
Fujairah Plant + Well fields	45
Total	118.2

3.2 Demand Forecast in Al Ain Water Utilities

A major difficulty in simulating and evaluating a water distribution system is how to model the actual demands; i.e. consumptions, and the losses at the level of individual nodes. In any real-life system this information is never fully available, even if the total demand and the total loss are known rather well. It is true that data on demand can be found in the literature (Bailey, Jolly, & Lacey, 1986; Edwards & Martin, 1995; Obradovic & Lonsdale, 1998), but only for the regular situations. Information about the possible influence of local pressure upon demand is sadly lacking. Note that the designer's position is easier since the demand is a given or selected value, pressures should be within the given range, and losses might be allocated in a rather arbitrary way, perhaps just by increasing the demand proportionally to include losses.

These assumptions are not valid for the operational management of real-life systems. If one compares daily diagrams for total demand of the whole system with corresponding data captured at the level of (relatively small) demand management areas, one will discover that the first has much smaller amplitude in comparison with the latter: the minimum night flow (MNF) is relatively higher and the morning/evening peaks are less prominent. Hence, the losses were included in the demand where these demands obtained from the water demand forecast issued in 2005 (AADC, 2005).

A Demand Forecast was prepared as per M/s Hyder activities for the project "Improvement of Water Supply in Al Ain". The objectives of the Demand Forecast were.

- To confirm the 2005 population Figures which will be adopted for population and demand projections;
- To confirm per capita consumption Figures through a demand survey which has been carried out in Al Ain,
- To confirm existing Unaccounted For Water (UFW) / Non Revenue Water (NRW) levels specific to Al Ain,
- To establish the current level of water usage (baseline position);
- To determine the factors that will influence the demand and how they will act over the forecast period; and,
- To establish demand forecasts for 2005 and 2010

The approach adopted for the Demand Forecast is set out in the Demand Forecast Methodology Report issued by AADC in November 2004 (AADC Annual report, 2004).

3.2.1 Factors influencing Water Demand

There are many influencing factors that will determine how demand will grow in the Al Ain region. The principal drivers to the demand forecast will be:

- Population Increases
- Increases in the proportion of Low Cost (Shabia) and Villa type dwellings
- Growth in bulk connection demands
- Changes in supply regime (move towards constant supply)

Based on the above mentioned study, it was agreed that the per capita consumption = 100 gal/capita/day, and the total demand for Shabia and Villa are 1500 gal/day and 3500 gal/day respectively including the demand required for landscaping purposes (AADC water demand forecast report, 2005).

Table 3.4 shows the water demand forecast for 2003, 2005 and 2010 respectively. In this Table the intermittent demand (Unrestrained) and the Controlled one (Restrained) supply is illustrated for 2003, where the major difference between the two patterns is applying the demand management tools as illustrated in section 3.5.5. For 2005 and 2010 the controlled demand is shown, along with the demand management concept (DM), this concept is illustrated in details in section 3.7. The Table shows the differences in the water demand for the domestic as well as the villas and shabia landscaping categories for years 2003, 05 and 2010. The remaining categories will not be affected by applying or achieving any demand management level, since the demand is requested and specified by the concern departments i.e. municipality, agriculture, military works and the private department which controls the VIP palaces and farms.

Table 3.4 Water Demand forecast under different demand management scenarios (M/s Hyder water demand forecast report, 2004)

Demand Category	2003		2005			2010		
	Restrained	Unrestrained	100% DM	50% DM	0% DM	100% DM	50% DM	0% DM
	mgd	mgd	mgd	mgd	mgd	mgd	mgd	mgd
Domestic Purposes	18.9	41.5	21.1	43.7	46.2	27.9	43.8	59.6
Villas Landscaping	13.4	29.3	16.2	33.5	35.4	23.1	36.8	50.5
Shabia Landscaping	4.07	8.91	4.68	9.69	10.3	6.05	9.65	13.3
General Parks	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
IndustrialAreas	10.4	10.4	12.2	12.2	12.2	15.3	15.3	15.3
Commercial and VIP Farms	11.1	11.1	26.4	26.4	26.4	62	62	62
Palaces	29.2	29.2	31.7	31.7	31.7	37.5	37.5	37.5
Military Camps	3.16	3.16	4.1	4.1	4.1	6.08	6.08	6.08
NRW	6.39	34.4	28.7	40.4	41.7	37.3	46.2	54.5
Totals	**96.6**	**168**	**145**	**202**	**208**	**215**	**257**	**299**

3.3 Water Distribution code issued by AADC

Below is ADWEA specifications and standards regarding different technical parameters i.e velocities, pressure, peak demand.. etc. These standards will be utilized to assess the data and results obtained from the fields and the hydraulic modeling in chapters 4 and 5 respectively.

3.3.1 Demand Factors

Demand is the required portable water needed to meet Customer use, adjusted to take into account Water Demand Management policies, which may include water conservation efficiencies of use and water leakage.

Since the sources of production, the trunk mains, and the service reservoirs must all be designed to cope with maximum expected daily consumption, the mains and pipes which convey the water from the service reservoirs to the consumer is designed to cope with the maximum hourly rate of consumption. This maximum hourly draw-off rate varies from 1.1 to 4 times the average daily rate, depending on the area being served. The smaller the area the greater is the ratio. Hence, for a single house, the maximum hourly rate can work out at over 300% of the average daily rate.

Adams (Adams J.I.W.F, 1955) investigated the areas and estimated the peak flow factors shown in Figure 3.3 below according to population size. These factors apply to the average daily consumption for the year and show that maximum hourly rates vary from 3 times for areas of 500 population to 1.9 times for areas of ½ million population.

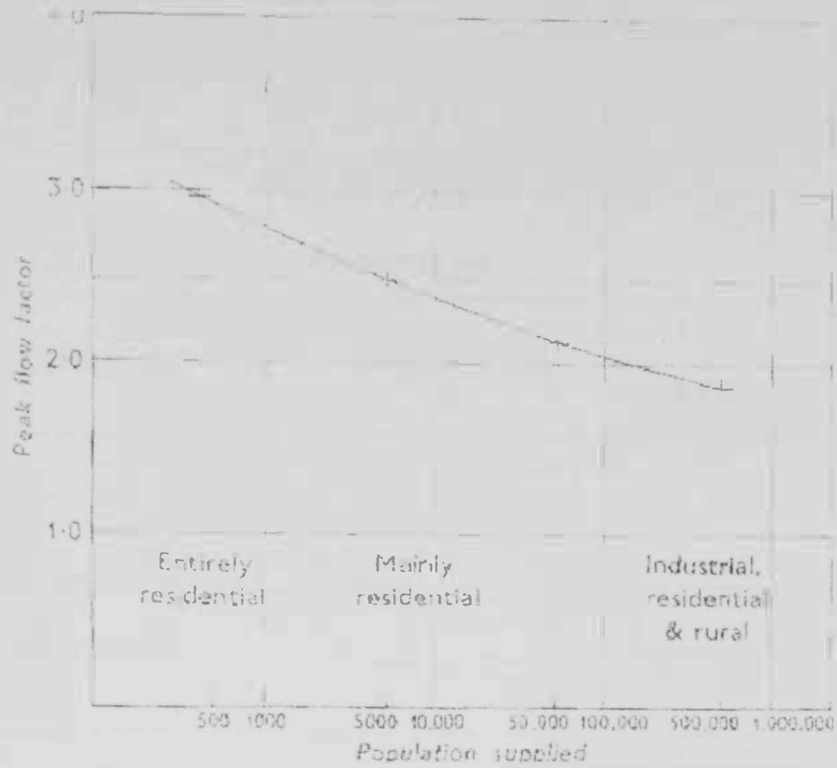


Figure 3.3 Ratio of peak hourly flow to annual average flow

The following relations then apply:

- Average Daily Demand (ADD) = Total Yearly consumption / 365 days
- Peak Daily Demand (PDD) = Maximum Demand in a single day
- Peak Instantaneous Demand (PID) = Maximum flow rate at any time

The demand forecast prepared by AADC shown in Tables 3.5 and 3.6 considers the average flow for the peak month. Within the peak month there will be variations in daily flows and diurnal peaks arising from varying demands during the day. These peaks are influenced by many factors, such as levels of leakage and background flow, the proportion of commercial and industrial demands and social patterns. As such, peaking factors are specific to the network under consideration. A factor of 2.5 can be applied to the average hourly flow for the peak month to obtain the peak hour design flow. This factor is based on population size.

Table 3.5 Peak Factors

Peak Factors, PF (range)		Peak Daily Demand (PDD)
1.25	1.5	PF x Average Daily Demand

Table 3.6 Peak Factors (According to the population)

Population	**Peak Factors, PF**	
50,000 or more	Low	2.0
	High	2.5
1,000 to 50,000	Low	2.5
	High	3.0
Less than 1,000	Low	3.0
	High	3.5

- Peak Instantaneous Demand (PID) = PF x Average Daily Demand
- PF for Intermittent Water Supply (6-8 hrs.) = 3.5

3.3.2 Pipeline Sizing

- All water distribution pipelines shall be sized to deliver the Peak Daily Demands (PDD) and allowing for fire fighting demand and leakage factors whilst ensuring head loss not exceeding 10 m/km.
- Secondary and Primary Pipeline (Sub-mains and mains) shall be sized to deliver peak daily demand (PDD) at connection nodes and head loss not exceeding 15 m/km.
- Special attention to be given to the fire flow requirements as capacity stipulated by the relevant Fire / Civil Defense Department.

Table 3.7 Pipeline Dimensions

Sub Clause	Pipeline Type	Minimum mm	Maximum Mm	Remarks
1	Distribution pipeline	100 150 (Recommended)	300	Pipelines smaller than DN150 may be used in locations where few consumer connections are required.
2	Secondary Pipeline(sub-main)	300	600	
3	Primary / Pressure Ring Pipeline(mains)	600	900	

Table 3.7 is applicable to DI pipes; other pipeline material made of GRP, Steel, HDPE or UPVC usually have different maximum sizing categories tolerated due to various technical and practical issues.

It should be noted that there will be other considerations such as available head and asset condition that will affect the replacement strategy for the individual pipes. There are conflicting criteria for the sizing of the distribution mains. A lower design velocity will allow a greater provision for an increase in demand beyond the design horizon. However, the transmission and distribution system covers a large area and larger mains will increase the retention time in the pipes, which may lead to water quality problems.

3.3.3 Water Network Pressures

Pressure zoning is arranged wherever possible to limit the maximum static pressure at any point to 60 meters head (6.0 bar) unless otherwise higher pressure is needed in the network for elevated areas or varying ground elevations. The pipeline test pressure shall be 1.5 times the design pressure or as directed by the AADC. Water distribution system is sized to ensure that the minimum residual pressures at the customer connection boundary shall be not less than 10 meters head (1 bar) at all times.

The distribution system currently operates on ground tanks for storage due to the intermittent supply. The minimum guideline pressure of 1 bar provisionally set by the RSB will be sufficient to fill the ground tanks. However, as one of the main objectives of the

AADC is to change over to a 24 hour continuous supply, this will make the ground storage tanks redundant. Removal of ground storage tanks should also be considered as part of a wider demand management strategy as considerable losses are likely to be experienced following the introduction of 24 hour supply due to leaking / overflowing tanks.

As design standards, it is recommended that the minimum pressure at the exit of the distribution system should be sufficient to fill a tank on the roof of a typical villa or shabia. A minimum pressure of 10m (1 bar) at the exit of the distribution system has therefore to be adopted.

This will be insufficient to serve the higher buildings in the city centre and therefore the existing ground or basement tanks with booster pumps will have to be retained in these areas. Increasing the pressure to 3 or 4 bar to serve these buildings is not recommended as the existing distribution system will have increased losses and breakout rate of bursts.

3.3.4 Water Flow Velocity

Ideally, the velocity in water pipelines should range between 0.5m/s and 2.0m/s depending on the pipeline size and material of construction. However, under extreme conditions (e.g. fire flows in high fire risk areas) velocities up to 4 m/s are acceptable depending on the type of construction material and hydraulic model output. The velocities in Table 3.8 are applied for the design and operational purposes.

According to the research published by the Ductile Iron Pipe Research Association (DIPRA, 2003), satisfactory performance with portable water has been achieved with cement/cement mortar lined ductile iron pipes with velocities of 6.1m/s to 12.2m/s. Based on this research and allowing for a factor of safety, DIPRA recommend a maximum design velocity of 4.3 m/s.

Pipeline manufacturers Saint Gobain advise that the maximum design velocity for their pipes is 7.0 m/s (DI pipes report, 2004).

It is recommended that the trigger level for the replacement of existing pipes due to excess velocity at 2010 peak month peak hour flows, be set at 4.5 m/s.

For new pipes, a lower design velocity should be considered to allow for spare capacity for future increases in demand. By extrapolating the rate of growth used in the demand forecast between 2005 and 2010, the total demand is predicted to double by the year 2030 (Hyder Master plan, 2005). If this is considered to be applied to an individual pipeline, the velocity would double within 20 years, therefore a design Figure of 2 m/s can be adopted for the design of new pipes when using the peak hourly flow. Whilst this approach does not consider individual pipe lengths, it does provide reasonable confidence that pipe lengths designed to 2 m/s velocity will typically not reach 4.5m/s until 2030 and will be well within the minimum design horizon of 10 years for hydraulic design of the distribution system.

The demand forecast predicts a 2.5 times increase demand in the Al Ain Region to 2010 compared to the current intermittent supply demand. However, this does not represent a 2.5 times increase in velocity in all the distribution system since the supply will be for 24 hrs compared to the intermittent supply for 6 hrs only. Demand in the existing pipes serving existing developments will only increase in line with growth in domestic consumption and therefore the pressure increase will be limited.

Certain pipes and in particular the transmission system will experience velocity increases approaching 2.5, though this is dependant on the location of new developed areas.

Table 3.8 Design/ Operational Velocities (m/s)

	Pipelines Type	Minimum (operational)	Maximum (Design/operational)	Remarks
		m/s	m/s	
1	Distribution Pipeline	0.3	1.2	Design velocity vary for pipeline material other than Cement Coated DI pipes
2	Secondary Pipeline	0.30	1.5	
3	Primary / Pressure Ring Pipeline	0.30	2	

3.3.5 Reservoir Sizing

Service reservoirs shall be designed to serve mainly water storage for operational balancing purposes and can be summarized as follows:

- To balance downstream diurnal variations in demand with relatively constant rates of inflow mainly during high demand flows,
- To balance pumped inflows and pumped outflows at forwarding stations;
- To provide contingency storage in the event of a failure in transmission upstream or during maintenance
- To provide damping effect so that small fluctuations are not reflected.

The Design standards include:

- Volume should be calculated based on Average Daily Demand (ADD) including fire reserve and the volume of storage so calculated shall be usable and exclusive of any unusable top or bottom water storage.
- All water storage facilities should have a minimum of two tanks, or one storage tank with minimum of two section or more that can be isolated, at each location;
- The volume of storage tanks at Distribution pumping station acting as forwarding station to other pump stations and to the network should be based on the Average Daily Demand (ADD) including fire reserve in addition to 10% of the design output to the pump station.
- All reservoirs should have a by-pass arrangement.

3.3.6 Water Network Configuration

- The water distribution network shall be designed preferably as loop/grid network without dead ends.
- In location where only single or few consumer connections exist, permanent blow-off arrangement (for flushing purposes) is required .
- The water supply system shall be preferably designed to provide two alternate sources to a sector or area. If an area have a single consumer (bulk), single feed line may be considered.

- Feed lines for a sector / area / zones shall have meters installed for demand management purposes
- The fire hydrant location must be in accordance with Fire Department requirements.

3.4 Al Ain Water Network

The trunk mains network and associated pumping stations from the Sweihan and the Remah Pumping station are operated by Transmission and Dispatch company (Transco), while Al Ain Distribution Company (AADC) operates Al Ain well fields and the associated pumping stations and distribution areas (Figure 1.1).

The Transco network starts upstream of the Sweihan and the Remah Pumping stations. The two main reception complexes in the city of Al Ain are the Remah and Al Ain Reception Pumping stations. From these pumping stations, water is supplied through a network of transmission mains to 15 pumping stations (Table 1.1). These pumping stations within the city directly supply the distribution networks and other pumping stations downstream, which in turn supply distribution networks. Along the trunk mains, there are various take-offs to the Al Maqam Palaces, tanker filling stations, bulk connections and directly fed distribution areas. The distribution networks operated by the Al Ain Distribution Company (AADC) are in the most part discrete districts. Within each of the district boundaries, a large number of valve operations are undertaken to ensure that all consumers receive an adequate daily supply. The water distribution network across Al Ain region is currently operating on an intermittent basis for the most part with most areas receiving water for a period approximately 6-8 hours, and only around 30 % of the city is supplied continuously.

Since, the above mentioned criteria have to be considered in designing any water network, the characteristics of Al Ain Distribution network is illustrated as per ADWEA specification, where the network has to be laid in a proper way to avoid any breakages and maintenance problems which may lead to a water quality troubles especially with the intermittent supply pattern. In this regard, normal standards to be followed after finalizing the design and getting the formal approvals to start the construction of the network. The following standards address the material and accessories specifications (ADWEA, 2003)

Table 3.9 Key data of Al Ain water system

Al Ain Region System Components	System Totals
Total Model Area Demand for 2005	133 mgd
Number of Properties	39,832
Population	404,732
Length of Mains	4,414km
Number of Districts	57
Number of Valves	11,327
Number of PRV's	21
Number of FCV's	26
Number of Reservoirs	85
Number of Pump Stations	35

3.4.1 Pipe Trenches

All types of pipes are laid in corridors allocated for water pipes in accordance with road cross section. The backfill on top of pipes is not less than 0.9 m to protect them from external pressures. Concrete blocks of suitable size support special pieces, valves and bends.

A bedding of 15-20 cm thick of soft soil or sand is placed under pipes in case of rocky soil, and in cases of backfilled areas, the backfill is removed and then replaced with sand refill on layers with thorough compaction to provide the required level.

3.4.2 Pipe Materials

- Ductile Iron Pipes (D.I)

Ductile Iron pipes (D.I) have proved to be more durable and can stand high external and internal pressures as well as tensile stress. Accordingly, D.I pipes are used with internal cement mortar lining and metallic zinc plus bituminous paint external coating.

- High Density Polyethylene Pipes (HDPE)

High Density Polyethylene Pipes are used for the house connection and as an alternative for main pipe with diameter less than or equal to 300 mm in case of shortage in DI pipes

3.4.3 Valves on Water Network

- Sluice and Gate Valves

The water network is provided with sluice and gate valves to control the flow of water in pipes and to ease the periodical maintenance of the network. All valves shall have a hand wheel or an operating switch. The valves are able to stand all the pressures in the distribution networks

- Washout Chambers

Washout chambers are installed on the main pipelines at the low points. The diameter of the wash branch pipe on which the wash valve is installed shall be according to ADWEA standard. The valve is erected inside a reinforced concrete chamber. Fire hydrants on the distribution network can act as wash valves for all smaller diameter pipes.

- Air Valves

Air valves are installed on the main pipelines at the high-level points. It is preferable to use the double ball valves, as the big ball allows air to escape from the pipe when being filled with water, and the smaller ball allows the escape of air collected at high-level points of pipes during operation. The air valve diameter shall be 0.2 of the main pipe diameter. A sluice valve is erected below the air valve on its pipe connection with main pipe to allow for its repair and maintenance. The air valve and its accessories are installed in an R.C. Chamber. House connection on the distribution network acts as air release valves for all smaller diameter pipes

- Fire Hydrants

Fire hydrants are spaced at intervals not more than 200 meters as per Civil Defense Regulations. The hydrants could be of the underground type or pillar type

3.5 Intermittent Water Supply And End Users Storages

3.5.1 Change in Supply Regime

Intermittent water supply may be defined as a piped water supply service that delivers water to users for less than 24 hours in 1 day. This section examines the consequences of intermittent water supply and evaluates its performance in real practice. It is anticipated that there will be a change in supply regime in 2007/ 2008 where Al Ain area will be supplied on a continuous basis as opposed to the current situation of intermittent supply.

When such change occurs, it is anticipated that customer wastage/usage will increase. Customer wastage/usage is defined as the unnecessary uncontrolled use of water beyond the customer meter or control valve. Customer wastage/usage should be considered as Authorised Consumption.

The intermittent supply data and 24 hours supply data were compared for the areas which were converted to continuous supply. The percentage increase in the supply for each area was 119%.

The most visible evidence of this increase was observed during the active leakage detection exercise done by AADC operation section where numerous customer tanks started to get more water quantities when the supply was changed from intermittent to 24 hour supply.

Hence, from the experience of AADC operation section, the advantage and disadvantage of the intermittent water supply pattern can be identified below as following:

Advantages

- Leakage of water is reduced.
- Available water is distributed equally.
- There is time for repairs and maintenance.

Disadvantages

- Systems do not operate as designed
- Reservoir capacities are underutilized.
- There is frequent wear and tear on valves.
- More manpower is needed.
- Water Contamination due to long residence in storage tanks in worm conditions.
- Contaminated water requires consumer to use the bottled water.
- Higher doses of chlorine are needed.

- Oversizing of networks is needed to supply the necessary quantities in a shorter time.
- Consumers have to pay more for storage and pumping
- Water meters malfunction, which can lead to a loss of revenue and customer disputes
- Accountability per subzone is not provided.
- In case of fire, immediate supply is unavailable.

3.5.2 Requirements of 24-Hour Supply Change

To move from an intermittent to a 24-hour supply, it must be accepted that governance and tariffs are at the core of the problem, and those issues must be addressed first. Then it will be necessary to embark on extensive stakeholder awareness programs to convince people that 24-hour access to piped water in the home is possible for all. For this type of service to become standard, moratoriums must be imposed on new connections while distribution systems are being hydraulically improved. This is best done by starting with 24-hour supply zones and gradually expanding them. Higher tariffs can be imposed on those with 24-hour supply, and the extra funds can be used to improve the system. When tariffs are sufficiently high, there will also be less water used in 24-hour zones, making more water available for use when these zones are extended. Twenty-four-hour zones must be 100% metered, and meters must be accurate (if they are found to be inaccurate, they must be replaced). District metering can be installed to pursue non revenue water (NRW) goals, and full computerization of accounts in 24-hour zones should be accomplished. Illegal connections must be pursued in the field as this is a fundamental governance issue.

There is a high prevalence of intermittent supply in Al Ain region where some problems can be associated with the intermittent supply pattern as clarified below:

- Intermittent supply is caused by extending distribution networks beyond their hydraulic capacities.
- Low tariffs and poor collection contribute to intermittent supply.
- Compared with 24-hour supply, intermittent supply uses more water, if the supply wasn't equally distributed among the customers.
- Intermittent supply leads to higher costs and great inconvenience for consumers and utilities.
- Expectations of consumers (due to a lack of awareness) are low.

The expected solutions for the above problems are as following

- Promote awareness among stakeholders.
- Address governance issues related to the autonomy of utilities.
- Introduce higher tariffs for 24-hour zones.
- Place moratoriums on new connections.
- Invest in hydraulic modification of distribution systems.
- Start with 24-hour zones, and then expand these.
- Enforce strict metering and collection.
- Reduce NRW.

3.5.3 The Costs (CAPEX & OPEX) to the Service Provider

An intermittent system needs to supply the same quantity of water in fewer hours. As a result, pipes in the primary distribution system have to be of a larger diameter to deliver a higher peak flow and maintain the necessary distribution pressure. Hydraulic calculations study done for Abu Dhabi water distribution system (UMS report, 2003) demonstrates that the cost of the primary distribution system to provide service for 6 hours a day is about 35% higher than for 24 hours service. The study found cost savings ranging from 12 and 39 percent for distribution systems under continuous supply.

Rationed supplies often lead to building norms that require private storage tanks or else households install them on their own. Small, individual storage tanks are much costlier than large, public storage reservoirs of the same aggregate volume since the economies of scale are very large. In addition, large storage reservoirs make the operations of the water distribution system easier. In case of assessing the situation for Al Ain city customers, while they are getting their demands, more quantities delivered to their storage tanks due to the pressure available in the distribution system.

A higher incidence of pipe bursts has been reported, as a result of pressure changes when valves are opened or closed to allocate water to rationed consumers. Pipe bursts increase both maintenance and water losses. In Al Ain for example, the number of pipe leaks are 21 times higher than in well operated 24-hour water services (Thames Water report, 2004).

Utilities also need additional manpower to manage the closing and opening of valves to rotate rationing between different areas of the city, a multi-regression analysis showed that the number of staff per thousand water connections increased in inverse proportion to the hours of service. If the company provides 6-hour service, then it needs 25 percent more staff than one providing 24-hour service (UMS report, 2003)

Intermittent and poor services also frustrate consumers who may feel morally justified in connecting illegally to the water supply system, and the utility loses revenue as a result. The revenue lost was estimated at 170% of the revenue collected by the company (AADC Annual Sales report, 2003)

3.5.4 The Costs of Unsafe Water

Several studies have documented that those who understand the health hazard of unsafe water quality, regardless of family income level, have a higher willingness to pay than those who do not. The willingness to pay make them pay for water filter systems, for the costs of boiling water or for bottled water.

Intermittent supplies are prone to contamination in distribution pipes that are often under no or negative pressure.

The bacteriological quality of an intermittent water supply is substantially lower than of a continuous service. In four districts in Al Ain city between 27% and 76 % of samples tested positive for fecal coliforms under intermittent supply versus only 10% of the samples under continuous water supply (AADC Laboratory report, 2005).

Due to the high temperature in the Gulf, in-house storage tanks in an intermittent supply also risk bacteriological deterioration of water. For example, extensive tests in four sections of the city in 2003 indicated that 96% of the samples from the water supply network tested negative for coliforms in contrast to 76% of the samples from consumer storage tanks.

3.5.5 Demand Management

As was mentioned before; excess customer demand will increase once the 24 hours flow regime for Al Ain region commences in 2007/ 2008.

Unobserved wastage/ usage is likely to include the following.

1 Tank overflows
2 Increased uncontrolled landscape irrigation
3 Increased controlled landscape irrigation
4 Changes in population behaviour
5 Accurate and reliable metering will have a key role in the management of demand.

Present metering arrangements are not adequate for the accurate analysis of consumption and leakage and the implementation of metering proposals will be essential in order to obtain reliable data on which actual demand can be monitored. Installing meters on all consumers will enable the early identification of excessive users and will permit the AADC to adopt the correct strategy for reducing excess wastage.

The level of increase will be dependant on the degree to which demand is controlled. Methods of demand management can be considered in terms of four key influences illustrated in Figure 3.4.

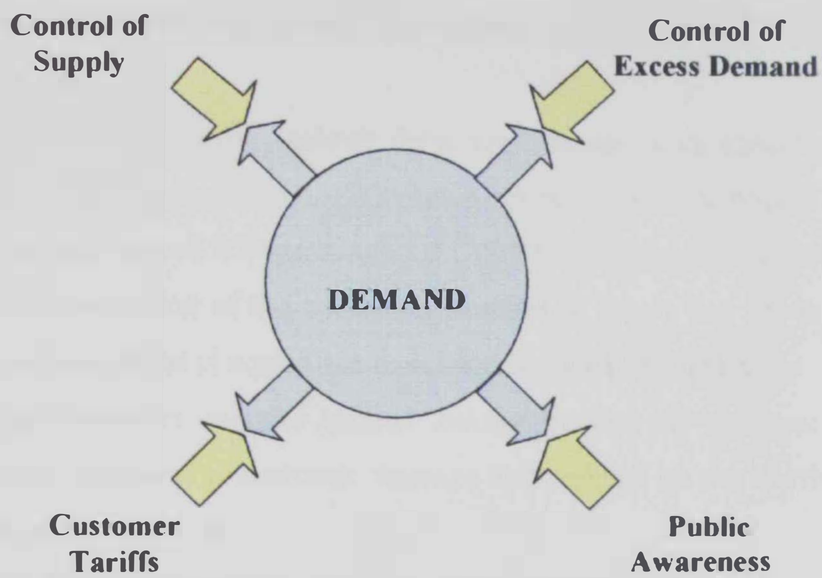


Figure 3.4 Key influences on Demand

These influences and their methods of application are described in more detail within the following sections.

3.5.6 Active Leakage Detection

A Leakage Detection survey was completed by the operation section for the areas getting 24 hours supply. The purpose was to assess:

- The level of leaks/leakage that could be identified in each zone;
- Assess the breakout of new leaks – in particular the reoccurring burst frequency.

The methodology involved an intensive leak detection exercise in two separate "sweeps" of the pilot areas using visual detection, sounding, and correlation methods.
The essential steps during the active leakage detection were as follows:

- Visual Inspection – walking route of main;
- Location of buried valve boxes using metal detector;
- Sound all fittings with listening stick;
- Trace out mains using mains tracing equipment (where necessary);
- Correlation of all mains (fitting to fitting) using accelerometer or hydrophones;
- Investigate areas of interest using ground microphone or listening stick;
- Mark area for repair

The results from the active leakage detection exercise were considered in terms of Equivalent Service Pipe Bursts (ESPB). An average service pipe burst flow at a 40m pressure is equivalent to a flow rate of 359 gallons/hour (AADC Leakage report, 2004). The ESPB gives an immediate indication of the maximum number of bursts that are to be looked for, recognising that a mains burst is equivalent to several service pipe bursts.
The active leakage detection exercise located leakage beyond the consumer control valve and/or meter which is known as customer wastage and leakage on the distribution network which is considered as real losses.

During the active leakage detection, the observed real losses found on the distribution network equates to a 3.1% on average, of the volume of water supplied. The customer side wastage observed during the active leakage detection equates to 4.3%, on average, of the water supplied. The Customer side Wastage above includes tank overflows during the time of inspection only. The volume of losses is based on an estimate by the leakage inspector undertaking the active leakage detection.

3.5.7 Non Revenue Water

Leakage detection exercises carried out observed a relatively small amount of real losses (3.1% of total inflow). The remaining volume of NRW must have an alternative destination. Illegal use is sometimes there and operational uses tend to be of very small volumes, therefore, a large proportion of the NRW will actually be consumed by the customers

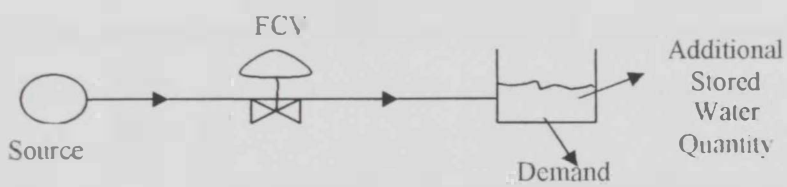
On average 13% of NRW has been calculated from data derived from measurement under a permanent supply regime (AADC Leakage report, 2004). A similar comparison must be made for calculating the figure for the entire Al Ain region to ensure consistency. It is not practical or possible to either test the entire region or implement a short-term permanent supply regime to this end. Therefore, the NRW value has been calculated using existing Al Ain regional data.

3.6 Control The Water Supply

3.6.1 Intermittent Supply Regime

The physical control of supply relies on limiting the provision of water to the customer. This system of operation is currently being employed in Al Ain to control the quantity of water consumed.

Customers are able to satisfy their requirements for water outside of the supply period by storing water in ground tanks. These tanks typically provide one to two days storage. It is reported that some customers have also installed suction pumps to obtain more water from the network during periods of supply. By providing sufficient storage, customers are able to satisfy their water needs and the impact of intermittent supply on demand in the long term is reduced as illustrated herein.



Ginen the drawbacks of intermittent supply discussed before, the preferred strategy for the AADC is to move to permanent 24 hour supply to all areas within the Al Ain region

3.6.2 Supply Control Devices

Devices can be fitted to supply to customers to either restrict or stop flow. The simplest cut-off device is a valve which can be operated by the water provider to temporarily prevent further flow if a customer fails to pay their bill. In the case of Al Ain, cutting off supply could be prompted in response to unusually high consumption.

One of the ideas to limit the consumption is the pre-paid water meters which are also available and require consumers to pay for water before consumption by purchasing a prepaid card. Consumers can then draw water from the meter by inserting the prepaid card into the meter. As service is delivered, the balance is adjusted, and the remaining credit displayed. These devices tend to be used for the provision of water via standpipes where domestic connections are limited.

The maximum flow available to a customer can also be limited in a number of ways, the simplest and most cost effective being through the provision of a restriction in the supply, such as an orifice plate. This approach would have benefits in Al Ain as flows to tanks could be restricted which would result in an attenuation of the peak flow and so help to reduce the maximum load on the system.

Whatever system is considered, the provision of equipment must ensure that the customer is unable to tamper with or remove the device, therefore location and security will be important considerations.

3.7 Control The Excess Demand (Consumption)

3.7.1 Installation of Ball Float Valves

Almost all Shabia and Villa type dwellings within Al Ain have there own galvanised (or similar) ground tank with sufficient storage to last 24 to 48 hour period between water within the given area being turned on. Tank sizes also vary in size with the smallest being 3000 gallons in the low cost housing areas and the largest of 7000 gallons for the villas. It is common for the villas to have two or more tanks – for domestic use, and for irrigating the garden.

The majority of customer tank inlets are not fitted with float valves and therefore overflow to waste during pumping times. This appears to be a major source of water wastage in Al Ain region.

Regulations have recently been introduced which require all new ground storage tanks to be fitted with ball valves. It is recommended that, for control to be effective, ball valves are also fitted to all existing ground storage tanks.

As Al Ain water supply system moves to a continuous supply, the effect of overflowing ground tanks will only serve more water losses. In the short to medium term it is essential that customer demand is reduced by ensuring that all customer connections are equipped with effective inlet control to the customers ground storage tank i.e. ball float valves. There is likely to be a great deal of resistance to this policy initially and it will need to be implemented with a Public Awareness programme, if the customers storage tanks will not be removed.

3.7.2 Flow Limiting Devices

Whilst the implementation of physical control to prevent wastage at ground storage tanks will help to offset this increase, there is much potential for investigation of innovative approaches to reducing domestic consumption.

Excess irrigation has potential to be an area of significant wastage, particularly in domestic properties where there is likely to be limited control of application rates. Providing clock operated valves to domestic irrigation system can provide significant savings in over-watering; such a scheme is currently being piloted in Bahrain whereby customers with high demands are being offered installation of time switching devices, which are set by the Water Distribution Department. The timed irrigation regime is carefully designed for each dwelling, based on a survey of irrigation requirements, and the owners of the property must rely on the WDD to make any changes required. The system has significantly reduced over-consumption for many households, resulting in more water being available for other consumers and providing savings on water charges to the householder.

3.7.3 Public Awareness

The implementation of any Demand Management programme does not have to impose restrictions on its customers, or compromise health and hygiene. However, it does require recognition by the water utility and its customers that water is a scarce resource, and efforts on both sides to reduce waste, misuse and undue consumption.

One of the main barriers to the introduction of demand management policies is the customers' attitude, consequently user education provides a key role in demand management programmes. This will be essential for the policy of installing ball float valves, leakage management progress and changing to a continuous supply.

Achieving the necessary level of understanding requires different approaches according to the organisation but will include, Internal seminars; Literature drops; Group Meetings; Presentations and discussions; Television, radio and press – advertising and education;

3.7.4 Customer Tariffs

Most countries have some form of charging structure or household metering for water used. In the case of Al Ain region, there is no charge for UAE Nationals and a fixed monthly charge for non Nationals.

In many instances, the very low or flat rate tariffs are subsidised or even free (frequently in the interests of low income customers) and often become the expected norm by many customers. However there are several disadvantages to a low or zero rated tariff structure and not charging the economic rate for water:

- It does not encourage sensible sustainable use;
- It does not encourage the repair of customer tanks leaks or overflow
- With little or no revenue there is no incentive to install active customer metering and replacement;
- Insufficient revenue is generated to provide a sustainable operation, maintenance and repair programme;
- Capital spend is higher due to increased demand.

Correcting the metering policy and tariff structure, in conjunction with other demand management and water conservation policies, has a major impact on reducing customer demand.

3.7.5 Demand Management Targets

Some areas in Al Ain region have demonstrated that the majority of the increase in demand following the introduction of 24 hour supply is likely to be due to over-irrigation and losses through over spilling ground tanks.

The preparation of the demand forecast has assumed that demand management will have a gradual impact on curbing excess demand, with 10% reduction in 2005 and 50% reduction by 2010. A break down of demand management for the main components of additional demand is presented in Table 3.4.

3.7.6 Network Zoning and Metering

In addition to the options proposed, there will be a need to split distribution network of the entire city into smaller zones to assist in its management and operation. It is proposed that each pumping zone would be made up of smaller discrete areas. Each area would have a meter installed to allow effective monitoring over 24 hours per day, 365 days per year. It is envisaged that the boundaries for these zones would be similar to postal sector boundaries where hydraulics permit

District Meter Areas (DMA's) need to be established across the Al Ain region to allow effective operational management of the water distribution network and to assist in the following

- Leakage Management – DMA's assist the operator to manage leakage effectively through prioritization of DMA's leading to water savings and reduced operational costs
- Mains rehabilitation – Mains Rehabilitation planning can be developed at the DMA level if discrete areas are established which allow the collation of flow, complaint and repair data.

To ensure that the two activities above are successful, an international specification for the establishment of the DMA's can be adhered, using the following criteria where possible:

- Single input into the DMA monitored by a mechanical or electromagnetic meter of appropriate size
- The installed meter should be loggable to allow collection and recording of data for downloading to a computer to allow analysis of minimum, average and maximum flows.
- Permanently "valved-in" to maintain discrete areas without causing poor pressure problems or customer complaints.
- Where possible, use natural boundaries such as wadi's, major highways, etc as natural end points for DMA's to ensure secure and discrete boundaries
- Where possible, DMA's should consist of the same age mains to assist in mains rehabilitation planning
- Ground levels should be relatively consistent across the DMA's to allow effective pressure management
- Pressure reduction valves should be installed where pressures are excessive and where possible near to the inflow meter to help simplify leakage calculations

A pragmatic approach can be taken to assign the district meter (DM) location, encompassed in the following considerations:

- Central DM location – a central supply point has been adopted to promote an efficient distribution of water, where distribution network is dense and sufficient to permit.

- Supply point selection – assessment of the key pipework within the DMA that will carry the majority of flow in the DMA to understand from which direction the DMA should be supplied
- Orientation to the supply main – minimisation of the distance travelled to the DM location to avoid unnecessary headloss in the supply main

The Critical Point (CP) within the DMA is defined as the point at which the lowest pressure will be experienced. Several factors, such as the elevation and the layout of the pipework affect the location of the CP. It is necessary to understand the CP within each DMA to correctly set levels of service, allow appropriate design of PRVs, where needed, and aid operation of the system.

In the case of Al Ain, the most influential factor is the elevation within the DMA. The highest elevation within the DMA, frequently confers the CP.

3.7.7 Pressure Reducing Valve (PRV) Design

Pressure management within a DMA is one method of reducing levels of leakage in the distribution network, since leakage rates from bursts are related to pressure. Thus, by reducing pressure, water losses are also reduced. Pressure Reducing Valves (PRVs) can also installed at the inlet of the DMA to protect aging or delicate pipe work.

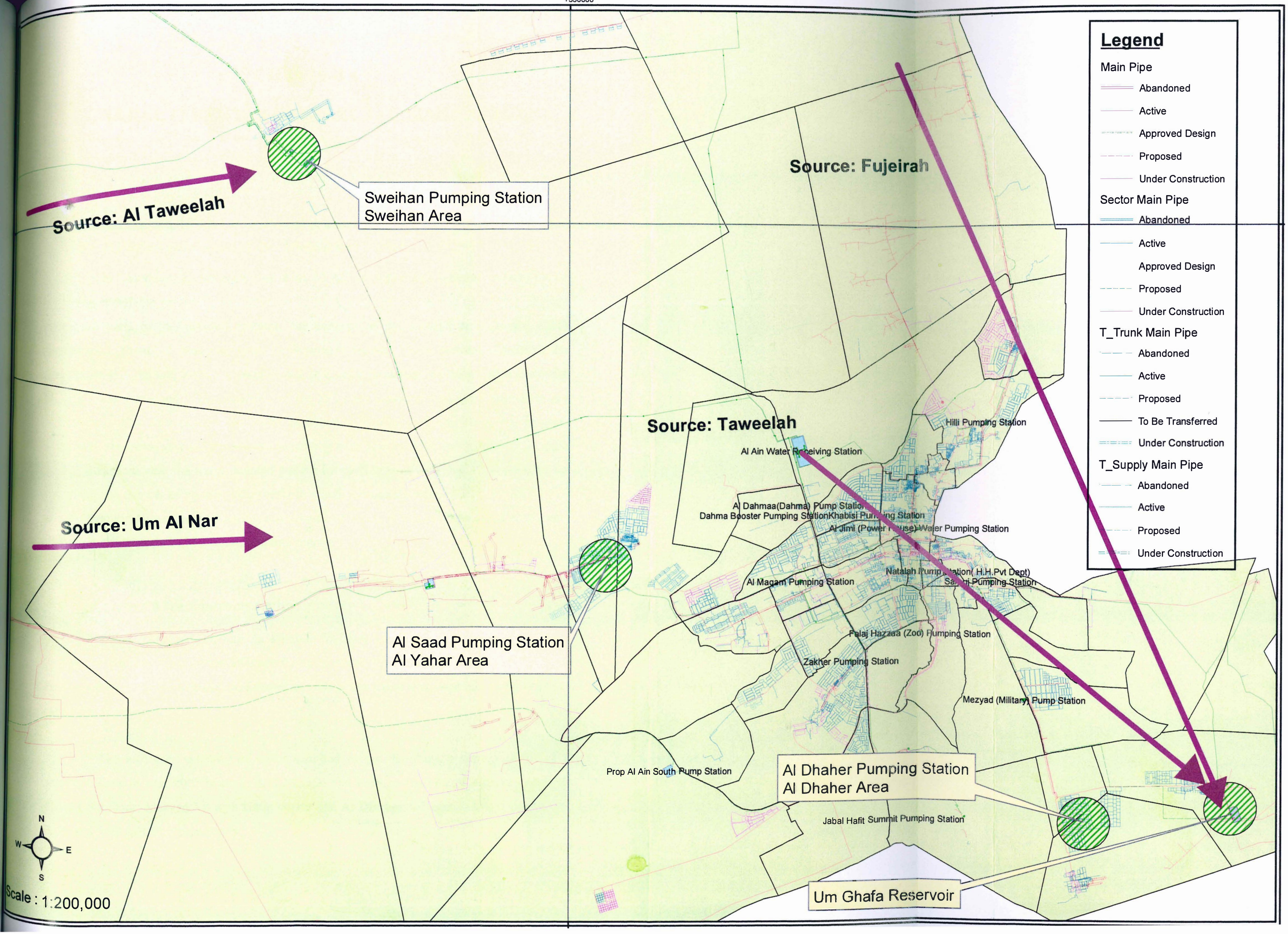
PRVs have been recommended for DMAs where pressure at the CP exceeds 10m under peak hour, peak month demands. This design will maintain a pressure of at least 10m across the entire DMA.

3.8 Selected Water Distribution Systems

As mentioned in the objectives of this thesis, the simulation will be carried out for a portion of Al Ain water system. The areas selected for this purpose are Al Dhaher, Al Yahar and Swiehan. The main reasons behind selecting these areas are:

1. Each area consists of low cost houses and villas which is considered an advantage in finding/ analyzing different water demand patterns and different storage tanks capacities.
2. All selected areas are equipped with FCV's that can define exactly the water consumption for each area (DMA). Also data loggers installed for 1 complete year (2005) are available and provide the pressure readings for some specific points to be compared with the regulations specified in the water distribution code.
3. Each area receiving its water from one particular source. So, the water quality simulation in each area will be assessing the quality variation of water originated from that particular source. For example, Al Yahar area is fed from Um Al Nar 1000 mm line (refer to the introduction) as a first location in Al Ain region receiving this source. Swiehan area is fed from Tawillah 1200 mm line as a first location in Al Ain region receiving this source. Al Dhaher area is the last location in Al Ain region which is supplied from two sources (Tawillah desalination plant via Al Ain reception pumping station and Fujirah desalination plant via the existing 600 mm line).

The following GIS sketch illustrates the locations of the three selected areas.



Legend
Main Pipe
Abandoned
Active
Approved Design
Proposed
Under Construction
Sector Main Pipe
Abandoned
Active
Approved Design
Proposed
Under Construction
T_Trunk Main Pipe
Abandoned
Active
Proposed
To Be Transferred
Under Construction
T_Supply Main Pipe
Abandoned
Active
Proposed
Under Construction
Source: Al Taweelah
Sweihan Pumping Station
Sweihan Area
Source: Fujeirah
Source: Taweelah
Source: Um Al Nar
Al Ain Water Receiving Station
Hilli Pumping Station
Dahma Booster Pumping Station
Al Maqam Pumping Station
Falaj Hazzaa (Zoo) Pumping Station
Zakher Pumping Station
Mezyad (Military) Pump Station
Al Saad Pumping Station
Al Yahar Area
Prop Al Ain South Pump Station
Al Dhaher Pumping Station
Al Dhaher Area
Jabal Hafit Summit Pumping Station
Um Ghafa Reservoir
N
W
E
S
Scale : 1:200,000

CHAPTER 4

CHARACTERISTICS OF SELECTED SIMULATION SYSTEMS

In this Chapter, the three areas selected to assess their water distribution systems from hydraulic and water quality points of view are described in details.

Data was collected for each area from Al Ain Distribution Company responsible of providing a reliable service to its customers in Al Ain region. Since the customers' storage tanks are parts of the selected systems, the concept of the internal storage tank (IST) represents a group of customer tanks is illustrated herein. Such concept simplifies the simulation process and avoids a complicated hydraulic modeling utilizing certain relevant criteria as is explained. The numbers of the ISTs are assumed and their capacities are calculated. The data reports the water distribution system characteristics; i.e. lengths, diameters, the elevations and the sources supplying each system. The water demand for each area is presented with the water demand pattern for each type of property. The information regarding the number of houses, villas and other bulk consumers were collected from Al Ain Town Planning Department. Field measurements of flows, pressure, residual chlorine, and THM are also reported and evaluated in this chapter

4.1 Al Dhaher Area Characteristics

Mazyad, Umm Ghafa, and **Al Dhaher** Villages and Coca Cola plant are located within the pressure Zone of the Military Pump Station. New distribution network is being developed this year for Mazyad, Al Dhaher and the labors camp areas that belong to different industrial companies,.

The estimated quantity of water supplied to the distribution system of Military pressure Zone in year 2005 is about 5.77 MIGD. The average quantity of water supplied to Coca Cola plant, Mazyad Village, Umm Ghafa and **Al Dhaher** Villages is about 0.30, 1.42, 3.08 and 0.98 MIGD; respectively.

Since the water production from the well field was considerably reduced in year 2005, the Villages of Mazyad, **Al Dhaher** and Umm Ghafa were all supplied by desalinated water from Military reservoir. In order to cater for the additional flow requirement, a pipeline of DN600 mm from Military reservoir to **Al Dhaher** Junction is now supplying Umm Ghafa and Al Dhaher Villages. So, **Al Dhaher** Village distribution network is lately supplied directly from the Military reservoir. New DN400 mm pipeline branched off from the DN600 mm pipeline to supply this Village was installed.

The Second Source is 600 mm line from Um Ghafa reservoir which has two sources from Fujairah transmission line and Tawillah desalination plant via Al Ain reception pumping station. This line supplies directly the elevated portion of Al Dhaher area as it has enough flow and pressure.

4.1.1 Estimate Water Demands & Demand pattern

In chapter 3, the demand forecast for the whole region was discussed. The general concept of estimating the demand for the whole region is illustrated herein. The domestic demand for shabia's and villas are 1500 gal/day and 3500 gal/day respectively. The per capita consumption estimated is 100 gal/capita/day (ADWEC annual report, 2005). Al Ain town planning department estimated that an average of 10 people live in each house or villa, so the total demand for the in house use is 1000 gal/day/each property. The out house uses is the difference between the total demand and the in house demand i.e. 500 gal/day and 2500 gal/day for shabia and villa; respectively. As an illustration to the mentioned Figures, the area of each property is playing a major role, since the dimension set by Town Planning Department (TPD) for the house (Shabia) = 25 x 25 m and for the villa = 60 x 60 m. This explains the need of more quantity which is used for the irrigation purposes for villas. For the multi story buildings, the demand for each apartment is less than that for shabia and villa, since no water demand is needed for the irrigation purposes. Similar per capita consumption is used for the people living in each apartment. The above estimates of demands apply for other areas in Al Ain region as well, in particular Al Yahar and Swiehan subsystems. Two types of demands exist in Al Dhaher, domestic and farm. Both types are described in details below. The farm demand is 100,000 gal/day.

Domestic Demand Pattern

As mentioned in the water distribution code in chapter 3, the demand pattern is usually defined as per the population of each area. The approximate population for Al Dhaher area is 1500 (TPD census, 2004), so it is within the range mentioned in section 3.4. The number of houses and villas are 100, 24 units respectively. Figure 4.1 presents the demand pattern for Al Dhaher area.

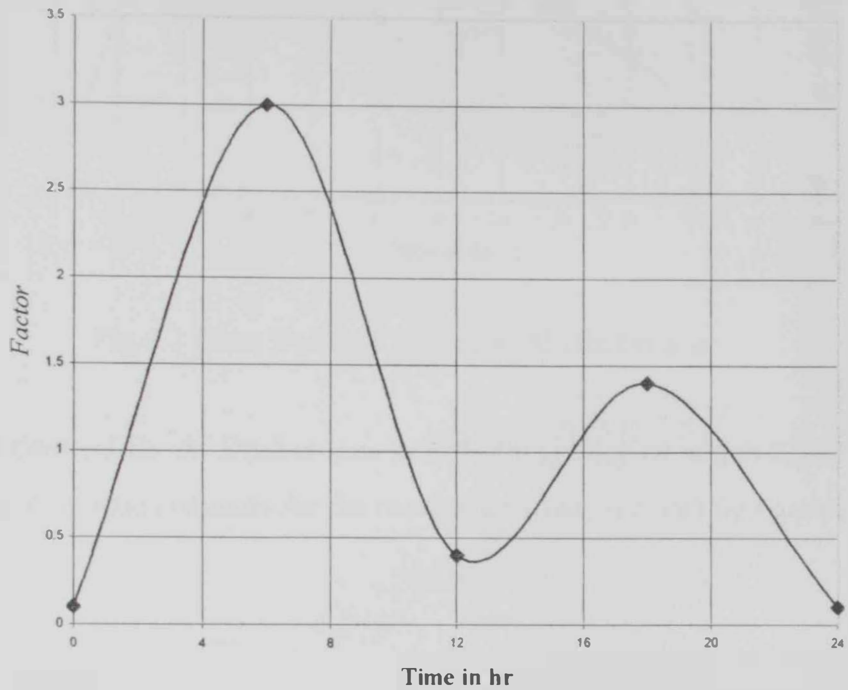


Fig 4.1 Domestic Demand Pattern for Al Dhaher Area

Farm Demand Pattern

The Farm demand pattern is agreed upon by both the AADC operation section and the applicant (consumer). The demand pattern reflecting only one peak during 48 hrs. The peak flow is expected to occur at 12 am as per AADC supply policies. The reason behind this phenomenon is that the applicant is requesting high demand during the day period. It is obvious that AADC operation section removes the conflict between the peak flow requested by different customers; i.e. the peak to the residential areas occurs two times per day at 6 am and 6 pm. On the other hand, the peak for the farm occurs at 12 am once every two days

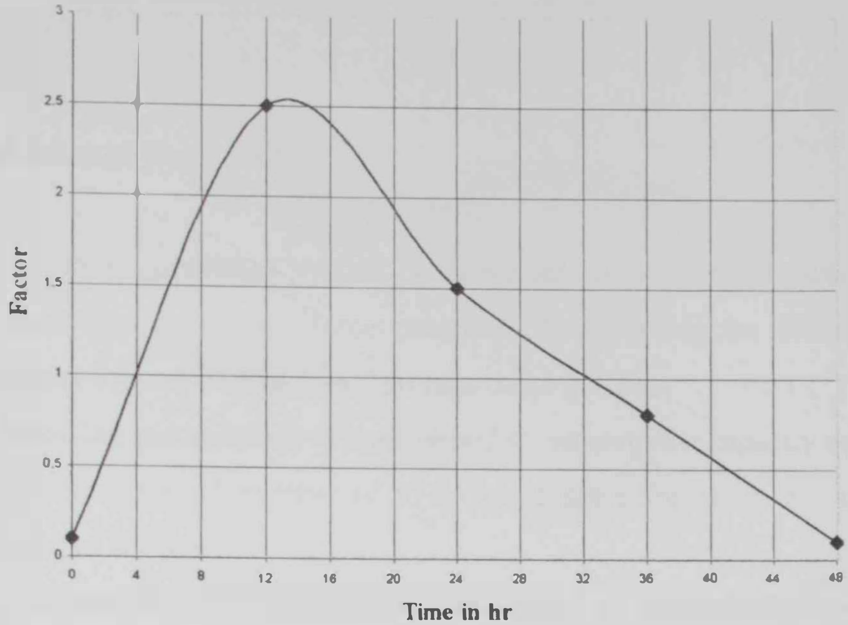


Fig 4.2 Farm Demand Pattern at Al Dhaher area

Hence, the total demand for Al Dhaher area is 334,000 gal/day of which 234,000 gal/day is used to fulfill the domestic demands for the residential areas, and 100,000 gal/day is the farm demand.

4.1.2 Supply Pattern

For Al Dhaher area, there are two sources of water as mentioned supplying an average flow is 0.96 MIGD. Two flow control valves are installed on each feed main, adjusted to supply a constant flow for the whole day. The flows are listed in Table 4.1 for Shabia's, villas and other bulk consumers.

Table 4.1 Supply pattern for Al Dhaher area

Consumers	First day MGD	Second day MGD
Shabia's + Farm	0.75	0.625
Villas	0.294	0.252
Total	1.044	0.877

It iS clear from the Table that the supply is 3 times the estimate demand for each type of properties

4.1.3 Internal Storage Tank (IST)

For the three simulated system, a skeletonization approach was followed to determine the water flow stored in different properties, by assuming one internal storage tank (IST) that represents a group of customers' storage tanks (CSTs).

The following methodology is approached to estimate the capacity and demand for each IST as well as the total number of ISTs , as these information are utilized in the hydraulic modeling of this area.

- The ISTs capacities and demands are calculated based on real existing customer storage tanks in each particular portion of the network,
- Each IST represents a group of real (CSTs) having their similar ground elevation and is supplies from the same feed main.
- The number of the ISTs are calculated based on the total number of properties since each property is considered to have one storage tank. This approach is followed to simplify the hydraulic modeling for the whole network and to easily obtain reasonable results representing the network behavior in the real life.
- Each bulk consumer tank (i.e. Farm, Palace , industrial area , Tanker Filling Station and Commercial building) is represented by one IST with the same physical capacity .

Generally, in flat area such as swiehan, the distributions of ISTs depend on the number of houses and villas to facilitate the hydraulic modeling process. On the other hand in the areas where the elevation is changed between the source and the last point in the network, the ISTs number is estimated based on the number of houses and villas which have the same elevation. The mentioned points described the way of selecting the ISTs. In addition to that, the real house connections are represented by assuming a 100 mm line with a check valve, connecting each IST to the network, this issue is discussed in chapter 5 in details. The main reason behind this is that, too many sizes of house connections found in the real life ranging from 3/4 – 1 inch for Shabia and 1 - 2 inches for villa. The exact connection size decided by AADC for each type of property depends on the number of family members (AADC house connections policy, 2004). The average length of these house connections as per ADWEA specifications is 20 m from the feed main upto the customer storage tank the assumed house connection to each IST has a length of 20 m

ISTs For Shabia

- No. of Shabia is 100 (low cost houses)

✓ The total demand is around 150,000 gal /day.

✓ Assumption has been made that 1 IST is representing 5 storage tanks in the customers houses (Shabia's)

✓ As per the experience of AADC, each house has 1 or more storage tanks with a total capacity = 3000 gal, then the total capacity for each one IST = 15000 gal

✓ The base demand for each IST = 7500 gal/day since it has the same demand of 5 houses.

✓ Based on the capacity of the IST, the diameter has been calculated assuming the max level = 5 m (As shown in the Table 4.3)

ISTs For Villas

- No. of Villas is 24 unit

✓ The total demand based on what is discussed above is around 84,000 gal /day

✓ Assumption has been made that 1 IST is representing each 3 storage tanks in the customers houses (Villa's)

✓ As per the advice of AADC operation section, each house has 1 or more storage tanks with a total capacity = 7000 gal, then the total capacity for each IST = 21000 gal

✓ So, the base demand for each IST = 10500 gal/day

✓ Based on the capacity of the IST, the diameter has been calculated assuming the max level = 5 m (As shown in the Table 4.3)

Farm IST

✓ Agricultural farm is supplied from the 400 mm Distribution main

✓ The IST capacity is = 200,000 gal

✓ The required demand = 100000 gal/day (As per the application submitted to AADC)

✓ Based on the capacity of the IST, the calculated diameter is 10.8 m, assuming the max level = 10 m.

Table 4.2 ISTs Capacities and Demands for Al Dhaher area

Property Type	No. of Properties	Demand gal/day /Property	Total Demand gal/day	Storage Tank capacity gal/ Property	Assumed no. of Properties / IST	No. of ISTs [1]	IST capacity (gal) [2]	IST demand (gal/day) [3]
Shabia's	100	1500	150000	3000	5	20	15000	7500
Villa's	24	3500	84000	7000	3	8	21000	10500
Farms	1	100000	100000	200000	1	1	200000	100000

1 No. of ISTs = No. of properties/No. of properties/IST
2 IST capacity = Storage tank capacity * No. of ISTs
3 IST demand = Total demand/No. of ISTs

More details about the 29th ISTs are described in details in appendix B.

4.1.4 Pipe Network

The following Table describes Al Dhaher network

Table 4.3 Pipe Network Description for Al Dhaher area

Pipe diameter	Pipe lengths in m
100.0 mm	27,140
150.0 mm	38,381
200.0 mm	10,115
300.0 mm	12,000
400.0 mm	3,260
600.0 mm	2,239
Total Length	93,135

Al Dhaher pipe network (Figure 4.3) has 192 pipes, 119 junctions and 29 tanks, the elevation of the junctions is shown in appendix B. Table 4.4 shows that around 70% of the pipes in the area are 100mm and 150mm The remaining 30% represents the other pipe sizes. The house connections are not a part of this Table. The bulk connection feeding the farm is branched form the 600 mm line, since this line has enough pressure, quantity of water, and in the same time it is considered the nearest line from the farm tank. The 100 mm lines are supplying the Shabia's since the house connections sizes for this type of properties are in the range of 0 5 – 1 inch (AADC house connections policy, 2004) where it is easy from operation point of view to be installed on the 100 mm line. The 150 mm lines are supplying the villas since it is easy to install the house connections sizes range from 1 – 2 inches. As per the water distribution code explained in chapter 3, the pipe lines ranged from 100 mm – 200 mm are considered distribution lines. The lines ranging from 300 mm – 400 mm are considered the secondary lines and the 600 mm line is considered the primary line. More details about the pipes are attached in appendix A. The elevations of the network vary from minimum of 295 m to maximum of 340 m above see level. The average elevation is around 303 m. Elevation of the two sources is 345 m above the see level as they both supply the area by gravity.

4.1.5 Field measurements

The following measurements are collected from AADC operation section. For the pressure measurement, only one data logger is installed in the black circle showed in Figure 4.3 which represents the lowest location for the network. The unavailability of another data logger or pressure gauge in the network restricts the detailed analysis. However, readings obtained for 1 year from that data logger are adequate to assess the performance of the current distribution network according to the water distribution code presented in chapter 3. Moreover, this data is also utilized to calibrate the hydraulic modelling for this network.

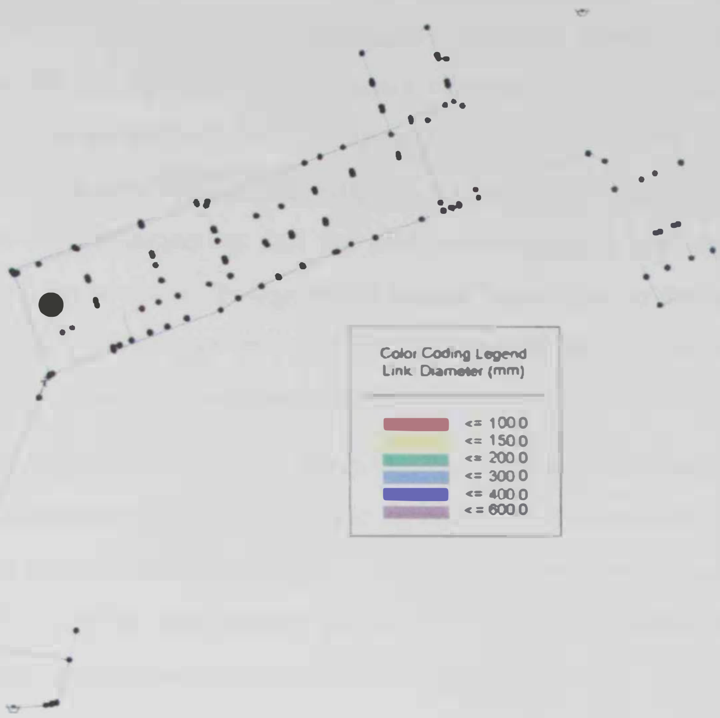


Figure 4.3 The location of the data logger at Al Dhaher area Network.

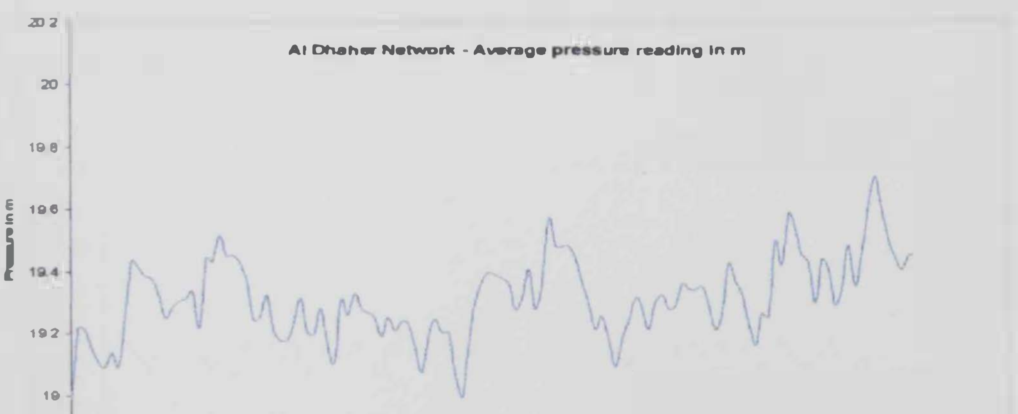


Figure 4.4 Pressure reading at the monitoring point of Al Dhaher Network.

In Figure 4.4, the average pressure is around 19.3 m. Comparing this with the residual pressure requirements as per the water distribution code, this average is found higher than the minimum limit of 10 m. This can be explained by the high source elevation above the network (more than 40 m). Since the head losses between the source and the measuring point can not reach 20 m for the estimated demands, it is perceived that such estimates are far below the actual consumption. This is supported by the fact that the estimated demands are much less than the supply indicating that the actual consumption approaches the current supply. The effect of the customer storage tanks is clear here since no demand management is practiced and the customer storage tanks are always supplied with surplus water that allow, the customers withdraw than their assigned demand.

The above graph (Figure 4.4) also shows that the reading was somewhat stable in that period. The seasonal consumption is not evident especially in the summer as it was expected due to the prevailing harsh weather conditions. Since most of the customers leave the area to spend their vacations outside the country during the summer seasons, the consumption remains almost unchanged and utilized mainly for irrigation purposes.

Residual chlorine measurement is also collected from the same point in Figure 4.3 to examine the current quality performance as well as to calibrate the chlorine simulation (Figure 4.5). An average reading of 0.37 mg/l is found within the international range of 0.2 – 0.5 mg/l as approved by AADC, Laboratory department. This indicates that at the monitoring point there is no problem with the water quality and it seems that the chlorine dose is acceptable.

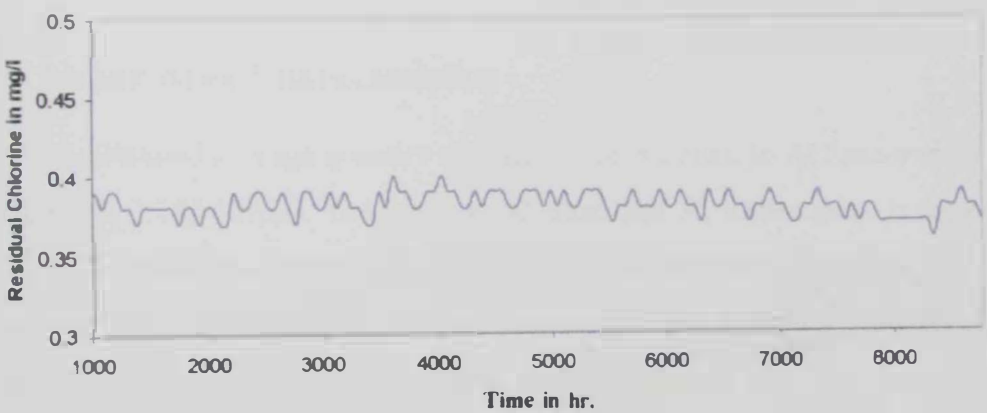


Figure 4.5 Residual Chlorine reading for Al Dhaher area for 1 year

In addition to the chlorine measurements recorded by AADC, Trihalomethane measurements were conducted over one day in the summer time to capture the worst condition of formed TTHM. Five samples were collected from the same monitoring point used in pressure and chlorine measurements. Five to six hours separated the sampling time of each two measurements. Table 4.4 lists the analysis results of there measurements for Al Dhaher area. It is noticed that Bromoform represents more than 90% of the THM compounds in all samples. Such finding is expected due to the high bromide present in sea water used to produced the supplied drinking water.

Table 4.4 TTHM field measurements for Al Dhaher area

Sample ID	Result (μg/l)				TTHM (μg/l)
	Chloroform	Dichlorobromomethane	Dibromochloromethane	Bromoform	
Sample 1	ND	0.9	3.6	31.9	36.41
Sample 2	ND	0.5	3.4	29.3	33.19
Sample 3	ND	0.7	3.2	27.2	31.04
Sample 4	ND	0.8	3.5	31.3	35.51
Sample 5	ND	0.4	3.8	32.8	36.92
Average	ND	0.64	3.48	30.50	34.61

Comparing the above figure with the international limits of 80 ppb for TTHM, one can find that TTHM levels in all areas are within the acceptable limits.

4.2 Al Yahar Area Characteristics

The estimated average quantity of water to be supplied to Al Saad and Al Yahar areas in year 2005 is 3.185 MIGD. The water to Al Saad and Al Yahar areas is supplied from Al Saad distribution Pump Station which is supplied directly from the 1000 mm Um Al Nar Transmission line. Al Saad tanker filling station and few industries are provided directly from 600 mm pipeline supplying Al Yahar area. The same pipe line is also supplying Al Salamat Farms.

New development of Shabiat and villas is expected to take place at Al Saad & Al Yahar in year 2007 (TPD Master plan, 2004), and the distribution networks to be constructed for this development will be supplied from Al Saad Pump Station.

Currently Al Yahar area is supplied from the 600 mm line branched from the 800 mm line supplying many other areas; i.e. Al Saad, Abu Samra and Remah, where this line is coming out of Al Saad Pumping Station. The 600 mm line is branched to two feed mains each diameter is DN300 mm line connected to Al Yahar distribution system.

4.2.1 Estimated Water Demands and Demand Pattern

The number of houses and villas are 140, 170 units respectively. One farm exists with a demand of 100,000 gal/day, an industrial area, tanker filling station and residential building with demands of 20,000, 30,000, and 25,000 gal/day respectively.

Domestic Demand Pattern

The approximate population for Al Yahar area was 5000 (TPD census, 2004), so it is within the range mentioned in the water distribution code in section 3.4. Accordingly the peak factor can be decided based on the international standards and demand pattern is determined to suite the life attitude of the citizens, where two peaks represent huge water quantities needed by the customer at specific times as shown in Figure 4.6.

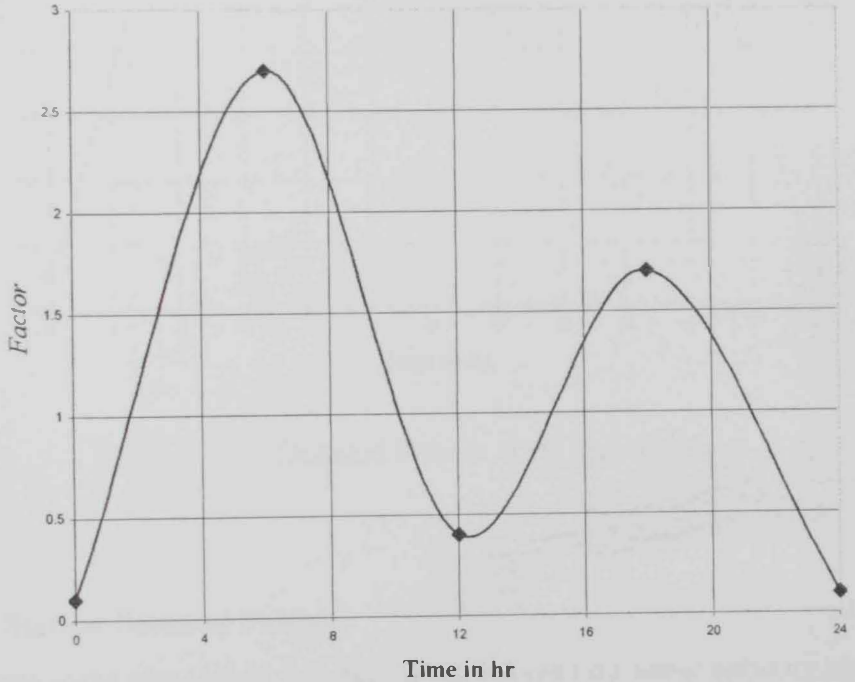


Fig 4.6 Domestic Demand Pattern for Al Yahar Area

Farm Demand Pattern

Similar to Al Dhaher area, the same conditions prevail for Al Yahar area, as the peak flow can be distributed on two times as agreed upon with the customer. The demand pattern for this farm reflecting tow peaks during 48 hrs. The peak flow is planned to take place at 12 am for the first and the second days. The reason behind that is that the applicant requests to get the peak flow two times through out the day period. It is obvious that conflict is removed between the peak flow requested by different customers i.e. the peaks to the residential areas occur two times per day at 6 am and 6 pm and the farm owner requested the same at 12 am for the first and the second day respectively.

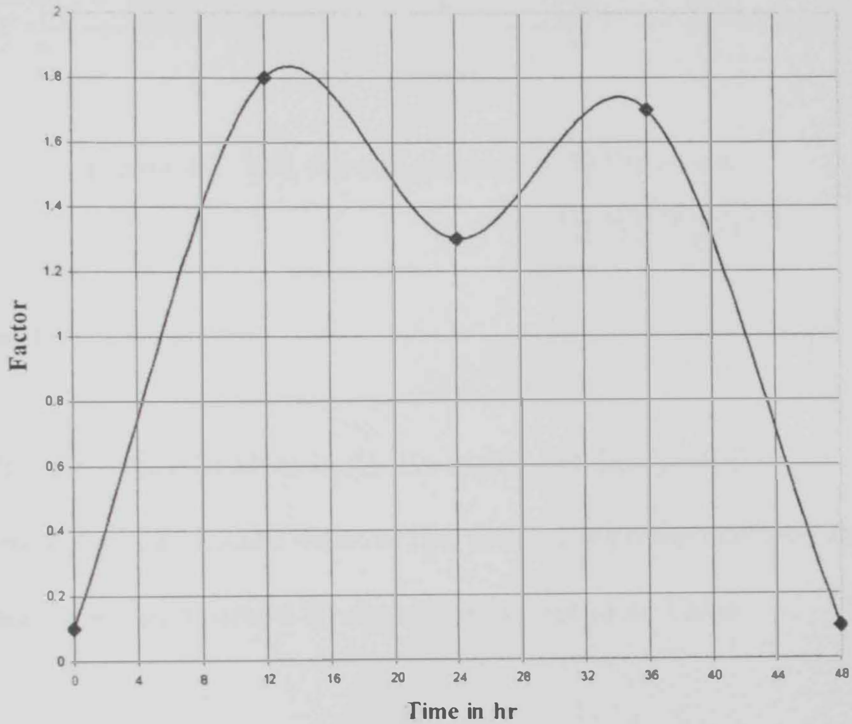


Fig 4.7 Farm Demand Pattern at Al Yahar area

Tanker Filling Station Demand Pattern

There are some areas in Al Ain region not covered by water network until now. So, as per AADC water division policy, the tankers which are supplying these areas, have to be filled at 6 am every day. Figure 4.8 shows the demand pattern for filling these tankers.

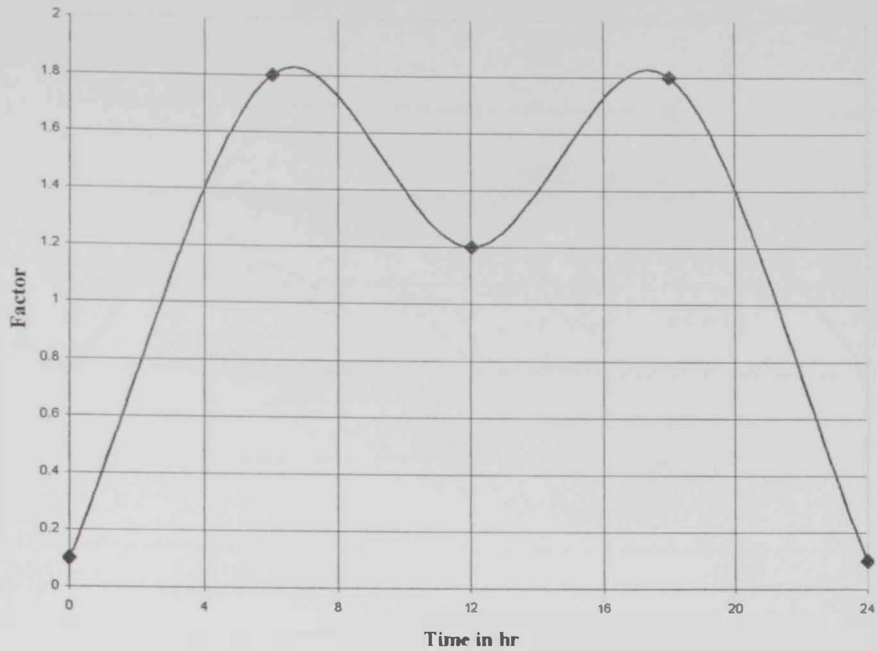


Figure 4.8 TFS demand pattern for Al Yahar area

Industrial area Demand Pattern

Usually, the industrial areas in Al Ain region get their peak flow for continuous 12 hours from 6 am to 6 pm as AADC believes that this is the production time for any factory. Figure 4.9 shows the demand pattern for the industrial area in Al Yahar.

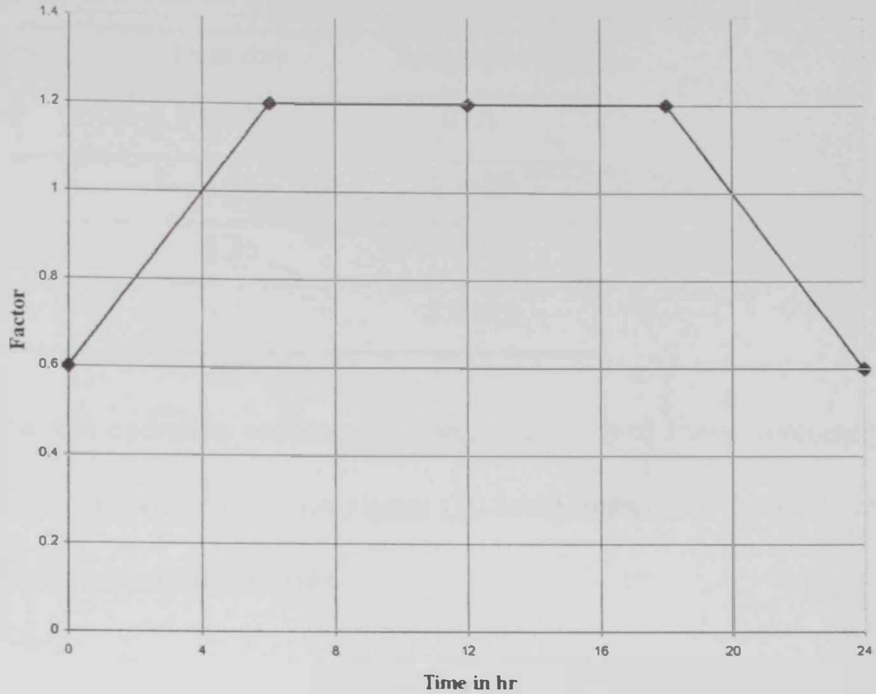


Figure 4.9 Industrial area demand pattern for Al Yahar area

Hence, as a conclusion to this section, the total demand for Al Yahar area is 930,000 gal/day, of which 830,000 gal/day is used to fulfill the domestic demands for the residential areas, and 100,000 gal/day is the bulk consumer's demand

4.2.2 Supply Pattern

There are two 300 mm branches supplying the area. Two flow control valves are installed on each feed main and adjusted to supply a constant flow for the whole day. The flows are listed in Table 4 5 for Shabia's, villas and other bulk consumers. The average supplied flow is 3.186 MIGD.

Table 4.5 Supply pattern for Al Yahar area in MIGD

Consumers	First day	Second day
Shabia's	0.63	0.63
Villas	2.38	2.0825
Bulk consumers	0.35	0.3
Total	3.36	3.0125

As per the operation section, the operators received many customers' complaints during 2004 and to alleviate these complaints (50 complaints/day), the supplied flow are 3-4 times the customer estimated demands.

4.2.3 Internal Storage Tanks

The methodology used for Al Dhaher area is also used for Al Yahar area. Table 4.6 reports the number of properties with their types (TPD Master plan, 2004). The demands of the residential areas as well as the bulk consumers were estimated utilizing the same approach presented in chapter 3 and in section 4.1.3. Finally the number, capacities and the demands for the ISTs are calculated based on similar assumptions used before.

Table 4.6 ISTs Capacities and Demands for Al Yaher area

Property Type	No. of Properties	Demand gal/day /Property	Total Demand gal/day	Storage Tank capacity (gal) / Property	Assumed No. of Properties/ 1 IST	No. of IST[1]	IST capacity (gal)[2]	IST demand (gal/day)[3]
Shabia's	140	1500	210000	3000	10	14	30000	15000
Villa's	170	3500	595000	7000	5	34	35000	17500
Farms	1	50000	50000	200000	1	1	100000	50000
Industrial Area's	1	20000	20000	20000	1	1	20000	20000
Tanker Filling Station's (TFS)	1	30000	30000	30000	1	1	30000	30000
Residential Buildings	1	25000	25000	25000	1	1	25000	25000

1 No. of ISTs = No. of properties/No. of properties/IST
2 IST capacity = Storage tank capacity * No. of ISTs
3 IST demand = Total demand/No. of ISTs

In appendix B, an attachment shows the details of the ISTs calculations and assumptions for different types of properties.

4.2.4 Pipe Network

The following Table is describing Al Yahar network

Table 4.7 Pipe Network Description for Al Yahar area

Pipe diameter	Pipe lengths in m
100.0 mm	56,994
150.0 mm	80,600
200.0 mm	21,242
300.0 mm	25,200
Total Length	184,036

Al Yahar pipe network (Figure 4.10) has 217 pipes, 121 junctions and 52 tanks; the elevation of the junctions is shown in appendix B. The elevations of the network vary from a minimum of 236 m to a maximum of 342 m above see level. The average elevation is around 239 m. The two sources elevation is 342 m above the see level and they are supplying the area with an average pressure of 3 bar.

The above Table shows that around 75% of the pipes in the area is 100mm and 150mm. The bulk connections feeding the multistory building, tanker filling station, and the industrial area are branched form the 150 mm line. The farm connection is branched from the 200 mm line, since this line has enough pressure to supply the requested 50,000 gal/day of water. The internal distribution network is supplying the remaining properties. Finally, since there are two branches of 300 mm lines supplying the whole area, they are considered the primary lines. More details about the pipes are attached in appendix A.

4.2.5 Field measurements

The following measurements are collected from AADC operation section. For the pressure measurement, one data logger is installed in the showed black circle in Figure 4.10, that represents a middle location for the network where the elevation is 240 m. According to the minimum pressure mentioned in the water distribution code, the efficiency of the area network can be assessed. Furthermore, the hydraulic modelling for this network will be calibrated using the pressure readings collected. Residual chlorine measurement is also collected to examine the current quality performance as well as to calibrate the chlorine simulation.

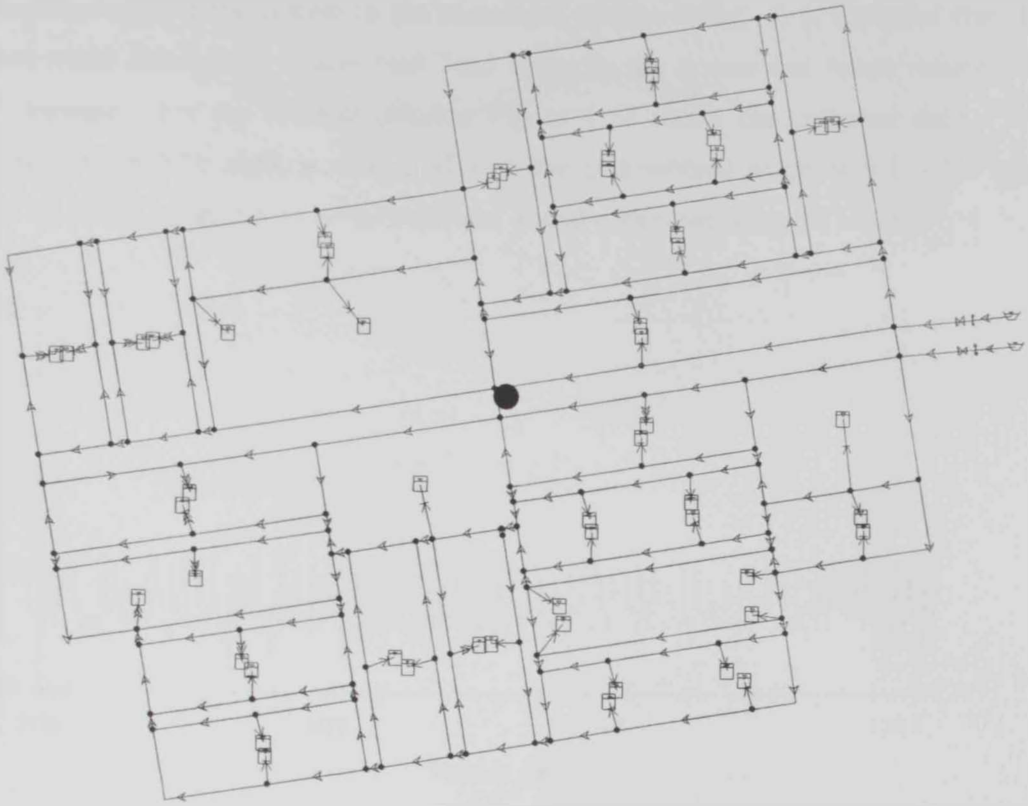


Figure 4.10 The location of the data logger at Al Yahar area Network.

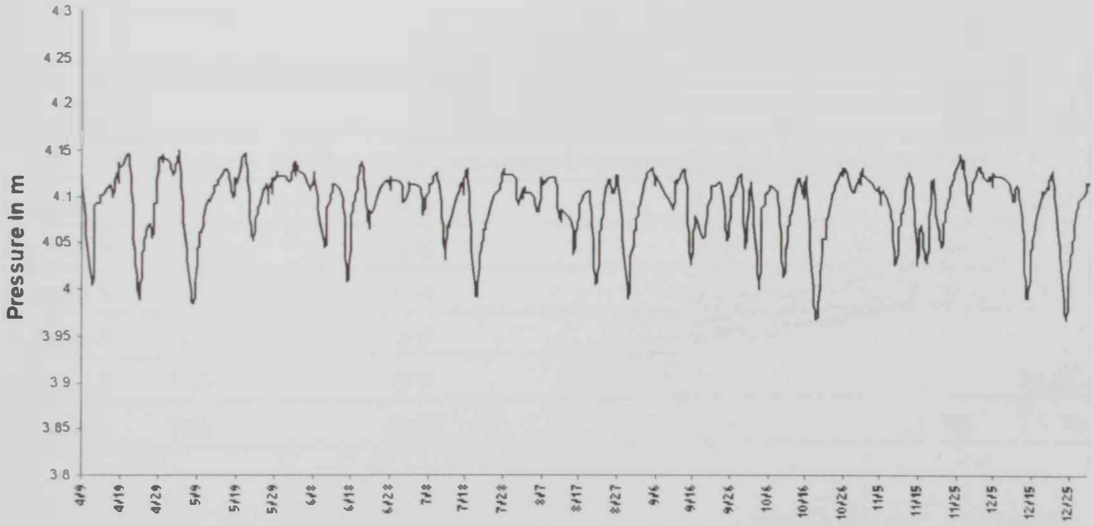


Figure 4.11 Pressure reading for the selected point at Al Yahar area Network.

In Figure 4.11, it is clear that the average pressure is around 4.07 m. In comparing this with the residual pressure requirements as per the water distribution code, this average is found lower than the minimum limit, of 10 m This can be attributed to the large quantity of water distributed through out the system to the customers storage tanks. It is expected that the mentioned water distribution creates high head losses in the system and hence reduces the residual pressure For the residual chlorine Figure 4 12 shows the collected data The average reading of 0.26 mg/l, is compared with the international range of 0.2 – 0.5 mg/l approved by AADC, Laboratory department and found within the accepted range.

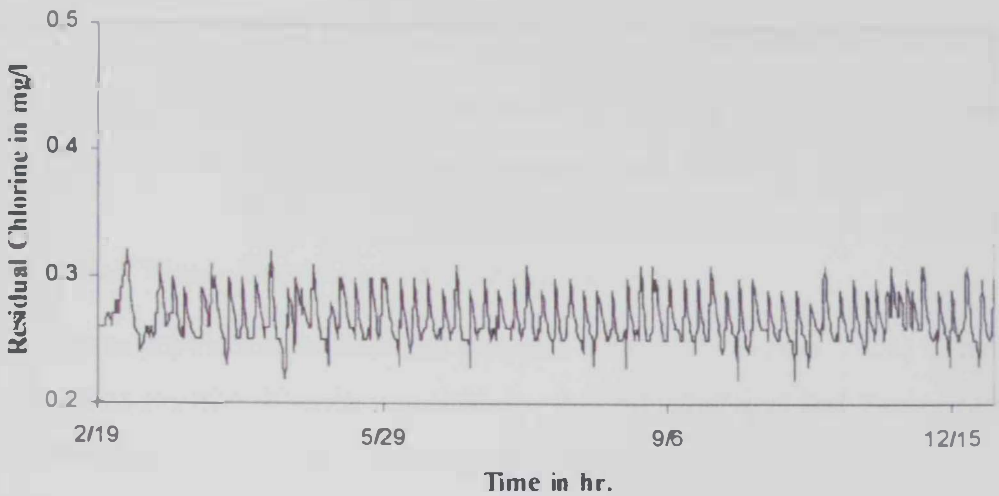


Figure 4.12 Residual Chlorine reading for Al Yahar area for 1 year

Similar sampling program for TTHM measurements was conducted and the results are reported in Table 4.8.

Table 4 8 TTHM field measurements for Al Yahar area

Sample ID	Result (µg/l)				TTHM (µg/l)
	Chloroform	Dichlorobromomethane	Dibromochloromethane	Bromoform	
Sample 1	ND	0.8	3.6	29.9	34.27
Sample 2	<0.30	0.8	3.8	31.8	36.42
Sample 3	ND	0.9	3.5	31.9	36.3
Sample 4	ND	0.8	3.5	31.5	35.87
Sample 5	ND	0.9	3.4	32.4	36.62
Average	ND	0.83	3.56	31.50	35.90

Comparing the above figure with the international limits of 80 ppb for TTHM, one can find that TTHM levels in all areas are within the acceptable limits

4.3 Sweihan Area Characteristics

The estimated quantity of water supplied to Sweihan and Nahel areas for year 2004 is 1.88 MIGD & 1.62 MIGD respectively. Sweihan area is being supplied from Sweihan Pump Station utilizing the distribution pump group. A 300 mm feeding the area is branched into two feed mains; one supplying the villa's side and the other supplying the Shabia's side

New development at Shabiat is expected to take place at Sweihan Village and the distribution network will be supplied from Sweihan Pump Station.

4.3.1 Estimated Water Demands and Demand Pattern

The approximate population for Swiehan area was 3000 (TPD census, 2004). The number of houses and villas are 200, 40 units respectively. An industrial area and Tanker filling station exist with demands of 20,000 and 50,000 gal/day respectively.

Domestic Demand Pattern

The peak factors and the demand pattern are shown in Figure 4.13

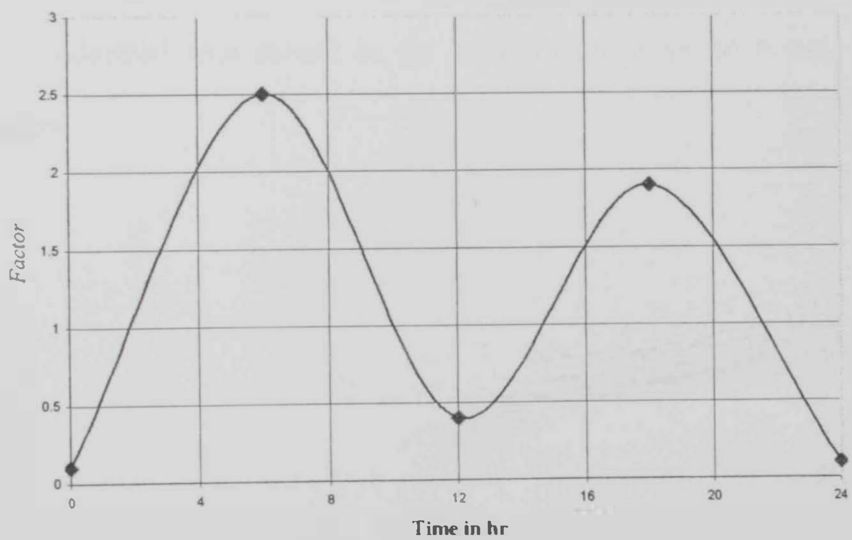


Figure 4.13 Domestic Demand Pattern for Sweihan Area

Tanker Filling Station Demand Pattern

Some areas under development outside Sweihan is supplied by tankers. So, these tankers have to be filled at 6 am every day. The peak flow needed to fulfill the new developed area needs is higher than that used for Al yahar area. Figure 4.14 represents the demand pattern for filling the tankers.

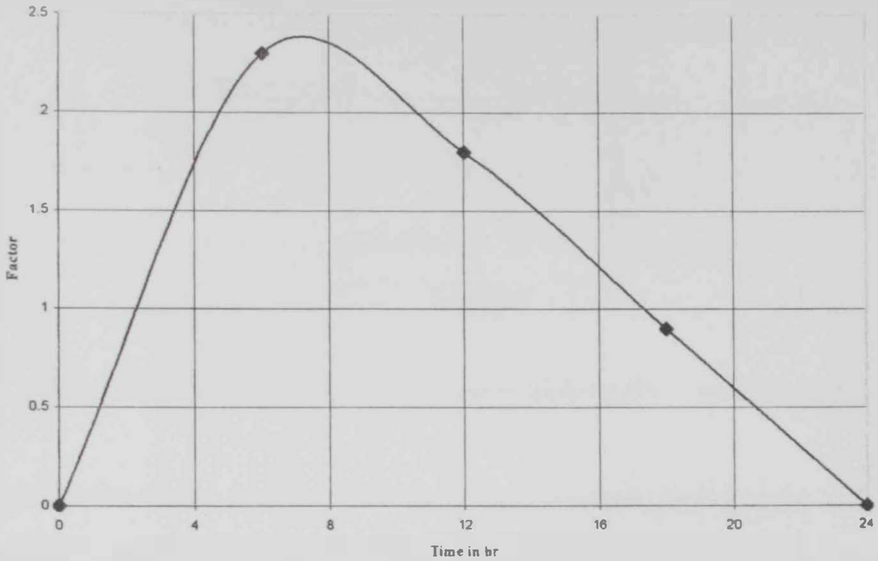


Figure 4.14 TFS demand pattern for Sweihan area

Industrial area Demand Pattern

Same as the demand pattern for Al Yahar area and as per the Al Ain municipality, the peak flow for Swiehan industrial area should be for 12 hrs from 6 am to 6 pm. Figure 4.15 illustrate that pattern.

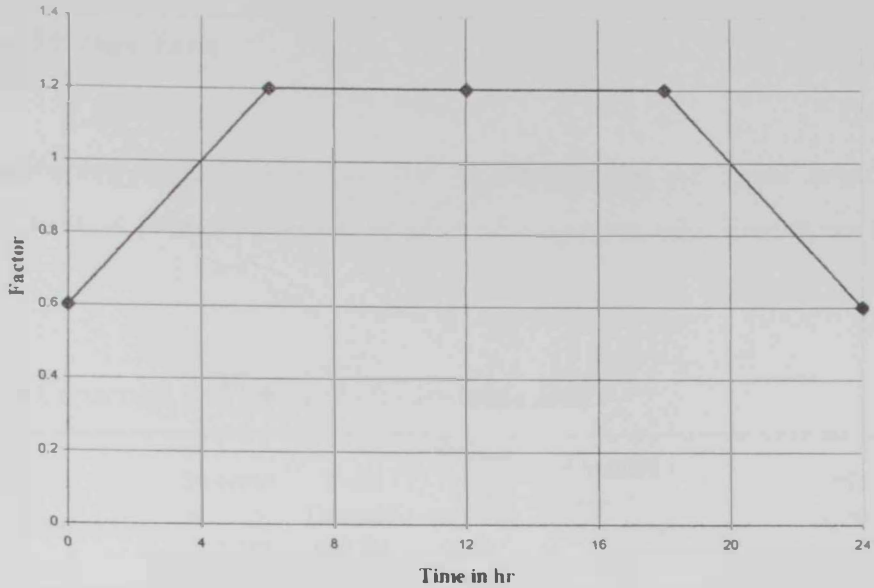


Figure 4.15 Industrial area demand pattern for Sweihan area

Hence, as a conclusion to this section. The total demand for Sweihan area is 510,000 gal/day of which 4400,000 gal/day is used to fulfill the domestic demands for the residential areas, and 70,000 gal/day is the bulk consumers' demand.

4.3.2 Supply Pattern

Two branches of 300 mm supply the area. Two flow control valves are installed on each feed main and adjusted to supply a constant flow for the whole day. These flows are listed in Table 4.9 for Shabia's, villas and other bulk consumers with average flow of 1.835 MIGD.

Table 4.9 Supply pattern for Swiehan area in MIGD

Consumers	First day	Second day
Shabia's	1.2	1.35
Villas	0.35	0.42
Bulk consumers	0.21	0.14
Total	1.76	1.91

4.3.3 Internal Storage Tank

The same methodology which was used for Al Dhaher and Al Yahar areas is used for Swiehan area. Table 4.10 represents the number of properties with their types (TPD Master plan, 2004).

Table 4.10 ISTs Capacities and Demands for Swiehan area

Property Type	No. of Properties	Demand gal/day /Property	Total Demand gal/day	Storage Tank capacity (gal) / Property	Assumed No. of Properties/ 1 IST	No. of IST[1]	IST capacity (gal)[2]	IST demand (gal/day)[3]
Shabia's	200	1500	300000	3000	10	20	30000	15000
Villa's	40	3500	140000	7000	2	20	14000	7000
Industrial Area's	1	20000	20000	40000	1	1	20000	20000
Tanker Filling Station's (TFS)	1	50000	50000	75000	1	1	30000	50000

1 No. of ISTs = No. of properties/No. of properties/IST
2 IST capacity = Storage tank capacity * No. of ISTs
3 IST demand = Total demand/No. of ISTs

The demands of the residential areas as well as the bulk consumers were estimated utilizing the same approach presented in chapter 3 and in section 4.1.3. Finally the number, capacities and the demands for the ISTs are calculated according to the illustrated assumptions in Table 4.10.

Supplementary features for the ISTs capacities and demands are attached in appendix B.

4.3.4 Pipe Network

The following Table is describing Swiehan network

Table 4.11 Pipe Network Description for Sweihan area

Pipe Diameter	Pipe lengths in m
100.0 mm	47,495
150.0 mm	67,167
200.0 mm	17,702
300.0 mm	21,000
Total Length	153,363

Swiehan pipe network (Figure 4.16) has 150 pipes, 82 junctions and 42 tanks. The above Table shows that around 75% of the pipes in the area is 100mm and 150mm. The bulk connections feeding the tanker filling station and the industrial area are branched form the 150 mm line. The 300 mm line supplies the whole area branched from 800 mm line coming out form Sweihan pumping station, and therefore considered the primary line, that is against the recommendations listed in the water distribution code. More details about the pipes are attached in appendix A. Swiehan city is flat area with most elevations 250 m above see level. The elevation of the junctions is shown in appendix B. The two sources elevation is 250 m above the see level and they are supplying the area with an average pressure of 4 bar.

4.3.5 Field measurements

For the pressure measurement, one data logger is installed in the showed black circle in Figure 4.17. This pint is located in the villas side where more consumption of water is expected. The performance of the network and the hydraulic modelling calibration can be done utilizing these readings.

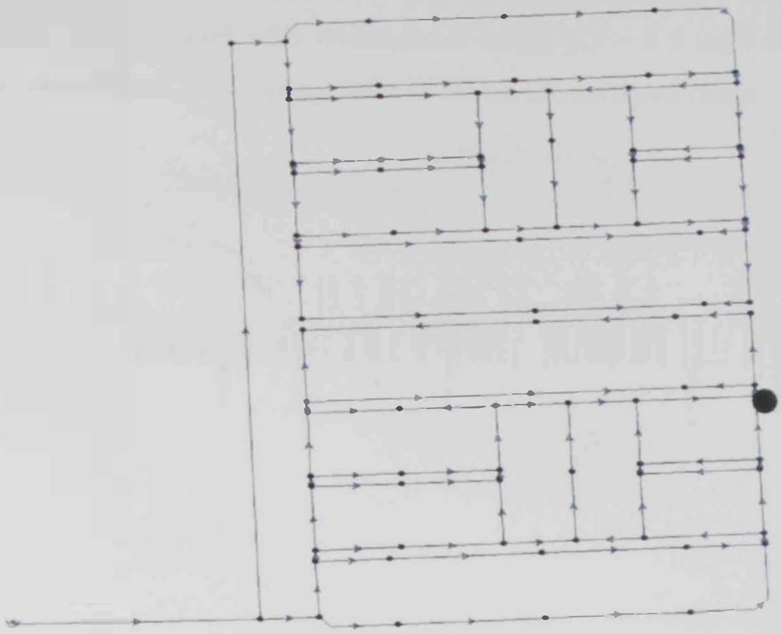


Figure 4 16 The location of the data logger at Sweihan area Network

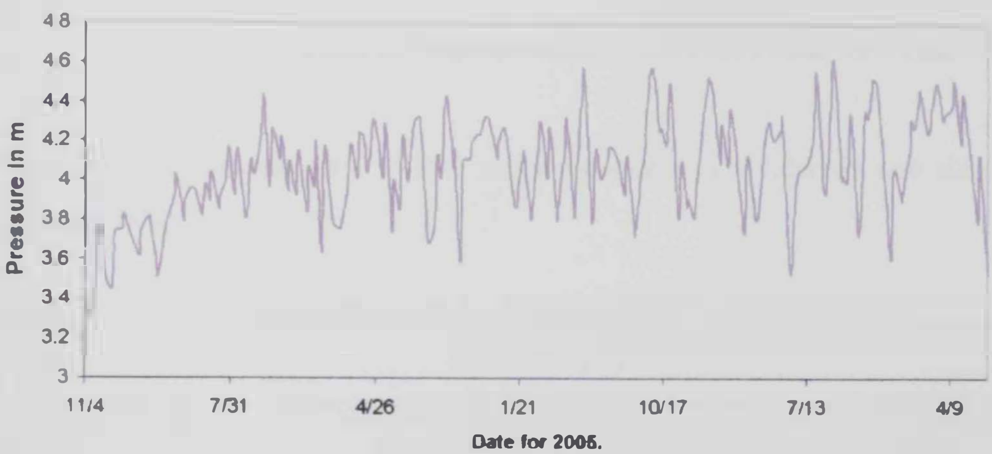


Figure 4.17 Pressure reading for the selected point at Sweihan area Network

Inspecting Figure 4.17, it is clear that the average pressure is around 4 10 m that is lower than the minimum pressure requirements as per the water distribution code. Again, this can be attributed to the large quantity of water distributed through the system to the customers storage tanks. It is expected that the mentioned water distribution creates high head losses in the system and hence reduces the residual pressure.

Residual chlorine measurements are also collected to examine the current quality performance as well as to calibrate the water quality simulation (Figure 4.18). Comparing the average reading of 0.26 mg/l with international range 0.2 – 0.5 mg/l as approved by AADC laboratory department, this reading is found within the accepted range.

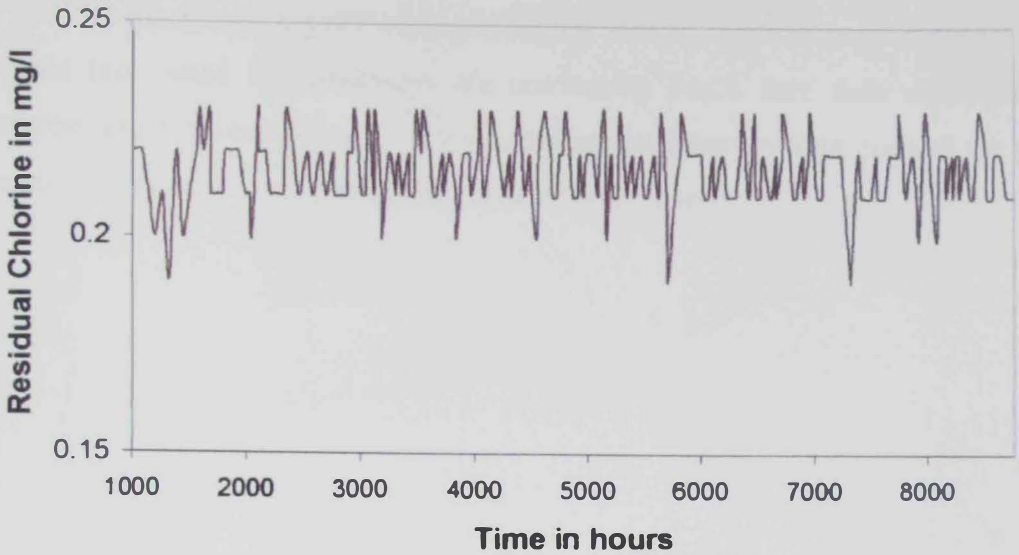


Figure 4.18 Residual Chlorine reading for Sweihan area for 1 year

Similar sampling program for TTHM measurements was conducted and the results are reported in Table 4.12.

Table 4.12 TTHM field measurements for Sweihan area

Sample ID	Result (μg/l)				TTHM (μg/l)
	Chloroform	Dichlorobromomethane	Dibromochloromethane	Bromoform	
Sample 1	ND	0.8	3.2	29.1	33.04
Sample 2	ND	0.7	2.9	25.8	29.43
Sample 3	ND	0.8	3.3	30.8	34.79
Sample 4	ND	0.8	3.3	30.3	34.43
Sample 5	ND	0.9	3.4	30	34.27
Average	ND	0.79	3.21	29.20	33.19

Comparing the above figure with the international limits of 80 ppb for TTHM, one can find that TTHM levels in all areas are within the acceptable limits.

Finally, as a brief conclusion to this chapter, it is noticed that the residual pressure measurements for Al Yahar and Sweihan are very low which is not accepted by the regulation and supervision office that's assessing the performance of any network according to its compatibility with the water distribution code. The existence of the customers storage tanks and the inaccurate supply management by AADC, are the main reason behind this observable fact, since the customers are consuming much than their estimated demand without regulating or controlling their house connections that are kept open all the time. For this reason, the system is not performing in acceptable manner.

CHAPTER 5

HYDRAULIC SIMULATION AND CALIBRATION

Hydraulic analyses are performed for the three selected areas comparing the simulation results with the data obtained from field measurements so that the calibration process is accomplished. The hydraulic modeling is performed for each system based on the demand driven approach by applying the estimated demand is allocated on the tanks (ISTS). The purpose of this modeling is to explain the difference between simulation results and collected field measurements especially the residual pressure for each system as well as comparing other results with the specifications and standards set by ADWEA (Water distribution Code). The energy cost is calculated for each system based on 0.05 Dhs/kwh. Due to the unavailability of the information from AADC, the calculated energy costs are not verified against the real one.

5.1 IST MODELING AND CALIBRATION

The aim of a computer model of the water-distribution system is to reproduce the behavior of a "real-world" hydraulic system as closely as feasible in terms of spatial and temporal characteristics. The collection of field data provides an opportunity to understand the operation of the real system at a specified number of locations and times. Such efforts are consistent with the findings of the American Water Works Association Engineering Computer Applications Committee which indicates that "true model calibration is achieved by adjusting whatever parameter values until a reasonable agreement is achieved between model-predicted behavior and actual field behavior" (AWWA Engineering Computer Applications Committee 1999). Once a model is calibrated, it can then be used to, among other purposes, estimate hydraulic characteristics of the real-world system at locations where measured data are unavailable or unknown, spatially and temporally.

As described in the Chapter 2, model calibration entails adjusting model parameter values until an acceptable match is achieved between measured data and model-simulated values i.e., pressures at the test hydrants and water levels in the storage tanks. The water-distribution system models for the 3 selected areas were calibrated to the collected hydraulic data. The model was run as an extended period simulation (EPS) using one hour hydraulic time step and demand-pattern factors derived from the water distribution code as illustrated in chapter 3.

According to the AWWA Engineering Computer Applications Committee (1999), 9 sources of possible error could cause poor agreement between simulated model values and measured field values. These sources of error, which provide a potential list of factors that can be adjusted during the model-calibration process, are: (1) errors in input data (measured and typographic), (2) unknown pipe roughness values (i.e., Hazen-Williams "C-Factors"), (3) effects of system demands (assigning consumption along a pipe to a single node), (4) errors in data derived from network maps, (5) node elevation errors, (6) errors introduced by time variance of parameter values such as storage tank water levels and pressures, (7) errors introduced by a skeletal representation of the network as opposed to modeling all small-diameter pipes, (8) errors introduced by geometric anomalies or partially closed valves, and (9) poorly calibrated measuring equipment including data loggers, tank water-level monitors, and SCADA systems.

These sources of error (Table 5A) also provide a list of potential model parameters that can be modified during the calibration process. To decide which parameters might require more, less, or no modification, investigators evaluated each parameter as to the qualitative magnitude of error (high, moderate, or low) that could result from uncertainty and variability of the parameter. These evaluations are also listed in Table 5A. Two of the sources of possible error were evaluated as having a qualitatively high or moderate error magnitude: (1) unknown initial, minimum and maximum water level, (2) effects of system demands and consumption. The initial estimates for these two parameters were subjected to variation during the calibration process and will be discussed below. The remaining 7 sources of possible error are believed to introduce minor to insignificant errors to model simulations, and therefore, were not modified during the calibration process.

Discussions with the water utility indicated that the network pipes were new and very clean, and inspections had shown very little debris. In addition, all pipes are made of ductile iron where the variation in "C-factor" is negligible since the networks installed 5 years ago. Taking into consideration that the water flowing through these networks is desalinated water of very good quality that is void of dust and sediments so that the interior wall of the pipes are not affected from that quality water. Therefore, initial estimates for "C-factor" obtained from published tabular values (DI pipes report, 2004) were kept unchanged during the calibration process. The calibration process was therefore directed to adjust the dimension of the tanks.

Table 5A. Qualitative evaluation of sources for model error (AWWA Engineering Computer Applications committee, 1999)

Error Type	Qualitative Estimate of Error			Notes
	High	Moderate	Low	
1. Input data			X	Measurement and typographical
2. Unknown pipe roughness values	X			No measured data, values assumed
3. Effects of system demands	X	X		Metered consumption data are not available
4. Data derived from network maps			X	Data from AADC databases, quality assured using GIS software
5. Node elevation data			X	Measuring point data determined from GIS and As built drawings
6. Time variance of pressures and water levels			X	Pressures monitored with continuous-recording data loggers
7. Skeletal representation of network			X	Not applicable--"street-level" network used
8. Geometric anomalies or partially closed valves			X	Areas of suspected partially closed valves reported to water utility and investigated
9. Poorly calibrated measuring equipment			X	Data loggers factory calibrated for each test; quality assured using manual pressure gauge

Generally, definitive standards to assess the accuracy of model calibration have yet to be agreed upon or established. However, the following calibration criteria have been suggested:

- An average pressure difference of ±2.2 psi (1 5 m) with a maximum difference of ±7.3 psi (5 m) for a "good" data set, and an average pressure difference of ±4.3 psi (3 m) with a maximum difference of ±14.2 psi (9.65 m) for a "poor" data set (Walski 1983); and
- The difference between measured and simulated values should be ±5 psi (3 4 m) to ±10 psi (6 8 m) (Cesario and Davis 1984).

These criteria have been used as general guidelines besides considering the availability and accuracy of the data for the water-distribution system serving the selected area. Therefore, a pressure difference at the test-hydrant locations (difference between measured and simulated pressure) of ±2.2 psi is considered as an upper limit for the calibration criteria for the model of the selected water-distribution system.

As discussed in the previous chapter, the IST concept is considered in the hydraulic simulation to model a group of customer storage tanks have the same features like they have the same elevation, they used to reserve the water quantity for a certain type of properties...etc. Hence the built model for each of the three selected system include the ISTs with the specifications and dimensions presented in chapter 4. Moreover, in that chapter it was mentioned that the house connection size and length considered to supply each IST was 4 inch and 20 m length respectively. The main reason behind the assumption made for the connection sizes is that for each area the number of ISTs per the number of properties were illustrated in tables 4.2, 4.6 and 4.10 for Al Dhaher, Al Yahar and Swiehan respectively showed that the a range of 8 – 10 number of customer storage tanks represented by one IST. In addition to that the sizes of the house connections used in reality are 0.5 – 2 inches, where actually there is no available information provided by AADC about the house connections sizes in the selected areas. Based on that 1 inch was assumed for each house connection, which represents 8-10 x1 inch connected to each IST. Hydraulic modeling task is conducted to proof that 8-10 connections each size is 1 inch are equivalent to one 4 inch connection considering 25 m length of each pipe (results are shown in appendix D).

In the conducted simulation, it was found that each 8 number of 1 inch connections are equivalent to 4 inch connection in term of delivering a certain quantity of water. Based on this result, 4 inch line for a length of 25 m was selected to represent the mentioned number of real house connections to each IST.

After that, the initial levels for the ISTs were adjusted approximately to reflect the real life adjustment of the internal gate valves for the network of each selected system. Taking the fact that the operators throttle more the gate valves located directly downstream the feed main used to supply each area to restrict the flow to the nearby houses/villas. The flow is expected to be very high to those properties near from the source and then this flow is reduced as a result of the increased headlosses to the properties at the far end from the source. Hence, to simulate this phenomenon, the initial levels for the nearby ISTs from the source were much higher than for those which are a way from the source, in order to "force" more water to flow to the remaining ISTs located at the middle and at the end of the network. HGL's are playing the major role in controlling the explained principle.

The validity of the assumptions made to estimate the capacities of the storage tanks for both low cost houses (Shabia's) and the villas are tested. Accordingly, an extended period simulation is performed with the estimated capacities of the ISTS presented in chapter 4. Initial modeling results were found not to match the measured parameters such as pressures. Also, the temporal behavior of WL in the ISTS .

Figures 5.1, 5.2 and 5.3 represent the time series curves for the percentage water levels in few selected ISTS for the three simulated systems.

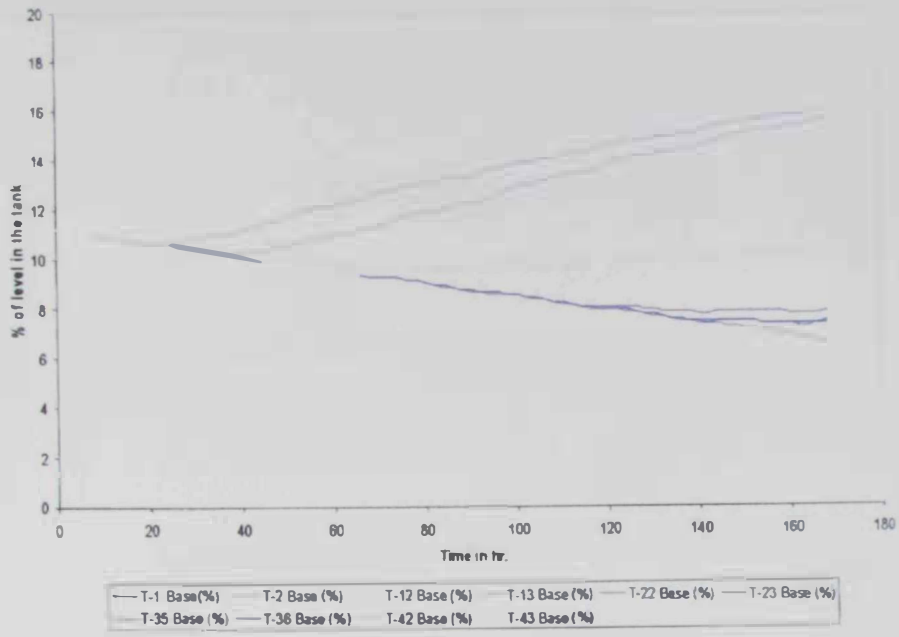


Figure 5.1 Al Yaher Network – Initial Setting Case - % of full level for some selected tanks

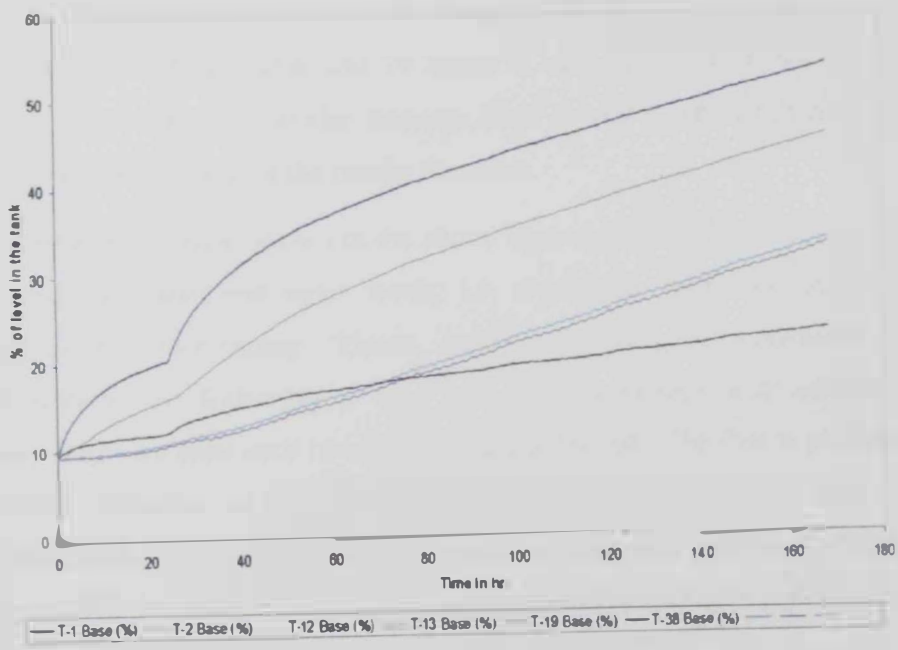


Figure 5.2 Sweihan network – Initial Setting case - % of full level for some selected tanks

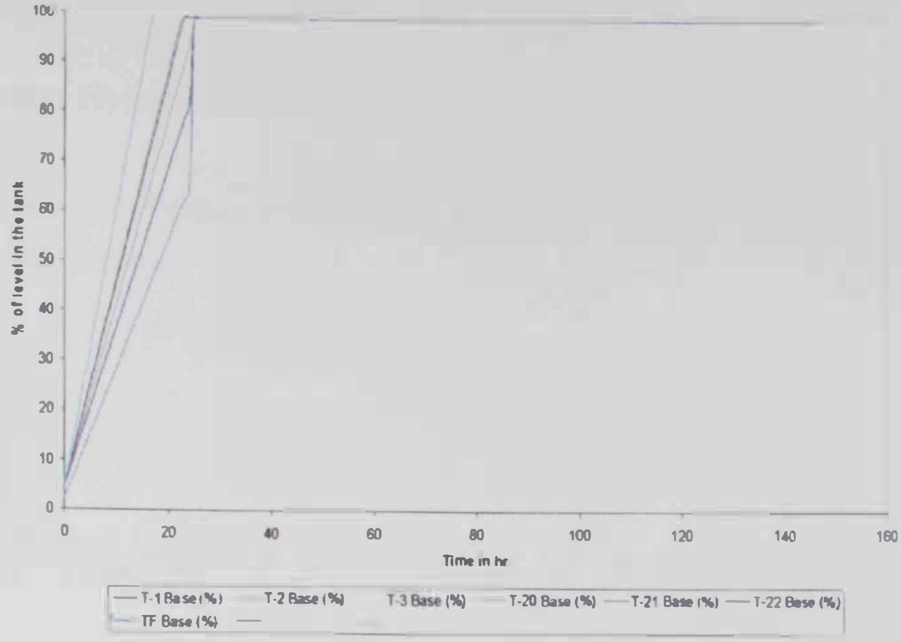


Figure 5 3 AL Dhaher network – Initial Setting case - % of full level for some selected tanks

The above figures show that some ISTS undergo monotonic rise of water level while some others undergo monotonic drop. Since the demands are changing in cyclic way; 48 hours in case of Al Dhaher and Al Yahar and 24 hours in case of Swiehan, water levels in ISTS should accordingly change in similar manner. This should be observed once the effects of estimated initial water levels on the results diminish.

The simulated results shown in the above figures suggest that both the estimated tank diameters and/ or considered water levels; i.e. maximum, minimum and initial, are not correct and need further tuning. Hence, major calibration was conducted for the three simulated systems by following a trial and error approach and adjusting the ISTS dimensions. This was done until two criteria were achieved. The first is produced cyclic (or close to cyclic) behavior of the water levels fluctuations in all ISTS and the second is attaining reasonable agreement between the measured pressures and the simulated ones at the monitoring points. This was conducted for a long period (one year) to eliminate the effect of initial conditions on the results. Yet, another criterion was also observed during that calibration process in which the percentage change in IST capacity was limited to 5 % of the original estimated value.

A large number of iterations were undertaken for each system and the results produced are shown and discussed below.

5.2 Al Yahar Hydraulic Calibration

Figure 5.5 shows the distribution of the ISTs for different locations to represent the actual customers' storage tanks

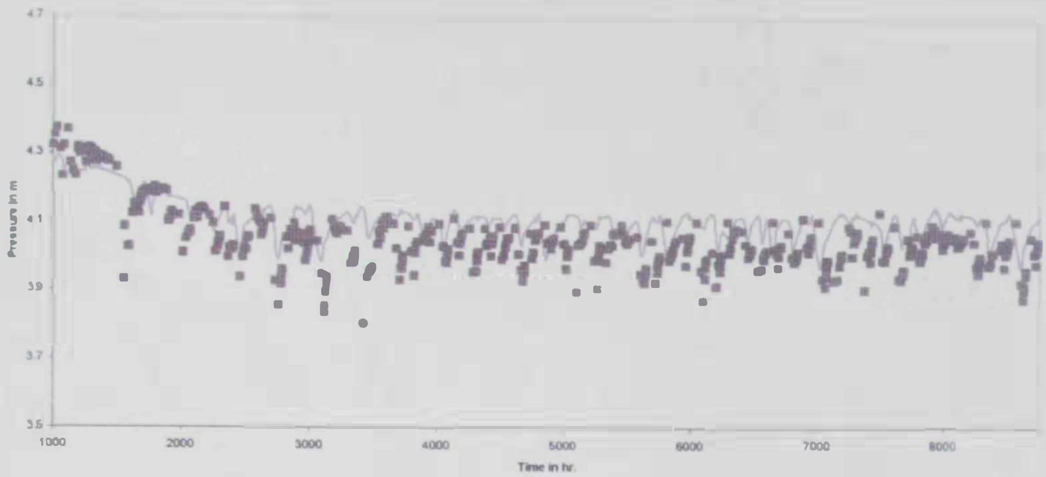


Figure 5.4 Residual pressure of J-41 in the model versus the data logger pressure reading for Al Yahar area

Figure 5.5 shows that the residual pressure of J-41 which has the same location of the data logger in the model showed in Figure 4.10 is matching with the pressure readings obtained from the field. This agreement has been reached after several trials to calibrate the assumed ISTs for Al Yahar area. The calibration was done on the initial and the maximum level of these ISTs, and the tanks' diameters were kept unchanged. This approach resulted in changing the calculated capacities of the ISTs Since the data assumed for the storage tanks capacities for Shabia's and villa's were practical and have been carefully selected.

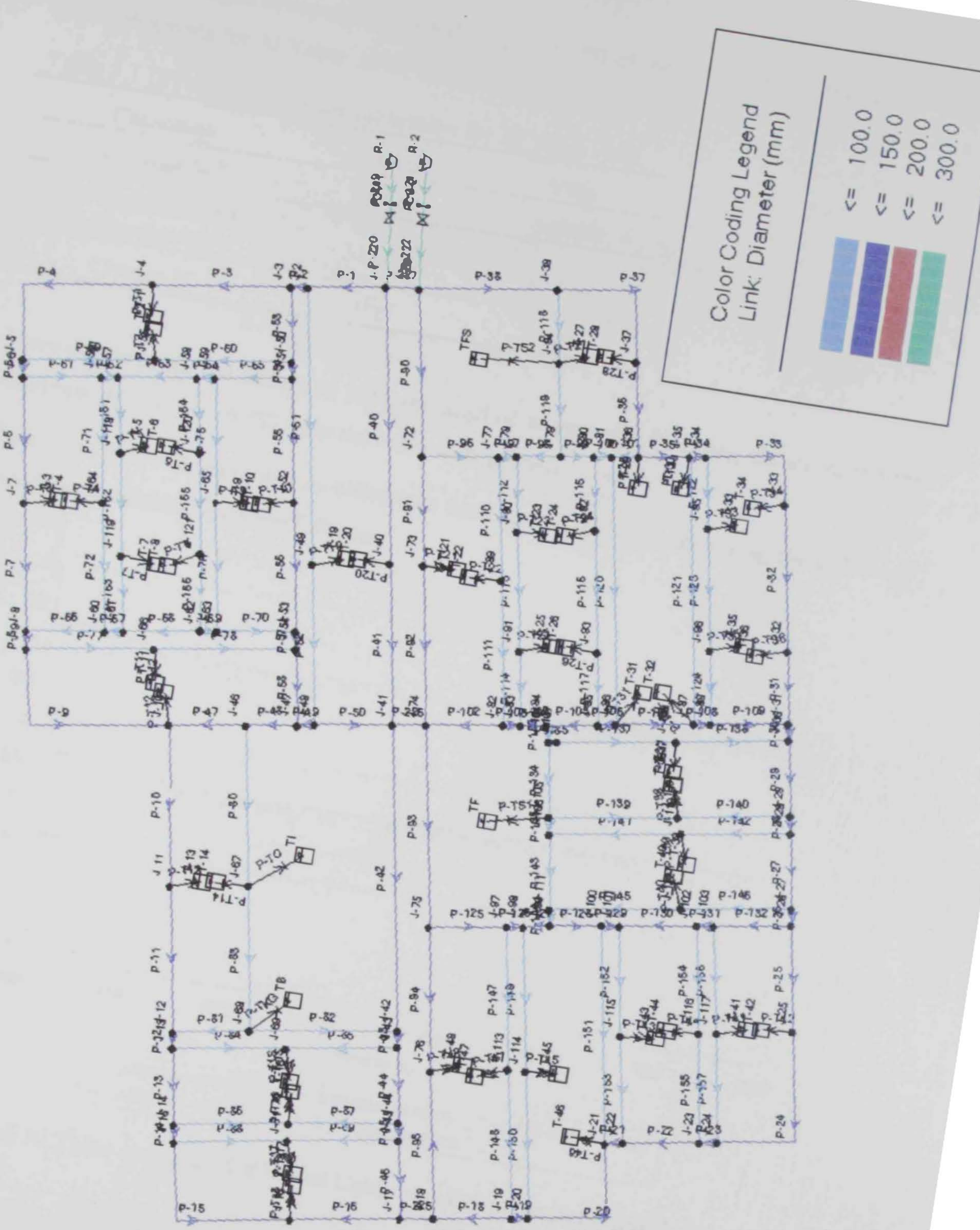


Figure 5.5 Al Yaher Network (Demand Allocation on ISTS)

Percentage changes in capacities were all limited to 5% of the estimated values of ISTs The new dimensions for Al Yahar ISTs are showed in table 5.1

Table 5.1 ISTs capacities after calibration for Al Yaher Area

Capacities	Shabia	Villa
Estimated (m^3)	30,069	35,382
New Capacities After Calibration (m^3)	31,603	36,616
Change %	5%	3.5%

More details about the calibrated ISTs are attached in appendix C, listing the initial and the maximum levels in each IST after the calibration process.

Based on the above tables and modifications done for the hydraulic modeling, the following results were obtained for a one full year.

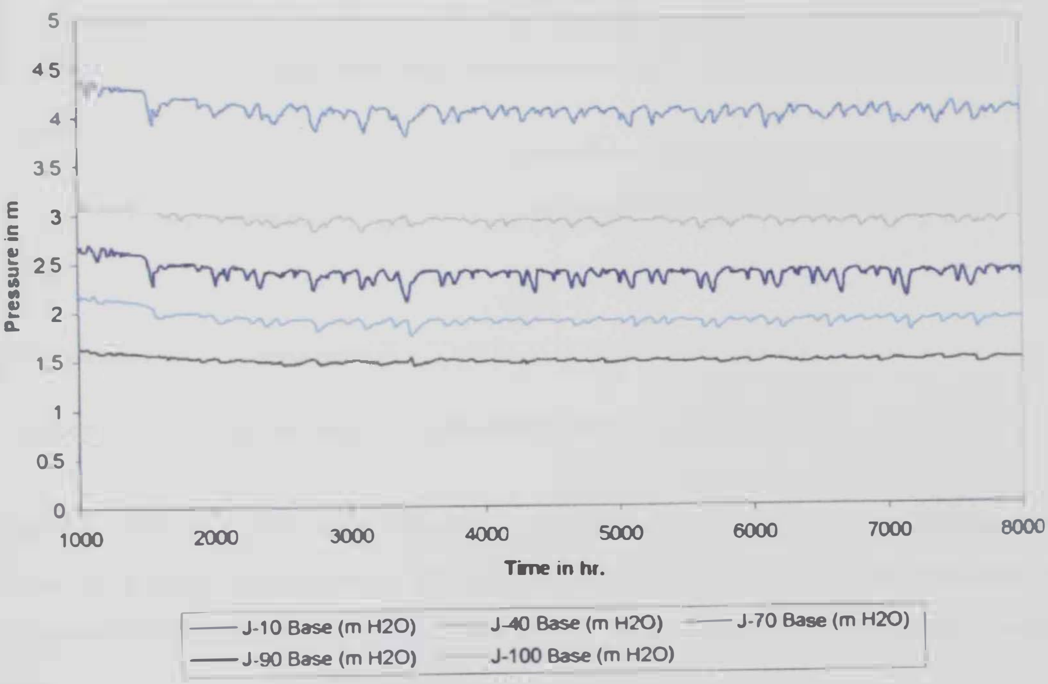


Figure 5.6 Al Yaher network – Calibrated Case – residual pressure for selected junctions

Figure 5.6 shows the calibrated pressure for a group of junctions scattered around different locations in the simulated network. The pressure ranges from 1.5 m to around 4.2 m Such pressures are not within acceptable limits described by the water distribution code discussed in chapter 3 where the minimum accepted pressure is 10 m. These results were discussed with the water utility and found accepted as per their expectations. As shown also, the pressure variation for each junction is very small; this is resulted from the constant flow rate supplied to this area. The pressure for J-90 is the lowest in the network where the water flowing through this junction is used to supply the farm tank as well as other ISTS. Hence, the pressure dropps due to the high head losses occurring in the pipelines upstream this junction.

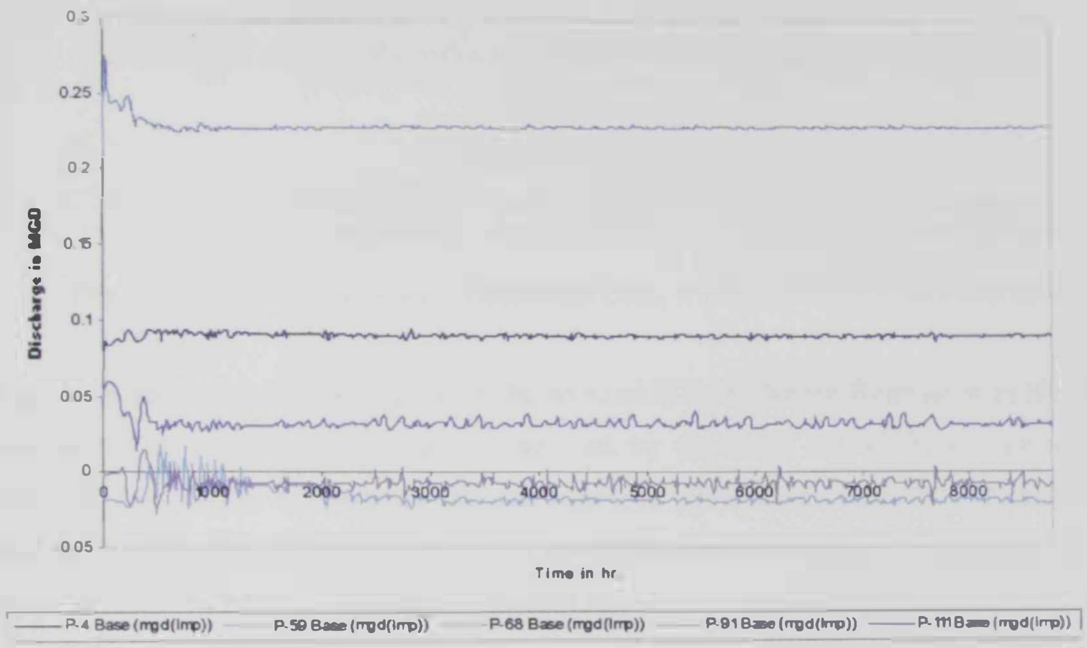


Figure 5.7 Al Yaher network – Calibrated Case – discharge for some selected pipes

Figure 5.7 illustrates the water discharge through the network. The highest quantity delivered to different customers or ISTS is through pipe P-91, where the demands for the farm as well as the remaining ISTS are catered. The maximum flow rate is around 0.23 MGD

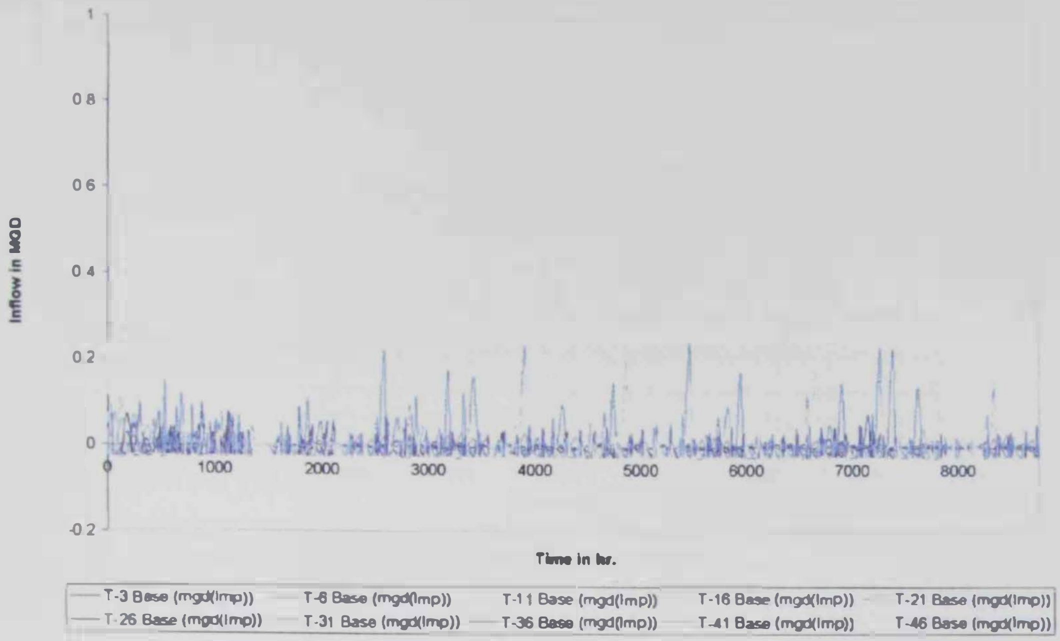


Figure 5.8 Al Yaher network – Calibrated Case – inflow for some selected tanks

Figure 5.8 shows the inflow to some of the selected ISTs where the fluctuation in the inflow represents the variable water quantity received by the ISTS in different periods. As mentioned in chapter 4, the water demand has been allocated on each IST. Hence, at the peak demand the water level drops due to the high consumption and the HGL for the IST drops as well. This allows the water system to deliver more water quantity to compensate the water diminution until the HGL in the system approaches that in the ISTS. This repetitive behavior has an impact on the water level in the tank as shown in Figure 5.9. This illustrates the fact that at all times and for any demand condition the water levels in the ISTS remain relatively constant with a cyclic (or semi cyclic) variation.

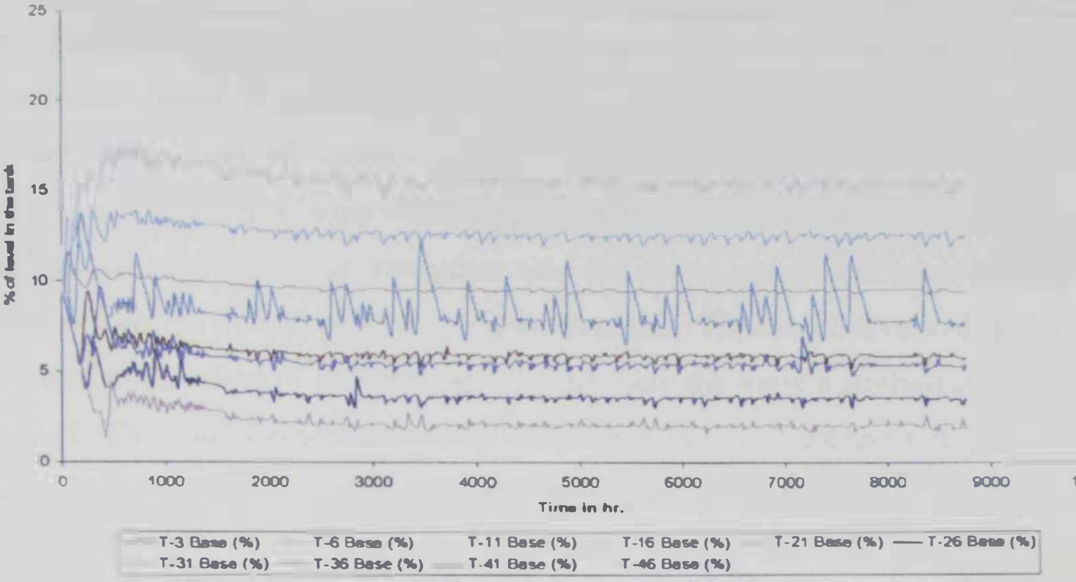


Figure 5.9 Al Yaher network – Calibrated Case - % of full level for some selected tanks

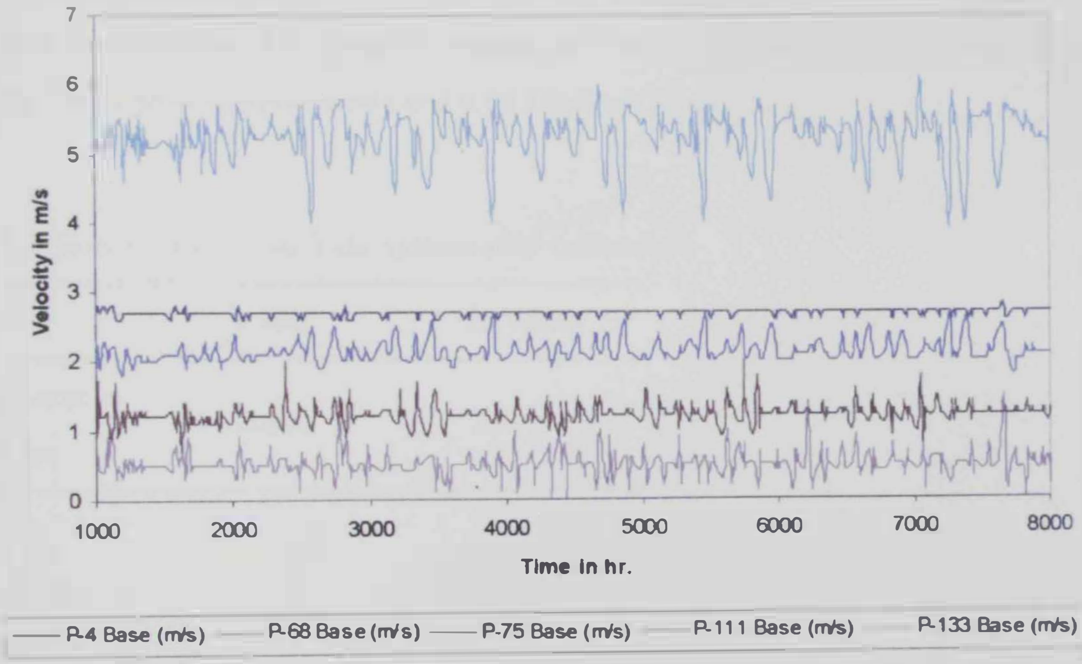


Figure 5.10 Al Yaher network – Calibrated Case – velocity in some selected pipes

Figure 5.10 shows that the velocities are in the range of 0.5 – 5.5 m/s and the average in some pipes is more than 1.2 m/s, indicating the supply is more than required as mentioned in chapter 4. The mentioned velocities are not accepted by the regulator according to the water distribution code As discussed with the water utility, the design of this network was done by considering the needed demands estimated in chapter 4. On the other hand, AADC operation section was forced to increase the supplied quantities to this area up to 3 times the needed demands after receiving several complaints from the customers as a result of shortage of supply. This increment in the water supply can be actually perceived in two ways. The first indicates that the customers' properties near the source usually get more quantity of water than the others as a result of receiving good pressure. The second is related to the unplanned and random throttling of valves to equally distribute the water quantities as per the required demand. The operation section usually limits the inflows into the tanks of properties near the source seeking unigorm distibtion to the entire system. This has been accomplished in the current simulation by increasing the intial water levels in the ISTs compared to other ISTs located toward the far end of the system. The cost of pumping depends on the flow and the head at the source. These two parameters are controlled for Al Yahar area, where the flow is constant due to maintaining a constant flow rate by means of FCV to deliver around 3 times the needed demand as well as constant pressure of 3 bars. The energy cost is then calculated assuming 100% power requirements and 0.05 Dhs/kwh (Table 5.2)

Table 5.2 Energy cost for Al Yahr system after calibration

	1 year	20 years
Energy cost in Dhs	10890	217487

5.3 Swiehan Hydraulic Calibration

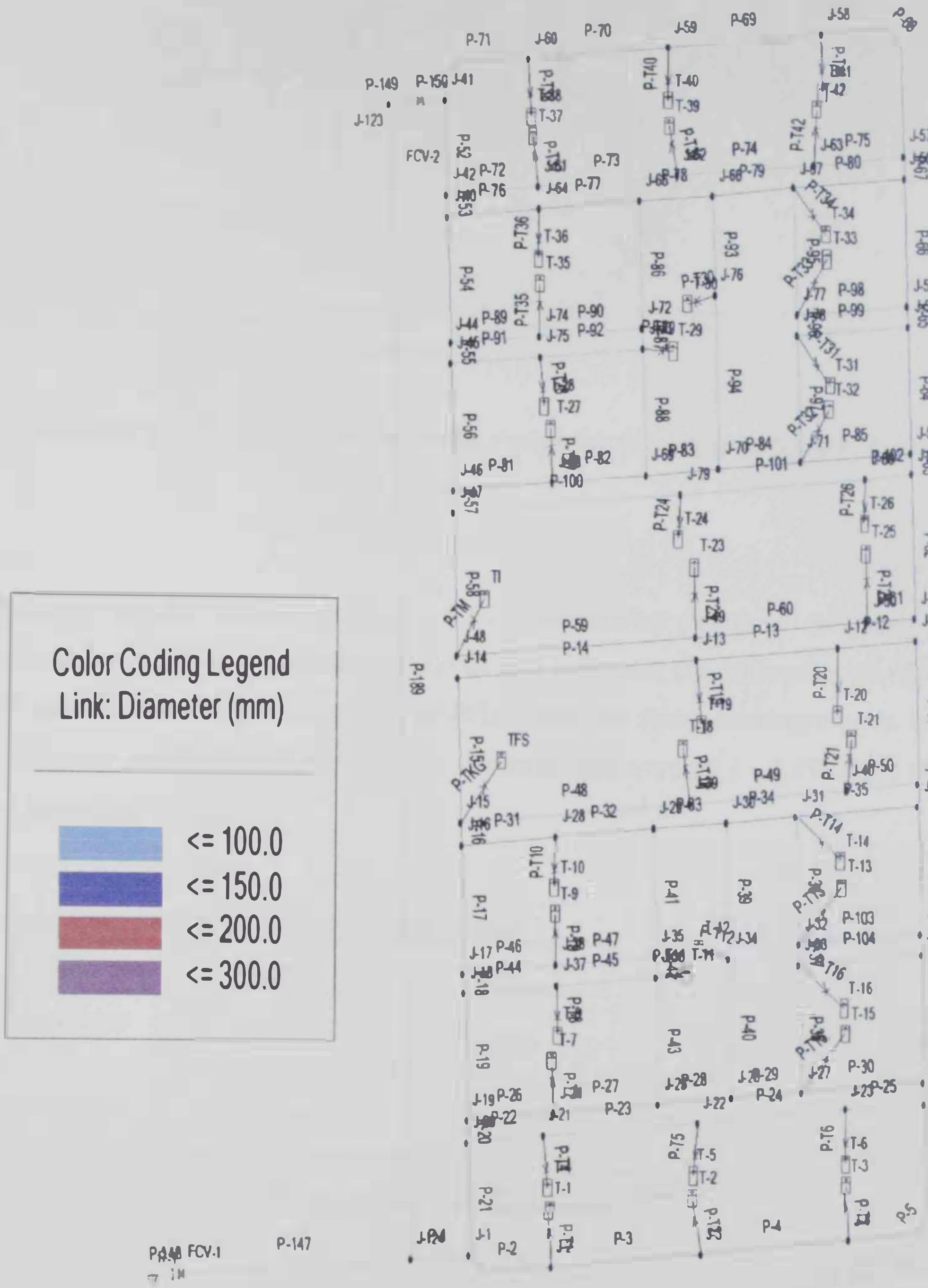


Figure 5.11 Sweihan network (Demand Allocation on ISTS)

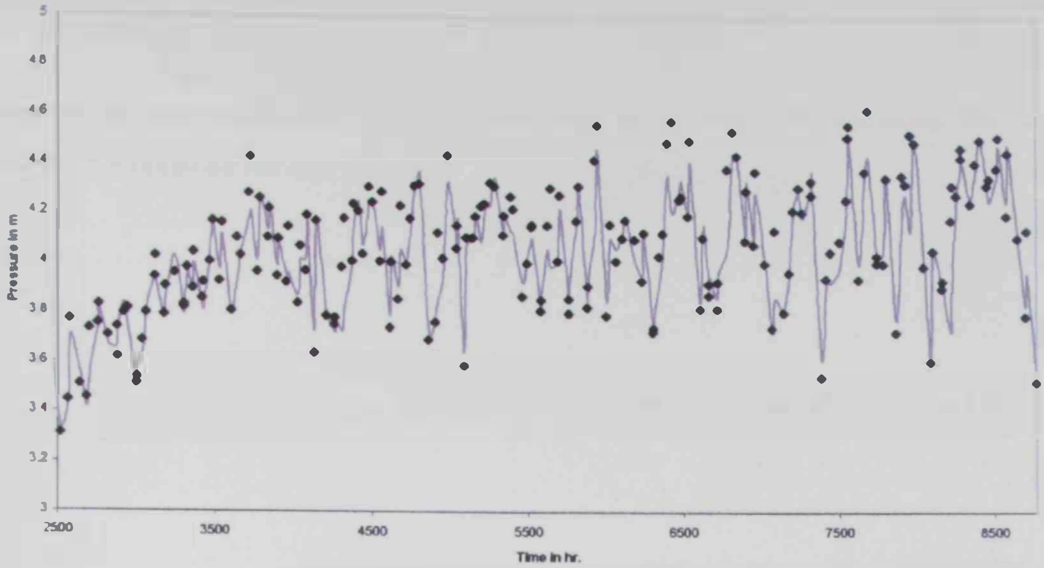


Figure 5.12 Residual pressure of J-10 in the model versus the data logger pressure reading for Sweihan area

Figure 5.12 shows that the residual pressure of J-10 (monitoring point), is matching the pressure readings obtained from the field. Similar to Al Yahar area, the calibration was done on the initial and the maximum level of these ISTs , and the tanks diameters were kept unchanged. After the calibration, the change in ISTs capacities were all 3 - 3.8% more than the estimated capacities (Table 5.3).

Table 5.3 IST capacity after calibration for Sweihan area

Capacities	Shabia	Villa
Estimated Capacities (m^3)	30,069	21,595
New Capacities After Calibration (m^3)	30,971	22,415
Change %	3%	3.8%

The initial and the maximum levels calibrated are shown in details for each IST in appendix C.

Based on the above tables and modifications done for the hydraulic modeling, the following results were obtained for one full year.

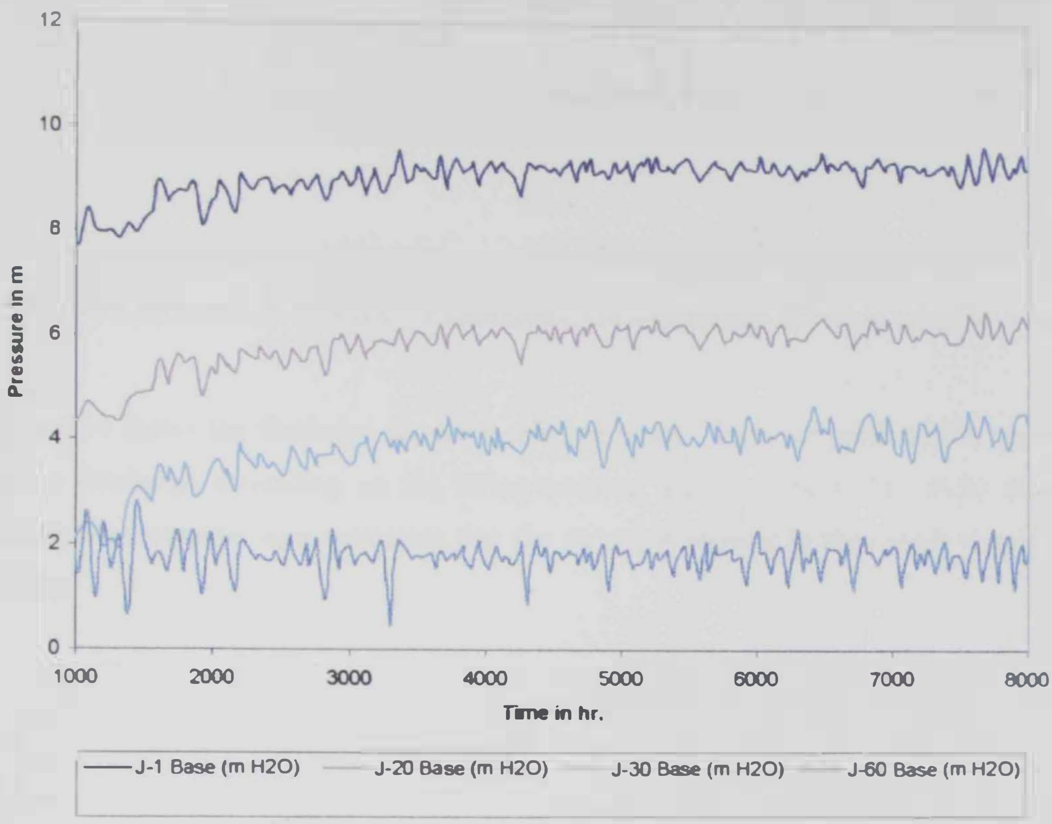


Figure 5.13 Sweihan network – Calibrated Case – residual pressure for some selected junctions

Figure 5.13 shows the residual pressure for different locations in the network where the pressure varied in average from 2 m to around 8.5 m. This pressure variation is depending on the water quantity flowing through each junction in the network. As mentioned for Al Yahar area, it is expected that the lowest residual pressure (J-60) is accomplished with high water flow to one of the bulk consumers and other ISTS in the network.

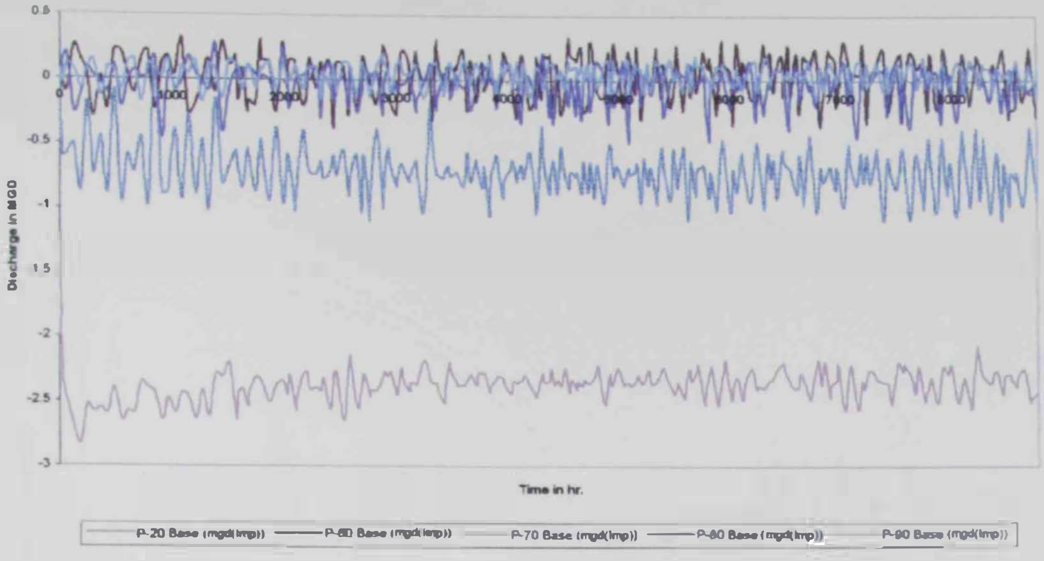


Figure 5.14 Sweihan network – Calibrated Case – discharge for some selected pipes.

Figure 5.14 shows the discharge for some selected pipes. They are presented in negative or positive discharge depending on the filling/draining status of the ISTs. P-20 shows the discharges in negative sign indicating that the direction showed in the model should be the opposite.

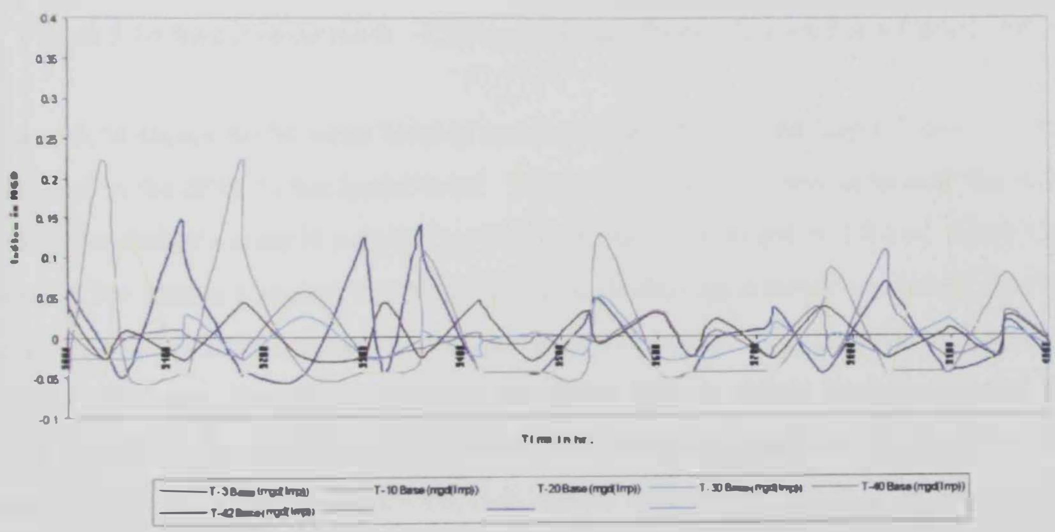


Figure 5.15 Sweihan network – Calibrated Case – inflow for some of the selected tanks

Figure 5.15 demonstrates a cyclic behavior for the ISTs inflow. As mentioned for Al Yahar area, the inflow increases to the ISTs when the level drops and vise versa and depending on the difference in the HGL between the ISTs and nearest junction.

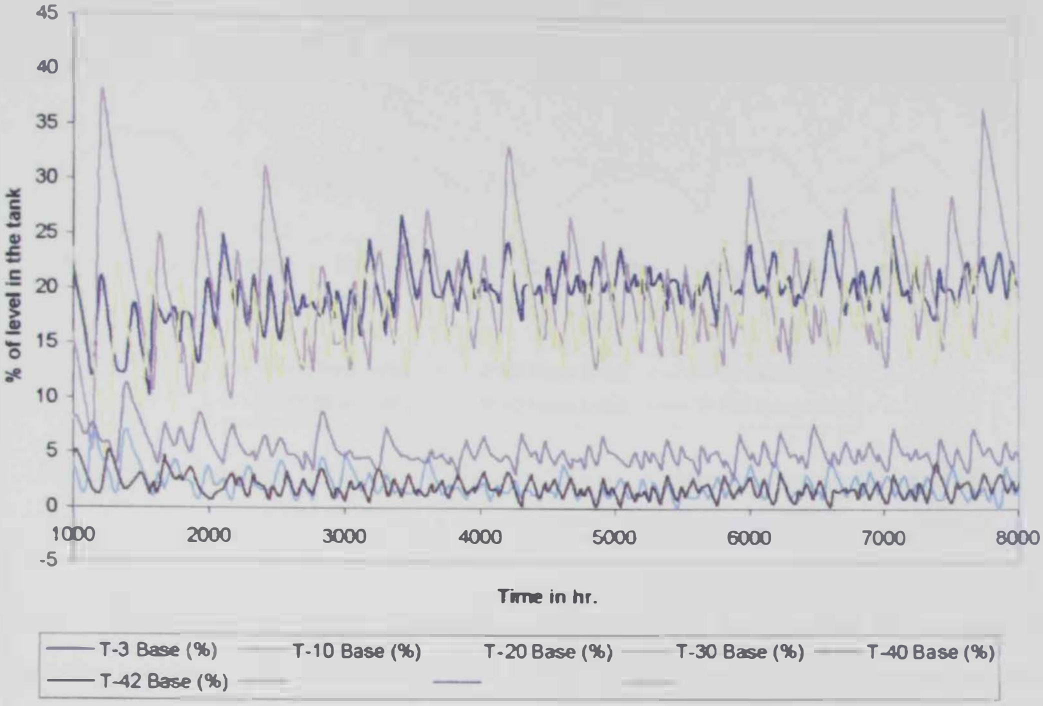


Figure 5.16 Sweuhan network – Calibrated Case - % of full level for some selected tank

Figure 5.16 shows the % water level in some selected ISTs. The major factor in controlling the level in the ISTs is the initial level. For Swiehan area the source is near the villas area which the shabia's areas is supplied by 200 mm line for a length of 2.8 km. Since the water demand for Shabia's around 0.3 MGD which is double the quantities required for the villas side. The head losses for this line may increase as a result of the mentioned flow as well as the long distance. Hence, the inflow to the shabia ISTs as shown for tanks labeled T-30, T-40 and T-42 is less although it is covering the needed demand but, on the other hand the water level in these tanks is much less than that for villas ISTS.

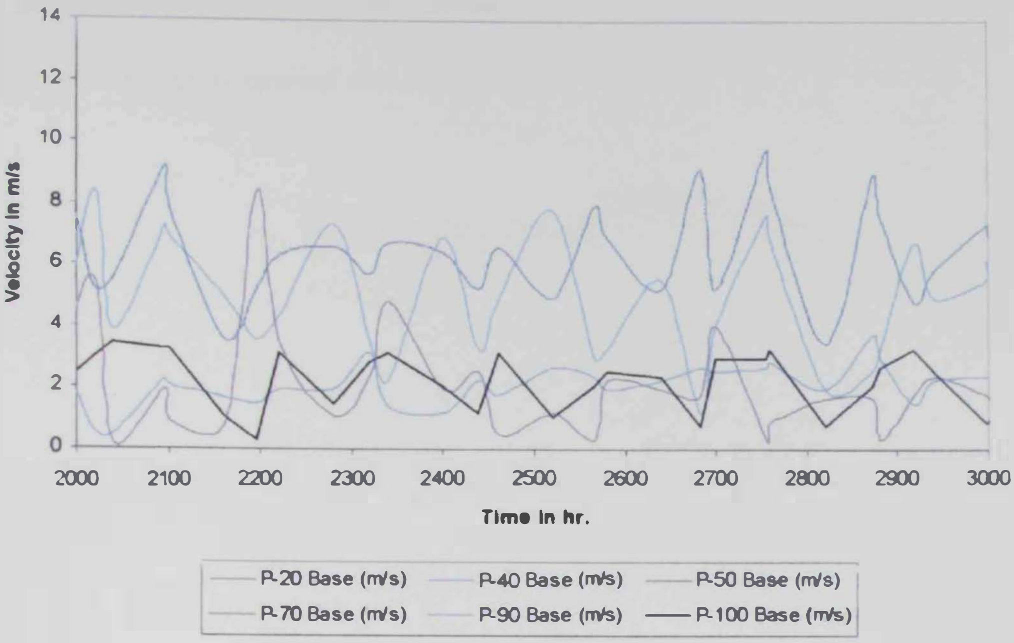


Figure 5.17 Sweihan network – Calibrated Case – velocity in some selected pipes

Figure 5.17 illustrates the velocity fluctuation in the system. As expected the in some pipes, the velocity reaches to around 8 m/s. This figure is found six times the maximum velocity described in the water distribution code.

Regarding the energy cost, the annual and 20 – year energy costs are listed in Table 5.4 based on constant inflow and head. However, it should be noted that the energy requirement of Al Yahar is supplied from Swiehan and Al Saad pumping stations that pump the water to both areas, Swiehan and Al Yahar

Table 5.4 Energy cost f or Sweihan system after calibration

	1 year	20 years
Energy cost in Dhs	8960	180980

5.4 Al Dhaher Hydraulic Calibration

The same methodology is applied also for this area

Figure 5.18 AL Dhaher network (Demand Allocation on ISTS)

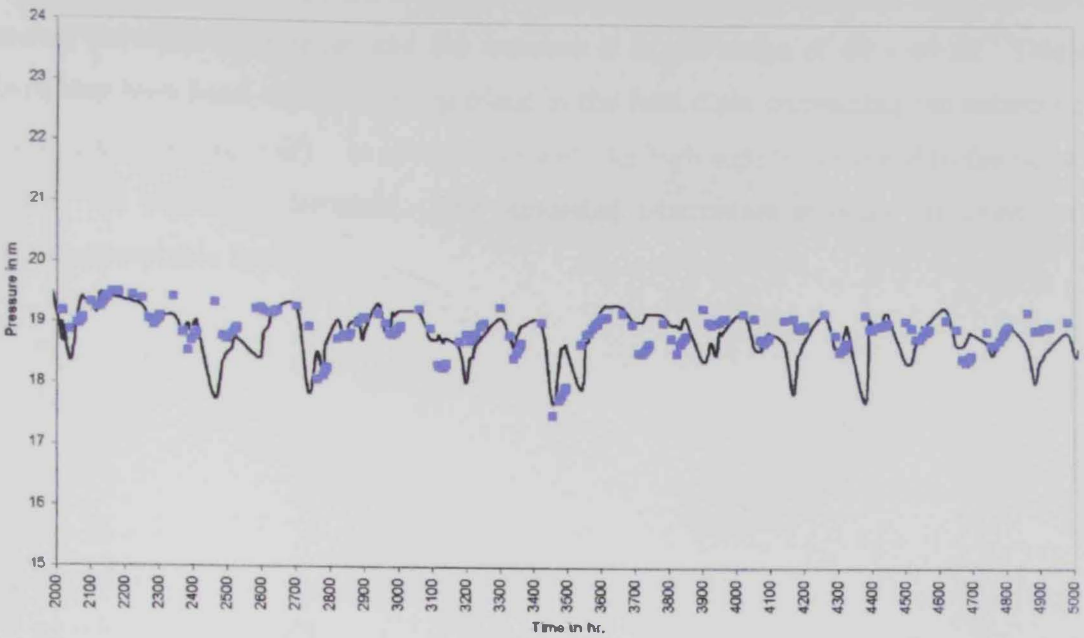


Figure 5.19 Residual pressure of J-20 in the model versus the data logger reading for Al Dhaher area

Figure 5.19 shows that the residual pressure of J-20; the monitoring point highlighted earlier in Figure 4.3. Similar to Al Yahar area, the calibration is done on the initial and the maximum level of these ISTs , and the other tanks dimensions are kept as estimated. After the calibration, the ISTs capacities were slightly increased (2 – 2.5%) above the assumed capacities (Table 5.5)

Table 5.5 IST capacity after calibration for Al Dhaher area

Capacities in m^3	Shabia	Villa
The Estimated Capacity	14,933	21,164
New Capacities After Calibration	15,306	21,587
Change %	2.5%	2%

In Chapter 4, it was clarified that all field measurements of Al Dhaher area are accepted as per the water distribution code. Even though, the pressure measurements at the data logger location are around 19 m, thy are still lower than expected for this system since the elevation difference between the sources and the network is in the range of 40 - 45 m. This result explains that high head losses is taking place in the feed main connecting the network to the source (UmGhafa reservoir). In association with the high supply delivered to the network in excess of the estimated demands. The simulated parameters at other junctions were all within the acceptable limit.

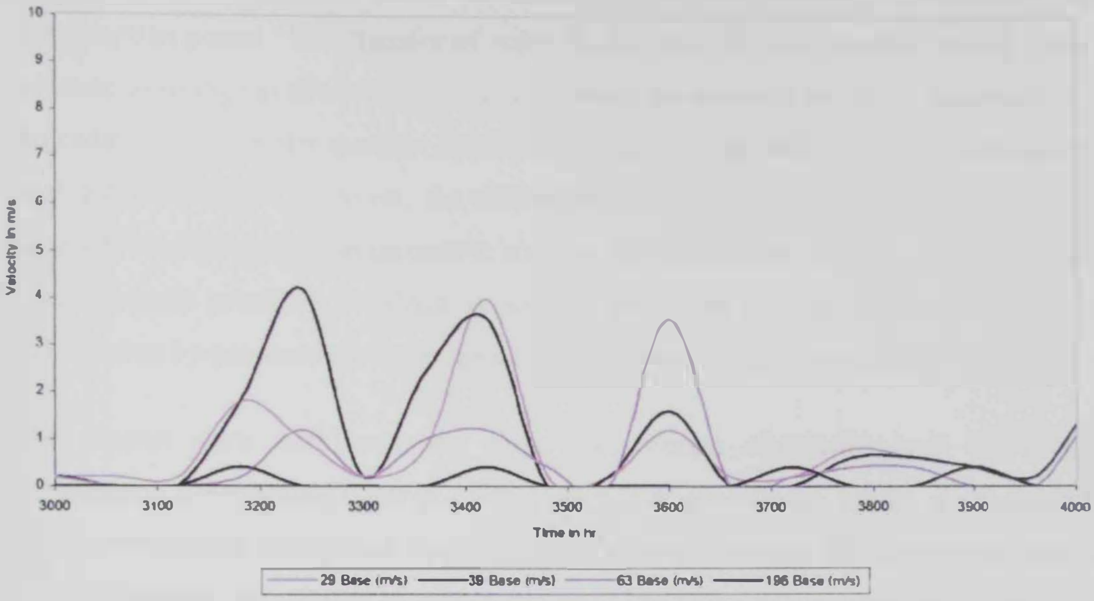


Figure 5.20 Al Yaher network – Calibrated Case – velocity in some selected pipes

In Figure 5.20 the velocities reach to of 4 m/s during the peak demands where in some pipes the average is more than 0.5 m/s. Though the high velocity mostly occurres in the pipes connected to the bulk consumer properties, this velocity can still be accepted due to the limited number of such connections and to also limit the flow to such locations.

Finally, the energy cost is not calculated for Al Dhaher since the it is supplied by gravity as mentioned in Chapter 4.

CHAPTER 6

QUALITY SIMULATION AND CALIBRATION

Attaining good quality of produced drinking water at the production point doesn't eliminate the need of investigating the water quality in the distribution system at different consumption points. The transfer of water in the system through mostly buried pipes in addition to storage in several tanks many of which are operated by the customers do cause degradation in the water quality. Quality investigation may include several chemical and biological elements. However, the disinfectant and possible hazardous disinfectant by-products are the most common ones to track in the distribution systems. Since chlorine is predominantly practiced in AlAin as well as the whole country, chlorine residual and chlorination by-products are investigated in this chapter for the three simulated areas.

The chapter starts with overview about the chlorine disinfectant and the Chorine Disinfection By-Products (DBPs). The chapter then presents the results of experimental work conducted to determine some kinetic parameters needed for simulating both the residual chlorine and THMs in pipe networks. The following sections then address the calibration and simulation results of chlorine residuals in the three simulated systems. Bromoform formation in these systems is discussed as well.

6.1 Chlorine Disinfection

The concept of Residual Concentration of Disinfectant is associated with disinfection durability. In fact, to guarantee the water supply system's disinfection, a residual concentration disinfectant is needed to prevent recontamination by pathogenic or indicator micro-organisms, which can originate in the biofilm formed inside the system, as well as in negative pressure areas (created by pipe cracks, fissures, etc.).

There is, however, a problem when the water distribution systems are considerably large where the chlorine concentration decays and the chlorine residual concentration tends to disappear in part of the system.

Knowing the aspects behind chlorine decay is in order if we are to develop a strategy capable of disinfecting a water supply system and, at the same time, preserving water quality until the point of use, without using more disinfectant than necessary. In this sense, mathematical modelling of the decay is essential to correctly project new systems or make changes in existing ones. The loss of chlorine residual concentration along the water distribution system is taking place in three separated mechanisms:

- Chlorine reactions in bulk fluid;
- Chlorine reactions with pipe and other system element's walls.
- Natural evaporation.

If, ideally, the chlorinated water was pure and the material of the pipes was inert, the only mechanism leading to the decay would be that of natural evaporation, especially in particular areas of the distribution system, namely reservoirs and other free surface flow. Mathematical modelling of chlorine decay along the water supply system is a problem whose solution is not yet absolutely mastered.

6.2 Chlorination By-products

Each type of disinfectant has both advantages and disadvantages in drinking water treatment. Free chlorine is very effective at inactivating pathogens but it produces some of the highest concentrations of DBPs. Chloramination is a weaker disinfectant compared to free chlorine but very few DBPs are formed when water system use chloramination. Ozone is an effective disinfectant and doesn't produce many DBPs of concern but ozone is not capable of providing a residual through the distribution system. Ultraviolet light has been shown to be effective at inactivating pathogens and it doesn't produce major DBPs that are yet regulated by the U.S. EPA but like ozone it does not produce a residual. Regarding chloramination, the best Cl_2:N ratio for minimizing DBP formation depends on raw water quality. The type and concentration of humic substances present in the raw water source are the most important parameters that dictate which Cl_2:N ratio is the best. In a study examining chloramine disinfection, Diehl *et al.* (2000) found higher TTHM levels when disinfecting with chloramines at a Cl_2: N ratio of 7:1. The experiment showed that a Cl_2: N ratio of 3:1 was ideal for controlling DBP formation, but this ratio might not be suitable for controlling bacterial growth.

The main byproducts formed with chlorine disinfection are THMs and HAAs. MX, a chlorinated hydroxyfuranone, is another chlorination by-product formed from the oxidation of organic matters. It is found in chlorinated drinking water with much lower concentrations than other by-products of potential concern. It should be noted that HAAs were investigated in few samples and no significant levels were found.

There are several factors affecting the formation potential of chlorination byproducts. Previous research studies have shown that the major variables that affect DBP formation are: residence time, temperature, pH, disinfectant type and concentration, and total organic carbon concentration.

Regulations of Chlorination By-Products

In the early 1970s, DBPs were first discovered to have harmful health effects to animals and humans. On November 29, 1979, the first legislation to limit the concentration of TTHMs in drinking waters was passed (U.S. EPA, 1979). This rule set a TTHM limit of 100 μg/l. The Stage I Disinfectants and Disinfection By–Product (D/DBP) Rule was promulgated by the U.S. EPA on December 16, 1998 (U.S. EPA, 1998). The Stage I D/DBP Rule addresses four main provisions: (1) lower TTHM limits; (2) contaminant limit for HAAs which had not yet been regulated; (3) maximum residual levels for four disinfectants; and (4) required removals of TOC based on source water quality. The rule affects all community water systems (CWSs) and nontransient-noncommunity water systems (NTNCWSs) that use a chemical disinfectant for any type of water treatment. The Stage I D/DBP Rule established maximum contaminant level goals (MCLGs) and maximum contaminant levels (MCLs) for TTHMs, HAA_5, chlorite and bromate. The MCL for TTHMs was set at 80 μg/L and the MCL for HAA_5 was set at 60 μg/L. Chlorite and bromate MCLs were set at 1,000 μg/L, and 10 μg/L, respectively. The MCLs for TTHM and HAA_5 compliance are based on a running annual arithmetic average that is formulated every quarter. The number of test sites in the distribution system is dependent on the size of community which the system is serving.

6.3 Laboratory Experiments for Parameter Identification

A number of laboratory experiments were conducted on drinking water samples collected from different points in AlAin water distribution system to determine the chlorine bulk reaction rate and to observe the formation of THM species under dynamic conditions in lab-controlled conditions (Temp.=21 °C). Figure 6.1 depicts the pipe-recirculation system used in observing the THM formation under dynamic conditions in which chlorine-spiked drinking water samples were pumped in three ductile iron pipes of different sizes for extended periods of times (24 hours) and considering spiking dosages of 5 to 20 mg/l. A number of samples were then taken from the system at different times and analyzed for THM species.

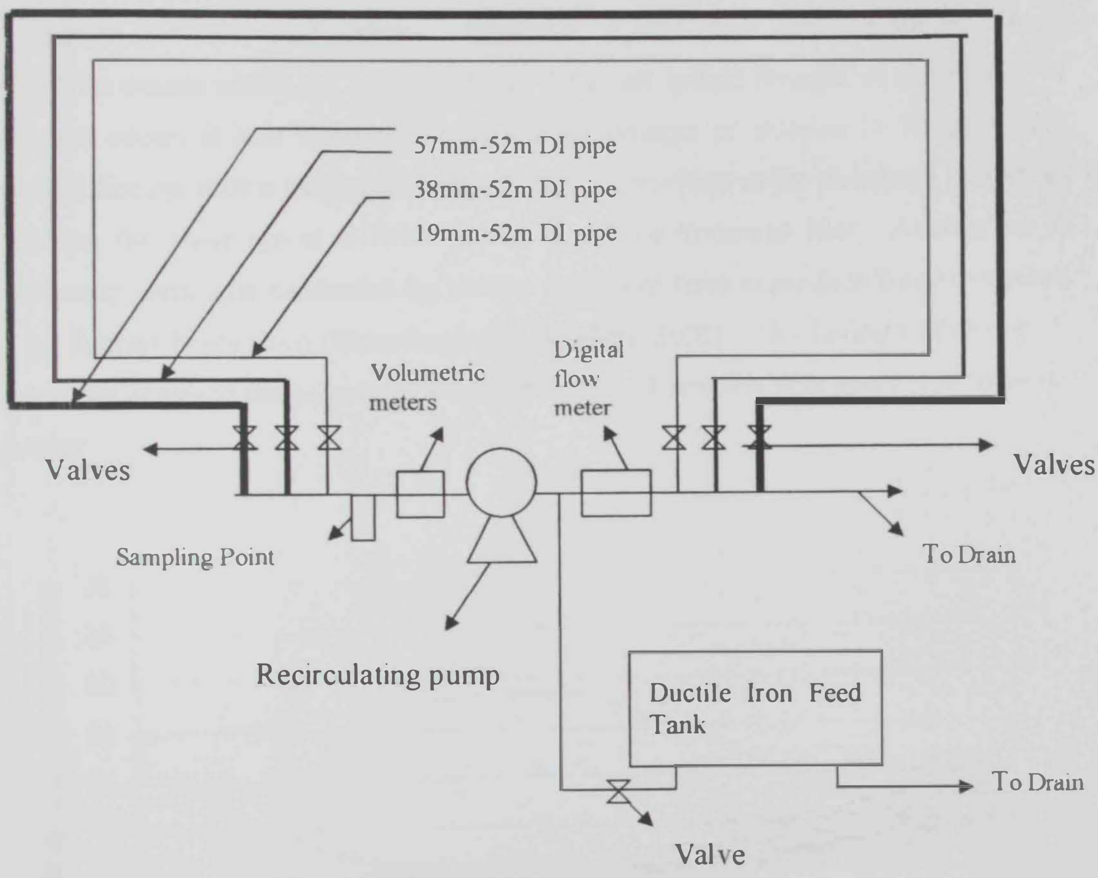


Figure 6.1 Experimental setting used in parameter identification (Elshorbagy and AlJaberi, 2006)

The results indicate that bromoform dominates the THM species in all samples and at all times. Dibromochlorofom is next but still with minor concentrations (less than 2 ppb in all tested samples). Same observations were reported in an earlier study (Elshorbagy et al., 2001) and attributed to high bromide levels in the raw sea water upstream the desalination plant (larger than 70 ppm). Therefore, the results reported here are only for bromoform. Opposite to most expectations, the bromoform was found to undergo major hydrolysis after some time in the recirculating pipes approaching zero levels. Figure 6.2 shows the bromform formation/decay with time in ten selected experiments. The observed hydrolysis phenomena was also noticed in an earlier study (Elshorbagy et al., 2001) in which the bromoform levels did significantly drop at distant locations from the production point. Bromforom formation is apparently reaching maximum levels at early times after spiking upon which the remaining natural organic matter (NOM) sharply depletes and the bromoform hydrolyses. Identifying growth or decay reaction rates was not feasible for such observations. However, it was clear that maximum formed bromoform occurs within the first two hours for small spiked dosages of chlorines (< 7 mg/l) and occurs at later hours for larger spiking dosages of chlorine (> 7mg/l). Such observed finding allows judging the peak formed bromoform in the simulated system by inspecting the water age at different points as will be discussed later. Another set of experiments were later conducted for shorter periods of time to produce kinetic relations for the formed bromoform (Elshorbagy and AlJaberi, 2006). The findings of this study support the observed phenomena of peak bromoform formation after short time followed by decay.

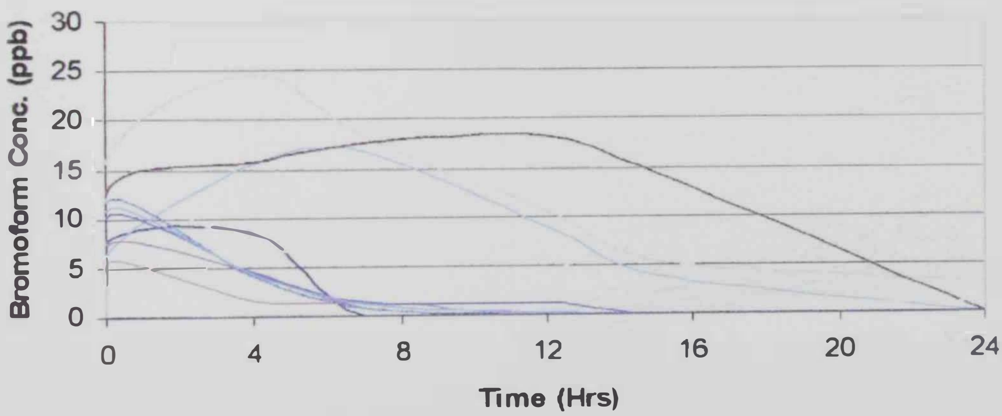


Figure 6.2 Bromoform formation/decay with time in different pipes

The above experiments involved identifying the residual chlorine concentrations in bulk water (samples from the still storage tank) at different times. Figure 6.3 shows the measured values plotted with time for all ten experiments. The set of measurements for each experiment were fitted against the first order decay equation in which the bulk reaction rate was obtained. K_b was found to range between 0.025 to 0.151 1/Hr in all experiments with an average of 0.088 1/hr. Using the Arhenius equation ($K_T = K_{20}\, \theta^{T-20}$) to estimate K_b at higher field temperature (32°C), θ=1.05, K_b is estimated as -0.152 1/Hr. This value was used as a basis or initial value in the numerical calibration process.

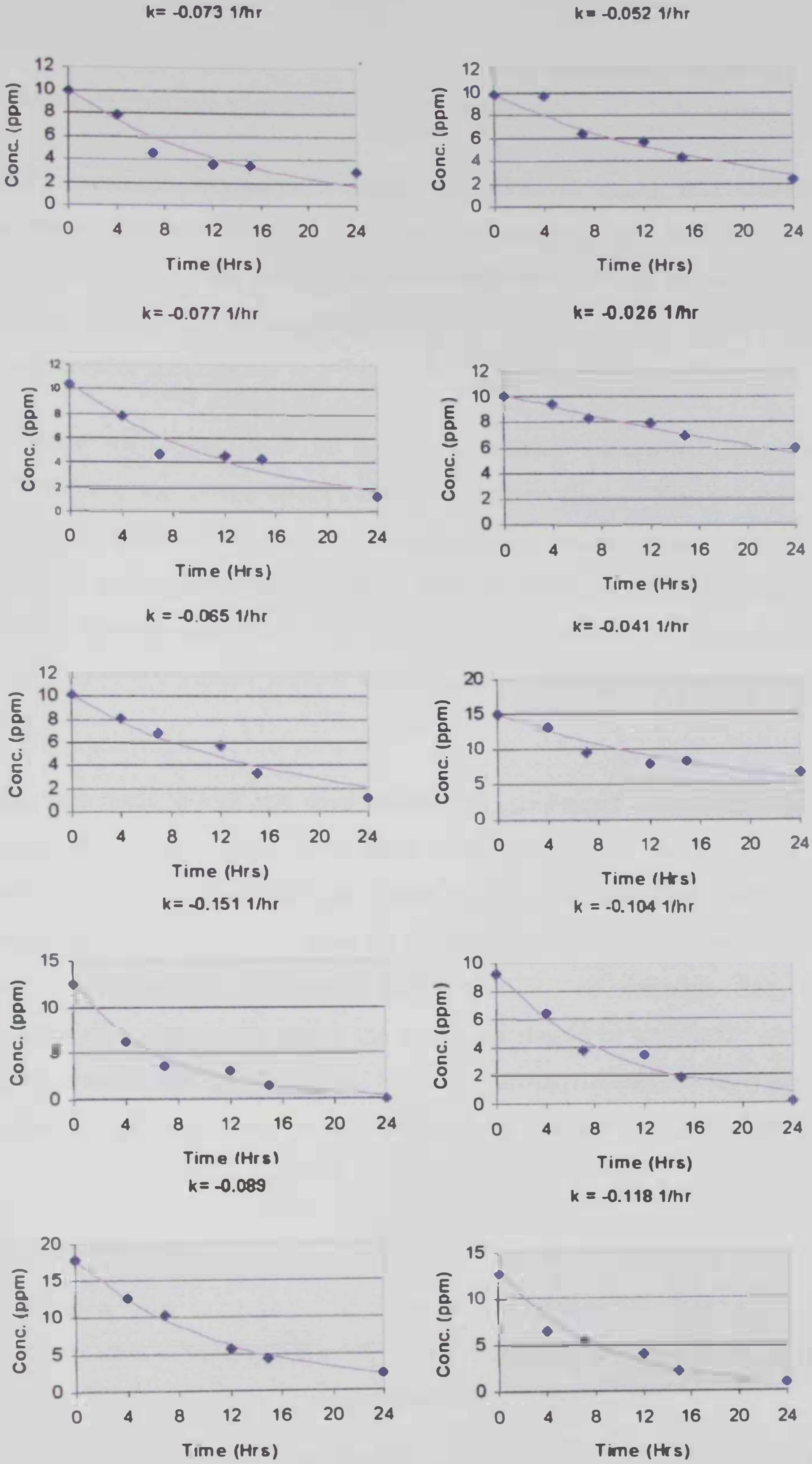


Figure 6.3 Experimental determination of bulk decay coefficient

6.4 Residual Chlorine Calibration and Simulation

Decay simulation conducted within WATERCAD takes into consideration the phenomena of chlorine reaction with chemical species at bulk fluid and with pipe walls. The contribution of bulk fluid is introduced into the software by means of a first order kinetics constant, K_b. The contribution of wall reactions is introduced into the software through another constant K_w.

To assure the correct use of the water quality simulator, incorporated in the WATERCAD, one must conduct a model calibration process. This consists of attributing the correct values to K_b and Kw coefficients. In most cases, model calibration is conducted, altering parameter values in order to obtain, in the model, values that match real ones. Nevertheless, ideal calibration must be conducted by taking samples of water and by studying pipelines nature. These samples allow predicting the real K_b and Kw values.

Within this study, a trial and error process was conducted to determine the K_b and Kw values of the water supplied to the three selected systems. The WATERCAD software models the chlorine decay through a first order kinetic law. This kinetic law takes the form of an equation which allows us to calculate the concentration of chlorine in the water, C, throughout the transportation time, t. To calculate this, the chlorine concentration at the source and at the beginning should be known for each system, C_o. The adjustment coefficients, K_b and Kw, are calculated based on knowing the residual chlorine in a certain location for the 3 selected areas as described in chapter 4.

6.4.1 Al Yahar Area

According to what is presented in Chapter 4 (Figure 4.12), it is noticeable that, the residual chlorine is varying from 0.25 – 0.3 which is considered in the acceptable range assigned by RSB and the international standards.

Hence, a quality simulation was done for Al Yahar system and the residual chlorine was modeled for some selected junctions and for some ISTs. The results are compared with the mentioned values and after numerous trails, the bulk reaction rate and the wall reaction rate are found: -0.05 1/hr and -0.012 ft/hr; respectively. These values correspond to the best attainable agreement between the measured and simulated values. The following graph (Figure 6.4) compares the simulated residual chlorine at J-41 that represents the monitoring location with the field measurements presented earlier in chapter 4.

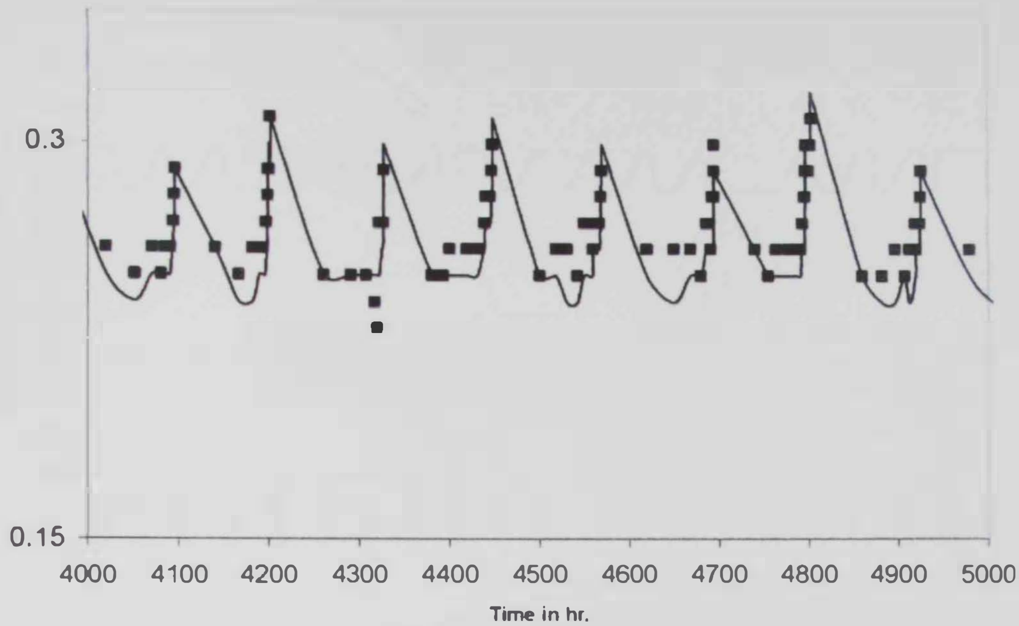


Figure 6.4 Al Yaher network – Simulated versus measured residual chorines at J-41

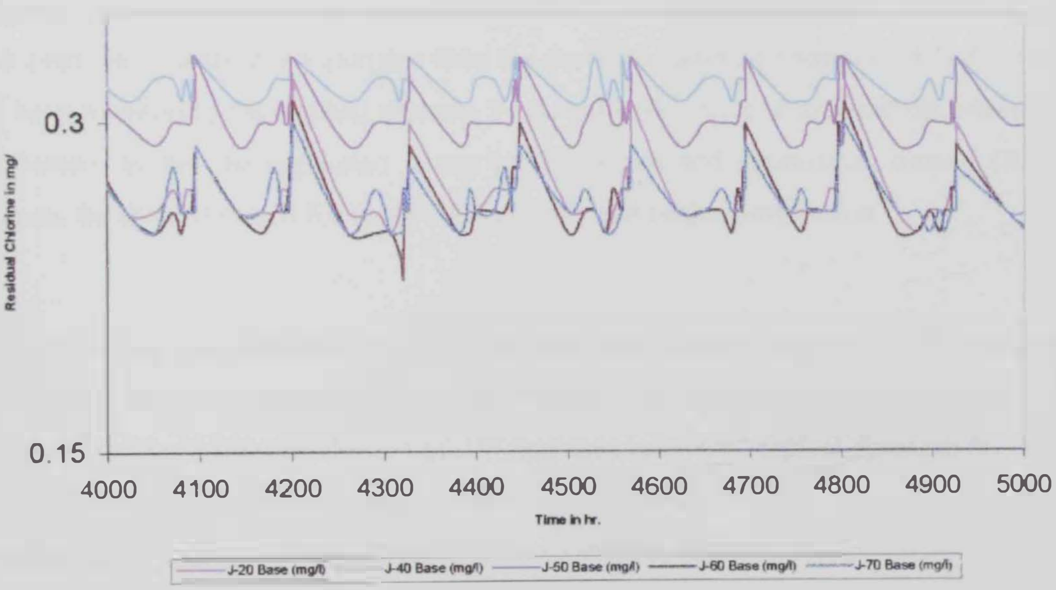


Figure 6.5 Al Yaher network – Calibrated case – Residual chlorine at some junctions

Simulation results at other locations (Figure 6.5) does also show a fluctuating cyclic behavior of residual chlorine. Yet, the entire fluctuating range in all junctions is found within the acceptable regulated range.

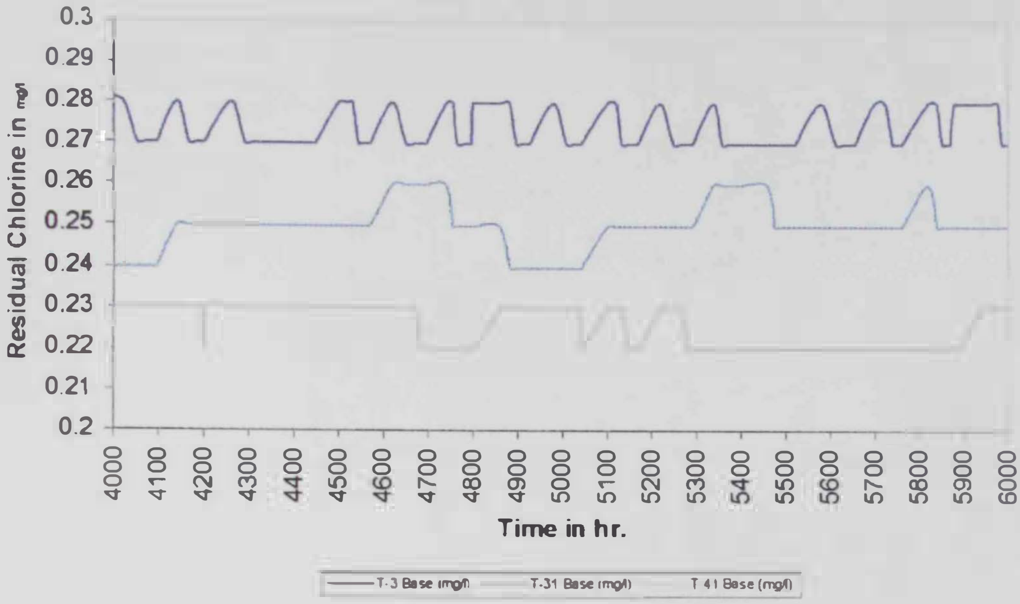


Figure 6.6 Al Yaher network- Calibrated Case- Residual chlorine in some selected ISTs

Figure 6.6 shows the simulated residual chlorine in some selected ISTs. In fact, it was not possible to analyze any samples from the customer tanks as a result of AADC policy to keep monitoring the residual chlorine in the network only and to avoid disturbing the customers as per the regulation issues set by record and supervision bureau (RSB). Hence, the showed results for the IST's may still need further verification.

A contour map was obtained from the simulation after about 6 months of the simulation in order to show the residual chlorine distribution over the whole network (Figure 6.7). Three spots located toward the end of AlYahar area have low residual chlorines less than 0.2 mg/l and even below 0.1 mg/l. However, these spots are few and cover tiny areas besides the duration of their occurrence are very short as reported in Figure 6.5.

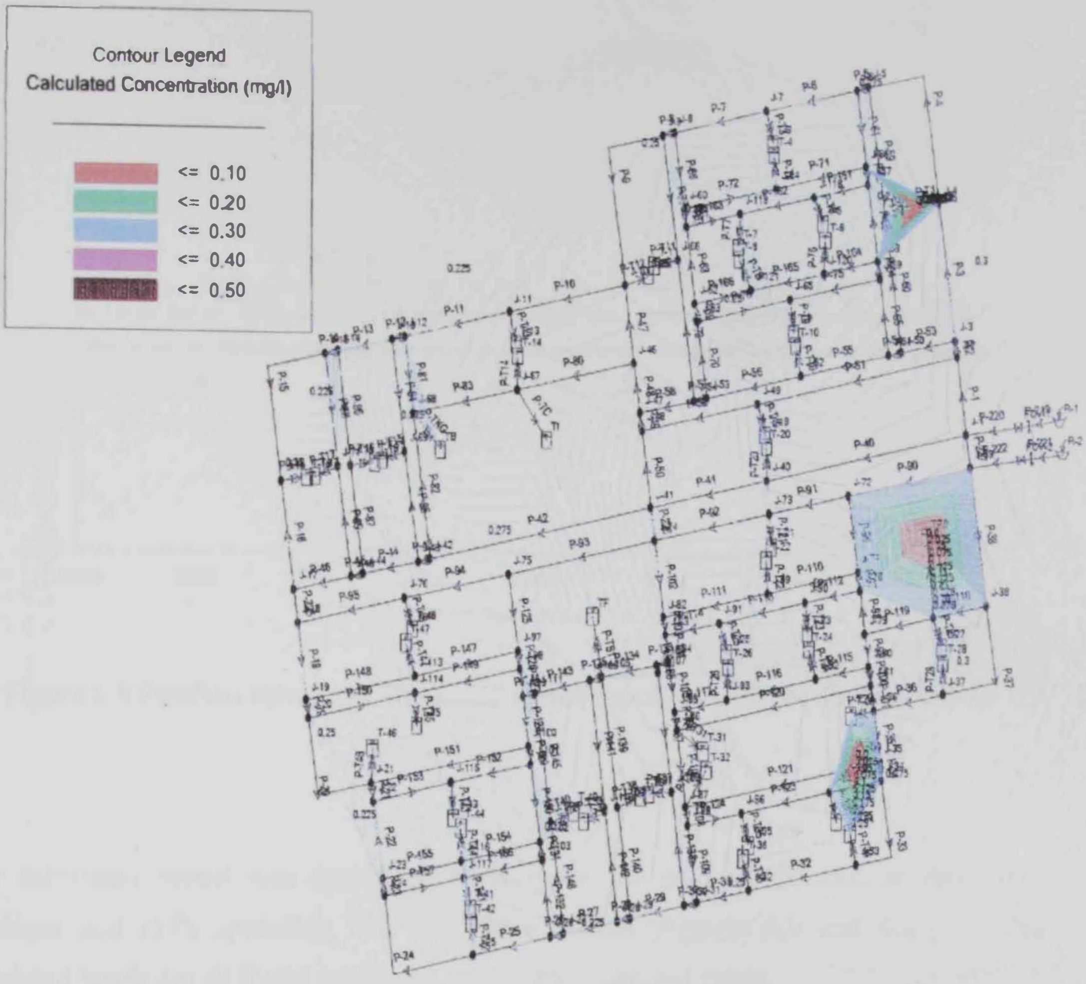


Figure 6.7 Contour Map for Al Yaher network – residual chlorine after calibration

6.4.2 Al Swiehan Area

A quality simulation was done for Al Sweihan system and the residual chlorine was modeled at the monitoring point. After many simulation trials, the best bulk reaction rate and the wall reaction rate were obtained: -0.07/hr and -0.023 ft/hr, respectively achieving the best agreement between the simulated and measured values at the monitoring point (Figure 6.8).

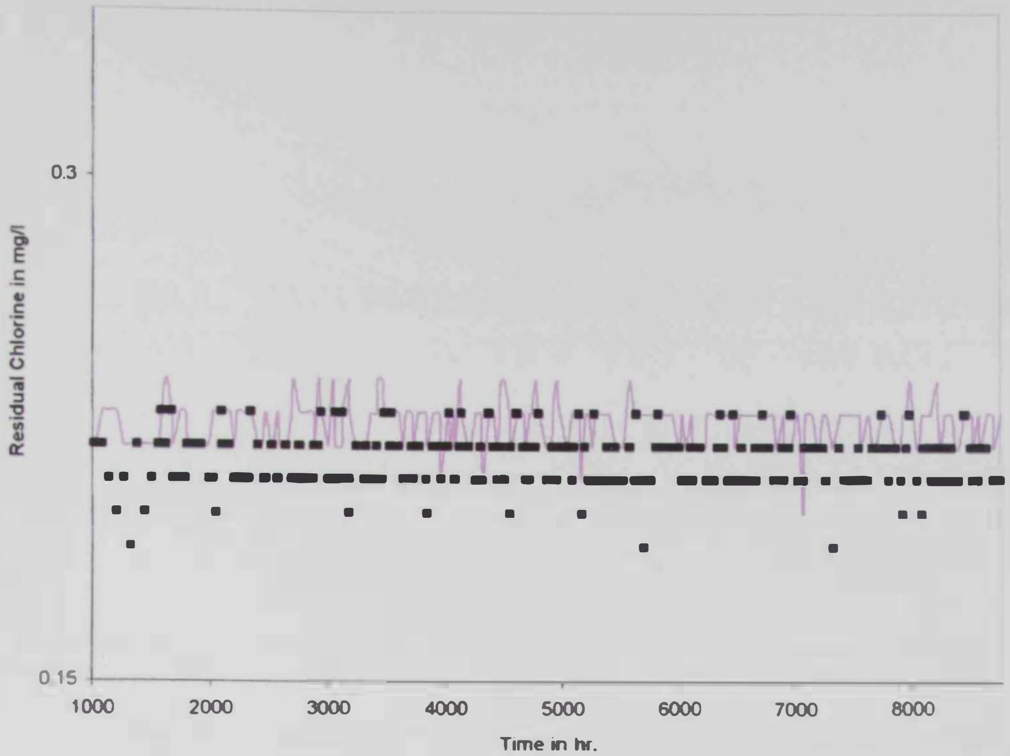


Figure 6.8 Sweihan network – Simulated versus measured residual chorine at J-10

The calibrated model was then used to simulate the residual chlorine at few other junctions and ISTs spreading over the entire system (Figures 6.9 and 6.10). The simulated levels are all found within the acceptable regulated range.

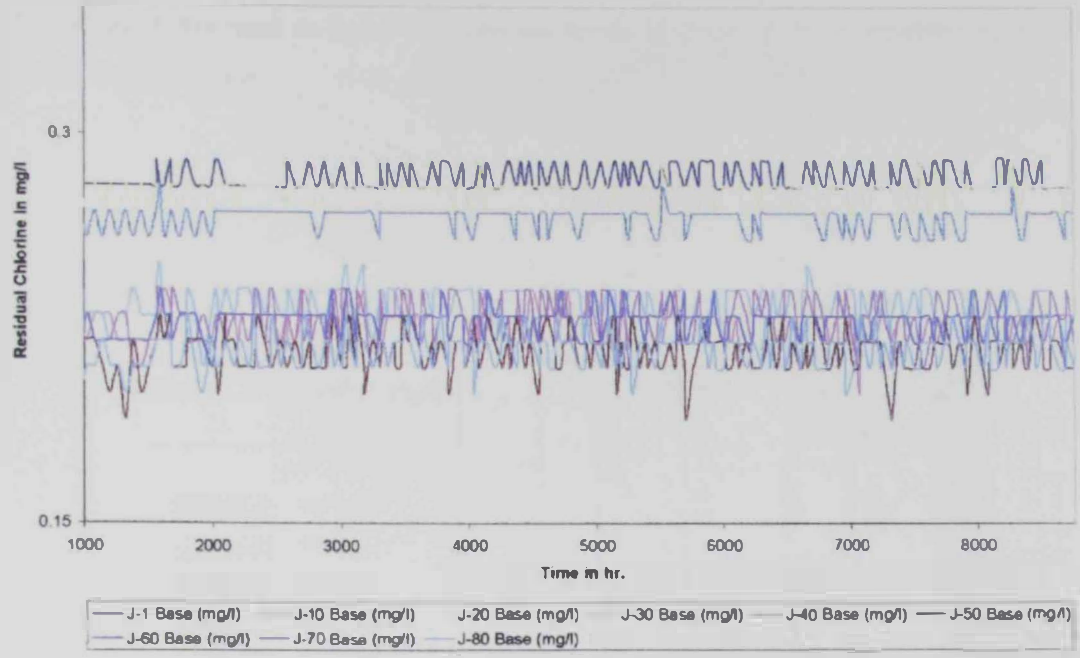


Figure 6.9 Sweihan Network – Calibrated case – Residual chlorine at some junctions

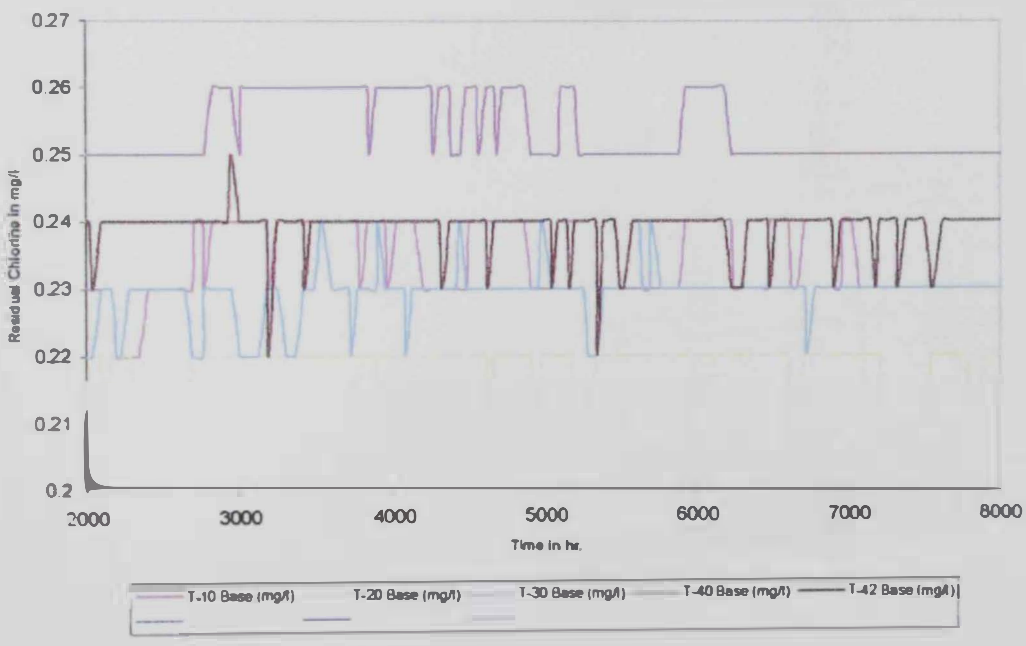


Figure 6.10 Sweihan network – Calibrated Case - Residual chlorine in some selected ISTs

A contour map was also obtained from the simulation model after about 6 months to report the spatial distribution of residual chlorine overall the whole network (Figure 6.11) The distribution depicts wider areas of low chlorine levels (<0.20 mg/l and <0.1 mg/l). This suggests the need to boost the chlorine levels in these spots by considering local booster injection units.

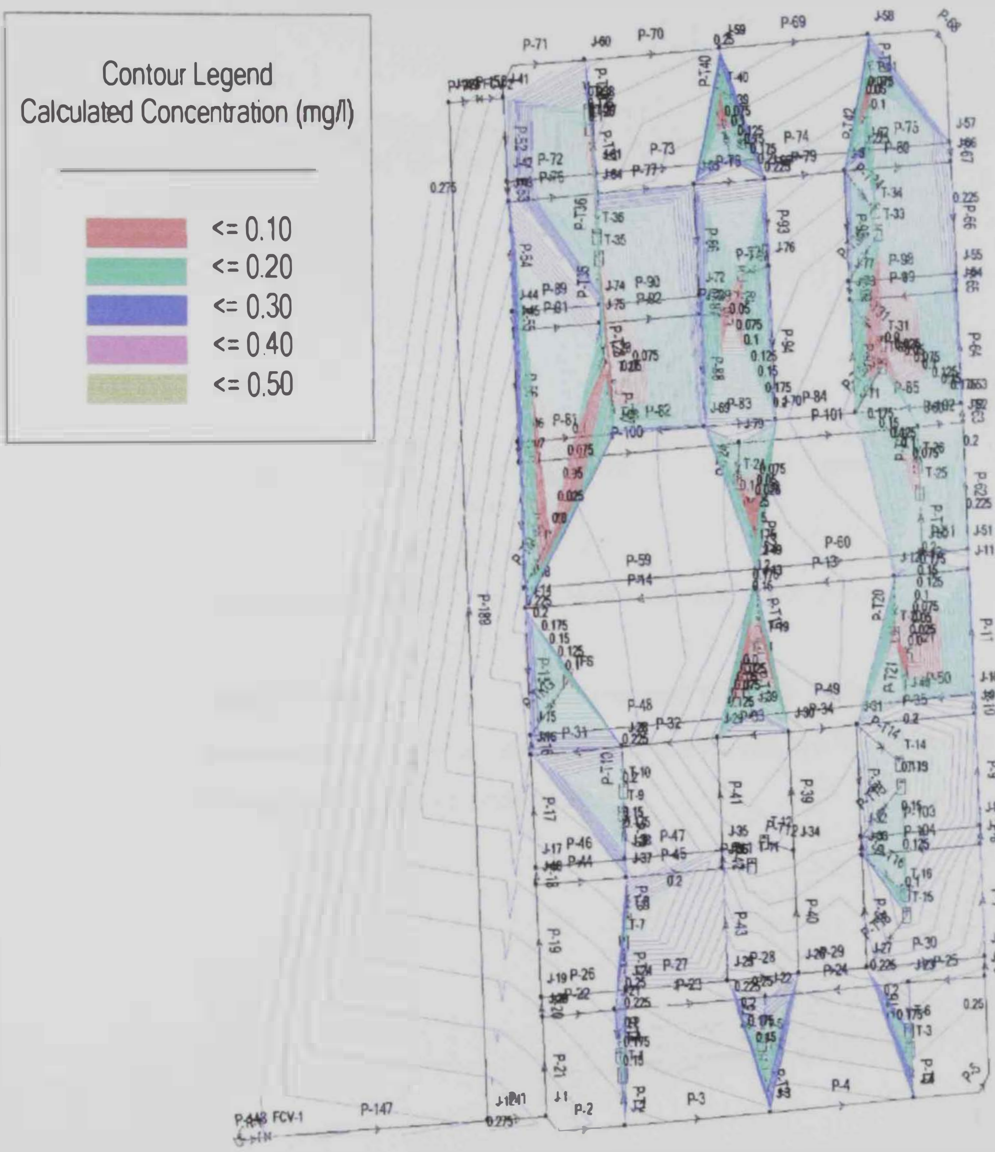


Figure 6.11 Contour map for Sweihan network area – residual chlorine after calibration

6.4.3 For Al Dhaher Area

Quality simulation was done for Al Dhaher system and the residual chlorine was modeled for few selected junctions and ISTs. Following many trails, the bulk reaction rate and the wall reaction rate were obtained as -0.065 /hr and -0.019 ft/hr, respectively. Figure 6.12 compares the simulated residual chlorine with the field measurements at J-20 that represents the installation location of the data logger.

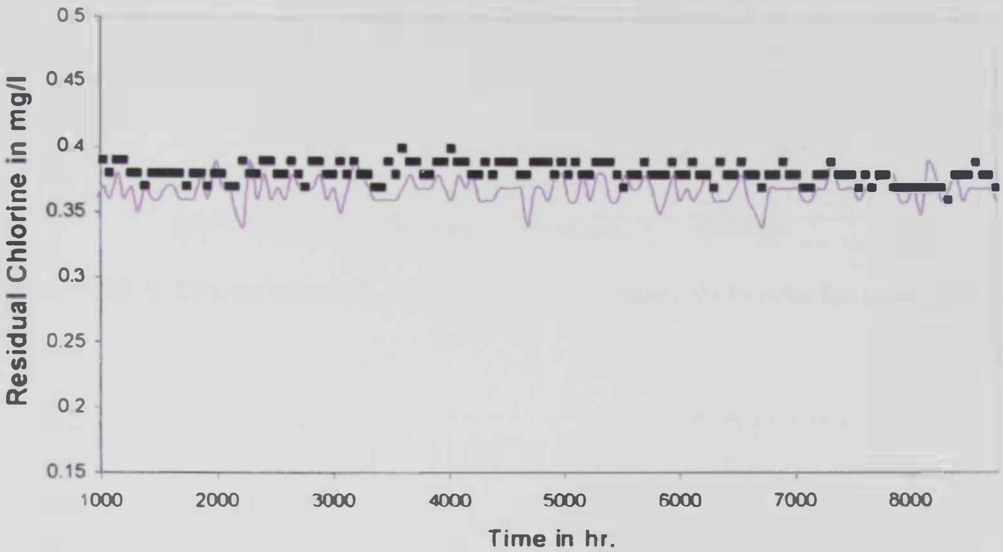


Figure 6.12 Al Dhaher network – Simulated versus measured residual chorine at J-20

As before, Figures 6.13 and 6.14 depict the simulated residual chlorine at few selected junctions and ISTs distributed over Al Dhaher network and the results are found within the acceptable limits described by the adapted code in AlAin

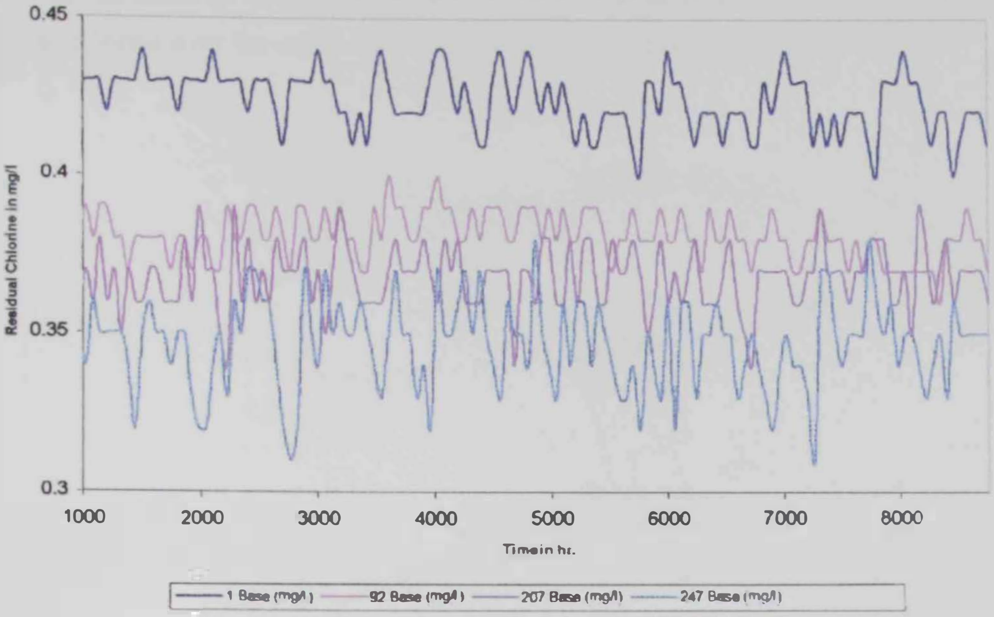


Figure 6.13 Al Dhaher network – Calibrated case – residual chlorine for some junctions

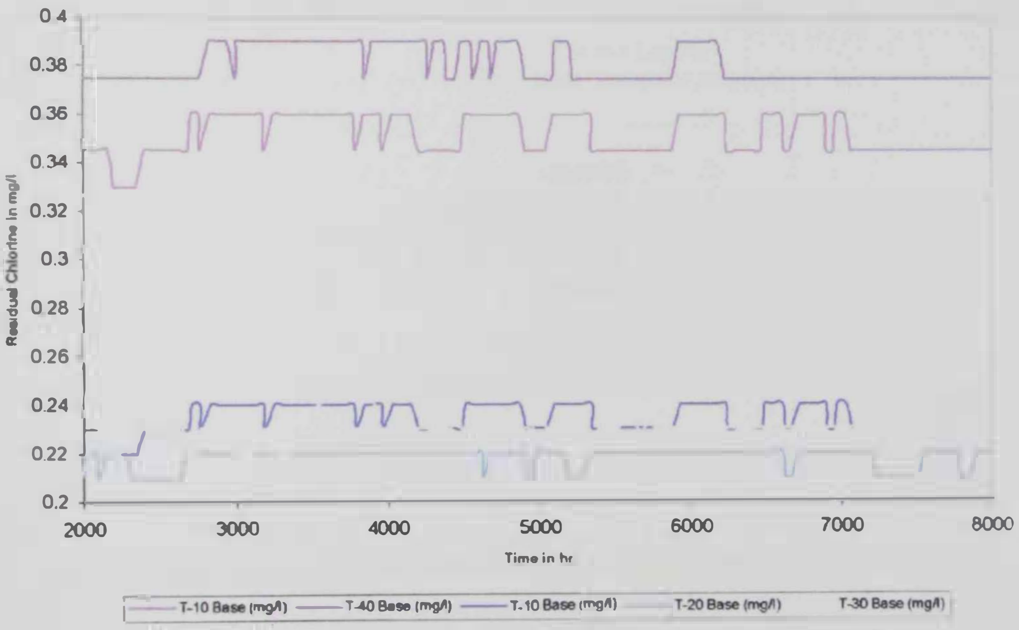


Figure 6.14 AL Dhaher network – Calibrated case- Simulated residual chlorine in some selected IST's (As a results of the calibrated Junctions)

The 6-month contour map obtained for Al Dhaher area (Figure 6.15) shows that the residual chlorine over the entire network and reports no spots of low chlorine levels.

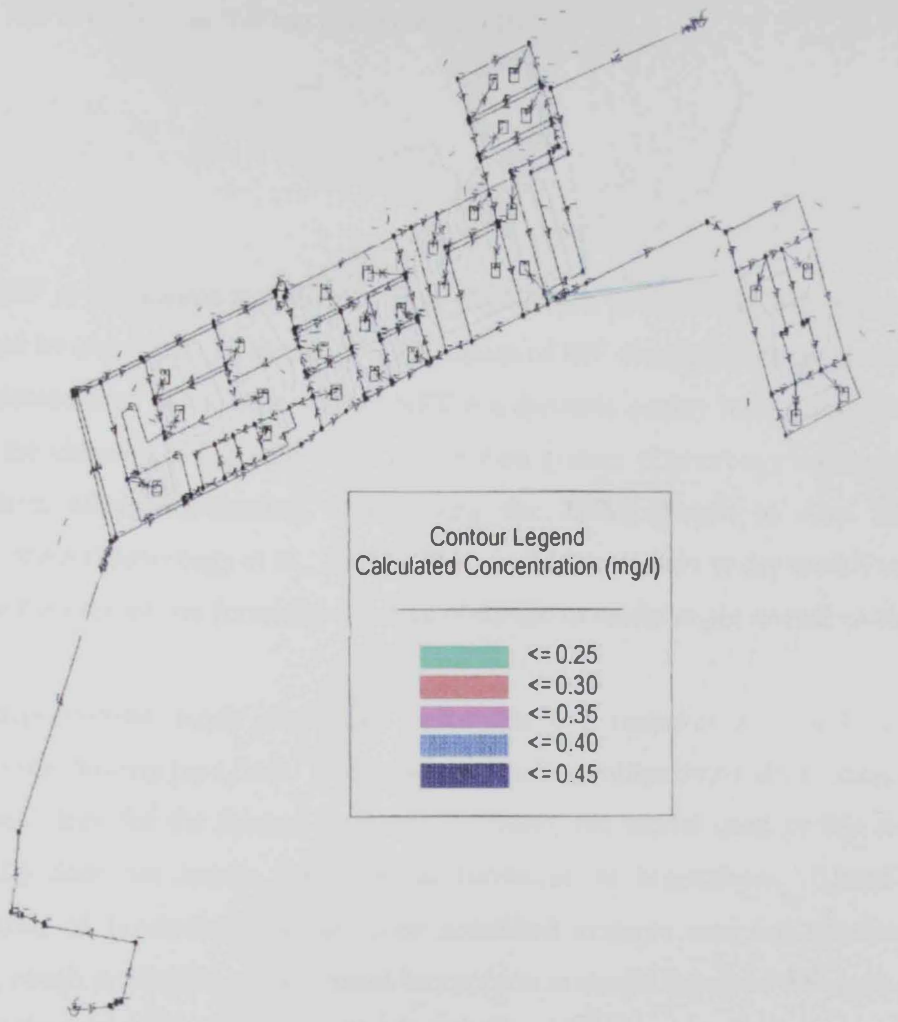


Figure 6.15 Contour map for AL Dhaher area – residual chlorine after calibration.

6.5 THM Formation in the Simulated Systems

Current modeling tools in water distribution systems consider either simple first-order growth reactions or limited (saturated) growth reactions of different substances including THMs. THM formation potential (THMFP) is also used to be modeled to represent the upper limit of formed THM in the distribution system. None of the current models consider formation of THM species nor consider non-simple formation kinetics. A recent study (Elshorbagy, 2000) models the kinetics of the THM species under representative extreme conditions employing the site-specific quality trends with stoichiometric expressions based on an average representative bromine content factor (BIF) where BIF is defined as follows (Gould et al. (1981):

$$BIF = \frac{\sum X[CHCl_{(3-X)}Br_X]}{\sum[CHCl_{(3-X)}Br_X]}$$

An average BIF at the source is estimated first. Significant variations in BIF with time can be handled by considering constant average values of BIF discretized on a reasonable number of selected time increments. QUALNET is a dynamic quality model used earlier in modeling the chlorine in a brushy-plain distribution system (Elshorbagy and Lansey, 1994) and later after modification in modeling the THM species in Abu Dhabi distribution system (Elshorbagy et al., 2001). This model is currently under modification to account for the bromoform formation kinetics observed in recent experimental work.

Conducted experimental work discussed in section 6.1 revealed a hydrolysis of bromoform in the flowing pipe lines. The reported kinetic profiles didn't allow obtaining simple reaction rates for the formed bromoform. Also, the model used in this study (WATERCAD) does not handle the observed formation of bromoform. Therefore, explicit tracking of bromoform in the three simulated systems was not conducted. Alternatively, rough prediction of the formed bromoform is sought based on the water age obtained from the calibrated simulation models.

Figure 6.16 shows the water ages in the three simulated systems at two locations. The first location represents the nearest location from the feeding mains, the second represents the monitoring point at which THM measurements are available while the third point represents the most distant point in each network with respect to the feeding source that capture the largest water age expected in the system during typical operation. The figure shows that the water ages at the monitoring points in all three systems range from 2 to 5 hours. This indicates that the maximum bromoforms are likely formed at points earlier than the monitoring points. Since the measured levels of bromoform were much less than the upper regulated level (30 ppb versus 80 ppb), the bromoform levels in the network potions upstream the monitoring point will unlikely exceed the regulated levels. For the areas downstream the monitoring points, representing the larger area of each system, the bromoform levels are expected to be less than the observed levels.

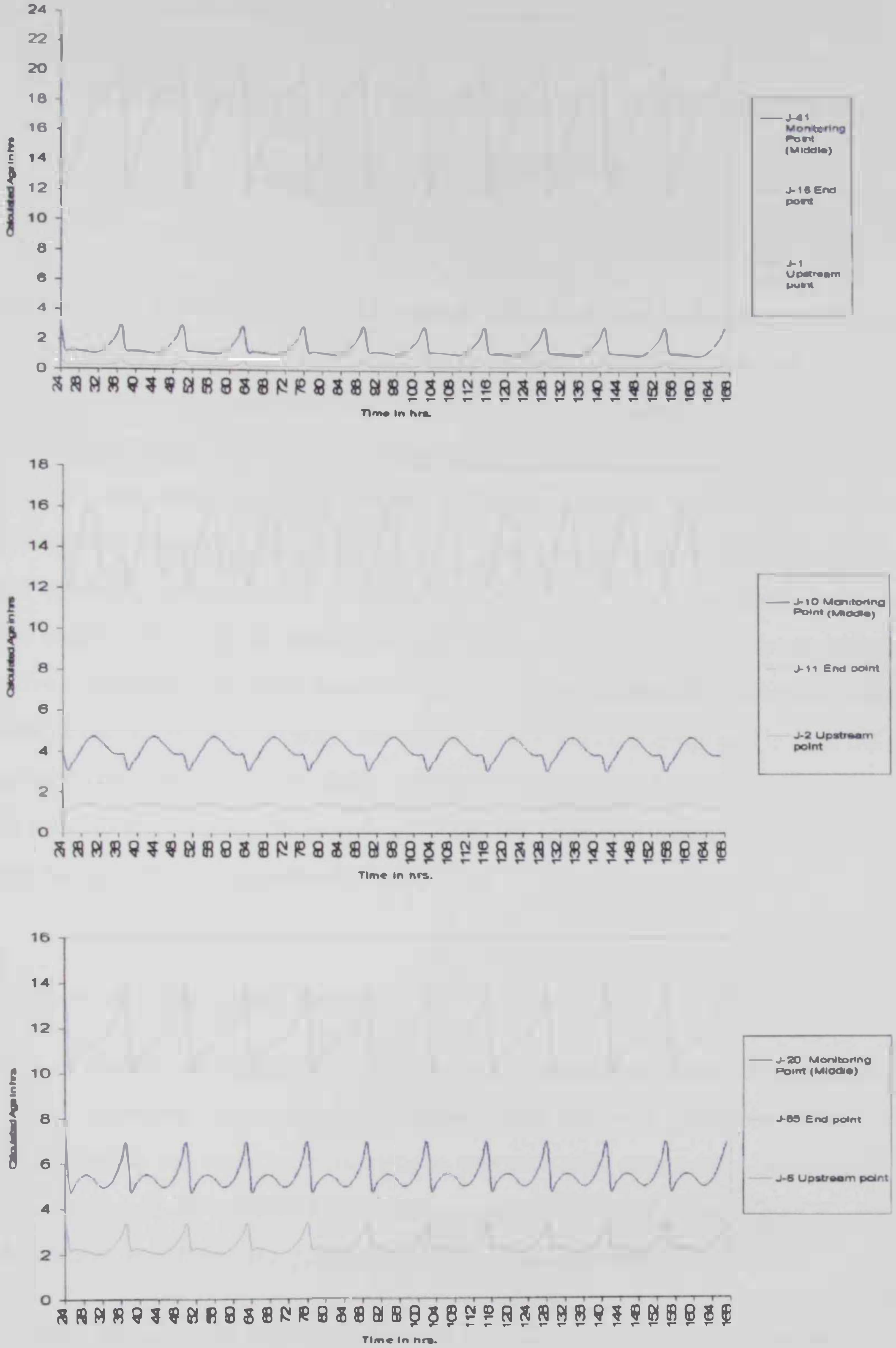


Figure 6.16 Water Age simulated for AL Yaher, Al Sweihan, and Al Dhaher systems, respectively

CHAPTER 7

PERFORMANCE OF ALTERNATIVE SYSTEM MODIFICATIONS

In chapters 5 and 6, the three selected systems were simulated and calibrated. Some of the results obtained from the conducted hydraulic modeling for Al Yahar and Sweihan areas were not accepted in comparison with the regulations presented in the water distribution code. The major tested parameters included residual pressure, the velocities, and the residual chlorine. In this chapter, the advantages of having elevated tanks in water distribution systems are first presented. One alternative is proposed to improve the performance of the selected systems, especially the mentioned parameters. This alternative considers the elevated tanks at the entrance of the system. The energy cost is calculated for the proposed alternative. Investigations were done for locating the elevated tank at different locations in particular at the end points. Finally, the system performance is evaluated in case the demand management program is successfully applied in Al Ain. This will allow the current consumption to significantly drop and approach the estimated demands allowing removing the large number of existing internal storage tanks.

7.1 Storage and Elevated Tank

Most people are neutral about storage tanks, considering them an acceptable part of the scenery. However, some individuals believe that tanks are unsightly and detract from views. Still others see tanks as a resource to promote their community or business. Tanks should be designed to satisfy the aesthetic considerations of stakeholders to the extent possible without sacrificing the purpose of the tank and efficiency of the system's operation.

Water storage is an extremely important element in a water distribution system. The principal advantages of distribution storage is that it can moderate the demands placed on the major supply sources, production works as well as major transmission mains. As a result, the sizes and/or capacities of each of these distribution system elements may be reduced. Additionally, distribution storage normally results in stabilized system pressures. Reserve supplies of water in distribution storage also provide a redundant source of water during emergencies, such as fires; water main breaks, and pump station power outages. The major purposes of storage are summarized as:

- Operational equalization (balancing storage)
- Fire flow provision
- Emergency needs

There are two types of distribution storage. These are in-ground reservoirs and elevated storage tanks. Generally, elevated storage tanks have been limited to a volume of approximately 9.0ML (Walski, 1998), whereas ground reservoirs are not limited in volume. Ground reservoirs may utilize either a direct pumping system (which requires a pumping system to pump water from the reservoir to system pressures) or an indirect pumping system (water "floats" on the system and flows by gravity from the tank to system pressures). Elevated storage tanks are virtually always indirect systems and float at the connected system HGL.

The indirect pumping system (with floating or elevated storage) has several advantages over a direct pumping system in a water distribution system among which:

- Lowers the peak pumping rates
- Stabilizes the pressure variations as demands fluctuate
- Maintains constant and reliable water supply and pressures (Figure 7.A)
- Increases operational flexibility, efficiency and convenience
- Balances and levels the pump operations
- Reduces the need for wide range of pump sizes
- Decreases the power costs – particularly for "time-of-day" energy pricing
- Provide an immediate emergency response (main break, power failure)
- Provide an immediate fire flow and pressure response
- Surge relief – dampens extreme low and high low pressures associated with hydraulic transients
- Ensures constant system pressure and helps prevent contamination from possible inflow

Based on the above, it is thus evident that distribution system storage is extremely beneficial to a water supply system. In particular, storage with indirect pumping (i.e. floating storage) provides clear advantages to the operation of the system and should thus be considered whenever and wherever practical and feasible.

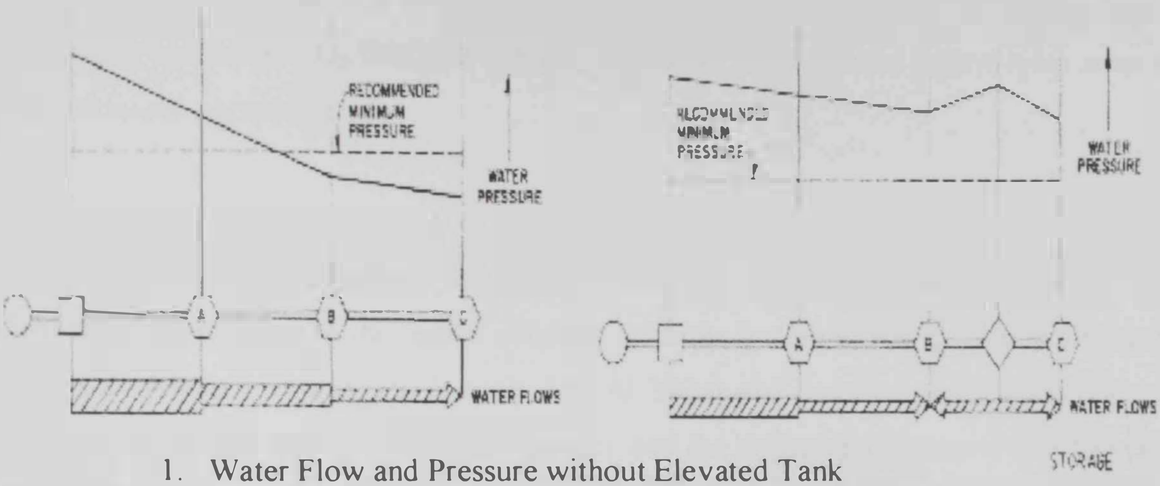


Figure 7.A

1. Water Flow and Pressure without Elevated Tank
2. Water Flow and Pressure without Elevated Tank

Water Source
Pump Station and storage
Demand Load Center
Elevated Tank

The major disadvantage with any distribution storage is the possibility of increased travel time from the source (chlorination point) “to the tap”. Ensuring adequate replenishment of water in storage facilities is a key factor in minimizing the potential chorine decay in the treated water. Two factors may be considered to improve this ensuring good mixing of fresh water with stored water in the tank and ensuring that the system is operated to maximize the replenishment of fresh water in the tank. It is noted that in-tank mixing is preferred for smaller tank volumes – “plug” flow may be preferred to ensure adequate water turnover in larger tanks.

Elevated storage tanks should be located in the areas having the lowest system pressures during intervals of high water use to be effective in maintaining adequate system pressures and flows during periods of peak water demand. These are those of greatest water demand or those farthest from pump stations. Elevated tanks are generally located at some distance from the pump station(s) serving a distribution pressure level, but not outside the boundaries of the service area, unless the facility can be placed on a nearby hill. Additional considerations for siting of elevated storage are conditions of terrain, suitability of subsurface soil and/or rock for foundation purposes, and hazards to low-flying aircraft. Elevated tanks are built on high ground to minimize the required construction cost and heights. The heights of elevated tanks should not be excessive. This will lead to higher pressures than required in low-lying areas as well as increased pumping costs.

The following alternatives are applied on some selected areas to enhance their performance and to improve the results obtained from the hydraulic and quality modeling of the original systems. This option is applied to Al Yahar and Sweihan areas but not for Al Dhaher area since this area is supplied by gravity and the system performance was accepted as the residual pressures for different junctions were more than the minimum accepted limit.

7.2 Elevated Tank Alternative

The next section evaluates the use of elevated tanks at the entrance of the systems. Such alternative showed major improvement in the pressures. More details for each particular system is discussed and analyzed as following.

7.2.1 Al Yahar Area

Due to the illustrated benefits of the elevated tank, an elevated tank is proposed for Al Yahar network, and located at the entrance close to the feeding point. The reason behind this selection is that the residual pressure provided by the source can be utilized in enhancing the pressure and the performance of the whole network. The tank capacity is selected to handle one day demand for Al Yahar area (0.93 MGD) to take advantage of securing the supply.

Consequently, the hydraulic analysis was done The proposed tank is elevated 20 m above the average elevation of the network with an operating range of water levels of 10 m

Figure 7.1 shows the pressures resolved from the hydraulic modeling taking into consideration that the whole area will be fed from the elevated tank directly

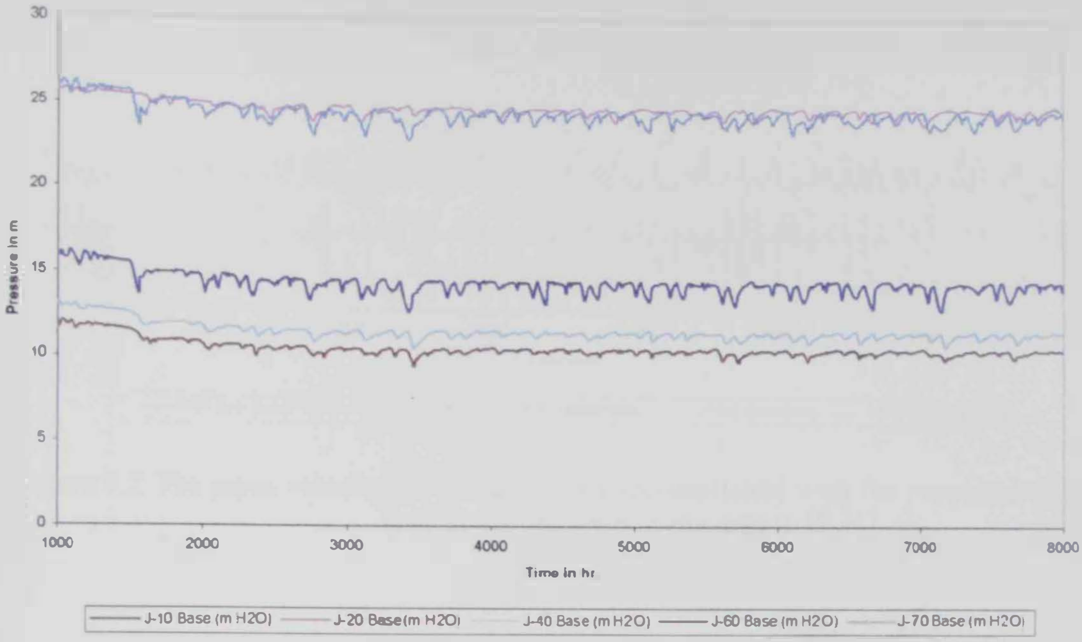


Figure 7.1 The residual pressure of Al Yahar area accomplished with the proposed elevated tank at the entrance of the area

From the figure, it is clear that the residual pressures are within the range of 10 - 25 m in many junctions with much less fluctuation than before. For the lowest points (J-20 and J-70) the residual pressure becomes around 25 m, and for the highest point in the area (J-60) the residual pressure is around 10 m. Both pressures are accepted by the water distribution code

On the other hand, the range of the velocity has been reduced on contrary to the range associated with the original setting (displayed in Figure 5.10). Figure 7 2 illustrates the velocities, where the maximum value is around 4.2 m/s. This high velocity is due to the relatively high demand required for the farm associated with the base configuration and as before, it can be tolerated as this is the only point in the system with that high demand Alternatively, the size of the connection from the network to this farm should be increased or another connection can be installed

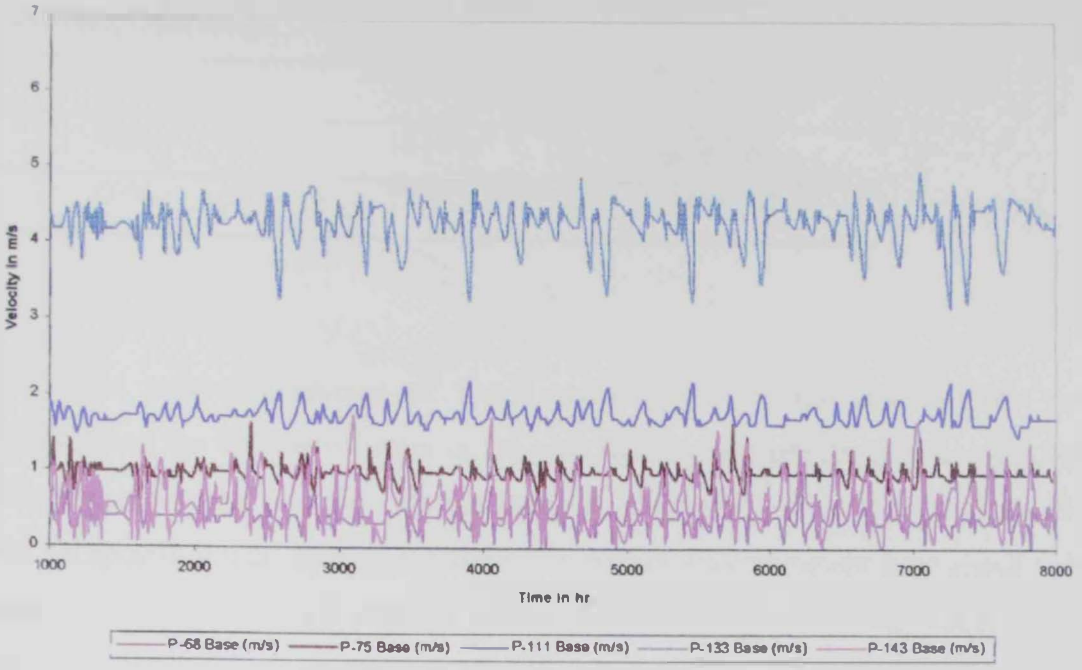


Figure 7.2 The pipes velocity of Al Yahar area accomplished with the proposed elevated tank at the entrance of the area

Tank operational levels for optimum Internal Mixing

To improve mixing of fresh water with "old" water inside the tank bowl, a two-riser pipe system with check valves has been incorporated in the simulation. The check valves ensure the tank fills through only one of the risers and is drawn down through the second riser pipe. The top of the "fill" riser pipe is angled to promote circulation in the tank and minimize the opportunity for "shortcircuiting" of flow in the bowl.

A range of normal operating volumes has been considered, varying from 20% to 60% of tank usage on a typical day (40% is generally considered normal for Al Yahar area). The volumes utilized in the tank for different operating percentages and simulated by the hydraulic model are shown below.

Daily balancing/Operating	Volume Used	Volume Remaining
20%	909 m^3	3636 m^3
40%	1818 m^3	2727 m^3
60%	2727 m^3	1818 m^3

The analysis seeks the amount of water remaining in storage (for different operating conditions) and the volume of water in the pipe section between the tank and the last water connection so that the residual chlorine levels in the tank as well as in near junctions are within acceptable limits. Such calculation determines approximately how much volume in the tank is refilled with fresh water on a daily basis, as opposed to "old" water ("old" water being the un-used volume in the tank plus the amount in the final pipe section between the tank and the last connection, which equals the total amount of "old" water in the tank after refilling).

Tank connecting pipe is 300 mm line with a length of 2000 m, from the main source supplying Al Yahar area. As the water quantity filling this line is 141.4 m^3, the amounts of fresh water getting in the tank are as follows.

Daily Balancing Use Tank (daily)	Fresh Water Volume	Fresh Water In
20%	909 – 141 = 768 m^3	16.9 %
40%	1818 – 141 = 1677 m^3	36.9 %
60%	2727 - 141 = 2586 m^3	56.9 %

The impact of varying amounts of "fresh" water in the tank related to the daily used amounts of storage on the water quality is reflected by the plots of residual chlorine in the network junctions. These plots, which are based on a chlorine decay rate illustrated in chapter 5, are shown in Figure 7.3 and suggest that maintain reasonable chlorine residual in the tank, the tank must be operated with more than 40% of the total volume being used on a daily basis.

As expected, the more water that is moved through the tank in a single day, the higher the chlorine residual in the tank.

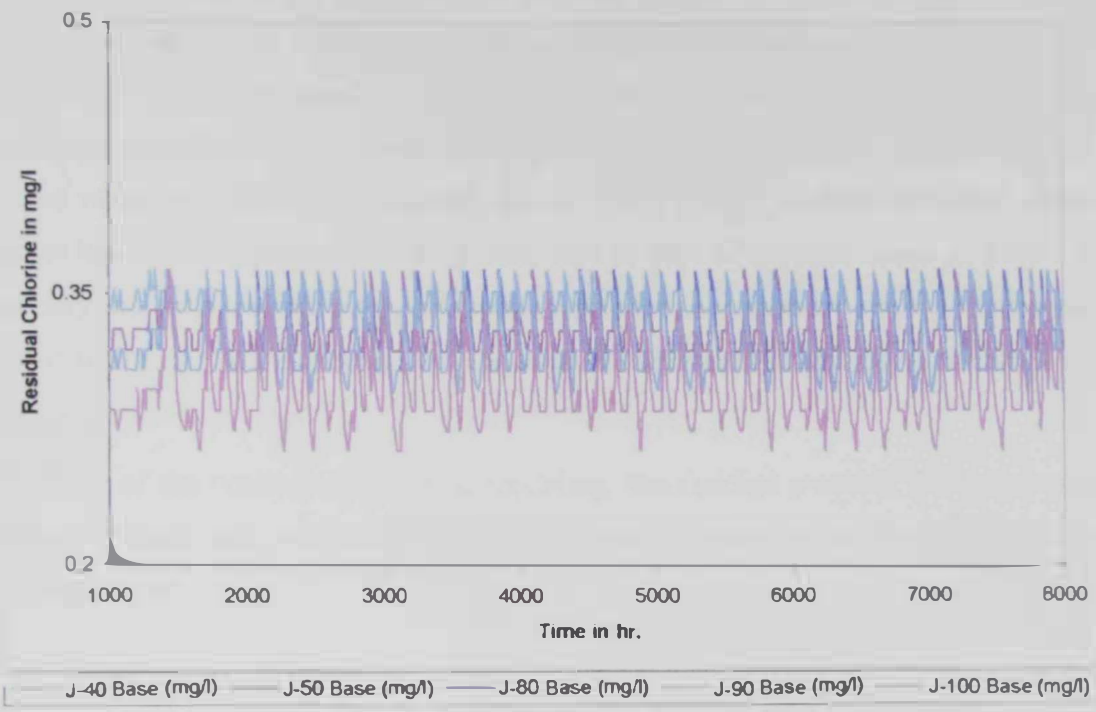


Figure 7.3 The residual chlorine of Al Yahar area accomplished with the proposed elevated tank at the entrance of the area

The energy cost for this alternative was calculated for one year and found 8760 Dhs

7.2.2 Swiehan Area

The same analysis is conducted for Swiehan area, where the elevated tank is proposed at the entrance of the area to get advantage of the inlet pressure addressed in chapter 4. The proposed tank has a total volume of 0.6 MG or 2727 m^3 to accommodate a minimum of one day demand for the area since its demand is 0.53 MGD. The tank elevation is 20 m above the average elevations of the whole network to also accommodate the high pressure at the entrance which is 4 bars. As proposed for Al Yahar area, a number of normal operating volumes have been considered, varying from 20% to 60% of the tank usage on a typical day to identify the optimum operating volume associated with maximum mixing. The proposed elevated tank for Swiehan area needs to be operated with more than 35% of the total volume being used on a daily basis.

As a result of the conducted hydraulic modeling, the residual pressure for the network is significantly improved, where the minimum pressure is accepted by the water distribution code (Figure 7.4).

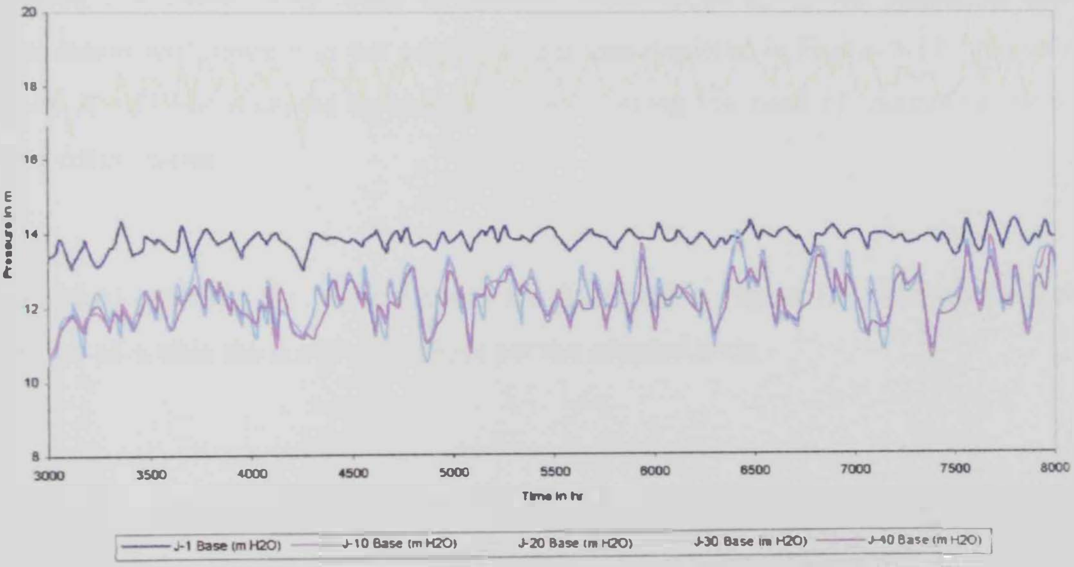


Figure 7.4 The residual pressure of Sweihan area accomplished with the proposed elevated tank at the entrance of the area

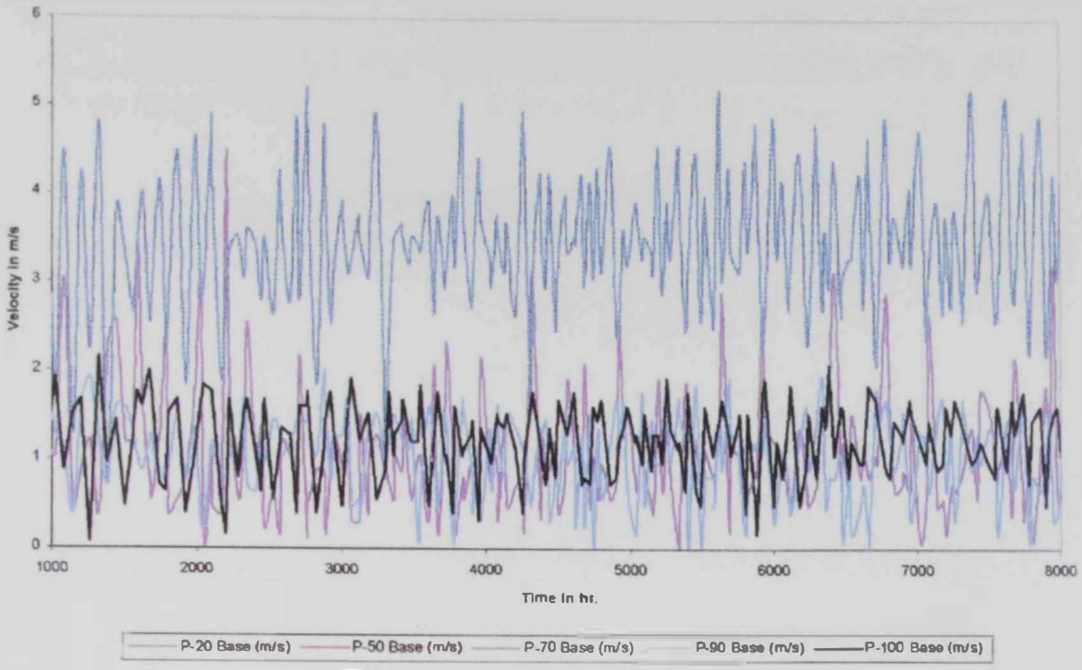


Figure 7.5 The pipes velocity of Swiehan area accomplished with the proposed elevated tank at the entrance of the area

In Figure 7.5, the velocity range is bounded from above by 4 m/s indicating noticeable improvement with respect to the original conditions depicted in Figure 5.17. However, this velocity is still not accepted by the regulator implying the need of increasing the sizes of same existing pipes.

The residual chlorines for selected nodes are illustrated in Figure 7.6. The resolved chlorine levels are all within the acceptable limits per the adopted code.

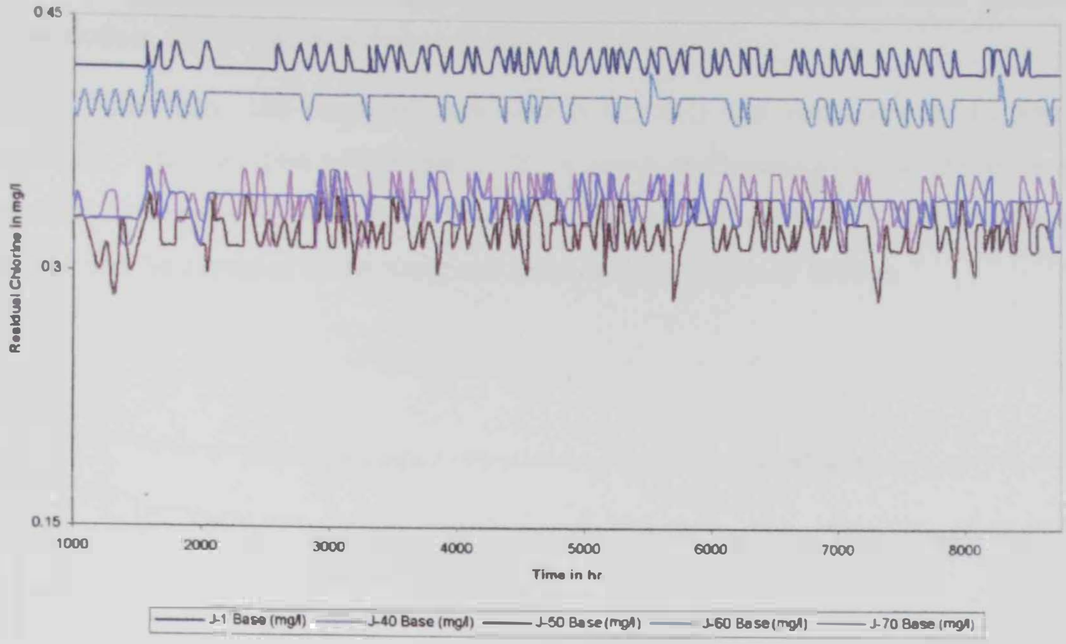


Figure 7.6 The residual chlorine of Sweihan area accomplished with the proposed elevated tank at the entrance of the area

The energy cost of this alternative was calculated and found 6310 Dhs.

7.2.3 Elevated tank at other points

Several other options and alternatives can be investigated and proposed to enhance the performance of the two considered networks. Sections 7.2.1 and 7.2.2 discussed the option of proposing the elevated tank at the entrance of AlYahr and Sweihan networks. This section discusses the analysis of the two networks after proposing an elevated storage tank either at the lowest point of the network or near the higher demand portion of the network. Since Swiehan area is flat with no elevation differences between the source and any point of the network, the proposed elevated tank may be located at the higher demand portion of the network (Shabia's area). For Al Yahar network, the proposed elevated tank is suggested to be located in the lowest portion of the network. Both points are located toward the end distant points with respect to the feeding source of each system.

Trying different elevations of the proposed elevated tanks at these locations, only parts of both systems were found to be fed such tanks while the remaining parts of the networks will be fed from current sources. As a result, the sizes of both tanks were considered to accommodate part of the total demands only for both areas

For Al Yahar area, the proposed tank size is 0.5 MG that can serve up to 50% of the customers. The elevation of the tank is 20 m above the lowest junction elevation and it is proposed to be supplied from one of the identical existing sources i.e. 300 mm line, where this line will be extended to the proposed location for a length of 1800 m.

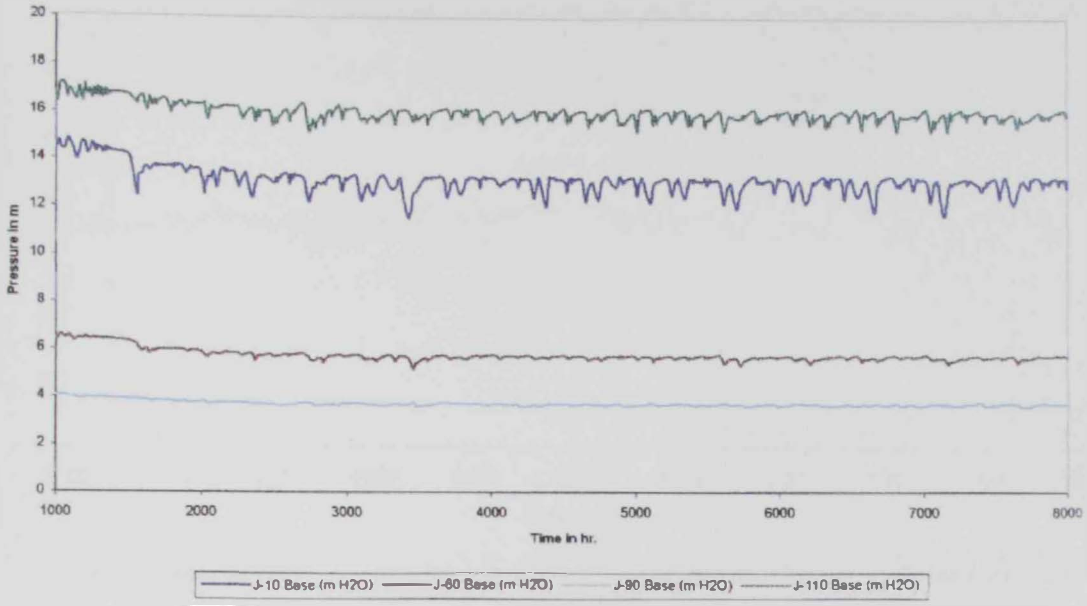


Figure 7.7 The residual pressure of Al Yahar area accomplished with the proposed elevated tank at the End point of the area

Figure 7.7 shows acceptable pressures in only few selected junctions as per the water distribution code. The junctions having pressures more than 15 m are mainly supplied from the elevated tank while the other junctions having low pressures, lower than the required minimum limit (10 m), are supplied from the existing source. So this option is not accepted and can't be considered a feasible alternative for the system improved performance.

For Swiehan area, the proposed elevated tank is located in the Shabia side (low cost houses) where the high demand is required at that portion of the network. The elevation of this proposed tank is 25 m with a size of 0.3 MG.

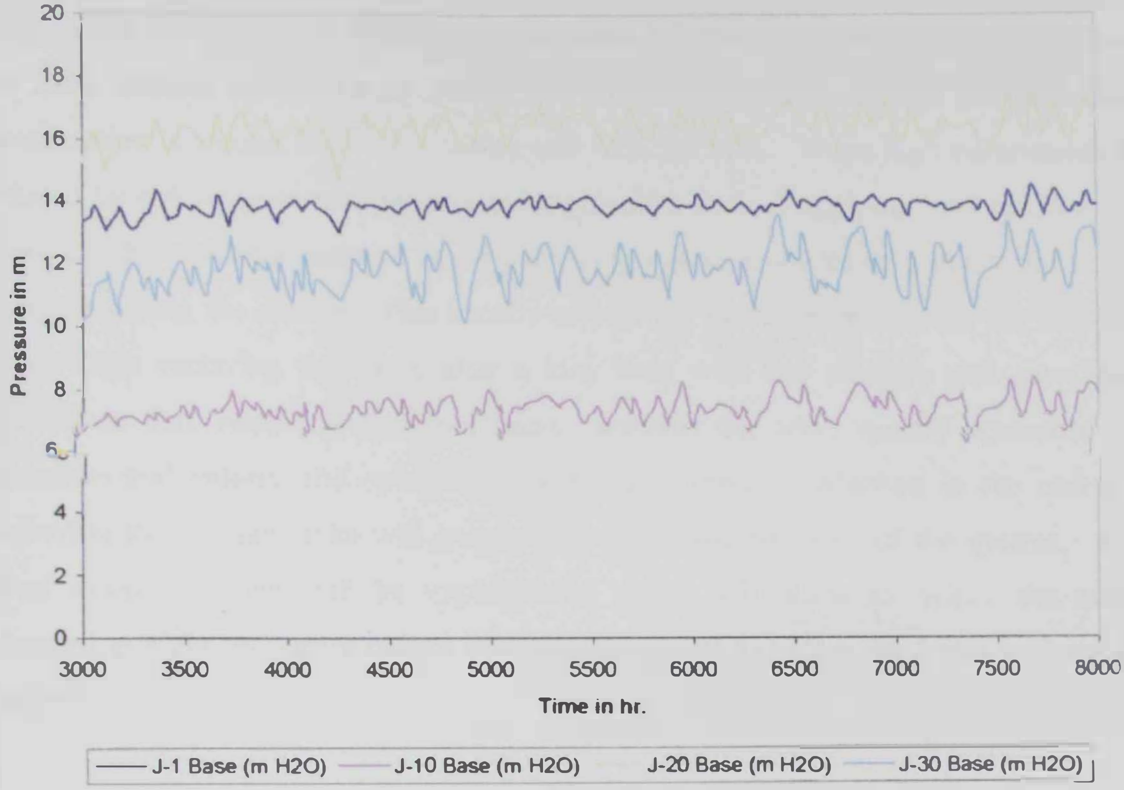


Figure 7.8 The residual pressure of Sweihan area accomplished with the proposed elevated tank at the end point

Figure 7.8 shows the residual pressure with a minimum of around 6.2 m to a maximum with 16 m head. In this regard, the previous option is better than this option, since it does not match the considered code. As a result, this option was not accepted too as it does not enhance the pressure for all junctions in the two systems. Velocities and residual chlorines were not investigated for the above elevated tanks alternative proposed at the end distant points.

7.3 Successful Demand Management Implementation

The purpose of this section is to implement the principle of the demand management concept illustrated in chapter 3. AADC is aware that the customers consume large quantities of water delivered to their tanks via redundant green areas irrigation in their properties. As shown in the water demand estimation for Shabia and Villa, the quantity required for the non human consumption is around 66.6% for shabia and 71% for villa. These high percentages can be reduced by enforcing certain policies and regulations such as applying high tariffs joined by removing the customer tanks to avoid delivering excess amounts of water to the houses or villas near from the source. This usually affects the distant properties in the network and causes them receiving the water after a long time with low pressure and quantities. In addition to that, removing such old tanks improves the water quality consumed by the customers and reduces the operating cost by the expected reduction in the energy cost. Removing the storage tanks will generally improve the behavior of the system. And the actual demand pattern will be implemented which will allow to reduce the quantities delivered at night and hence reduce the wastage as well as reduce the leakage if any during that period.

7.3.1 Al Dhaher Area

This area is supplied from two sources, the main of which is Al Ain reception pumping station named UmGhafa reservoir. The elevation difference between this reservoir and the network is around 40 – 45m. The demand of this area is 334,000 gal/day allocated at the junctions representing the villas, shabias and the other bulk properties as described in chapter 4.

Based on the Hydraulic Analysis, the following is recognized and concluded. The velocity inside the pipes is varying from 0.3 to 1.42 m/sec for the main pipe for minimum, average and peak demand scenarios. The velocities are slightly low for the minimum and average scenarios and may affect the Chlorine concentration at the end user connections. This happen in some areas but in general the velocities are within the accepted range set by AADC. Table 7.2 summarizes the results obtained from the system hydraulic modeling after removing the tanks.

Table 7.1 Summery of the Hydraulic Analysis for Al Dhaher

Items	Min.	Average	Peak
Total Discharge l/sec	12.55	17.57	50.1
Max.Velocity (Main) m/sec	0.36	0.5	1.42
Min. Velocity. (Main) m/sec	0.3	0.44	1.21
Max.Velocity (Branch) m/sec	0..17	0.24	0.67
Min. Velocity. (Branch) m/sec	0..11	0.16	0.53
Max. Pressure. M	36	34	31
Min. Pressure. M	33	25	21

The residual chlorine was also modeled with results of 0.31, 0.36 and 0.41 ppm for minimum, average and peak demands. Moreover, important parameters such as velocities and residual pressure are all accepted as they are noticed to be much better when compared to the analysis done for the original network having IST's. The residual chlorine is also in a good and accepted range in the network. Hence, it is recommended if this option is successfully implemented, to reduce the dosage quantity of the chlorine until the average in the network reach to 0.35 ppm in order to save the chemical cost expended currently. It is worth mentioning that the energy cost is not calculated for this system since it is supplied by gravity.

7.3.2 Al Yahar Area

This area is supplied from 2 sources with a total estimated demand of 0.93 MGD. The average pressure at the entrance is 3 bars and is almost constant during most of the day. The average residual chlorine reaching from Al Saad pumping station to the entrance of this network is around 0.41 ppm. Accordingly, the hydraulic modeling analysis was conducted producing the following results

Table 7.2 Summary of the Hydraulic Analysis for Al Yahar

Items	Min	Average	Peak
Total Discharge l/sec	34	49	124.4
Max. Velocity m/sec	0 34	0 56	1.33
Min. Velocity. m/sec	0.21	0 38	1.12
Max. Pressure. M	26.5	23.1	20.2
Min. Pressure. m	25.4	21.8	19.3

The residual chlorine ranged from 0.32 – 0.37 ppm, where this range is very accepted and even the customer will not feel the odor associated with it.

7.3.3 Al Sweihan Area

Swiehan area is supplied from a branch of 300 mm line with a total quantity of 510,000 gal/day and with an average pressure of 4 bars. The residual chlorine at the entrance is 0.35 ppm in average. Hence the results of the hydraulic modeling are as following.

o

Table 7.3 Summery of the Hydraulic Analysis for Al Sweihan

Items	Min	Average	Peak
Total Discharge l/sec	18.12	26.83	65.2
Max. Velocity m/sec	0.51	0.73	1.21
Min. Velocity. m/sec	0.43	0.69	1.08
Max. Pressure. m	37	33	30.5
Min. Pressure. m	35.6	31.7	20.7

The residual chlorine modeling results are 0.21, 0.29 and 0.31 for minimum, average and peak demands respectively.

As a conclusion to the mentioned demand management concept, the results of the residual pressure, residual chlorine and the velocities in all water systems for the 3 selected areas are compatible with the water distribution code. The energy costs for Al Yahar and Sweihan areas are 3630 and 3200 Dhs respectively. On the other hand no energy cost calculated for Al Dhaher area since it is supplied by gravity.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

Hydraulic models of the water supply networks in three selected areas have been developed in order to determine current operational performance and to assist in the development of options for improvement under 24 hour supply. The three selected areas are AlDhaher, AlYahar, and Sweihan and the selection was made considering the following three criteria:

1. Two types of housing zones with largely different demands (Shabia and Villas) exist in each area.
2. Flow control valves are available in each area providing accurate estimate of the supply and actual consumption of all areas.
3. Different sources supply the three systems so that the model quality calibration and evaluation is done for different sources.

The models have been developed to run extended period simulation conditions representing the time-dependent demands and other relevant conditions. WaterCad 7.0; a Network Modelling Software, was used in the network modelling. The approach adopted for hydraulic modelling of 2005 flows comprised two main elements:

- Information about the systems' demands and supplies, customer storage tanks capacities to establish solid assumptions for the internal storage tanks dimensions, and the systems' networks. The data obtained from the asset management regime were also used to determine roughness values applied to the model of each system and to understand network problems at the micro level.
- Construction of detailed hydraulic models using the collected data and calibrate them using field measurements for each system. ISTs capacities data, estimated water demands data from the water demand Forecast and demand pattern from Al Ain Distribution Company.

Figure 8.1 illustrates how the above two components side other sources of information compliment each other in the construction and configuration of the hydraulic model.

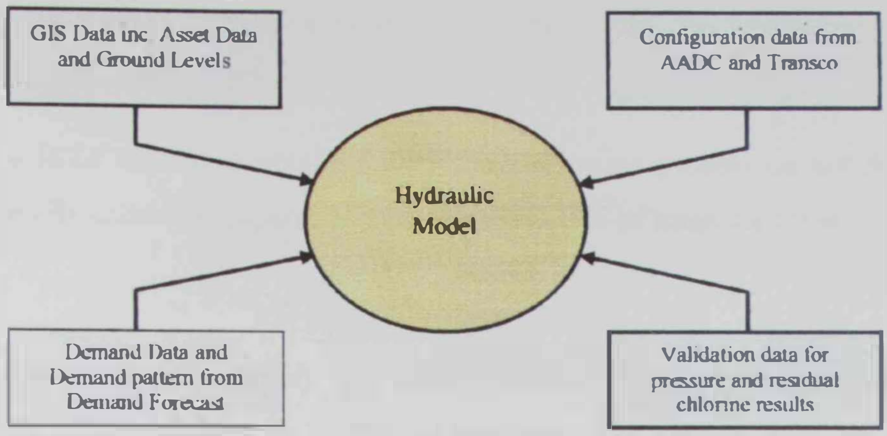


Figure 8.1 Overview of data sources for the hydraulic model

The model data like pips diameter, length and elevations were checked before commencement of the model building then loaded into WaterCad hydraulic models. The built models were verified against the GIS platform available in Transco and AADC. AlDhaher network (Figure 4.3) comprises of 192 pipes, 119 junctions and 29 tanks, Al Yahar pipe network (Figure 4.10) comprises of 217 pipes, 121 junctions and 52 tanks, while Swiehan network (Figure 4.16) has 150 pipes, 82 junctions and 42 tanks. Model configuration and asset attribute data were verified through site inspections and cross reference to other asset information provided by AADC and Transco. Levels were also taken at strategic locations within the network to verify elevation data. The demand forecast of 2005 as presented in chapter 4 has been divided into its component elements and allocated to the hydraulic model using a system of allocation parameters. Validation of the models was undertaken by comparing the modelled system performance with known data obtained from the field measurements.

Calibration of the three models was made following a trial and error simulation approach and targeted tuning the sizes and water levels of the ISTs. As the three areas are all new and receiving good quality desalinated water from different sources, a C value of 120 was used in simulation and kept unchanged during the calibration process. The following criteria were observed during the calibration:

- Producing close to-cyclic behaviour of the water level fluctuations in all ISTs.

- Maintaining a maximum deviation of 1.5 m between the pressure measurements and simulated pressure for each network
- The new ISTs' capacities obtained from the calibration process do not deviate from the originally estimated capacities with more than 5% of these capacities.

The model has been used to identify key aspects of network performance particularly in relation to levels of service and hydraulic performance. The results were investigated to identify locations of low pressures, high velocities, and low residual chlorine concentrations. Specific conclusions drawn from such results are as follows:

1. The modelling of Al Yahar and Sweihan areas resulted in pressures less than the minimum limit of 1 bar. Pressures for Al Dhaher area were accepted as they were around 19 m.
2. The velocity in several pipes was found to exceed the limit. Most of these pipes are connected to bulk properties; such as the farms, where large quantities of water is used to irrigate the corps without any specified demand management. Even though this situation can be tolerated by the adopted water distribution code, they are recommended to be replaced by larger sizes as well as installing flow control valves to regulate the consumed quantities.
3. The field measurements besides the simulated residual chlorine of many selected junctions in the three areas were accepted as per the international standards since the minimum limit is 0.2 ppm.
4. Some locations in the three simulated areas where still experiencing low residual chlorine (Refer to the Chlorine Contour Maps in Chapter 6). Hence, it is needed to boost the residual chlorine by injecting chlorine at these specific locations or increase the dosage of the chlorine from the source, to at least reach to the minimum limit 0.2 ppm without affecting the other locations for each area. Optimum dosage that satisfy all locations can be identified through further simulation.
5. In some areas the chlorine concentration exhibits cycles with a period of 5 days, this can be justified that the sampling point was downstream of the home storage tank, which is undoubtedly the reason for the cyclic behavior (see figure 6.4)

6. THM measurements as well as experiments indicated that bromoform dominates the THM species in all samples and at all times. Average of TTH measurements for Swiehan, Al Yahar and Al Dhaher were 33.19, 35.9 and 34.61, respectively. Comparing these figures with the international limits of 80 ppb for TTHM, one can find that TTHM levels in all areas are within the acceptable limits.

7. Laboratory experiments conducted to analyze the propagation of TTHM species in pipelines showed that the bromoform undergoes major hydrolysis after reaching its peak and ultimately approaches zero levels. The reported kinetic profiles didn't allow obtaining simple reaction rates for THM simulation in the simulated networks. Water ages in the three simulated networks were alternatively obtained to roughly predict the bromoform formation in the three networks. The water ages at the monitoring points for all networks were ranging from 2 to 5 hours indicating that the areas downstream such points are likely subjected to low bromoform levels while the upstream areas are likely subjected to higher levels. This needs further verification and future research is needed to explore the effect of temperature on the bromofom propagation in pipe lines.

8. Installing elevated storage tank at the entrance of Al Yahar and Swiehan systems is recommended to boost the pressures of these systems. This option produced good results for the pressure as well as the residual chlorines.

9. Demand Management techniques discussed in this thesis can be favored by removing the customer storage tanks in each system. This will allow each system to receive its needed demands at the customer tap. The systems will be pressurized all the time and the customers will receive their demands only without receiving excess amounts and eventually lowering the high head losses observed under current operation schemes. This scenario was simulated for the three selected systems and the results of pressures, velocities, and residual chlorines were all within acceptable limits.

10. Energy cost of the current systems or their potential modified alternatives can be significantly reduced through installing variable speed in the sources supplying the selected systems. Managing the operation of these pumps according to the actual time-dependent demands of downstream zones will eliminate the permanent high pressures that sometimes exceed the actual needs. This can be numerically investigated through

simulating larger systems that enclose the sources hosting the supplying pumping stations

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APPENDIX A

NETWORK DATA

AL DHAHER AREA

Label	Length (m)	Diameter (mm)	Label	Length (m)	Diameter (mm)	Material Description
27	29.58	400	186	308.92	100	Ductile Iron
192	203.96	100	70	92.1	150	Ductile Iron
191	30	100	89	103.73	150	Ductile Iron
194	30	100	169	406.89	150	Ductile Iron
110	14.17	150	85	18.2	150	Ductile Iron
109	167.22	150	93	254.13	150	Ductile Iron
107	18.2	150	94	129.39	150	Ductile Iron
108	210.15	150	92	18.2	150	Ductile Iron
116	155.96	150	91	254.13	150	Ductile Iron
114	18.2	150	83	251.94	150	Ductile Iron
112	155.96	150	74	120.56	150	Ductile Iron
106	210.15	150	73	135.76	150	Ductile Iron
100	155.96	150	72	119.59	150	Ductile Iron
145	134.8	150	132	542.9	150	Ductile Iron
119	18.2	150	182	214.5	150	Ductile Iron
208	321.3	150	53	41.56	150	Ductile Iron
193	146.97	150	88	254.13	150	Ductile Iron
188	131.96	150	87	52.23	150	Ductile Iron
14	393.47	150	153	365.79	150	Ductile Iron
205	83.2	200	47	26.97	150	Ductile Iron
206	54	200	45	117.05	150	Ductile Iron
199	572.8	200	46	393.31	150	Ductile Iron
12	18.2	200	160	406.89	150	Ductile Iron
111	197.02	400	86	620.23	150	Ductile Iron
113	93.99	400	75	135.44	150	Ductile Iron
99	106.64	400	71	384.38	150	Ductile Iron
97	150.06	400	80	133.08	150	Ductile Iron

26	33.7	400	185	86.98	150	Ductile Iron
23	586.22	400	183	161.95	150	Ductile Iron
30	252.85	400	158	406.89	150	Ductile Iron
4	110	400	167	406.89	150	Ductile Iron
5	32	400	204	616.6	150	Ductile Iron
10	380.37	400	202	398	150	Ductile Iron
56	492	600	48	393.37	150	Ductile Iron
98	155.96	150	81	32.15	200	Ductile Iron
105	155.96	150	78	340.82	200	Ductile Iron
198	131.95	150	180	145.16	200	Ductile Iron
195	161.95	150	170	23	200	Ductile Iron
31	61.93	150	168	131.96	200	Ductile Iron
196	512.87	200	173	93.82	200	Ductile Iron
15	240.57	200	181	161.96	200	Ductile Iron
13	130.7	200	187	161.95	200	Ductile Iron
201	283	200	200	572.8	200	Ductile Iron
104	156.39	400	76	18.2	200	Ductile Iron
24	18.2	400	184	512.88	200	Ductile Iron
32	475.3	400	155	406.89	200	Ductile Iron
P-77b	0.3	400	161	21	200	Ductile Iron
P-77a	0.3	400	159	162	200	Ductile Iron
11	18.2	400	203	485	200	Ductile Iron
197	203.95	100	176	683.67	300	Ductile Iron
121	173.29	150	79	190.5	300	Ductile Iron
68	18.2	150	P-31	8.5	300	Ductile Iron
55	652.01	400	P-32	13	300	Ductile Iron
115	210.15	150	90	128.66	400	Ductile Iron
96	130.1	150	82	599.67	400	Ductile Iron
118	510.1	150	1	300	400	Ductile Iron
126	21	150	164	5	400	Ductile Iron
123	134.8	150	P-29	0.5	400	Ductile Iron
57	72.56	150	P-30	0.5	400	Ductile Iron
28	586.25	150	154	135.96	400	Ductile Iron
140	23	200	151	521.6	400	Ductile Iron
142	115.14	200	84	24	400	Ductile Iron
122	98.87	400	166	1,375.20	400	Ductile Iron
29	377.78	400	150	21.3	400	Ductile Iron

2	55.45	400	165	54.4	400	Ductile Iron
189	308.92	100	59	272	600	Ductile Iron
128	287.93	150	60	520	600	Ductile Iron
64	156.25	150	P-33	2	600	Ductile Iron
61	118.59	150	P-34	18	600	Ductile Iron
103	11.22	150	P-6	0.5	1,000.00	Ductile Iron
101	18.2	150	P-7	0.5	1,000.00	Ductile Iron
102	179.27	150	P-5	0.5	1,000.00	Ductile Iron
135	365.8	150	P-12	0.5	1,000.00	Ductile Iron
148	21	150	P-9	0.5	1,000.00	Ductile Iron
127	287.93	150	P-13	0.5	1,000.00	Ductile Iron
95	84.53	150	P-25	0.5	1,000.00	Ductile Iron
69	51.93	150	P-24	0.5	1,000.00	Ductile Iron
67	102.44	150	P-17	0.5	1,000.00	Ductile Iron
65	102.76	150	P-18	0.5	1,000.00	Ductile Iron
66	137.1	150	P-16	0.5	1,000.00	Ductile Iron
54	19.1	150	P-14	0.5	1,000.00	Ductile Iron
43	18.2	150	P-15	0.5	1,000.00	Ductile Iron
44	393.31	150	P-22	0.5	1,000.00	Ductile Iron
40	156.25	150	P-19	0.5	1,000.00	Ductile Iron
137	131.96	200	P-11	0.5	1,000.00	Ductile Iron
133	161.95	200	P-10	0.5	1,000.00	Ductile Iron
134	21	200	P-8	0.5	1,000.00	Ductile Iron
41	393.31	200	P-23	0.5	1,000.00	Ductile Iron
117	144.97	400	P-3	0.5	1,000.00	Ductile Iron
136	287.95	400	P-4	0.5	1,000.00	Ductile Iron
149	214.5	400	P-2	0.5	1,000.00	Ductile Iron
120	12.56	400	P-1	0.5	1,000.00	Ductile Iron
125	149.78	400	P-28	0.5	1,000.00	Ductile Iron
124	423.9	400	P-27	0.5	1,000.00	Ductile Iron
63	339.32	400	P-26	0.5	1,000.00	Ductile Iron
39	403.91	400	P-21	0.5	1,000.00	Ductile Iron
36	20	400	P-20	0.5	1,000.00	Ductile Iron

AL YAHAR AREA

Label	Length (m)	Diameter (mm)	Label	Length (m)	Diameter (mm)	Label	Length (m)	Diameter (mm)	Material
P-142	144	100	P-137	156.5	100	P-50	102	150	Ductile Iron
P-141	173	100	P-120	363.5	100	P-8	23	150	Ductile Iron
P-143	102	100	P-101	35	100	P-7	171.5	150	Ductile Iron
P-139	169.5	100	P-114	100.5	100	P-6	170.5	150	Ductile Iron
P-138	149	100	P-115	99.5	100	P-3	181.5	150	Ductile Iron
P-156	146	100	P-117	98.5	100	P-56	177	150	Ductile Iron
P-151	294	100	P-116	166	100	P-5	23	150	Ductile Iron
P-155	147.5	100	P-112	102	100	P-4	272.5	150	Ductile Iron
P-164	100.5	100	P-99	44	100	P-30	23	150	Ductile Iron
P-154	146.5	100	P-113	161	100	P-47	102	150	Ductile Iron
P-140	147.5	100	P-11	196.5	150	P-29	102	150	Ductile Iron
P-111	194.5	100	P-10	216.5	150	P-33	166.5	150	Ductile Iron
P-146	147	100	P-14	23	150	P-28	23	150	Ductile Iron
P-110	169	100	P-9	285	150	P-93	273	150	Ductile Iron
P-135	23	100	P-1	102	150	P-48	67	150	Ductile Iron
P-134	102	100	P-18	102	150	P-91	148	150	Ductile Iron
P-148	201.5	100	P-16	144	150	P-53	102	150	Ductile

									Iron
P-147	193	100	P-15	259.5	150	P-92	215.5	150	Ductile Iron
P-149	195.5	100	P-19	23	150	P-222	40.5	250	Ductile Iron
P-145	170	100	P-20	202.5	150	P-219	32	250	Ductile Iron
P-144	23	100	P-21	23	150	P-220	41	250	Ductile Iron
P-51	375.5	100	P-132	102	150	P-221	34	250	Ductile Iron
P-52	216.5	100	P-128	67	150	P-T3	1	1000	Ductile Iron
P-68	102	100	P-129	23	150	P-T7	1	1000	Ductile Iron
P-66	102	100	P-225	43	150	P-T4	1	1000	Ductile Iron
P-67	23	100	P-226	43	150	P-T2	1	1000	Ductile Iron
P-59	171.5	100	P-227	43	150	P-T1	1	1000	Ductile Iron
P-124	100.5	100	P-130	102	150	P-T10	1	1000	Ductile Iron
P-185	11.5	100	P-131	23	150	P-T9	1	1000	Ductile Iron
P-161	102	100	P-12	23	150	P-T11	1	1000	Ductile Iron
P-157	148	100	P-13	102	150	P-T8	1	1000	Ductile Iron
P-71	168	100	P-125	102	150	P-T6	1	1000	Ductile Iron
P-64	23	100	P-126	23	150	P-T33	1	1000	Ductile Iron
P-65	102	100	P-127	35	150	P-T32	1	1000	Ductile Iron
P-63	102	100	P-2	23	150	P-T34	1	1000	Ductile Iron
P-152	148.5	100	P-22	102	150	P-T30	1	1000	Ductile Iron

P-153	145.5	100	P-102	102	150	P-T29	1	1000	Ductile Iron
P-69	23	100	P-105	67	150	P-T28	1	1000	Ductile Iron
P-70	102	100	P-27	102	150	P-T27	1	1000	Ductile Iron
P-62	23	100	P-43	23	150	P-T23	1	1000	Ductile Iron
P-60	180.5	100	P-25	144.5	150	P-T26	1	1000	Ductile Iron
P-61	102	100	P-26	23	150	P-T24	1	1000	Ductile Iron
P-72	174	100	P-104	35	150	P-T31	1	1000	Ductile Iron
P-75	167	100	P-24	251.5	150	P-T21	1	1000	Ductile Iron
P-76	175	100	P-31	98.5	150	P-T14	1	1000	Ductile Iron
P-77	166	100	P-32	200.5	150	P-T22	1	1000	Ductile Iron
P-78	186	100	P-23	23	150	P-T19	1	1000	Ductile Iron
P-80	218.5	100	P-103	23	150	P-T18	1	1000	Ductile Iron
P-81	102	100	P-95	200	150	P-T16	1	1000	Ductile Iron
P-82	192	100	P-94	194.5	150	P-T15	1	1000	Ductile Iron
P-83	194.5	100	P-37	200.5	150	P-T17	1	1000	Ductile Iron
P-84	148.5	100	P-38	183	150	P-T13	1	1000	Ductile Iron
P-85	145.5	100	P-40	374.5	150	P-T12	1	1000	Ductile Iron
P-86	149	100	P-108	23	150	P-T38	1	1000	Ductile Iron
P-87	145	100	P-109	102	150	P-T45	1	1000	Ductile Iron
P-88	148	100	P-106	23	150	P-T37	1	1000	Ductile

									Iron
P-89	146	100	P-107	102	150	P-T42	1	1000	Ductile Iron
P-162	138.5	100	P-34	23	150	P-T36	1	1000	Ductile Iron
P-163	101.5	100	P-46	106.5	150	P-T46	1	1000	Ductile Iron
P-166	104.5	100	P-41	217.5	150	P-T44	1	1000	Ductile Iron
P-165	137.5	100	P-42	413	150	P-T5	1	1000	Ductile Iron
P-150	199	100	P-45	23	150	P-T43	1	1000	Ductile Iron
P-133	23	100	P-35	67	150	P-T47	1	1000	Ductile Iron
P-96	102	100	P-36	129.5	150	P-T41	1	1000	Ductile Iron
P-97	23	100	P-44	102	150	P-T40	1	1000	Ductile Iron
P-98	58	100	P-58	102	150	P-TS1	1	1000	Ductile Iron
P-123	165	100	P-54	23	150	P-TS2	1	1000	Ductile Iron
P-118	100	100	P-55	165	150	P-T35	1	1000	Ductile Iron
P-121	363.5	100	P-57	23	150	P-T25	1	1000	Ductile Iron
P-122	98	100	P-90	228.5	150	P-T20	1	1000	Ductile Iron
P-119	128.5	100	P-49	23	150	P-T48	1	1000	Ductile Iron
P-100	23	100	P-T39	1	1,000	P-TC	1	1000	Ductile Iron
			P-TKG	1	1,000				Ductile Iron

SWEIHAN AREA

Label	Length (m)	Diameter (mm)	Label	Length (m)	Diameter (mm)	Label	Length (m)	Diameter (mm)	Material Description
P-99	199	100	P-82	168.5	150	P-21	116	150	Ductile Iron
P-95	132	100	P-67	23	150	P-19	132	150	Ductile Iron
P-103	220	100	P-76	165.5	150	P-17	132	150	Ductile Iron
P-102	83.5	100	P-77	180.5	150	P-150	44	200	Ductile Iron
P-104	220	100	P-66	132	150	P-149	56.5	200	Ductile Iron
P-97	132	100	P-83	132	150	P-1	108	200	Ductile Iron
P-47	180.5	100	P-68	267	150	P-189	1197	200	Ductile Iron
P-98	199	100	P-69	276.5	150	P-147	408	300	Ductile Iron
P-94	181	100	P-64	132	150	P-148	24.5	300	Ductile Iron
P-96	21	100	P-65	21	150	P-TM	1	1,000	Ductile Iron
P-100	408	100	P-62	149	150	P-T41	1	1,000	Ductile Iron
P-92	185	100	P-59	429	150	P-T42	1	1,000	Ductile Iron
P-89	160.5	100	P-60	309.5	150	P-T40	1	1,000	Ductile Iron
P-93	104	100	P-61	85	150	P-T39	1	1,000	Ductile Iron
P-91	160.5	100	P-58	149	150	P-T38	1	1,000	Ductile Iron
P-90	185	100	P-63	23	150	P-T37	1	1,000	Ductile Iron
P-87	21	100	P-56	132	150	P-T36	1	1,000	Ductile Iron
P-101	332.5	100	P-57	23	150	P-T35	1	1,000	Ductile Iron

P-88	132	100	P-54	132	150	P-T28	1	1,000	Ductile Iron
P-36	132	100	P-35	220	150	P-T27	1	1,000	Ductile Iron
P-86	132	100	P-52	98.5	150	P-T30	1	1,000	Ductile Iron
P-38	132	100	P-53	23	150	P-T29	1	1,000	Ductile Iron
P-42	21	100	P-34	126	150	P-T34	1	1,000	Ductile Iron
P-37	21	100	P-70	251.5	150	P-T33	1	1,000	Ductile Iron
P-41	132	100	P-10	23	150	P-T31	1	1,000	Ductile Iron
P-39	141.5	100	P-8	21	150	P-T32	1	1,000	Ductile Iron
P-43	132	100	P-6	23	150	P-T26	1	1,000	Ductile Iron
P-75	161	100	P-9	132	150	P-T25	1	1,000	Ductile Iron
P-74	247	100	P-3	270	150	P-T24	1	1,000	Ductile Iron
P-73	250	100	P-2	153	150	P-T23	1	1,000	Ductile Iron
P-72	165.5	100	P-7	132	150	P-T20	1	1,000	Ductile Iron
P-40	143.5	100	P-29	126	150	P-T21	1	1,000	Ductile Iron
P-45	180.5	100	P-31	169	150	P-T18	1	1,000	Ductile Iron
P-49	284	100	P-32	177	150	P-T12	1	1,000	Ductile Iron
P-44	165.5	100	P-30	220	150	P-T19	1	1,000	Ductile Iron
P-48	411	100	P-26	157.5	150	P-T7	1	1,000	Ductile Iron
P-46	165.5	100	P-28	132	150	P-T10	1	1,000	Ductile Iron
P-50	128.5	100	P-27	188.5	150	P-T8	1	1,000	Ductile

									Iron
P-22	135.5	100	P-14	429	150	P-TKG	1	1,000	Ductile Iron
P-23	279	100	P-12	138.5	150	P-T9	1	1,000	Ductile Iron
P-24	266.5	100	P-13	256.5	150	P-T5	1	1,000	Ductile Iron
P-25	142.5	100	P-16	23	150	P-T11	1	1,000	Ductile Iron
P-79	147	150	P-71	178.5	150	P-T14	1	1,000	Ductile Iron
P-84	147	150	P-33	132	150	P-T13	1	1,000	Ductile Iron
P-85	199	150	P-15	149	150	P-T16	1	1,000	Ductile Iron
P-78	132	150	P-20	23	150	P-T15	1	1,000	Ductile Iron
P-55	21	150	P-11	149	150	P-T2	1	1,000	Ductile Iron
P-80	199	150	P-4	266	150	P-T3	1	1,000	Ductile Iron
P-81	177.5	150	P-5	267	150	P-T1	1	1,000	Ductile Iron
P-T4	1	1,000	P-18	21	150	P-T6	1	1,000	Ductile Iron

APPENDIX B

IST DIMESION AND ELEVATIONS

AL DHAHER AREA

Label	Type	Base Elevation (m)	Initial Level (m)	Min Level (m)	Max Level (m)	Tank Dia (m)	Base Flow (mgd(Imp))	Total Volume (gal(Imp))
T-25	Shabia	295	0.1	0.1	5	4.2	0.0075	14,933
T-5	Shabia	297	0.1	0.1	5	4.2	0.0075	14,933
T-28	Shabia	297	0.1	0.1	5	4.2	0.0075	14,933
T-4	Shabia	297	0.1	0.1	5	4.2	0.0075	14,933
T-1	Shabia	297	0.1	0.1	5	4.2	0.0075	14,933
T-2	Shabia	297	0.1	0.1	5	4.2	0.0075	14,933
T-3	Shabia	297	0.1	0.1	5	4.2	0.0075	14,933
T-26	Shabia	298	0.1	0.1	5	4.2	0.0075	14,933
T-12	Shabia	300	0.1	0.1	5	4.2	0.0075	14,933
T-13	Shabia	300	0.1	0.1	5	4.2	0.0075	14,933
T-11	Shabia	300	0.1	0.1	5	4.2	0.0075	14,933
T-7	Shabia	300	0.1	0.1	5	4.2	0.0075	14,933
T-6	Shabia	300	0.1	0.1	5	4.2	0.0075	14,933
T-8	Shabia	300	0.1	0.1	5	4.2	0.0075	14,933
T-10	Shabia	300	0.1	0.1	5	4.2	0.0075	14,933
T-9	Shabia	300	0.1	0.1	5	4.2	0.0075	14,933
T-27	Shabia	302	0.1	0.1	5	4.2	0.0075	14,933
T-15	Shabia	304	0.1	0.1	5	4.2	0.0075	14,933
T-14	Shabia	304	0.1	0.1	5	4.2	0.0075	14,933
T-16	Shabia	304	0.1	0.1	5	4.2	0.0075	14,933
TF	Farm	305	0.1	0.1	10	10.8	0.1	199,496
T-18	Villa	307	0.1	0.1	5	5	0.00105	21,164
T-17	Villa	307	0.1	0.1	5	5	0.00105	21,164
T-20	Villa	307	0.1	0.1	5	5	0.00105	21,164
T-19	Villa	307	0.1	0.1	5	5	0.00105	21,164
T-23	Villa	333	0.1	0.1	5	5	0.00105	21,164
T-24	Villa	333	0.1	0.1	5	5	0.00105	21,164

T-21	Villa	333	0.1	0.1	5	5	0.00105	21.164
T-22	Villa	333	0.1	0.1	5	5	0.00105	21,164

AL YAHAR AREA

Label	Type	Base Elevation (m)	Initial Level (m)	Minimum Level (m)	Maximum Level (m)	Tank Diameter (m)	Base Flow (mgd(Imp))	Total Volume (gal(Imp))
T-10	Shabia	247.1	0.1	0.1	5.1	5.9	0.015	30,069
T-9	Shabia	247.2	0.1	0.1	5.1	5.9	0.015	30,069
T-8	Shabia	246.5	0.1	0.1	5.1	5.9	0.015	30,069
T-11	Shabia	245.9	0.1	0.1	5.1	5.9	0.015	30,069
T-20	Shabia	246.5	0.1	0.1	5.1	5.9	0.015	30,069
T-19	Shabia	247.1	0.1	0.1	5.1	5.9	0.015	30,069
T-12	Shabia	245.4	0.1	0.1	5.1	5.9	0.015	30,069
T-3	Shabia	246.9	0.1	0.1	5.1	5.9	0.015	30,069
T-2	Shabia	248.6	0.1	0.1	5.1	5.9	0.015	30,069
T-1	Shabia	248.6	0.1	0.1	5.1	5.9	0.015	30,069
T-4	Shabia	247.1	0.1	0.1	5.1	5.9	0.015	30,069
T-7	Shabia	246.9	0.1	0.1	5.1	5.9	0.015	30,069
T-6	Shabia	247.2	0.1	0.1	5.1	5.9	0.015	30,069
T-5	Shabia	247	0.1	0.1	5.1	5.9	0.015	30,069
T-22	Villa	246.7	0.1	0.1	5.1	6.4	0.0175	35,382
T-21	Villa	246.4	0.1	0.1	5.1	6.4	0.0175	35,382
T-25	Villa	246	0.1	0.1	5.1	6.4	0.0175	35,382
T-26	Villa	246.3	0.1	0.1	5.1	6.4	0.0175	35,382
T-23	Villa	246.4	0.1	0.1	5.1	6.4	0.0175	35,382
T-24	Villa	248	0.1	0.1	5.1	6.4	0.0175	35,382
T-15	Villa	243.7	0.1	0.1	5.1	6.4	0.0175	35,382
T-14	Villa	245.5	0.1	0.1	5.1	6.4	0.0175	35,382
T-13	Villa	244.8	0.1	0.1	5.1	6.4	0.0175	35,382
T-18	Villa	243.5	0.1	0.1	5.1	6.4	0.0175	35,382

T-17	Villa	243.4	0.1	0.1	5.1	6.4	0.0175	35,382
T-16	Villa	243.4	0.1	0.1	5.1	6.4	0.0175	35,382
T-41	Villa	244.1	0.1	0.1	5.1	6.4	0.0175	35,382
T-42	Villa	244.1	0.1	0.1	5.1	6.4	0.0175	35,382
T-40	Villa	244.8	0.1	0.1	5.1	6.4	0.0175	35,382
T-38	Villa	244.8	0.1	0.1	5.1	6.4	0.0175	35,382
T-39	Villa	244.8	0.1	0.1	5.1	6.4	0.0175	35,382
T-43	Villa	243.7	0.1	0.1	5.1	6.4	0.0175	35,382
T-47	Villa	243.7	0.1	0.1	5.1	6.4	0.0175	35,382
T-48	Villa	243.5	0.1	0.1	5.1	6.4	0.0175	35,382
T-46	Villa	243.5	0.1	0.1	5.1	6.4	0.0175	35,382
T-44	Villa	244.1	0.1	0.1	5.1	6.4	0.0175	35,382
T-45	Villa	243.7	0.1	0.1	5.1	6.4	0.0175	35,382
T-30	Villa	248.2	0.1	0.1	5.1	6.4	0.0175	35,382
T-31	Villa	245.8	0.1	0.1	5.1	6.4	0.0175	35,382
T-29	Villa	248.2	0.1	0.1	5.1	6.4	0.0175	35,382
T-27	Villa	249	0.1	0.1	5.1	6.4	0.0175	35,382
T-28	Villa	248.8	0.1	0.1	5.1	6.4	0.0175	35,382
T-32	Villa	245.8	0.1	0.1	5.1	6.4	0.0175	35,382
T-36	Villa	246.3	0.1	0.1	5.1	6.4	0.0175	35,382
T-37	Villa	245.5	0.1	0.1	5.1	6.4	0.0175	35,382
T-35	Villa	246.4	0.1	0.1	5.1	6.4	0.0175	35,382
T-33	Villa	247.3	0.1	0.1	5.1	6.4	0.0175	35,382
T-34	Villa	247.5	0.1	0.1	5.1	6.4	0.0175	35,382
TI	Industrial	243	0.1	0.1	10.1	3.4	0.02	19,971
TB	Building	243	0.1	0.1	10.1	3.8	0.025	24,947
TFS	Tanker filling Station	247	0.1	0.1	10.1	4.16	0.03	29,898
TF	Farm	243	0.1	0.1	10.1	7.6	0.05	99,788

SWEIHAN AREA

Label	Type	Base Elevation (m)	Initial Level (m)	Minimum Level (m)	Maximum Level (m)	Tank Diameter (m)	Base Flow (mgd(Imp))	Total Volume (gal(Imp))
T-3	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-1	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-2	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-6	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-4	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-7	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-8	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-9	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-10	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-11	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-13	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-16	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-12	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-14	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-15	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-19	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-5	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-18	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-20	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-21	Villa	255	0.1	0.1	5.1	5	0.007	21,595
T-23	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-24	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-25	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-26	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-27	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-28	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-29	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-30	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-31	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-32	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069

T-33	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-34	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-35	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-36	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-37	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-38	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-39	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-40	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-41	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
T-42	Shabia	255	0.1	0.1	5.1	5.9	0.015	30,069
TI	Industrial	255	0.1	0.1	10.1	4.8	0.02	39,805
TFS	Tanker Filling Station	255	0.1	0.1	10.1	6.6	0.05	75,256

APPENDIX C

IST DIMENSIONS AFTER CALIBRATION

AL YAHAR AREA

Label	Base Elevation (m)	Tank Diameter (m)	Initial Level (m)	Minimum Level (m)	Maximum Level (m)	Base Flow (mgd(Imp))	Total Volume (gal(Imp))
T-1	241.6	5.9	2.5	0.1	5.36	0.015	31,603
T-2	241	5.9	2.4	0.1	5.36	0.015	31,603
T-3	239.9	5.9	2	0.1	5.36	0.015	31,603
T-4	240.1	5.9	2.3	0.1	5.36	0.015	31,603
T-5	240.2	5.9	2.5	0.1	5.36	0.015	31,603
T-6	240.2	5.9	2.5	0.1	5.36	0.015	31,603
T-7	239.9	5.9	2.5	0.1	5.36	0.015	31,603
T-8	239.5	5.9	3	0.1	5.36	0.015	31,603
T-9	240.2	5.9	2	0.1	5.36	0.015	31,603
T-10	240.2	5.9	2.4	0.1	5.36	0.015	31,603
T-11	239.3	5.9	2.6	0.1	5.36	0.015	31,603
T-12	238.4	5.9	2.7	0.1	5.36	0.015	31,603
T-13	237.8	6.4	2.5	0.1	5.36	0.0175	37,186
T-14	238.5	6.4	2.5	0.1	5.36	0.0175	37,186
T-15	236.7	6.4	2.5	0.1	5.36	0.0175	37,186
T-16	236.5	6.4	2.4	0.1	5.36	0.0175	37,186
T-17	236.5	6.4	2.7	0.1	5.36	0.0175	37,186
T-18	236.5	6.4	2.5	0.1	5.36	0.0175	37,186
T-19	240.1	5.9	2.4	0.1	5.36	0.015	31,603
T-20	239.5	5.9	2.5	0.1	5.36	0.015	31,603
T-21	239.4	6.4	2.8	0.1	5.36	0.0175	37,186
T-22	239.7	6.4	2.5	0.1	5.36	0.0175	37,186
T-23	239.7	6.4	2.5	0.1	5.36	0.0175	37,186
T-24	241	6.4	2.5	0.1	5.36	0.0175	37,186
T-25	239	6.4	2.7	0.1	5.36	0.0175	37,186
T-26	239.3	6.4	2.56	0.1	5.36	0.0175	37,186
T-27	242	6.4	2.67	0.1	5.36	0.0175	37,186

T-28	241.8	6.4	2.8	0.1	5.36	0.0175	37,186
T-29	241.2	6.4	2.9	0.1	5.36	0.0175	37,186
T-30	241.2	6.4	3	0.1	5.36	0.0175	37,186
T-31	238.8	6.4	2	0.1	5.36	0.0175	37,186
T-32	238.8	6.4	1.9	0.1	5.36	0.0175	37,186
T-33	240.3	6.4	1.8	0.1	5.36	0.0175	37,186
T-34	240.5	6.4	2.4	0.1	5.36	0.0175	37,186
T-35	239.4	6.4	2.5	0.1	5.36	0.0175	37,186
T-36	239.3	6.4	2.5	0.1	5.36	0.0175	37,186
T-37	238.5	6.4	2.45	0.1	5.36	0.0175	37,186
T-38	237.8	6.4	2.35	0.1	5.36	0.0175	37,186
T-39	237.8	6.4	2.5	0.1	5.36	0.0175	37,186
T-40	237.8	6.4	2.5	0.1	5.36	0.0175	37,186
T-41	237.1	6.4	2.6	0.1	5.36	0.0175	37,186
T-42	237.1	6.4	2.7	0.1	5.36	0.0175	37,186
T-43	236.7	6.4	2.8	0.1	5.36	0.0175	37,186
T-44	237.1	6.4	2.4	0.1	5.36	0.0175	37,186
T-45	236.7	6.4	3.4	0.1	5.36	0.0175	37,186
T-46	236.5	6.4	2.5	0.1	5.36	0.0175	37,186
T-47	236.7	6.4	2.5	0.1	5.36	0.0175	37,186
T-48	236.5	6.4	2.7	0.1	5.36	0.0175	37,186
TB	238	3.8	2.9	0.1	10.61	0.025	26,207
TF	237.9	7.6	3	0.1	10.6	0.05	104,828
TFS	242	4.16	3.5	0.1	10.61	0.03	31,408
TI	238.5	3.4	3.8	0.1	10.6	0.02	20,980

SWEIHAN AREA

Label	Base Elevation (m)	Tank Diameter (m)	Initial Level (m)	Minimum Level (m)	Maximum Level (m)	Base Flow (mgd(Imp))	Total Volume (gal(Imp))
T-1	250	5.15	2.5	0.1	5.1	0.007	22,911
T-2	250	5.15	2.45	0.1	5.1	0.007	22,911
T-3	250	5.15	2.4	0.1	5.1	0.007	22,911
T-4	250	5.15	2.55	0.1	5.1	0.007	22,911
T-5	250	5.15	3	0.1	5.1	0.007	22,911
T-6	250	5.15	2.46	0.1	5.1	0.007	22,911
T-7	250	5.15	2.48	0.1	5.1	0.007	22,911
T-8	250	5.15	2.8	0.1	5.1	0.007	22,911
T-9	250	5.15	2.41	0.1	5.1	0.007	22,911
T-10	250	5.15	2.5	0.1	5.1	0.007	22,911
T-11	250	5.15	2.53	0.1	5.1	0.007	22,911
T-12	250	5.15	2.9	0.1	5.1	0.007	22,911
T-13	250	5.15	2.46	0.1	5.1	0.007	22,911
T-14	250	5.15	2.49	0.1	5.1	0.007	22,911
T-15	250	5.15	2.56	0.1	5.1	0.007	22,911
T-16	250	5.15	2.51	0.1	5.1	0.007	22,911
T-18	250	5.15	2.5	0.1	5.1	0.007	22,911
T-19	250	5.15	2.5	0.1	5.1	0.007	22,911
T-20	250	5.15	3	0.1	5.1	0.007	22,911
T-21	250	5.15	2.45	0.1	5.1	0.007	22,911
T-23	250	6.08	2.52	0.1	5.1	0.015	31,901
T-24	250	6.08	2.53	0.1	5.1	0.015	31,901
T-25	250	6.08	2.55	0.1	5.1	0.015	31,901
T-26	250	6.08	2.54	0.1	5.1	0.015	31,901
T-27	250	6.08	2.47	0.1	5.1	0.015	31,901
T-28	250	6.08	3.2	0.1	5.1	0.015	31,901
T-29	250	6.08	2.58	0.1	5.1	0.015	31,901
T-30	250	6.08	2.49	0.1	5.1	0.015	31,901
T-31	250	6.08	2.46	0.1	5.1	0.015	31,901
T-32	250	6.08	2.47	0.1	5.1	0.015	31,901
T-33	250	6.08	2.39	0.1	5.1	0.015	31,901
T-34	250	6.08	3.2	0.1	5.1	0.015	31,901
T-35	250	6.08	2.57	0.1	5.1	0.015	31,901
T-36	250	6.08	2.5	0.1	5.1	0.015	31,901
T-37	250	6.08	2.5	0.1	5.1	0.015	31,901
T-38	250	6.08	2.5	0.1	5.1	0.015	31,901
T-39	250	6.08	2.46	0.1	5.1	0.015	31,901

T-40	250	6.08	3.5	0.1	5.1	0.015	31,901
T-41	250	6.08	2.5	0.1	5.1	0.015	31,901
T-42	250	6.08	4	0.1	5.1	0.015	31,901
TFS	250	6.8	2.5	0.1	10.1	0.05	79,839
TI	250	4.94	3.4	0.1	10.1	0.02	42,229

AL DHAHER AREA

Label	Base Elevation (m)	Tank Diameter (m)	Initial Level (m)	Minimum Level (m)	Maximum Level (m)	Base Flow (mgd(Imp))	Total Volume (gal(Imp))
T-1	297	4.54	3.2	0.1	5	0.0075	17,418
T-10	300	4.54	2.6	0.1	5	0.0075	17,418
T-11	300	4.54	2	0.1	5	0.0075	17,418
T-12	300	4.54	2.6	0.1	5	0.0075	17,418
T-13	300	4.54	2.1	0.1	5	0.0075	17,418
T-14	304	4.54	3.1	0.1	5	0.0075	17,418
T-15	304	4.54	2.55	0.1	5	0.0075	17,418
T-16	304	4.54	2.55	0.1	5	0.0075	17,418
T-17	307	5.4	2.4	0.1	5	0.00105	24,685
T-18	307	5.4	2.6	0.1	5	0.00105	24,685
T-19	307	5.4	2.25	0.1	5	0.00105	24,685
T-2	297	4.54	3.1	0.1	5	0.0075	17,418
T-20	307	5.4	2.35	0.1	5	0.00105	24,685
T-21	333	5.4	2.95	0.1	5	0.00105	24,685
T-22	333	5.4	3.1	0.1	5	0.00105	24,685
T-23	333	5.4	2.1	0.1	5	0.00105	24,685
T-24	333	5.4	1.9	0.1	5	0.00105	24,685
T-25	295	4.54	2	0.1	5	0.0075	17,418
T-26	298	4.54	2.4	0.1	5	0.0075	17,418
T-27	302	4.54	3.2	0.1	5	0.0075	17,418
T-28	297	4.54	1.9	0.1	5	0.0075	17,418
T-3	297	4.54	2.9	0.1	5	0.0075	17,418
T-4	297	4.54	3	0.1	5	0.0075	17,418
T-5	297	4.54	2.1	0.1	5	0.0075	17,418
T-6	300	4.54	2.85	0.1	5	0.0075	17,418
T-7	300	4.54	2.87	0.1	5	0.0075	17,418
T-8	300	4.54	2.9	0.1	5	0.0075	17,418
T-9	300	4.54	2.45	0.1	5	0.0075	17,418
TF	305	11.66	3.5	0.1	10	0.1	232,692

APPENDIX D

IST HOUSE CONNECTION CALCULATION

Pressure Pipes Inventory					
1.50 in	200	m	4.00 in	20	m
Total Length	220	m			

Pressure Junctions @ 0.00 hr

Label	Calculated Hydraulic Grade (m)	Pressure (m H2O)	Pressure Head (m)	Demand (Calculated) (gal(Imp)/d)
J-1	5	4.99	5	0
J-2	5	4.99	5	0
J-3	5	4.99	5	0
J-4	5	4.99	5	0
J-5	5	4.99	5	0
J-6	5	4.99	5	0
J-7	5	4.99	5	0
J-8	5	4.99	5	0
J-9	5	4.99	5	0
J-10	5	4.99	5	0
J-11	5	4.99	5	0

Pressure Pipes @ 0.00 hr

Label	Control Status	Discharge (gal(Imp)/d)	Velocity (m/s)	Upstream Structure Hydraulic Grade (m)	Downstream Structure Hydraulic Grade (m)	Calculated Friction Headloss (m)	Calculated Minor Headloss (m)	Pressure Pipe Headloss (m)	Headloss Gradient (m/km)
P-1	Open	87,666	4.05	10	5	5	0	5	500
P-2	Open	87,666	4.05	5	0	5	0	5	500
P-3	Open	87,666	4.05	10	5	5	0	5	500
P-4	Open	87,666	4.05	5	0	5	0	5	500
P-5	Open	87,666	4.05	10	5	5	0	5	500

P-6	Open	87,666	4.05	5	0	5	0	5	500
P-7	Open	87,666	4.05	10	5	5	0	5	500
P-8	Open	87,666	4.05	5	0	5	0	5	500
P-9	Open	87,666	4.05	10	5	5	0	5	500
P-10	Open	87,666	4.05	5	0	5	0	5	500
P-11	Open	87,666	4.05	10	5	5	0	5	500
P-12	Open	87,666	4.05	5	0	5	0	5	500
P-13	Open	87,666	4.05	10	5	5	0	5	500
P-14	Open	87,666	4.05	5	0	5	0	5	500
P-15	Open	87,666	4.05	10	5	5	0	5	500
P-16	Open	87,666	4.05	5	0	5	0	5	500
P-17	Open	87,666	4.05	10	5	5	0	5	500
P-18	Open	87,666	4.05	5	0	5	0	5	500
P-19	Open	87,666	4.05	10	5	5	0	5	500
P-20	Open	87,666	4.05	5	0	5	0	5	500
P-21	Open	1,156,600	7.51	10	5	5	0	5	500
P-22	Open	1,156,600	7.51	5	0	5	0	5	500

Reservoirs @ 0.00 hr

Label	Calculated Hydraulic Grade (m)	Inflow (gal(Imp)/d)	Outflow (gal(Imp)/d)
Source supplying 10 house connections each has a size of 1 inch	10	-876.658	876.658
Source supplying one house connection its size is 4 inch	10	-1,156,600	1,156,600

Tanks @ 0.00 hr

Label	Calculated Hydraulic Grade	Calculated Level (m)	Pressure (m H2O)	Calculated Percent Full (%)	Calculated Volume (m^3)	Inflow (gal(Imp)/d)	Outflow (gal(Imp)/d)	Current Status

	(m)							
Storage tank 1	0	0	0	0	0	876,658	-876.658	Filling
Storage tank 2	0	0	0	0	0	1,156,600	-1,156,600	Filling

والتي تم الحصول عليها من شركة العين للتوزيع . في ذات الوقت أخذت بعض العينات من كل شبكة لعمل تحليل مخبري لمركب (THM) كل النتائج التي تم الحصول عليها كانت أقل من 80 جزء من البليون وهو الحد الأعلى الذي تنص عليه المعايير و المقاييس العالميه. هذا التحليل تم اجراءه عن طريق تجارب مخبريه اثبتت وجود مادة البروموفورم كمادة أساسية في مركب (THM) وفي كل الأوقات.

وعلى الجانب الآخر وجد أن مادة (Bromoform) تأخذ منحى انحداري بعد وقت معين من بدء التجربه وحتى تصل تقريبا إلى العدم. كل النتائج التي تم الحصول عليها من ضغوط وسرعات تدفق المياه ونسب الكلور في كل الشبكات قورنت مع الحدود المطلوبة في تقرير معايير توزيع المياه الذي تم الحصول عليه من الشركة. ومن وجهه نظر اقتصادية فقد تم التطرق لتكلفة توزيع المياه لكل المشتركين في المناطق الثلاث المشار إليها ولذلك بإعتبار سعر 5 فلس لكل كيلووات في الساعة وعلى مدى 20 سنة.

أخيرا تم مناقشة بعض التوصيات من أجل اجراء تحسينات على انظمه توزيع المياه وبخاصة الغاء الخزانات الموجودة في كل عقار والتي تسبب في فقدان الكثير من كميات المياه والتي تعمل ايضا على تقليل جودة المياه.

ملخص الأطروحة

يعتبر الماء بالنسبة للأنسان عصب الحياة، وبناء عليه أصبح من الضرورة تحليل شبكات توزيع المياه للمستهلكين من الناحية الفنية على الصعيدين النوعي والكمي المرتبطان بجودة المياة والضغوط المطلوبة في الشبكات لايصال كميات معينه من المياه لكل مستهلك حسب الطلب وبناء على ماهو متوفر من مصادر المياه.

في هذه الأطروحه تم تحليل و مناقشة بعض شبكات توزيع المياه بمنطقه العين الواقعة في دولة الإمارات العربية المتحدة من حيث قدرتها على توزيع كميات المياه للمستهلكين وذلك بالإشارة إلى الضغوط المتوفرة من مصدر كل شبكة على حده، بالإضافة إلى تحليل جودة المياه وخاصة ما يتعلق بعمليات التعقيم والكلوره.

عليه تم عمل محاكاة ونمذجة كمية ونوعية لشبكات المياه الواقعة في مناطق اليحر والظاهر وسويحان أخذا بالإعتبار خزانات المياه الموجودة في العقارات المملوكة للمستهلكين. وقد تم اختيار هذه المناطق الثلاث المشار اليها وذلك بسبب حصولها على المياه من ثلاث مصادر مختلفة وهي محطة تحليه أم النار ومحطة تحليه الطويلة ومحطة تحليه الفجيرة و تقع هذه المناطق على أطراف مدينة العين حيث أن لكل منطقة بيانات خاصة بها تم الحصول عليها من شركة العين للتوزيع، من ناحية الضغوط وكميات تدفق المياه بها.

وقد تمت عمليات المحاكاة عن طريق اقتراح أحجام ذات أبعاد معينة للخزانات الموجودة في كل عقار وذلك بعد تعديل هذه الأحجام بواسطة المحاولة والخطأ وخاصة عن طريق تغيير مستويات المياه في هذه الخزانات. وبناء عليه و أخذا باعتبار أن الشبكات المشار إليها جديدة الإنشاء فقد تم اعتبار معامل هازن وليم ثابت بقدر 120 .

بعد الإنتهاء من عملية النمذجة الكمية تم إجراء نمذجة نوعيه وخاصة للمادة المعقمة (الكلور) وقد تم الحصول على نتائج جيده بعد مقارنتها بالنتائج الحقيقية المأخوذة من نقاط معينة في كل شبكة

بإشراف الدكتور: وليد الشوربجي- استاذ مشارك – قسم الهندسة المدنية والبيئه – جامعة الإمارات

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جامعة الإمارات العربية المتحدة

عمادة الدراسات العليا

نمذجة كمية ونوعية لشبكات توزيع المياه بمنطقة العين

اطروحه مقدمة من الطالب
عوبث صالح سالم الجابري

بكالوريوس هندسه كيميائيه
جامعة الإمارات العربية المتحدة(2000)

استكمالا لمتطلبات الحصول على درجة الماجستير في علوم موارد المياه

يونيو 2006