Optimisation of Water Distribution Systems using Genetic Algorithms for Hydraulic and Water Quality Issues

By

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To my Parents

Rosslyn and Michael Hewitson

For the love and support they have shown

To my sisters

Joanne, Angela and Caroline

For keeping me out of trouble along the way

and

To my love

Lucinda

For the care she has shown, and the future we might share

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Abstract

The design and management of water distribution systems is a crucial component in providing adequate sanitation for consumers. Unless continuous supply satisfying water quality guidelines is available, the public health and aesthetics of the water are at risk. This study develops a framework balancing water quality costs resulting from waterborne disease, disinfection by-product exposure and aesthetic concerns, against hydraulic costs, which include pipes, pumps and tanks. The model solutions satisfy the hydraulic pressure and capacity requirements, while achieving an acceptable water quality level, by selecting the infrastructure combination, and dosing regime that allow a balanced system. To obtain efficient solutions, genetic algorithms (GAs) were integrated with the EPANET simulation package to optimise the solution space.

The model was checked for accuracy and efficiency by applying it to the New York tunnels problem. The GA developed in this research successfully obtained the current optimal hydraulic solution for this problem, before adapting the model to incorporate the water quality issues. The research concluded that the current optimal solution for hydraulic analysis is not necessarily the preferred solution when water quality is included in the cost structure.

To test the model on a more demanding system, the research was applied to the Yorke Peninsula network, South Australia, which is controlled by several pumps and tanks. The GA determined good solutions to this complex system, satisfying the system hydraulics, and producing an acceptable water quality profile for the system.

The research found that the inclusion of water quality into the analysis forced the solutions to reduce the residence time in the system, thereby reducing the amount of disinfection consumption in the system. The social cost framework incorporating all of the different system costs proved to be an excellent structure, producing efficient, feasible solutions. The findings from this research have implications in delivering a better quality supply to the community, and ultimately, assisting in preventing health risks associated with water.

Statement of Originality

This thesis contains no material which has been accepted for the award of any other degree or diploma at any university or other tertiary institution and, to the best of my knowledge and belief, this thesis contains no material previously published or written by another person, except where due reference is made in the thesis text.

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Chapter 1 Introduction

One of the most important aspects in the provision of acceptable health standards is the supply of a continuous source of potable water that meets desired quality criteria. The World Health Organisation has estimated that 80% of all sickness can be attributed to inadequate water supplies or poor sanitation (Gleeson and Gray, 1997). This illustrates the role of the water supply in public health and amenity.

There are several aspects that need to be considered when examining how to provide a water supply that caters for public health, ranging from a good hydraulic design to the type of disinfection and dosing regime. The design produced that incorporates these factors dictates the level of acceptability of the supply to the consumer.

With advancements in computer technology in recent times, mathematical models are becoming increasingly important in the development of good design practice. Not only can they simulate a system based on a specified design, they can also be applied to an optimisation model to determine efficient solutions for a specific system. Although the majority of the previous research in this field has been centred on hydraulics, recent research has begun to incorporate water quality as an important facet in system design.

This study examines the application of genetic algorithms, an optimisation technique that has been proven for the hydraulic design of pipe network systems (Simpson *et al.*, 1994). This method has been applied to the layout and operation of distribution system design to provide an overview on what effect the inclusion of water quality as a design factor will have on the system infrastructure.

1.1 Objectives and Scope

The objectives of this research include:

- To examine both the hydraulic and water quality issues that affect the design and/or operation of water distribution systems;
- To provide a structure that allows the quantification of these issues so that a balanced approach towards system design can be achieved;
- To produce an efficient optimisation program and link it with a recognised simulation package to allow the analysis and optimisation of the total cost structure;
- To examine whether the design of a distribution system with the inclusion of water quality has an effect on the layout of the network as opposed to a design based purely on hydraulic aspects;
- To discuss the different relationships between the cost components; and
- To examine the effectiveness of the different methods allowing the determination of an efficient system.

1.2 Layout and Content of Thesis

Chapter 2 in this thesis addresses the analysis and modelling of hydraulic systems, outlining the different methodologies available for simulation and optimisation of water networks. Both real time and steady state analysis methods are examined in this chapter.

The overview of modelling is extended in Chapter 3, which incorporates water quality into the structure. This chapter discusses the different facets that may affect the water quality costs incurred by the system and examines the process of quantification of these issues. The potential risks due to disinfection by-products and waterborne disease are addressed in an even discussion providing a balanced overview in the role disinfection has in water supply. Chapter 3 concludes with examples of modelling methods used to not just analyse water quality in supply systems, but to provide optimal practice for provision of good water quality.

The modelling technique of genetic algorithms has been used to optimise the information from Chapters 2 and 3. Chapter 4 gives an overview of how genetic algorithms operate, and provides comparisons with other optimising methods. This chapter also discusses how this method will be applied towards the hydraulic and quality framework aforementioned.

Chapter 5 provides the bulk of the formulation for the model. It examines the cost structures for the infrastructure costs, operation and maintenance costs, and potential water quality costs. This chapter details the methods used to help confine the solutions to a feasible range and describes how all of the different aspects fit together to form the total cost structure.

Chapter 6 introduces the New York Tunnels system, the first case study for this thesis. This chapter examines the history of the system, the research history of the study, and the makeup and expectations of the current system. It provides specific information relating to hydraulics and water quality for this system, including calibration of decay rates and the infrastructure cost structure.

Chapter 7 is the results chapter for the New York Tunnels study. This chapter compares the hydraulic results produced against those of previous researchers as a means of checking the potential and accuracy of the model. Chapter 7 then includes the various water quality aspects into the optimisation, looking at how they inter-relate, and what effect the water quality components have on the layout of the network. This chapter outlines some conclusions drawn from these results that provide an interesting insight into design of gravity fed systems.

Yorke Peninsula is introduced as the second case study in Chapter 8 of this thesis. This chapter provides a more specific formulation and calibration of this system for model application. Chapter 8 discusses the assumed decay rates and seasonal system operation used in determination of a water quality overview.

Chapter 9 provides the results from the Yorke Peninsula study. This chapter produces efficient solutions that provide adequate water at good quality throughout the network. This chapter also looks at the potential of this technique in optimising for complex systems with controls as opposed to relatively simple gravity fed systems as discussed in Chapter 7.

Chapter 10 discusses and provides conclusions from the two studies and looks at how this research may be extended by other researchers in the future.

Chapter 2 Water Supply Systems

2.1 History of Water Distribution

Engineering is the art of using natural resources for the benefit of humankind. As such, engineering has been practised since the beginnings of human existence. Engineering practices became particularly important around 8000BC, when the development of techniques for cultivating crops enabled the human population to increase significantly beyond that of a hunter-gatherer society. New tools were needed to plant, harvest and process crops as well as to supply water to the crops and to human populations.

Early forms of water distribution were irrigation canals. As populations became more settled, greater water supply was required to assist in food provision. By 3000BC the Sumerians carried out irrigation using a system of canals (Sprauge de Camp, 1963).

The early Roman engineers (around 100BC) transferred water from water sources to townships using vast aqueduct systems, many of which are still standing today. During the Roman era the science of hydraulics was not understood, as they firmly believed that the flow in their aqueducts was proportional to the cross-sectional area of the channel and ignored the slope. Lead poisoning, due to the lead lining in the Roman distribution systems, was detrimental to the consumers, although calcification in many systems prevented the lead from entering the water supply (Hodge, 1992).

The pumping of water was not widely practised until coal became a major fuel source around the turn of the 14^{th} century. As the coal shafts deepened, groundwater began to fill the diggings. This state of affairs led to the invention of horse and water-powered mechanical pumps to keep the water out. The Dutch also incorporated pumps at the turn of the 15^{th} century to de-water land from seawater using wind-powered pumps and dykes (Sprauge de Camp, 1963).

During the Middle Ages, municipal waterworks slowly began to regain ground since the fall of the Roman Empire. Taking the city of Paris as an example, in the 9th century, Norse invaders destroyed the Roman aqueduct built under Emperor Julian century's prior, and for three centuries after its destruction, Parisians took their water supply entirely from wells and the Seine River. In the 12th century, abbeys began to take their water through a series of pipes from springs as they found the river supply too inconvenient. The abbeys turned the waterworks over to the city after Parisians started demanding more wells. The aristocracy began to take private lines to their houses and before long, the infrastructure no longer existed to provide a reliable supply to the public.

This problem of insufficient supply continued until 1600AD. The French king, Henri IV, authorised a Flemish engineer to install a pumping station under the Pont Neuf. Using a water wheel, which worked 4 pumps, the river water was raised to a height where it could be gravity fed into the royal houses and public domain. Hereafter, Paris saw the evolution of a modern water system.

The development of water supply systems has evolved in several ways since Paris in 1600AD. Current trends in water distribution engineering involve significant water treatment processes to make the water satisfy health regulations, then transport the water through complex networks of pumps, pipes, valves and tanks to the end user. This practice is itself an art, as it forms a range of different options that must be configured to provide an efficient service at reasonable cost, thus contributing to an increased standard of living of society as the ancient engineers started to do thousands of years ago.

2.2 Analysis of System Hydraulics

Water distribution systems consist of an interconnected set of pipes, pumps, valves and tanks. The aim is to transport water of suitable quality to the consumer in the quantities needed at suitable pressures.

For the analysis of pipe networks, it is assumed that the following quantities are known:

- The pipe layout;

- The demands at all nodes;
- The diameters of all pipes (and their corresponding roughness factors);
- Heads at one or more nodes (ie. tanks and reservoirs);
- The characteristic curves of all pumps; and
- The head loss relationships for all valves.

The basic geometry for a pipe network comprises of several pipe sections with constant diameter that may contain pumps, fittings or bends. The end points for each section are the pipe junctions or fixed grade nodes such as reservoirs. Primary loops can be identified, which include all closed pipe circuits that contain no additional circuits within them. Once the junction and fixed nodes are identified, along with the primary loops, the primary relationship between the number of pipes, nodes and loops is given by Equation 2.1 (Wood and Funk, 1993).

$$nP = nJ + nL + nF - 1 \tag{2.1}$$

Where nP = number of pipes, nJ = number of junction nodes, nL = number of primary loops and nF = number of fixed grade nodes.

The basic equations in the analysis of pipe systems are the continuity equations (Equation 2.2) and the head loss equations (Equations 2.3 and 2.4).

$$\sum Q_{in} - \sum Q_{out} = Q_e \tag{2.2}$$

Where Q_{in} = flow entering node, Q_{out} = flow exiting node and Q_e = external inflow (negative) or demand (positive) at node.

In the analysis of pipe networks, velocity heads and minor losses are assumed to be negligible, and the losses at the junctions in the network are also assumed to be negligible (Dandy *et al.*, 1993). In a loop, with i pipes, the head loss around the loop should be zero as shown in Equation 2.3.

$$\sum_{\substack{i=1 \\ i \in K}}^{Pk} \left(h_f\right)_i = 0 \tag{2.4}$$

Where $(h_f)_i$ = head loss in pipe *i*, *K* in the set of all pipes in loop *k*.

To quantify loss in head through the pipe due to the friction, the Hazen-Williams equation for water at ordinary temperatures (Streeter and Wylie, 1983) is used. This is seen in Equation 2.4.

$$\frac{h_f}{L} = \frac{RQ^n}{D^m} = \frac{10.675Q^n}{C^n D^m} \quad \textbf{(2.4)}$$

Where h_f = head loss in the pipe, L = length of pipe, R = resistance coefficient, Q = discharge in the pipe, D = diameter of the pipe, C = the Hazen Williams roughness coefficient, n = 1.852, m = 4.8704. The constant R is for metric units.

The Hazen Williams roughness coefficient is based on the type and condition of pipe, as listed in Table 2.1 (Streeter and Wylie, 1983).

Table 2.1 Hazen-Williams roughness coefficients

Roughness Coefficient (C)	Description
140	Extremely smooth, straight pipes; asbestos cement
130	Very smooth pipes; concrete, new cast iron
120	Wood stave; new welded steel
110	Virified clay; new riveted steel
100	Cast iron after years of use
95	Riveted steel after years of use
60-80	Old pipes in bad condition

Another method to calculate the head loss in pipes is the Darcy-Weisbach equation (Douglas *et al.*,1985), which for steady, uniform flow is given by Equation 2.5.

$$h_f = \frac{fL}{D}\frac{v^2}{2g} \quad \textbf{(2.5)}$$

Where f = Darcy-Weisbach friction factor of pipe, v = velocity in pipe and g = acceleration due to gravity.

For this calculation, the friction factor (*f*) of the pipe is the important coefficient. This is a function of the Reynolds number of the flow and the roughness of the pipe. It may be determined from a Moody Diagram (Streeter and Wylie, 1983).

The level of accuracy with the determination of the head loss due to friction is important in the overall pressure analysis of the system, and the provision of an adequate hydraulic grade-line catering for the system constraints.

Some quantities in the data require estimation, such as the pipe roughness levels. The model undergoes calibration to check that the simulated pressures agree with the observed nodal pressures in the system (Walski, 1992).

The unknown quantities in Equations 2.2 and 2.3 are the nodal heads and the flows throughout the system (Dandy *et al.*, 1993).

Applying loop or node equations can reduce the number of unknowns. Common solution techniques include:

- The Hardy Cross Method (Cross, 1936)
- The Newton-Raphson Method (Shamir and Howard, 1968); and
- The linear theory method (Wood and Charles, 1972).

2.3 Optimisation of Water Distribution Systems

Application of the mathematical models to different optimisation methods provides the means to achieve cost efficiency for specific networks. Optimisation models may be used to identify minimum cost designs for water distribution systems. When applied to controlled systems, they may be required to increase the efficiency of the controls, which include leak reduction with 15%-40% of water normally unaccounted for, and more efficient pump practice allowing 5%-20% cost reduction in a typical system (Jarrige, 1993).

For a typical gravity fed system, the cost function to be optimised may have the expression given in Equation 2.6.

$$\min Z = \sum_{j=1}^{NP} C(D_i)L_j \tag{2.6}$$

Where $C(D_j)$ = cost per unit length of diameter D_j for pipe j, L_j = length of pipe j, and NP = total number of pipes.

A number of techniques have been used in the optimisation of pipe systems, they include:

1. Partial enumeration (Loubser and Gessler, 1990);
2. Linear Programming (Jarrige, 1993);
3. Dynamic Programming (Ormsbee, 1992);
4. Non-linear programming (El-Bahrawy and Smith, 1985); and
5. Genetic Algorithms (Simpson *et al.*, 1994).

2.3.1 Partial Enumeration

Partial enumeration was proposed to overcome many of the practical considerations that make network optimisation difficult. Unlike complete enumeration, which evaluates every possible solution, partial enumeration evaluates a portion of the total search space. Complete enumeration, although guaranteeing an optimal result requires extensive computer time, even for relatively simple systems (Dandy, *et al.*, 1993). For example, a system with 16 pipes each with 8 possible diameters would have to be subjected to 8^{16} or 2.8×10^{14} evaluations, which (assuming that each evaluation takes one second) would take 9 million years.

By having an understanding of the systems flow pattern, decisions can be made that reduce the search space to be enumerated. Taking the example of pipes, some of the pipes are assumed to be of the same size, hence allowing a reduction in the number of diameters to be found. Other pipes may require fewer options than the default list of options, hence reducing the search space. This process is also applicable for tank and pump sizing.

Partial enumeration requires experience with the pipe system in question. However, even though intuition and experience can prove invaluable in reducing running time, they may also lead to the exclusion of the optimum solution from the search space thereby losing cost-efficient solutions.

2.3.2 Linear Programming

Linear programming (LP) is used to determine the optimal value of a linear function of a certain number of variables, given a set of linear constraints on these variables. Linear programming has been applied to a large number of practical applications in real problems, and has often been used in industry, government organisations, ecological sciences and engineering to optimise constrained objective functions.

The most common solution approach for LP is the Simplex method, which offers a very efficient means of determining the optimal solution by repeatedly increasing the decision variable causing the greatest improvement in the objective function (Ormsbee *et al.*, 1992).

LP consists in solving the linear problems, and developing algorithms and software able to find the optimal solution to a problem. Two methods have been developed to linearise the network optimisation problem for gravity fed systems. One involves only considering discrete pipe sizes and using pipe length as the principle variables (Alpertovitis and Shamir, 1977). As the cost function and head loss equation are linear with length, this produces a linear optimisation model. The objective function for this method is given by Equation 2.7 (Dandy *et al.*, 1993).

$$\min Z = \sum_{j=1}^{NP} \sum_{m=1}^{M} C_{jm} X_{jm} \tag{2.7}$$

Where X_{jm} = the length of pipe j of size m, C_{jm} = cost per unit length of size m for pipe j, and M = number of available pipe sizes.

The other method involves defining a new variable X for each pipe equal to the diameter rasied to the power of 2.63. This effectively linearises the head loss equations (Quindry, 1981).

In the application of LP to a problem incorporating pumps, valves and tanks (Jarrige, 1993), it was assumed that pump and valve operation are not effected by reservoir levels, and that the hydraulic effects and costs of any command (pump, valve settings) are able to be pre-computed.

The linear formulation was then obtained by using reservoir volumes and command application (pump and valve settings) as variables, then solved using a simplex-like method.

Jarrige (1993) found that although the linear formulation showed its ability to solve medium-size problems with a good accuracy for pump interaction and hydraulic capacity, the method was unable to model reservoir equilibrium accurately.

2.3.3 Dynamic Programming

Dynamic programming (DP) is best suited for problems that can be broken down into sub-sections. However, it does not require linearity of the objective function of constraints. This technique is based upon the theory that to achieve an optimal solution from an initial decision, there must be an optimal path between the first stage and the last stage to find the optimal solutions based on Bellman's principle of optimality (Ormsbee *et al.*, 1992):

'An optimal policy has the property that whatever the initial state and decisions are, the remaining decisions must constitute an optimal policy with regard to the state resulting from the first decision.'

This principle enables an optimal solution to be found by evaluating all state transitions between adjacent stages, instead of all transitions along all stages. It is an easier task to locate an optimal curve in a search space than an optimal point. A positive feature of DP is its ability to provide a global optimum.

The disadvantage with DP is the calculation time required as it increases exponentially with the number of system facets being optimised (Dandy and Warner, 1989). Jarrige (1993) found that this method was applicable to distribution systems in a limited form. Due to the computational requirements, it was determined that this method could optimise for:

- Single reservoir systems of any complexity; or
- Dual-reservoir systems with a simplified hydraulic model.

For larger systems, further simplification is required.

2.3.4 Non-Linear Programming

Non-linear programming (NLP) is a compromise between the powerful linear programming method and the robust dynamic modelling. This method is suited to problems that have a non-linear objective function and/or constraints.

NLP differs from LP in three main areas (Dandy and Warner, 1989):

1. The feasible search space for a non-linear problem may be convex or no-convex;
2. The optimum solution to a non-linear problem may occur at any location within the feasible region (not necessarily at an extreme point); and
3. It is possible for more than one optimum to exist, with the global optimum encompassing the entire search space, and local optima being the best solutions for only a potion of the total search space.

An initial estimate of the optimal solution is determined by changing the decision variables that affect the associated objective function. This, like the simplex method from LP is not a random method, as the variables are altered based on a change in the objective function and constraints. This process is constantly updated until no further improvement in the objective function is achievable (Ormsbee *et al.*, 1992).

The objective function for the application of NLP for optimisation of gravity fed systems can take the form given in Equation 2.8 (Dandy *et al.*, 1993).

$$\min Z = K\sum_{j=1}^{NP} L_j D_j^{\alpha} \qquad \textbf{(2.8)}$$

Where *Lj* are known K and α are determined from fitting a cost curve to pipes of known diameters. The constraints are given by Equations (2.2) and (2.3).

As a local optimum can be found using this method rather than the global optimum, it is preferable to solve the problem several times starting at several different initial solutions (Dandy *et al.*, 1993).

2.3.5 Genetic Algorithms

Genetic Algorithms (GAs) are a robust optimisation method that has been successfully applied to the optimisation of distribution systems (Murphy *et al.*, 1996; Simpson *et al.*, 1994). Chapter 4 describes the process behind GA optimisation, and compares this technique to other methods.

Other methods such as simulated annealing have also been applied to water distribution systems (Cunha and Sousa, 1999).

2.4 Summary

Hydraulic analysis of a water distribution system is an excellent tool in determining the operational aspects of a specific network. Optimisation models combined with analysis packages provide an excellent opportunity for cost-efficient design and operation of water supply systems.

The advances in modelling of system hydraulics have enabled research into water quality analysis to provide a more detailed overview of the network. It is this element of research that has been taken up in this thesis.

Chapter 3 Water Quality in Distribution Systems

In the past, the major focus in designing distribution systems has been to meet the hydraulic requirements, then address the water quality concerns. Chapter 3 examines the possible effects that the distribution system can have on water quality, addressing both the infrastructure and treatment aspects of the network.

A major issue for water managers is determining the potential problems with the water in terms of consumer confidence in the supply, and how concerns, such as waterborne disease, taste and odour and disinfection by-products (DBPs) can be assessed adequately. Risk assessment is a means of quantifying these issues, however, due to the lack of definitive knowledge on the factors causing the concerns and the potential risks from them, the process, whilst useful, has limitations in its current form. Once quantified, an examination of the potential benefits and costs from different operation options relating to infrastructure selection and dosing levels is discussed, culminating in a discussion on the balancing of the concerns arising from the different options.

Some of these concerns can be traced using modelling techniques, which have the ability to not only simulate the water quality conditions, but optimise them. This study will take this optimisation capability a step further by examining a framework that balances water quality concerns, while satisfying the system hydraulics.

3.1 Effect of System on Water Quality

The system infrastructure can have major implications on the water quality aspects of the network. As the water quality can be affected by the residence time in the system, an accurate model of the hydraulics, which determines the residence times, becomes very important.

The importance of the distribution system in relation to water quality can be seen from a study carried out by the United States Environmental Protection Agency (USEPA) for Peru (Clark *et al*, 1993) following a major outbreak of cholera. The following recommendations were made to improve the system quality:

1. Flushing of systems should occur in the cities where the outbreak was identified to remove pathogens and reduce chlorine demand;
2. Contaminated water must be disinfected; and
3. Water system operation should provide continuous supply and not allow intermittent flow.

The third recommendation required a positive pressure barrier to be in place, preventing backflow of contaminated surface water runoff or intrusion of sewage.

The longer-term recommendations, listed below, focussed more on the system infrastructure:

1. Reduction of pipe leaks and unauthorised connections, which present water losses;
2. Implement a policy to maintain water pressure in the system; and
3. Develop protocols for public water system operation and design.

Improving the system to reduce leaks and maintain pressure involves major infrastructure alterations, with replacement of inadequate pipes and duplication in areas to relieve the pressure concerns.

3.1.1 Pipe Effects

As pipes usually represent the majority of the infrastructure in a water distribution system, analysis of their effect on the water quality is an area of interest.

3.1.1.1 Pipe Diameter

If the pressure requirements at the nodes are not met, pipe backflow and dead water pockets can occur in the system, both of which can present quality issues for the supply.

The larger the diameters of a specific pipe type, the lower the head loss throughout the pipe. This is due to the reduced shear force by the pipe wall per unit volume of water. Diameter is inversely related to the loss of head throughout the system. Therefore, selection of pipe diameter can have ramifications on the network's pressure situation.

3.1.1.2 Pipe Material

The type of pipe has an effect on the roughness, which physically affects water quality by increasing the head loss, potentially adding to pressure problems in the system. Often the pipe material itself can have adverse effects on water quality from corrosion and leaching, which can have a chemical effect on the supply. Generally, the level of health concerns is inversely related to the smoothness of the material. A completely smooth pipe would not attract biofilm and dependent on pipe material, be unlikely to corrode.

The early water engineers, the Romans, often used lead piping in their expansive networks (Hodge, 1992). Historians have alluded to lead poisoning from piping and cooking implements as one of the reasons for the breakup of the Roman Empire. While contemporary pipelines are not made of lead, potential quality concerns can arise from some of the current pipe materials such as leaching in metal and asbestos-cement pipes.

The leaching of metal and asbestos-cement pipe materials, and pipe corrosion can cause water quality changes in distribution systems (Logsdon *et al.*, 1991). Corrosion can result in depressions forming where bacteria can attach themselves. To stop this leaching, the pipes are usually lined with cement.

The reason for pipe lining is to discourage pipe corrosion or taste and odour problems (Geldreich, 1996). The amount of chlorine required varies significantly with pipe material. For example, iron pipes require up to ten times the chlorine than other materials to maintain a residual. This additional chlorine demand presents problems with waterborne disease, disinfection by-product formation, and taste and odour. The relationship between chlorine demand and these water quality concerns is discussed later in this chapter.

Cement mortar lining involves spraying cement on the inside of iron pipes. It is commonly used for the protection of new pipes, and the rehabilitation of old pipes. A potential problem with this application is the leaching of lime under excessive pH levels. Extraction of aluminium (Al), iron (Fe), sodium (Na), potassium (K), and calcium (Ca) could also become an issue (Conroy *et al.*, 1993).

Epoxy resin linings are an alternative to concrete lining. As the epoxy lining is formed by a complex chemical reaction between the resin and hardener components, a complex mixture of chemicals can result. Concern of this technique is centred on the potential leaching of the chemicals bound up in the lining (Conroy *et al.*, 1993).

3.1.1.3 Pipe Age

The age of the pipeline in the system can have adverse effects on the head loss, which can force duplication, hence increased residence time. A typical relationship between pipe age and roughness coefficient (Hazen-Williams C value) can be seen in Figure 3.1 (Eggener and Powlowski, 1976).

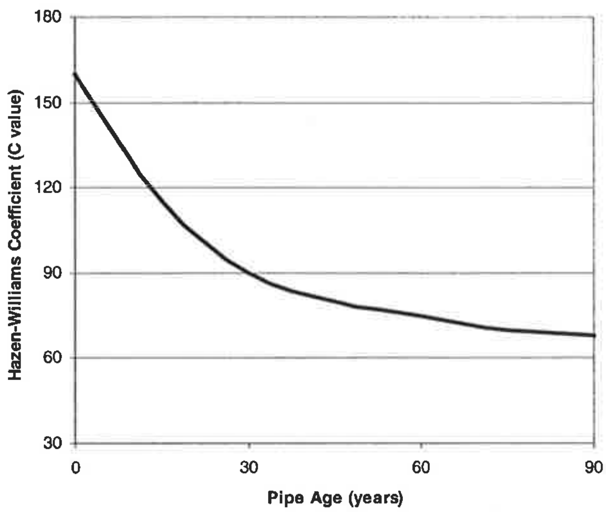

Figure 3.1 Effect of pipe age on roughness

The age of the pipelines can also have adverse effects on the reliability, hence, bursts, and leaks are more likely, creating potential entry points for microbial contamination. The greater pipe roughness also provides a more protected environment for microorganisms attached to the pipe wall.

The greater the level of biological build-up on the pipe wall can lead to an increase in the required pumping time and costs. This is caused by the reduction in capacity through the pipe as the biological material effectively reduces diameter and increases the resistance to flow (Baker, 1986).

3.1.2 Tank Effects

One of the major factors that determine the residence time of the network, are the balancing storage points. Tanks and reservoirs are usually designed to cater for the hydraulic requirements in the system. The water quality aspects are only considered secondarily. However, storages have been considered to have a likely role in waterborne disease outbreaks (Grayman *et al.*, 1996). So, the effect that tanks have on the system is important in the network design.

Long residence times in a system can have an adverse effect on the quality in the system as this causes a reduction in disinfectant residuals, DBP increase and potential bacterial regrowth. Although due to a lack of technology in the past to assess and evaluate such issues, long residence times were seen to be justified for hydraulic purposes (Boulos *et al.*, 1996).

3.1.2.1 Mixing in Tanks

If the mixing in tanks is inadequate, pockets of dead water can be created that can affect both public health and the aesthetic nature (taste and odour) of the supply. Rossman and Grayman (1999) found that for common storage structures, the time it took to achieve complete mixing followed Equation 3.1:

$$Tt_{mix} \propto \frac{V_i^{2/3}}{M_f^{\ 1/2}} \qquad \textbf{(3.1)}$$

Where Tt_{mix} = time taken to achieve mixing, V_i = initial volume of water in storage, and M_f = inflow momentum flux.

If the system storages achieve acceptable mixing, the consistency and the quality of the supply is increased.

3.1.2.2 Sizing of Tanks

As residence time is proportional to tank volume, selection of the correct tank size is important in the provision of a good quality supply. The selected tank must provide adequate emergency storage for hydraulic purposes (EWS, 1986(b)), while not storing any excess that increases detention time.

3.1.2.3 Location and Operation of Tanks

As with the sizing of tanks, the location and operation levels of the tanks can have an effect on water quality. The location of the storage is important, as the system should use it efficiently, and should not be cut off from the supply.

If the operating levels are too close together, the water at the bottom of the tank may stratify and age, causing poor quality supply. If they are too far apart, the tank could run out of water, which would not only impact on the hydraulic capability, but may also adversely affect water quality. Grayman and Clark (1993) highlighted the importance of tank design and operation in reducing the potential degradation of water quality due to the long residence times. The operating levels also control the pumps that supply water to the tanks.

3.1.3 Pumping Effects

Pumping is the process that mechanically adds head to the supply. It can affect the quality of the supply in the following ways:

- If pumps are too small, they may not be able to provide a pressure balance, hence backflow can occur;
- The pumps can reduce the residence time in the system, reducing the disinfection demand;
- If the pumps force the head too high, the stress may cause leaks in the network; and
- If there is a section of the system i.e. a tank supplying a region which is experiencing poor water quality, a more powerful pump could get water to that location more quickly and thus improve its water quality.

Operation of the pumps is very important for an efficient supply, both hydraulically and for quality purposes. Optimisation of system pumping has been researched for both hydraulic alone (Murphy *et al.*, 1994) and combined with water quality (Sakarya and Mays, 1999; Goldman and Mays, 1999).

3.1.4 Treatment Process

The treatment process controls the water coming into the network, and the discouragement of microbial growth throughout the network.

3.1.4.1 Treatment Plant

The treatment plant attempts to provide a pristine supply entering the network. Using a series of processes such as coagulation, flocculation, filtration, and dosing, the water entering the system should meet health regulations. In some rural areas, water treatment can be little more than dosing with chlorine.

3.1.4.2 System Flushing

Flushing is used to remove the accumulation of organic nutrients, inorganic corrosion particles and biofilm and to restore disinfectant residuals (Geldriech, 1996). It is a process to be undertaken after obtaining a good knowledge of the system. Knowing where any high coliform counts are, or regions of taste and odour concern are important, along with the hydraulic knowledge of street valves, fire hydrants, water pressure zones and flows to each area.

Flushing cleans the pipe system, which allows a regular treatment process to maintain a good level of water quality. The common method used applies different dosing levels throughout the year in response to residence times and seasonal decay rates.

3.1.4.3 Dosing Regime

The dosing regime is the regular dosing pattern throughout the year in the system. This acts as the protective barrier against regrowth of microbial activity. The makeup of the system will dictate the dosing regime as the pipe type, pipe age, storage locations and pump size will all contribute to determine what level of disinfectant should go in to allow proper disinfection.

The type of disinfectant is also very important. As they have varying decay and success rates against a variety of microorganisms, the selection of disinfectant will have a marked effect on system quality. Again the disinfectant choice can be dictated by the makeup of the system. If the system has a long residence time, a slow decaying disinfectant such as chloramine could be used. If it is a modern system with low residence times, a residual may not even be required, with UV or ozone disinfection at the treatment plant considered adequate.

The effectiveness of the dosing regime is dependent on the system operators. Application of tools such as modelling to assist in the dosing regime will inevitably aid the improvement of the quality of the water supply (Rodriguez *et al.*, 1997).

3.1.5 Assessing Effect of System on Public Health and Amenity

Whether due to infrastructure type and location, or water treatment methodology, the distribution system can have a marked effect on water quality. The issue then becomes how the water quality levels in the system affect the consumers. Whether due to aesthetic concern, potential health problems caused by waterborne disease or disinfection by-products, the system has a large part to play. In determining the scale of the potential quality concerns in a given system, assessment of the quality issues is important.

3.2 Risk Assessment of Water Quality

Research into the potential risks of illness from drinking water consumption is still in its relative infancy. Many of the waterborne disease outbreaks come from unknown origins, and there is still uncertainty as to the role disinfection by-products play in cancer contraction. Assessment of the potential risks based on the current level of knowledge is a difficult task, however, even in a limited form it is a useful tool in water quality analysis.

3.2.1 Methodology of Risk Assessment

The common methods of risk determination can be positioned into three main sectors as listed in Table 3.1 (Thomas and Hrudey, 1997).

Table 3.1 Categories of health risk evidence and inference

Factor	Direct	Indirect	Predictive
Knowledge Source	Evidence	Evidence + inference	Inference
Information Source	Death certificates	Epidemiology	Toxicology
Coverage	Everyone	Population sample	Laboratory animals
Information Type	Age, gender and cause of death	Exposure & effect association for risk factor exposure	Estimate risk associated with a specific exposure

Direct evidence is the summarised data from death certificates providing the apparent cause of death which is statistically analysed and the relevant risks assessed. Indirect evidence determines risk factors based on health risks and can be applied to morbidity studies as well as mortality. Predictive evidence is usually produced from laboratory studies on animals.

3.2.1.1 Direct Evidence

Direct evidence stems from death certificates, which list a cause of death. If a person falls off a building, or drinks water with a high level of cyanide contamination, the certainty of the evidence is good. If the deceased died in their sleep, or died of a long-term illness, it may be difficult to ascertain what was the cause of death. As Hrudey (1999) maintains, knowing that about one out of three Canadians will die of cancer does not tell us the factors causing the cancer. For example, if a coal miner contracts lung cancer, it could have been alternative factors such as smoking that caused the illness rather than the occupation.

3.2.1.2 Epidemiology

Merriam-Webster's dictionary (Merriam-Webster, 1998) defines epidemiology as:

1. a branch of medical science that deals with the incidence, distribution, and control of disease in a population
2. the sum of the factors controlling the presence or absence of a disease or pathogen

Epidemiological analysis of drinking water quality aims to assess the possible changes in morbidity (illness) or mortality (death) linked to the consumption of drinking water. As is the case with drinking water analysis, it is difficult for environmental epidemiologists to associate low-level exposure or a long duration to irregular health effects (Aldrich and Griffith, 1993).

Epidemiological studies develop evidence for association between a cause and an outcome. More simply, they determine risks based on the possible outcomes listed in Table 3.2 (Thomas and Hrudey, 1997).

Table 3.2 Epidemiological theory

	Disease	No Disease
Exposure	A	B
No Exposure	C	D

This could represent the analysis of a drinking water system for the potential health risks caused by use of a disinfectant. The exposure in this example is from consumption of supply using the disinfectant whereas no exposure is for consumption of water supply not using the disinfectant. The options of disease or no disease for this example are for the contraction of the studied illness. The four possible outcomes are:

- A – The consumer has contracted the disease and is exposed to the disinfected supply;
- B – The consumer has not contracted the disease but is exposed to the disinfectant;
- C – The consumer has contracted the disease but isn't exposed to the disinfectant; and
- D – The consumer neither contracts the disease nor is exposed to the disinfectant.

Epidemiology determines a relative risk (RR), which indicates the change in risk an individual may encounter if they change from an unexposed lifestyle to one that is exposed. The formulation for the relative risk is given in Equation 3.2 (Thomas and Hrudey, 1997).

$$RR = \frac{\left(\dfrac{a}{a+b}\right)}{\left(\dfrac{c}{c+d}\right)} = \frac{\left(\dfrac{\text{disease (exposed)}}{\text{total exposed}}\right)}{\left(\dfrac{\text{disease (not exposed)}}{\text{total not exposed}}\right)} \tag{3.2}$$

The greater the knowledge of the history of the population, the greater the accuracy of the study, as other factors including work, lifestyle, race, gender and even recreational pursuits often have to be factored in to the analysis.

3.2.1.3 Toxicological Risk Assessment

Toxicology is the study of how substances act as poisons in a human system. This is often done by rigorous laboratory testing and has the potential to be the least accurate form of analysis.

One of the reasons for the reduced accuracy stems from the analysis of the substance on its own, where human exposure always involves a complex combination of different substances (Thomas and Hrudey, 1997). In addition, the testing is often undertaken on rodents, which do not necessarily respond in the same way as humans.

3.2.2 Accuracy of Risk Assessment for Water Quality

Determination of the accuracy of risks for drinking water is a very difficult task. With many of the waterborne diseases, the source of the illness could be traced to food, person to person contamination, pets or even recreational water activity. Limiting the analysis to drinking water is difficult, especially considering the many different types of waterborne illness that exist. Data that places direct risks to certain levels of water quality is not extensive, as the area of study is very broad with the discovery of new microorganisms and strains of illness.

3.2.3 Establishing a Framework to Consider Risk

Although limited by the certainty that can be placed on the potential issues of water related illness, the establishment of a framework that considers the different concerns is a potential method for the quantification of water quality in distribution systems. Though basic in form, such a framework will allow approximate relationships to be formed. These relationships can then be built upon as a greater knowledge of the risks associated with drinking water are identified.

3.2.3.1 Risk Quantification

To provide the basis for a framework, quantification of the potential water-related risks is necessary. This allows the quality aspects of the network to be valued so that they can be considered with the hydraulic components to provide a more complete system overview.

3.2.3.2 Value of Life and Safety

Once a risk level has been ascertained, a cost is placed on this risk by including the potential costs of morbidity or mortality of the illness. This is discussed in greater depth in Section 3.10.

3.2.3.3 Interaction of Different Risks

After the risks have been valued and quantified, they can be balanced to allow operational assistance for the system. Although there is still much more research required in this field to help tighten the values from risk determination of drinking water, the information to date allows some analysis to take place. The issues surrounding the interaction and balancing of risks are discussed further in Section 3.11.

3.3 History of Consumer Concerns

This section aims to place the different aspects of water quality included in this study into a loose chronological order, which will provide the structure for the remainder of this chapter. The different aspects to be considered are:

1. Waterborne disease;
2. Disinfection;
3. Taste and odour; and
4. Disinfection by-products.

3.3.1 Waterborne Disease

Probably the oldest risk factor associated with a water supply is waterborne disease. Although not officially recognised until the 19th century, disease affecting human health dates back as far as humanity itself.

Prior to the 1850's, there had been considerable theorising concerning the cause and transmission media of disease. Proof of the theory had yet to be established, as the science of bacteriology was not known. In 1854, London experienced a localised outbreak of Asiatic cholera, which prompted two men, John Snow and John York to investigate. They demonstrated that the source of infection was from a local pump. They further discovered that the well had been contaminated from a nearby sewer. This case study proved to be a benchmark for engineering practice as it established that water was a vehicle for disease transmission.

Scientific advancements have increased the awareness of the extent to which waterborne disease affects humans. Not only have many more causes for illness been exposed, but knowledge of the transfer mechanisms and origins of these diseases has improved greatly.

3.3.2 Disinfection

Regular chlorination of water supplies was not introduced until 1904, when a public water supply in England employed continuous chlorination. In 1908 and 1909, hypochlorite was used to disinfected supplies in Chicago and New Jersey respectively. In a legal case concerning the practice of disinfection, it was ruled that the city had the right to chlorinate supply in the interests of public health.

It was not until the development of facilities for feeding gaseous chlorine into the supply during 1912 that disinfection of supply using chlorine started to become commonplace. The improved sanitation method of disinfecting water supply along with other sanitary practices including milk pasteurisation greatly reduced some of the common diseases of the period such as typhoid as shown in Figure 3.2 (Sawyer and McCarty, 1978).

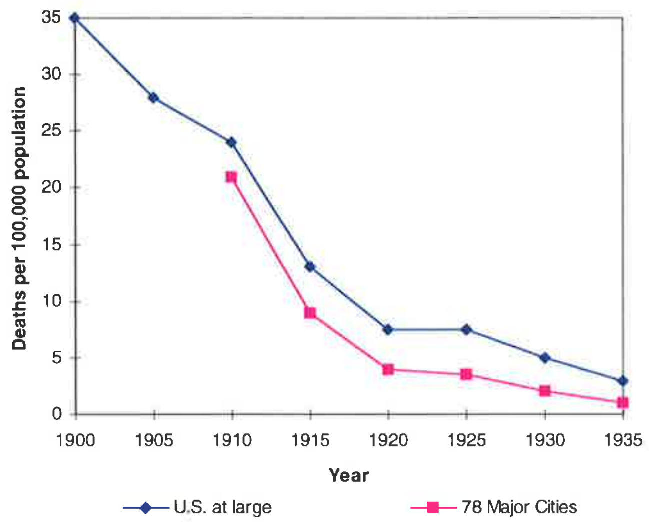

Figure 3.2 Reduction in Typhoid mortality in the United States

The use of disinfection has had a drastic effect on waterborne disease, as Section 3.6 will discuss.

3.3.3 Taste and Odour

Taste and odour is a concern in supply systems as long as there is an authority to question, as found in the 18th century in the old New York supply (Weidner, 1974). Even so, it was not until disinfection was introduced at the start of the 20th century that taste and odour became linked with disinfection practice.

3.3.4 Disinfection By-products

Disinfection by-products were a direct result of the interaction between the disinfectant and the organics in the supply. Their existence in supply systems would have followed almost immediately after disinfection, although their relevance to the supply was not realised until much later.

Concerns with chlorinated compounds began to surface with pesticides in the 1960s, then with contaminants such as trichloroethylene (TCE). In 1974, Johnnes Rook published results implicating the use of chlorine in water supplies as the cause of trihalomethanes (THMs) (Minear and Amy, 1996). In 1979, legislation was implemented in the USA by assigning a maximum contaminant level (MCL) of 100µg/<L for THMs (Oxenford, 1996).

3.3.5 Summary of Consumer Concerns

The aspects from Section 3.3.1 to Section 3.3.4 make up a structure that needs to be balanced by operators of distribution systems. If risk assessment and quantification is applied, estimation of the potential water quality costs is possible.

3.4 Waterborne Disease

A considerable portion of the world's population is without access to potable water supplies or adequate sanitation. The World Health Organisation has estimated that 80% of all sickness can be attributed to inadequate water supplies or poor sanitation (Gleeson and Gray, 1997). Outbreaks of waterborne disease are not just isolated to developing countries with basic system structures. In 1993 and 1994, 30 waterborne disease outbreaks were reported throughout the United States affecting an estimated 405,366 people (Kramer *et al.,* 1996). The Centre for Disease Control and Prevention estimates that 900 to 1000 people die annually from microbial illness in U.S. drinking water (Gelt, 1998). With these levels of illness resulting from waterborne disease, its control is an important facet of drinking water supply.

3.4.1 Outbreaks of Waterborne Disease

Water related infections are classified into four different groups (Gleeson and Gray, 1997):

1. Waterborne disease: pathogen transmitted by ingestion of contaminated water;
2. Water-washed disease: Disease spread faecal-orally, or lack of adequate supply for washing;
3. Water-based infections: Disease caused by pathogenic organisms, which spend part of their life cycle in aquatic organisms ie. guinea worms, and parasitise snails; and
4. Water related disease: Due to insects that breed in water, such as mosquitoes.

This chapter examines the waterborne disease facet of the four, as drinking water is ingested.

3.4.2 Types of Waterborne Diseases

Waterborne disease commonly originates from faecal discharges from infected humans, pets, livestock or wildlife. The main forms of microorganisms that contribute towards waterborne disease are:

1. Bacteria;
2. Viruses; and
3. Protozoa.

A major health concern that causes significant morbidity and mortality throughout the world, is gastroenteritis, which can develop from exposure to microorganisms in drinking water supply. In developed countries, the rate of gastroenteritis has been estimated as between 0.7 and 1.5 cases per person per year, with around 70 deaths annually in Australia, and 10,000 deaths yearly in the United States of America. In undeveloped countries, an estimated 4.6 million to 6 million deaths from gastroenteritis result annually (Hellard, 1998).

Other estimates include Gelt (1998) who reported that from 7million to 30 million Americans develop gastrointestinal (GI) illness each year, and Payment (1991) who estimated the annual incidence of GI illness was 0.76/person for unfiltered drinkers and 0.5/person for consumers of filtered supply.

3.4.2.1 Bacterial Diseases

Along with the traditional waterborne disease threats of *Salmonella* (typhoid), *Shigella* (dysentery), and *Vibrio cholerae* (Cholera), the last twenty years has seen the emergence of new bacterial diseases spawned from opportunistic bacteria (Glesson and Gray, 1997).

These organisms target the immuno-compromised, the aged and very young (Bitton, 1994) and the health implications are difficult to assess due to the lack of data on the occurrence, infectious doses and incidence of human disease from these organisms. The range of susceptible individuals in the population is also uncertain, as is the effectiveness of current treatment practice.

The traditional pathogen causing problems in developed countries is the *Salmonella* genus. This is partially due to the improved sanitation removing the other potential illnesses. Typical symptoms of salmonellosis are gastroenteritis with diarrhoea, and vomiting, headache, even collapse and death.

3.4.2.2 Viral Diseases

Currently, over 120 types of human pathogenic viruses are known. The ones that are a concern for analysis of drinking water are known as the enteric viruses, which cause gastro-intestinal illness (Gleeson and Gray, 1997). Some of the better known include poliovirus, enterovirus, hepatitis, and rotavirus. Enteric viruses can be found in polluted surface waters and unprotected groundwater supplies used for potable water (Geldreich, 1996).

Outbreaks of many of the viral diseases, with the exception of hepatitis, are difficult to recognise due to the latency of the infections. Each year a number of reported cases of waterborne disease are of unknown identity (Herwaldt *et al.*, 1992; Kramer *et al.*, 1996).

3.4.2.3 Illness due to Protozoa

Giardia and *Cryptosporidium* are pathogens that are frequently isolated from stool samples of gastroenteritis, though, it is unknown to what extent they cause gastroenteritis (Hellard, 1998).

Giardia lamblia was first described in 1681 by Antonie van Leeuwenhoek, however the first official report was not until the 1800s (Hellard, 1998). *Giardia* cysts are found in water as a result of faecal deposition from humans and animals.

G. lamblia is the most common intestinal parasite throughout the world. Detection rates for supplies in the developed world occur in 2% to 5% of the supply systems, in the developing countries experience a higher range of 20% to 30% of the distribution networks being infected (Hellard, 1998). It is estimated that 60% of all *Giardia* infections occur from exposure to contaminated water (Rose *et al.*, 1991).

The term of illness from *Giardia* infection is usually 7-14 days, with intestinal uneasiness and nausea, followed by diarrhoea and abdominal cramps. The acute stage is 3-4 days with diarrhoea lasting for the remainder of the period.

Ernest Tizzer discovered *Cryptosporidium* in 1907, his work was not considered important until 1971, when *Cryptosporidium* was associated with diarrhoea in cows. In 1976, the first human cases were reported (Daniel, 1996). Relatively few cases were reported until 1982, when cryptosporidiosis was associated with diarrhoea in AIDS patients. The first outbreak in a water supply occurred in Texas, 1984, but it was April 1993, when 400,000 people contracted illness in Milwaukee that the scale of the potential health concerns was realised.

Reported infection rates from contaminated supply range between 0.6% and 20%, and *Cryptosporidium* is thought to be one of the common enteropathogens causing diarrhoeal illness. The duration and severity of the disease can act as long as 7 days with immunocompromised persons the most at risk of mortality (Gerba *et al.*, 1996). The sources of the *Cryptosporidium* oocysts can be found in sewage effluent and cattle wastes (Carrington and Miller, 1993). Surface water, not affected by human waste can still contain the *Cryptosporidium* oocysts, and thus present a health concern.

3.4.3 Determination of Cause of Waterborne Disease

The recognition that a sick person contracted their illness through contact with water requires either epidemiological data or water quality data as back up. As previously mentioned, many of the diseases found in water can be transferred through other means than ingestion. Table 3.3 (Kramer *et al.*, 1996) lists the classification system in use in the United States for deciding if an illness can be attributed to contact with water. This surveillance is not flawless, but provides a means of analysis.

Table 3.3 Classification of investigations of waterborne disease outbreaks

Class	Epidemiologic Data	Water Quality Data
I	Adequate	Provided and adequate
II	Adequate	Not provided or inadequate
III	Provided but limited	Provided and adequate
IV	Provided but limited	Not provided or inadequate

In this classification system, epidemiologic data is favoured to the water quality data. Adequate epidemiologic data requires that:

- Data was provided about exposed and unexposed individuals; and
- The relative risk (RR) was ≥2.

If the data is considered "provided but limited", it implies that the data could not meet the requirements for Class I, or that the claim was made that exposure to water was the connecting factor between effected individuals, but no data provided. If water quality data was deemed "provided and adequate", it could be historical data such as a main breaking, or laboratory data indicating high levels of coliforms in the water.

3.4.4 Development of Water Treatment Practice to Combat Illness

Investigations into the disinfection potential against *Cryptosporidium* have shown that chlorine and chloramine have virtually no effect at typical dose levels. Ozonation has shown to be a successful option, however, additional filtration is likely to be the most practical method for removal (Li *et al.,* 1997). *Giardia* is also resistant to chlor(am)ine disinfectants, and similar approaches have been adopted to remove it from the water supply.

It has also been mentioned that a risk with ozonation is that while it inactivates the protozoa, it may increase the amount of biodegradable organic matter for the benefit of opportunistic pathogens such as *Legionella* (Daniel, 1996).

3.5 Potential Risks of Waterborne Disease

The determination of risks associated with waterborne disease is an area that requires further research. The common approach to the subject is to have an acceptable risk represented by a dotted box (seen in Figure 3.3), where there are no severe health effects deemed to occur. This approach (Cabelli, 1978) assumes a quantifiable relationship between the potential health risks and the indicator organism.

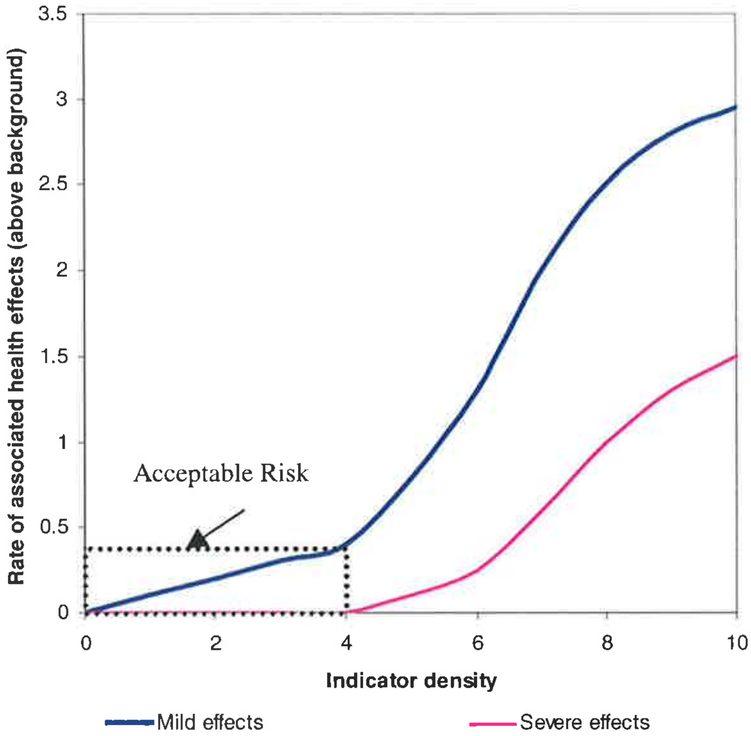

Figure 3.3 Determination of water quality criteria and guideline derivation

The level of acceptable risk for water quality consumption has been suggested as 1×10^{-4} annually (Haas *et al*, 1996), which is also used as the acceptable level for accidental consumption of small amounts of reclaimed water (Ashbolt, 1998; Gerba *et al.*, 1996; Gardner *et al.*, 1998). This corresponds to one illness per 10,000 people per year.

The indicator organisms widely used for determination of water quality are faecal coliforms such as *E.coli* and total coliforms. The use of coliforms has been an important method in water quality assessment throughout the last quarter of a century. Despite this, there is some speculation on its application to the current water quality situation with different microorganisms causing new strains of waterborne disease (Gleeson and Gray, 1997).

The South Australian Health Commission discussed that while monitoring for microbiological quality is based on enumeration of total coliforms and *E.coli*, there is no absolute correlation between numbers of indicator organisms and numbers of enteric pathogens, or risk of illness (SA Water, 1995). The following points on the use of coliforms were highlighted from this discussion:

- The detection of total coliforms suggests possible faecal contamination and therefore some potential health risk; and
- The detection of faecal contamination represents a greater health risk due to the possible presence of enteric pathogens.

The presence of coliforms does not mean that the water contains waterborne disease. There is also the arrangement where disinfectants can kill the indicator bacteria, but spare pathogenic protozoa. Although useful in measuring the success or failure of a treatment process, a more adequate way to address the issue would be to test for pathogens/indicators directly in the source water. Current detection methods may not be adequate in testing for quality, other methods such as molecular genetic approaches may be required (Ford and Colwell, 1998). To gain an appreciation of the potential health concerns associated with different waterborne diseases, Table 3.4 lists several types of water-related illnesses and the risk of infection from exposure to a single organism (Rose and Gerba, 1991).

Table 3.4 Risk of waterborne disease infection due to different organisms

Microorganism	Risk of infection per organism	Minimum dose for 1% infection rate for consumers
Campylobacter	7×10^{-3}	1.4
Salmonella	2.3×10^{-3}	4.3
Salmonella typhi	3.8×10^{-5}	263
Shigella	1.0×10^{-3}	10
Vibro cholera	7×10^{-6}	1428
Poliovirus 1	1.49×10^{-2}	0.67
Poliovirus 3	3.1×10^{-2}	0.32
Echovirus 1,2	1.7×10^{-2}	0.59
Rotavirus	3.1×10^{-1}	0.03
Entaboeba coli	9.1×10^{-2}	0.1
Entamoeba histolyctia	2.8×10^{-1}	0.04
Giardia lamblia	1.98×10^{-2}	0.5

Table 3.4 also provides the number of organisms required resulting in a 1% infection rate amongst an exposed community (Rose and Gerba, 1991).

In early research into risk estimation due to low doses of microorganisms, Haas (1983) found it impossible to rule out the hypothesis that a single microorganism has the potential to induce infection. Through further research, approximate risk factors as given in Table 3.4, were applied based on the single organism.

A study into the risks due to ingestion of water containing different forms of disease based on ingestion of 100mL of water found that water containing enteric viruses and *Giardia* pose the highest risks of around 2×10^{-4} (Ashbolt, 1998).

If a distribution system is poorly managed, and the water has a low, or has no residual, the continual exposure to waterborne problems increases the potential risk. Consumers who are based in systems with long residence times between the dosing station and the tap may experience greater levels of risk, as the time without adequate treatment is longer.

The risk of infection to water of poor quality can be high. For example, Milwaukee, with a population of 628,000 (execpc, 1999) had over 400,000 (65%) infected with *Cryptosporidosis* during the 1993 outbreak.

Approximate levels taken from Rose *et al.* (1991) indicate that for *Giardia* alone, risks of about 5×10^{-3}/year occur for polluted systems, and 10^{-4}/year for pristine supply systems. A polluted system is one that takes surface water from a watershed that is exposed to animal or human faecal contamination from a sewage treatment plant, or dairy. A pristine system considers water from a protected catchment.

The total risks due to waterborne disease would exceed these values, as there are many other forms of illness. Based on these figures, annual waterborne disease risks for an adequately treated system should not exceed 10^{-4}, whereas the annual risk of a polluted system could range from 10^{-2} to 10^{-1} or even higher as the contraction rate from Milwaukee indicates. The level of risk is highly dependent on the nature of the system and the quality of the source water.

The common approach to reducing the levels of waterborne disease risk in current distribution systems is the application of disinfectants to kill the microorganisms before they are able to cause consumer infection. An adequate disinfection regime combined with an effective pre-treatment process allows for good system sanitation.

3.6 Disinfection

Disinfection has had proven success in lowering the risk levels associated with waterborne disease in water supply. The process is a sanitation exercise to kill any microorganisms in the supply. It is usually the last stage in the water treatment process, and with disinfectants that have residuals, the process continues to the consumer's tap.

3.6.1 Background of Disinfection

Disinfection has become an important component of public health. Traditional methods including chlorination and chloramination are used alongside more recent techniques such as ultraviolet radiation and ozonation in achieving effective water treatment.

3.6.2 Examples of Common Disinfectants

There are a variety of different disinfectants used for sanitation of municipal water supply. The disinfectant types examined in this section are:

- Chlorination;
- Chloramination;
- Ozone;
- Chlorine Dioxide; and
- Ultra Violet Irradiation.

3.6.2.1 Chlorination

Karl Scheele discovered chlorine in its gaseous state in 1774. It has been used as a bleaching agent since 1785 and was first recognised as a germicide in 1846. The earliest application of chlorine as a possible disinfectant for water was in 1835, when Robey Dunlingsen published a proposal to make marsh water potable through the addition of small levels of chlorine (White, 1980).

Chlorination is the most used form of system disinfectant. Easy application, lack of sensitivity to pH and temperature and relatively low physiological effects at normal doses make it a robust option for water system treatment. It has reasonable persistence in the system and is also cost effective.

Chlorine is effective in systems with reasonably short retention times, but has difficulty in longer systems as the residual decays relatively rapidly. It is also known to produce by-products, notably trihalomethanes (THMs), which are a suspected carcinogen (Morris *et al.*, 1992).

3.6.2.2 Chloramination

When ammonia and chlorine are mixed in water, chloramines are produced. Chloramination is being used increasingly in Australian supplies due to its ability to provide a residual in long residence systems. The lower decay rate makes chloramination an important consideration in country areas, as the supply has to connect locations over a wide area.

Both chlorination and chloramination can produce taste and odour concerns, though the use of chloramine reduces these issues. Chloramination produces significantly lower levels of DBPs, as the chlorine component chooses to react with the ammonia in the system.

Chloramine is not a strong virucidal agent and has a decay rate that fluctuates with pH.

3.6.2.3 Ozone

Ozone is a very effective disinfectant against chemicals and pesticides, and is often used to remove phenols and organics to increase the efficiency of a system disinfection process. It is widely used in Europe, and reduces taste, odour, and colour problems in the supply. It removes odour concerns due to its oxidising ability, by oxidising the sulfides, iron and manganese, the taste is improved (Leitzke, 1999).

The disinfection ability of ozone has been recognised since 1886. The first full-scale plant for drinking water disinfection was in 1893 in the Netherlands. The number of ozone installations increased throughout Europe until 1914, when World War I (WWI) started. Research into poisonous gases during WWI led to the development of inexpensive chlorine, stimulating the use of chlorine as a disinfectant and constraining the use of ozone (Langlais *et al.*, 1991).

Ozone has recently expanded its use, which can be represented by the U.K., which prior to 1990 had only four operating ozone plants. During 1996, 25 additional ozone plants had been constructed, with 10 more under construction (Giannopoulos, 1996).

The advantages of ozone include the ability to kill resilient microorganisms, including *Giardia,* and *Cryptosporidium*, as well as all known algal toxins (SA Water, 1995). A detrimental aspect of ozone is its lack of residual, which exposes the system to regrowth problems in the event of contamination. It is also a complicated and costly disinfection technique, which may restrict it to certain systems. Although no THMs are formed from ozonation, it has the ability to form its own DBPs, such as bromate and aldehydes.

3.6.2.4 Chlorine dioxide

Chlorine dioxide is more toxic than chlorine and due to its volatility, cannot be transported. It is mainly used for industrial purposes in Australia though it is used in water treatment in many other parts of the world.

Taste and odour concerns are negligible with chlorine dioxide, and THM formation is not a factor, it is also reasonably independent of pH. Chlorine dioxide does not react with ammonia and is a powerful oxidising agent (Petrucci *et al.,* 1998).

There is some concern of health risk due to the toxicological effects from the chlorite ion, caused from the lack of complete efficiency in the chlorine dioxide making process from the constituents of chlorine and sodium chlorate.

3.6.2.5 Ultra Violet Irradiation

Ultra Violet (UV) irradiation has been used frequently in the beverage industry, hospitals, and pharmaceutical production. It has met with considerable success in its application to wastewater disinfection, but has not achieved similar heights as a disinfectant for potable water.

The lack of residual makes UV disinfection more of an instantaneous disinfectant rather than a distribution option. It can however, be used in conjunction with a primary disinfectant to enhance the treatment efficiency in the system. A drawback of it is that it will break down any existing chlorine residual which could have ramifications on the dosing rates.

UV does not create any by-products, and does not react with pH. It is non-chemical and provides strong virucidal properties. In fact, at certain wavelengths, UV works as a powerful germicide and alters the genetic materials in cells so that bacteria, viruses, algae and other microorganisms can no longer reproduce (UVTA, 1999).

3.6.2.6 Comparison of Disinfectants

Issues ranging from plant type and complexity through to pH dependency are listed in Table 3.5 (NHMRC/ARMCANZ, 1995) to fully compare the different disinfectants.

Table 3.5 Disinfectant Comparison

Consideration	Chlorine	Chloramine	Ozone	Cl Dioxide	UV
Plant Size	All sizes	All sizes	Med-Large	Small-Med.	Small-Med.
Equip Reliability	Good	Good	Fair-good	Good	Fair-good
Complexity	Sim-Mod.	Sim-Mod.	Complex	Moderate	Sim.-Mod.
Safety concerns	Yes	Yes	Moderate	Yes	Minimal
Bactericidal	Good	Good	Good	Good	Good
Virucidal	Moderate	Poor	Good	Good	Good
DBP concerns	Yes	Fewer	Unresolved	Yes	No
Persistent res.	Moderate	Long	None	Moderate	None
Contact time	Moderate	Moderate	Short	Moderate	Short
pH dependent	Yes	Yes	Slight	Slight	No
Process control	Developed	Developed	Developing	Developing	Developing

When comparing costs of different disinfectants, the extra costs with ozonation are evident from a study by Clark *et al.* (1998). The results of disinfection costs for different disinfection processes from this study are shown in Figure 3.4 (Clark *et al.*, 1998).

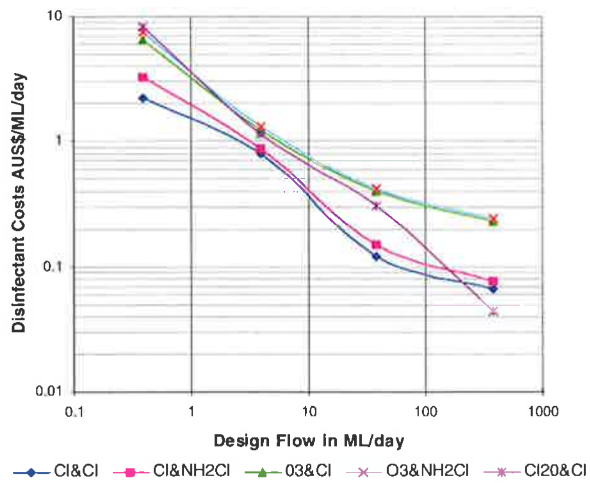

Figure 3.4 Cost comparison of different disinfection options

The cost issue associated with disinfection depends on the 'willingness to pay' of the consumers for water quality. Section 3.10.2 discusses this issue.

3.6.3 Examination of Chlorination

The chemistry of chlorination, by addition of gaseous chlorine is represented by Equation 3.3. The hypochlorous acid (HOCl) is the most effective of the chlorine residual fractions, this fraction is known as the free available chlorine residual (White 1980).

$$Cl_2 + H_2O \leftrightarrow HOCl + H^+ + Cl^- \quad \textbf{(3.3)}$$

The formation of the hypochlorous acid (HOCl) is the important part of Equation 3.3. Due to HOCl being a 'weak' acid, it suffers partial dissociation as shown in Equation 3.4.

$$HOCl \leftrightarrow H^+ + OCl^- \quad \textbf{(3.4)}$$

Although chlorine was in regular use at the start of the 20th century, it was not until 1944 that Chang determined HOCl as the disinfecting agent from the chlorine reaction, and that no further reaction to hydrochloric acid (HCl) existed. In 1946 Green and Stump concluded that chlorine killed the bacteria in the system by reacting irreversibly with them (White, 1980).

Taste and odour concerns do exist with chlorination, as do potential health concerns if the dosing levels are excessive. The health-related maximum guideline level from NHMRC for chlorine is 5mg/L. The recommended level for aesthetic benefit is 0.6mg/L (SA Water, 1995).

3.6.3.1 Decay Process for Chlorination

The chemical kinetics of chlorine decay reactions throughout distribution systems is a little known aspect of water quality modelling. However, chlorine decay is based on the generic reaction in Equation 3.5 (Chambers *et al.*, 1995).

$$Cl_2 + X \rightarrow products \qquad \textbf{(3.5)}$$

The X component represents all species reacting with chlorine, which includes the total organics in the source water. If it is assumed that there is a greater concentration of X than Cl_2, the reaction becomes pseudo-first order as seen in Equation 3.6 (Chambers *et al.*, 1995).

$$Cl_2 \rightarrow products \qquad \textbf{(3.6)}$$

There are different decay models that track this breakdown from Cl_2 to DBPs.

3.6.3.2 First Order Decay Model

The first-order decay model has the rate of decay proportional to the concentration of the chlorine remaining as shown in Equation 3.7. The integrated form of the formulation is also represented.

$$\frac{\partial Cn}{\partial t} = -k_1 Cn \;\Rightarrow\; Cn_t = Cn_0 e^{-k_1 t} \qquad \textbf{(3.7)}$$

Cn = chlorine concentration from initial time *0* to time t, k_1 = the first order decay rate.

3.6.3.3 Second Order Decay Model

The second-order decay model has the rate of decay proportional to the square of the concentration of the remaining chlorine as seen in Equation 3.8. The integrated form of the formulation is also represented. Second order decay has been shown to provide a more realistic match to observed data (Clark, 1998) as it caters for the initial rapid decay better than its first-order counterpart.

$$\frac{\partial Cn}{\partial t} = -k_2 Cn^2 \Rightarrow \frac{1}{Cn_t} = \frac{1}{Cn_0} + k_2 t \qquad \textbf{(3.8)}$$

There is also an n^{th} order decay, which has the rate of decay proportional to the concentration of the remaining chlorine raised to the power *n*.

3.6.3.4 Effect of the Pipe Wall

The pipe wall can have a marked effect on the overall decay rate due to a combination of the reactions with biofilms and tubercles that are attached to the pipe wall, and the corrosion of the pipe material. The two decay processes used to represent these factors are the first-order decay model (Equation 3.9) and the zero-order decay model (Equation 3.10) (Vasconcelos *et al.*, 1997).

$$\left.\frac{\partial Cn}{\partial t}\right|_{wall} = -\frac{k_{w,1} Cn_w}{r_h} \qquad \textbf{(3.9)}$$

Where Cn_w = chlorine concentration at the pipe wall, $k_{w,1}$ = first-order decay rate of chlorine at the pipe wall, and r_h = hydraulic radius of the pipe.

$$\left.\frac{\partial Cn}{\partial t}\right|_{wall} = -\frac{k_{w,0}}{r_h} \qquad \textbf{(3.10)}$$

Where $k_{w,0}$ = zero-order decay rate at the pipe wall.

The first-order decay model best represents the process where chlorine is the limiting reactant as is the case with biofilm reaction. The zero-order decay process better represents the case where the chlorine immediately oxidises some material, which is suited more to corrosion-induced reactions.

3.6.3.5 Comparison of Chlorination with other Disinfectants

Chlorination is the most widely used disinfectant for water supply treatment worldwide. As a result, it becomes a benchmark for comparison for alternative disinfectant options, and how it alters the water quality in the system is an important issue.

3.6.3.6 Decay Rate Determination

Decay rates vary from system to system. By examining water quality data from the system and knowing the dosing rates and residual levels, decay rates can be ascertained. Factors that can have an effect on the decay rate include:

- Temperature – as it increases, microbial growth and biofilm growth increase chlorine demand (Geldreich, 1995). It also has an effect on the level of HOCl (Table 3.6) in the system (White, 1980);
- Dissolved activated carbon (DOC) – the higher the level of DOC, the greater the level of material for the chlorine to react with, hence increasing chlorine demand;
- pH – affects the production of HOCl (Table 3.6) affecting disinfection ability; and
- Pipe diameter – the smaller the diameter, the greater surface contact, hence greater decay.

The water quality in the system is very important to the chlorine demand of the system, Table 3.6 indicates how the pH and temperature can affect the formation of HOCl in a reaction between Cl_2 and water. The table looks at the level of HOCl that has not disassociated into the considerably less potent OCl^-.

Table 3.6 Percentage formation of HOCl for different water quality

	Percentage of HOCl that has not disassociated to OCl^-	
pH	0°C	20°C
4.0	100	100
5.0	100	99.7
6.0	98.2	96.8
7.0	83.3	75.2
8.0	32.2	23.2
8.5	13.7	8.75
9.0	4.5	2.9
10.0	0.5	0.3
11.0	0.05	0.03

White (1980) states that the disinfecting ability of free available chlorine decreases significantly as the pH rises, and for pH levels above 9, there is little disinfecting power. The examination of factors affecting disinfection is extended in the chloramine analysis.

3.6.4 Chloramination

Since the 1980s, chloramination has been increasingly used as an alternative to chlorination in water distribution disinfection. Chloramine tends to have greater stability, which corresponds to an increased residence time, and can provide a greater long-term control against the growth of bacteria and protozoa. It can also reduce taste and odour characteristics of a water supply.

When chlorine and ammonia are combined in aqueous solution, they form inorganic chloramines as indicated in the 3-step reaction process in Equation 3.11.

$$\begin{aligned} &HOCl + NH_3 \rightarrow NH_2Cl + H_2O \\ &HOCl + NH_2Cl \rightarrow NHCl_2 + H_2O \\ &HOCl + NHCl_2 \rightarrow NCl_3 + H_2O \end{aligned} \qquad \textbf{(3.11)}$$

The successful transfer of chlorine and ammonia to chloramine depends on the pH and chlorine to ammonia ratio. The lower the pH and higher the $Cl:NH_3$ ratio, the more chlorinated the ammonia becomes (Wolfe *et al.*, 1984). The reaction is quite fast, with 99% conversion rates in water of pH 7 and temperature 25°C being achieved in 0.2 seconds (White, 1986).

The health-related maximum guideline level from NHMRC for chloramine is 3mg/L. The recommended level for aesthetic benefit is 0.5mg/L (SA Water, 1995). Achieving these levels in rural systems while still maintaining a residual at the end of the system can be difficult.

3.6.4.1 Chloramination Decay

There are three types of chloramines, monochloramine (NH_2Cl), dichloramine ($NHCl_2$) and trichloramine (NCl_3). Trichloramine and dichloramine rapidly decompose, causing a disproportionate level of monochloramine, which subsequently leads to its loss (Valentine and Jafvert, 1988). The two step loss reaction is shown in Equation 3.12.

$$NH_2Cl + NH_2Cl \rightarrow NHCl_2 + NH_3$$

$$NHCl_2 + NH_2Cl \rightarrow N_2 + 3Cl^- + 3H^+ \qquad \textbf{(3.12)}$$

Monochloramine is also lost through hydrolysis that follows a two step formulation seen in Equation 3.13.

$$NH_2Cl + H_2O \rightarrow HOCl + NH_3$$

$$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O \qquad \textbf{(3.13)}$$

From these two equations, the net reaction for monochloramine loss is found using Equation 3.14.

$$3NH_2Cl \rightarrow N_2 + NH_3 + 3Cl^- + 3H^+ \qquad \textbf{(3.14)}$$

3.6.4.2 Effect of pH on Chloramine Decay

The decay rate of chloramine is inversely relelated to pH (Robinson *et al.*, 1984), this is ratified by Thomas (1990) as seen in Table 3.7.

Table 3.7 Effect of pH on chloramination decay rate at 35°C

pH	Decay rate (days^{-1})	Half life (days)
6.27	0.198	3.5
6.74	0.103	6.7
7.26	0.058	12
8.22	0.019	37
9.12	0.008	87

At higher temperatures, the variation of decay rate with pH is even more pronounced. This is because temperature is directly related to decay rate.

3.6.4.3 Effect of Temperature on Chloramine Decay

The effect of temperature on decay rate is evident from Table 3.8 (Thomas, 1990). In this analysis, the pH was held constant at 7.5. The direct relationship between pH and temperature is evidenced in the table.

Table 3.8 Effect of temperature on chloramine decay rate for pH 7.5

Temperature (°C)	Decay rate ($days^{-1}$)
15	0.0166
20	0.0183
27	0.0306
35	0.0533
42	0.0776
50	0.1563

Another factor that has a directly proportional relationship with decay rate is the level of dissolved organic carbon (DOC) in the water.

3.6.4.4 Effect of DOC on Decay Rate

Chloramine is lost as it reacts with DOC to form by-products. This is seen in Table 3.9, which is for a constant temperature (35°C), and a constant pH 7.5.

Table 3.9 Effect of DOC on chloramine decay rate for 35°C and pH of 7.5

DOC (mg/L)	Decay rate ($days^{-1}$)
1.3	0.043
2.9	0.052
5.8	0.062
10.0	0.153
14.8	0.308
19.3	0.529
23.9	1.27

Other factors can contribute to the decay rate of chloramine, including bromide level, turbidity, and type of treatment process such as alum treatment and pre-disinfection (Thomas, 1990).

3.7 Taste and Odour

Taste and odour in drinking water is only an issue when there are consumer complaints regarding the aesthetic quality of the supply. Surveys conducted by Lyonnaise de Eaux in various regional centres in France found that 30% to 50% of recorded drinking water complaints arose from taste and odour concerns (Bruchet and Hochereau, 1999). As the measure of acceptability tends to be dependent on individuals olfactory senses, the level of the problem varies from person to person (SA Water, 1995).

Different approaches are possible to determine the taste and odour in a supply. The two most common methods are quantitative judgement and the difference judgement methods, which are discussed in detail by MacCrae *et al.* (1993). Quantitative judgement requires an assessment of one or more smell or flavour components, whereas difference judgement seeks to determine the lowest concentration with a detectable difference in taste or odour.

The preferred aesthetic levels in South Australia for chlorine and chloramine to limit taste and odour complaints are 0.6mg/L and 0.5mg/L respectively (SA Water, 1995). Krasner and Barrett (1984) mention that monochloramine solutions possess chlorine-like taste and odour characteristics with only a slight taste recorded at a concentration of 3mg/L. White (1980) states that taste and odours from monochloramine application are not likely to occur until a level of 5mg/L is reached.

Some of the methods that have been suggested by Kerslake (1989) that can assist in preventing the chemical and drying tastes of chloramination include:

- The addition of a strong oxidant such as ozone prior to chloramination; and
- The use of activated carbon with coagulation/filtration prior to disinfection.

Kerslake (1989) also discusses the role that DBPs can play in taste and odour concerns due to chloramination. Iodoform (CHI_3) was mentioned in a Government Chemical Laboratories report, as being the causative taste agent. Chlorine added to the supply prior to the ammonia apparently vastly reduced the level of iodoforms by forming chlorinated THMs instead. The role of DBPs in distribution systems extends past taste and odour, as they have become an issue in the management and operation of water distribution systems.

3.8 Disinfection By-Products

In 1981, the USEPA published guidance for the control of trihalomethanes (THMs). Aeration and activated carbon were discussed as effective THM removal techniques (Oxenford, 1996), whereas oxidation, clarification and adsorption of THM precursors can assist in lowering formation rates of THMs.

3.8.1 Formation of By-Products of Chlorination

Some of classes of by-products found in chlorinated supply (Hayes, 1986) include:

- Trihalomethanes (chloroform, bromoform etc.);
- Haloacids;
- Chlorinated ketones;
- Aliphatic aldehydes;
- Chlorinated alkenes and alkanes; and
- Aromatic compounds.

The DBP, which will be analysed in this study, is the trihalomethane (THM) component. This has become a major research issue of disinfection based on the suspicion that THMs are a suspected carcinogen.

One of the most significant aspects relating to DBP formation is its great dependence on physical aspects of the water supply. The organic composition dictates, to a degree, disinfectant consumption, and the removal of organic carbon from the supply can reduce the formation of DBPs (Black *et al.*, 1996). Other aspects, irrelevant of incoming water quality, such as pH and water temperature have a significant bearing on the production rate of DBPs.

3.8.1.1 Effect of Temperature on DBP Formation

Chemical reactions tend to proceed more rapidly at high temperatures when the other factors remain constant. In a study of several distribution systems in Dublin, Ireland, the increase in THM formation was plotted at a rate of 0.5μg/L per ^{o}C as seen in Figure 3.5 (Casey and Chua, 1997).

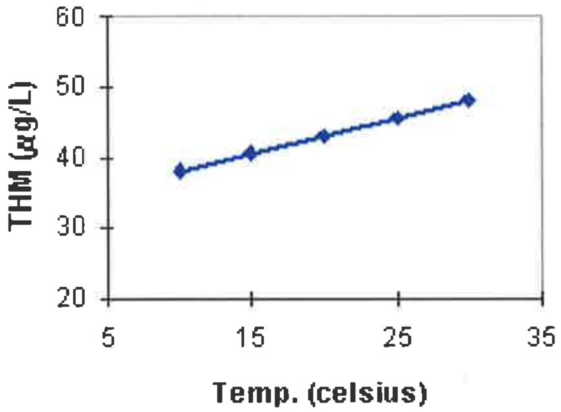

Figure 3.5 Effect of temperature on THM formation

It should also be noted from these results that limitations of reactant availability may influence THM formation.

It is perceived nonetheless that, at higher temperatures, the decay rate of the disinfectant will increase. Hence, the level of chlorine consumed for a given time will also increase causing an increase in by-product levels. The effects of the seasons cause not only a change in temperature, but also an alteration in the level of organic matter infiltrating the system.

Another study (Summers *et al*, 1996) examines the effect of temperature on THM formation. This study found that between the temperature range of 10°C and 30°C the THM levels produced more than doubled.

The same study also determined that at higher temperatures, the by-products would contain higher concentrations of the chlorinated DBPs.

3.8.1.2 Effect of pH on DBP Formation

The pH of the supply causes marked variation in DBP formation as seen in Table 3.10 (Bull, 1992). Some of the reasons for this include the decline in mutagenic activity in finished drinking water with higher pH and the instability of some by-products at higher pH levels.

Table 3.10 Effect of pH on formation of chlorination by-products

By-Product Class	pH		
µg/L	5	7	9.4
Trichloromethanes	44	84	138
Trichloroacetate	32	29	20
Dichloroacetate	24	29	28
Trichloroacetataldehyde	6	9	1
Dichloroacetonitrile	5	0.9	0.8

These conclusions are ratified in a more recent study (Summers *et al.*, 1996) which attributes the rise of THM level with pH to hydrolysis of chlorinated intermediates. This study examined three different waterways with the results as shown in Figure 3.6.

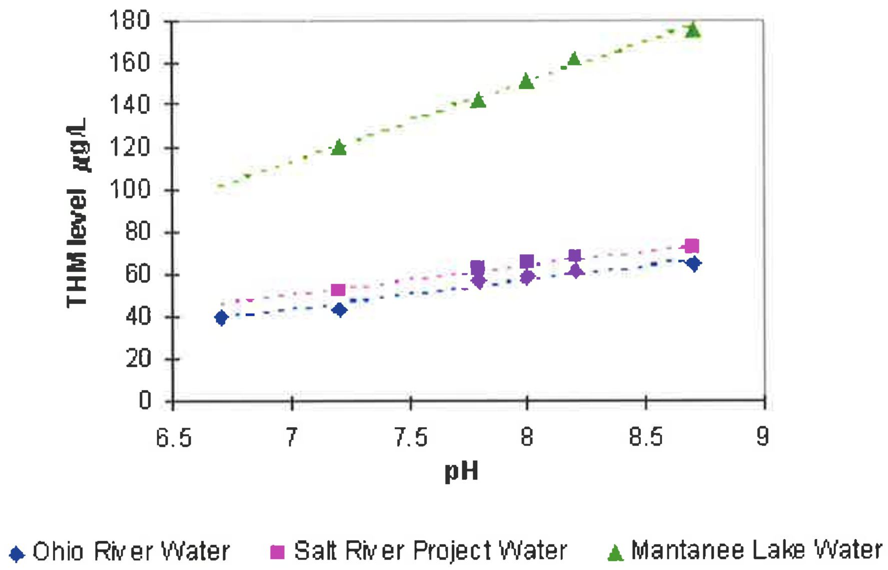

Figure 3.6 Effect of pH on THM formation

For water from the Ohio River, an increase in pH from 7 to 9 increased the level of THMs by 50% to 60%. It has even been reported that THM formation increases with a significant rise in pH, even in the absence of a chlorine residual (Summers *et al.,* 1996).

3.8.2 By-Product Formation from Chloramination

The by-products of chloramine are generally of similar nature to chlorination by-products. The amount of by-product can depend on how the chloramine is added to the system (Bull, 1992).

If the chlorine is in the system for a while before the ammonia is added, resultant levels are similar to chlorination levels. If the ammonia is added simultaneously with the chlorine, considerably lower levels of DBPs result, as shown in Table 3.11.

Table 3.11 THM levels dependent on application of disinfectant

Trihalomethane	Mean Concentration (μg/L)		
	Cl	Cl followed by ammonia	Cl plus ammonia
Chloroform	26.4 ± 3.0	20.8 ± 3.1	3.3 ± 1.8
Bromodichloromethane	9.1 ± 1.5	15.2 ± 1.1	3.5 ± 0.8
Dibromochloromethane	5.7 ± 1.2	14.6 ± 2.0	4.8 ± 1.3
Bromoform	4.4 ± 1.3	8.5 ± 2.4	12.1 ± 4.8

Chloramination, when correctly applied tends to produce significantly lower levels of DBPs (notably THMs) than chlorination. A comparison of the effects of chloramine and chlorine for seasonal conditions in a Canadian study is given in Table 3.12 (Williams *et al.*, 1998). The sample space for this study consisted of 35 systems using chlorine at the treatment plant and chlorine throughout the network, 10 systems using chloramine to maintain residuals while still implementing chlorination at the treatment plant.

Table 3.12 TTHM (μg/L) levels in Canadian drinking water

Treatment	Site	Winter	Winter	Summer	Summer
		mean	range	mean	Range
Chlorine-	Plant	16.8	2.0-67.9	33.5	1.6-120.8
Chlorine	System	33.4	2.8-221.1	62.5	0.3-342.4
Difference in means		16.6		29	
Chlorine-	Plant	12.1	0.6-40.3	31.2	2.9-80.1
Chloramine	System	13.7	1.5-42.1	32.8	4.3-85.2
Difference in means		1.6		1.6	

From Table 3.12, the mean increase in winter in TTHM from the treatment plant to the distribution system for chlorinated systems is 16.6μg/L and in summer 29μg/L. These compare to a mean increase of 1.6μg/L of TTHM due to the chloraminated supply in winter and 1.6μg/L in summer. This corresponds with the statement made by Dunnick and Melnick (1993) that chloramine produce levels of TTHM by up to a 90% lower rate than chlorine.

While chlorine experienced higher levels of TTHMs during summer than winter due to higher reaction rates of the chlorine, chloramine did not have any real seasonal variations. This may be explained by the significantly lower decay rate of chloramine, which causes little alteration to the rate of consumption when compared with residence time.

The following process to explains the different performances of chlorination and chloramination at a first-order decay level:

Imagine a typical system, which has its residence time 50% lower in summer but its decay rate is doubled. The breakup is shown in Table 3.13. It should be noted that the residence times for the chloraminated system are 10 times that of the chlorinated system. This is because chloramine is favoured in systems with significantly greater residence time.

Table 3.13 Differences in disinfectants for different seasons

Disinfectant	summer	winter	summer	winter	Summer	Winter	residual
	Decay rate (day)	Decay rate (day)	Res (days)	Res (days)	dose	dose	
Chlorine	2.0	1.0	1.0	1.5	3.7	2.2	0.5
Chloramine	0.05	0.025	10	15	0.82	0.73	0.5

Due to the greater consumption of chlorine to maintain the same residual, the TTHMs produced in summer will be higher. With chloramine, the increased dosing from winter to summer is still reasonably small (0.09mg/L) and is unlikely to significantly alter the TTHM formation levels.

3.8.3 Disinfection/Disinfection By-Product (D/DBP) Rule

In 1986, the USEPA introduced a disinfection/disinfection by-product rule (D/DBP) to regulate the use of disinfectants in water treatment. In 1994, this rule was amended to form two stages. The first stage requires maximum contaminant levels (MCL) of THMs and haloacetic acids (HAAs) to be kept below 80μg/L and 60μg/L respectively. Maximum residual disinfection levels (MRDLs) of 4.0mg/L were established for chlorine and chloramines, and 0.8mg/L for chlorine dioxide.

The second stage establishes levels of 40μg/L and 30μg/L for THMs and HAAs respectively. Although stages I and II were introduced together, further negotiation of stage two is an issue under the Information Collection Rule promulgated May 14 1996 (Arora *et al.*, 1997). The implementation of this rule is expected to be dictated by HAAs more than THMs in many systems (Singer *et al.*, 1995).

The reason for the regulation of DBPs is based on their potential relationship to cancer in many studies, though other studies have claimed that there is inadequate evidence to make such a link. The potential risk of cancer from DBPs (notably THMs) was deemed sufficient basis for extensive regulation.

3.9 Potential Risks due to Disinfection By-products

The benefits of disinfection are well known with massive reductions in the contraction rate of waterborne disease. What is not as well documented is the potentially harmful side effects from DBPs. One of the reasons for the lack of documentation is that DBPs were only recently thought to potentially cause harm.

Although there is some speculation as to whether links between DBPs and cancer can be defined, many researchers indicate that a correlation exists. This section examines studies supporting both viewpoints, discussing the results found and looking at the possibilities for application in the modelling phase of this thesis.

3.9.1 Early Research

Early studies on the links between cancer and DBPs examined differences in cancer contraction rates between regions with chlorinated water supplies and those without. Potentially affecting the results are that these regions may have other factors that are potentially carcinogenic, which could be unrelated to water supply. For example, if the chlorinated supplies tend to be located in urban regions, pollution from cars or nearby industry may affect the rates of contraction from cancer. By examining enough areas and by appropriate analysis, a worthwhile examination can take place.

A good example of researching to reduce unrelated factors is evident in a study by Kanarek and Young (1980). This study looked at white Wisconsin females during 1972-1977. As the majority of these females had an occupation listed as homemaker and less than 1% had high-risk occupations, they provided a sample space with reduced impact from other carcinogenic sources such as air pollution from traffic. This sample allowed the researchers to base their conclusions on the assumption that subjects spent the majority of their time consuming water from their home. Migration rates (people moving in and out of region) were also quite low (<10%) for the counties studied over a 20 year period making the affected population reasonably constant.

Kanarek and Young (1980) provided one of 5 studies from the USA examined in a report by Crump and Guess (1982). This report determined cancer relative risk ratios (as discussed in Section 3.2.1.2) based on these studies and drew comparisons, which are summarised in Table 3.14.

Table 3.14 Early studies on cancer association with chlorinated drinking water

Site	Alavanja *et al.*, 1978	Strauba, 1979	Brenniman *et al.*, 1980	Kanarek & Young, 1980*	Gottlieb *et al.*, 1981
Rectum	1.93	1.53	1.26	1.39	1.41
Colon	1.61	1.30	1.08	1.51	1.05
Bladder	1.69	1.54	1.04	1.04	1.07

*Only included the cancer ratios for the high chlorination areas, not medium or low from this study.

The first study associating drinking water with gastrointestional (GI) and urinary tract (UT) cancers was Alavanja *et al.* (1978). They concluded that subjects living in chlorinated areas were at greater risk of GI and UT cancer contraction than those in non-chlorinated regions. Brenniman *et al.* (1980) attempted to replicate the Alavanja study by examining whites in 70 Illinois communities. While the authors reported elevated estimated risks of colon and rectal cancer they described their findings as'tenuous'. Chlorination of surface water from the Mississippi River in 20 South Louisiana counties was the region studied by Gottlieb *et al.* (1981). The urban factor discussed earlier was posed as a factor that may have altered the results. No relationship of chlorinated drinking water to cancer was posed from this study.

The second study (Struba, 1979) looked at mortality data from subjects who had died from rectum, colon and bladder cancer at an age of greater than 45 within North Carolina. This age stipulation was used to ensure sufficient exposure to the drinking water from the region. Struba (1979) found that a correlation between cancer and drinking water chlorination was in evidence.

The Kanarek and Young (1980) study discussed earlier concluded that a direct link between colon cancer and exposure to chlorinated drinking water exists.

After examining recent epidemiological studies, Crump and Guess (1982) used animal data to provide estimates of human cancer risk from drinking water. By assuming that most of the cancer risk from chlorination was associated with the DBP chloroform, they developed a level of 1.8×10^{-4} as the added cancer risk in a chlorinated area. This would correspond to a risk ratio of about 1.02, which is significantly lower than the relative risks identified earlier.

They concluded that this discrepancy was caused by the epidemiological studies overestimating the effect of drinking water on cancer. It was posed that humans may have greater susceptibility to cancer contraction from DBPs than the animals tested. It also allows for the fact that undetected chemicals may be causing cancer in conjunction with the detected causes of contraction (ie. chloroform). The summary definitely indicated that the link between cancer and chlorination of drinking water was evident.

The report by Crump and Guess (1982) is contradicted to a degree by a study by Carlo and Mettlin (1980) who found that they could provide no support to the hypothesis that THM levels meeting 1980 standards were related to cancer incidence. The study looked at reported cancer cases from Erie County, New York State. It stated that the results of the research may be due to the modest THM range in the county (71μg/L) as well as other factors inhibiting the disease process.

An early study (Cantor *et al.*, 1978) determined that positive correlations between THMs and cancer contraction was evident, with bladder cancer showing the strongest association with a THM exposure index.

THMs and cancer have also become an object of recent interest with many publications examining the problem many years on. Again, some studies display hesitancy in ratifying the probable link between cancer and THMs.

3.9.2 Recent Research

Much of the early research was re-analysed in later studies (IARC, 1991; Morris *et al.*, 1992) providing different overviews as to the nature of the potential risks.

Since 1969, the International Agency for Research on Cancer (IARC) has run a program evaluating the carcinogenic risk of chemicals to humans. As there were many studies linking chlorinated drinking water to cancer risk through the consumption of disinfection by-products, the IARC embarked on producing a monograph examining the issue (IARC, 1991). This analysis included several of the previously discussed studies, and from analysis of the toxicological and epidemiological evidence it produced the following evaluations:

- There is *inadequate evidence* for the carcinogenicity of chlorinated drinking water in humans; and
- Chlorinated drinking water is *not classifiable as to its carcinogenicity to humans.*

The summary of the first evaluation is that the studies are unable to be interpreted as showing either the presence or absence of a carcinogenic effect because of major qualitative or quantitative limitations.

The second classification occurs when the exposure circumstances are not deemed to be possibly or probably carcinogenic to humans, but can not be classified as probably not carcinogenic to humans. This statement implies that the current level of research has been inadequate to either prove or disprove that a relationship exists between disinfection by-products and cancer.

Another study examining previous research was undertaken by Morris *et al.* (1992) providing a meta-analysis of 25 years of analysis in this field. A meta-analysis is an examination of studies relating to a topic by a panel of judges. Preferably, the adjudicators should have no knowledge of author or institution that produced the research. In the Morris meta-analysis, several articles on chlorination by-products and cancer, published between 1966 and 1991, were analysed by two independent readers blinded as to authors, institutions, and journals.

The results of this study suggested a positive correlation between consumption of chlorination by-products in drinking water and bladder and rectal cancer in humans. Relative risk estimates were 1.21 for bladder cancer and 1.38 for rectal cancer. Based on a conservative estimate of 54% of population consumption being chlorinated surface water, 4,200 cases of bladder cancer and 6,500 cases of rectal cancers in the USA can be attributed to drinking water consumption annually.

That chlorination may serve as a marker for another aspect of drinking water has not been discounted by the study. However, if even a portion of the cases is DBP related, the annual costs accrued are still significant.

Using a linear multistage model (LMS), the appropriate range of regulatory levels for the United States corresponds to risk range of 10^{-4}-10^{-6} (Craun *et al.*, 1994(a)). Another linear extrapolation model applied to drinking water placed the lifetime cancer risk in Canada as ranging from $10^{-5} – 10^{-6}$ (Krishnan *et al.*, 1997)

Craun *et al.* (1994(a)) also mentioned that there is a high level of uncertainty in the cancer risk estimates for the low exposure levels found in drinking water because risk assessment models have not been experimentally verified. The risk assessment models extrapolate results found from animal data in a linear fashion over several orders of magnitude without specific justification from the scientific data available.

The uncertainty surrounding the linear extrapolation approach for analysis of disinfectant by-product risk is an area of dispute in several studies (Golden *et al.*, 1997; Hrudey, 1998). Golden *et al.* (1998) suggests that the LMS model is an area of risk assessment that could be improved, as it is based on extrapolating the results of chronic high-dose studies in rodents to estimated potential risk in humans. Hrudey (1998) states that although the USEPA guidelines adopt the LMS model as the default model, the absence of adequate epidemiologic data led to the application of the model without some important qualifiers, including that risk estimates currently do not permit more than one significant figure to be presented.

Thomas and Hrudey (1997) highlight another area for concern with toxicological analysis. The objective of the cancer bioassay is to select the maximum tolerated dose (MTD), which is the maximum possible dose that does not produce substantial non-tumour effects, reduce the animals weight by more than 10%, or affect the longevity of the animals. Sub-chronic studies of ninety days or less are usually performed to establish the MTD. The doses applied to the test animals are then applied in levels lower than the MTD. The problem with this is that the carcinogenic level of the chemical may be greater than the MTD.

The use of laboratory animals for human comparison was examined in the Thomas and Hrudey study (1997). This discussion resulted in some very important differences between animals and humans:

- Rodents have a life span of two to three years compared with 75 for humans;
- Laboratory animals are bred to be genetically similar;
- The mass ratio for humans and rodents is ten times greater than the surface area ratio; and
- The route of exposure from a laboratory study may differ from the exposure route of greatest concern for humans.

Thomas and Hrudey (1997) also discuss the issue of a threshold. If no threshold exists, any amount of exposure to a carcinogen will pose some risk, as represented in the LMS model. If there is a threshold, the carcinogen must reach a certain level before a risk is observed. It is discussed that the LMS model does not necessarily give a realistic prediction of the risk.

Epidemiological studies are seen to be a preferable method of analysis. However, this approach also has concerns, including the realisation that DBPs may be linked to health effects in chlorinated supply that they did not in fact cause (Craun *et al.*, 1994(a)).

More recent studies have created greater scope for the application of the findings, as they are based on recent epidemiological studies. Represented in this analysis are three studies from Sardinia, Italy (Attias *et al.*, 1995), Colorado, U.S.A. (McGeehin *et al.*, 1993), and Norway (Flaten, 1992). All three of these studies have occurred since the IARC (1991) analysis and the Morris *et al.* (1992) meta-analysis.

The Italian study assigns risk levels per μg/L of THM instead of relative-risk. This allows a floating penalty for DBPs to exist. Research in Sardinia, Italy (Attias *et al.*, 1995) found that chlorinated surface water increased the cancer risk to consumers. The approximate risk level of 1×10^{-5}/μg/L THMs for the epidemiological analysis compared to approximately 1×10^{-6}/μg/L for a LMS model from animal testing. When applied to a system with 50μg/L of THMs, these extra risk levels are adjusted accordingly.

The Sardinia study is also of interest as it has a well-established dosing regime that has remained unchanged for several decades. This provides a sample space of consistent levels of chlorine added over time so the results obtained are not caused by large dose fluctuations. The study concluded by stating that while it found no reasons for concern from the risk values, further study in the area wass required.

The study based in Colorado (McGeehin *et al.*, 1993) considered 327 histologically verified bladder cancer cases against age, sex and 261 other cancer controls. The relative risk of contraction increased for longer levels of exposure (30 years) to chlorinated surface water to a ratio of 1.8. Unlike the Attias *et al.* (1995) study, levels of DBPs were not associated with bladder cancer risk as it was an epidemiological study based on chlorination of supply.

The Norwegian study (Flaten, 1992) focussed on an ecological epidemiological analysis based on nationwide data from the Cancer Registry of Norway. From this analysis, after variable adjustment, the chlorination of drinking water was linked with an increased incidence of colon and rectal cancer.

Wigle (1998) from Health Canada discusses this growing epidemiological evidence. This study mentions that disinfection by-products have been estimated to be potentially responsible for 14-16% of bladder cancers in Ontario, Canada. A workshop established by Health Canada (Mills *et al.*, 1998) concluded that it was possible (stated by 60% of group) or probable (40% of group) that chlorination by-products pose a significant risk to the development of cancer.

In this study, risk association for DBPs and cancer assisted in providing a social cost regime for water quality quantification. However, it is recognised that current practice methods require further research through several epidemiological studies before the level of accuracy is sufficiently tightened. The use of DBP risks allows a trade-off with the waterborne disease issues, thus providing a more complete overview of the water quality in the distribution system.

More extensive research into this area is required. Instead of maximum tolerated dose studies, lower exposures studied over a longer time period would provide a more realistic analysis of the potential risks (USEPA, 1997).

3.9.3 Debate on Risks due to Disinfection By-products

The debate on the potential health risks of disinfection by-products has not been confined to the purely academic or industrial level. Rachel's Environment and Health Weekly (Montague, 1988) published an article in 1988 condemning the use of chlorination as a disinfectant. It stated that "we should switch to ozone treatment, abandoning chlorine" and stated that chlorinated chemicals cause cancer in humans.

The Chlorine Chemistry Council disagreed with this article, and published under its banner a retort to this piece called Rachel's Folly: The End of Chlorine (Malkin and Fumento, 1996). This report was written by environmental journalists, and adamantly supported the use of chlorine as a disinfectant and dismissed the links to cancer.

The debate as to the association between DBPs and cancer in humans will continue until adequate evidence is determined from research proving an association or a lack of an association. The discussion raised by Thomas and Hrudey (1997) on the risk assessment process for determination of carcinogens is also important. If the method of determination is questionable, the degree of surety in the relationships formed from the assessment process become tenuous.

3.10 Quantification of Water Quality Risks

To produce a distribution system that balances the water quality issues, while optimising for hydraulic cost, it is important to assess the water quality in the system in a way that allows it to be compared with the infrastructure, operation and maintenance costs. To do this, quantification of the risks associated with water-related illness produces a potential cost structure. It is then possible to make the comparison with system hydraulics.

3.10.1 Background

Although more developed information on the status of water supply systems and infectious disease is required to adequately quantify risk (Ford and Colwell, 1998), cost-benefit analysis can be applied to the current level of knowledge to assist in assessing the potential risks of illness.

The social cost applicable to mild cases of infectious illness such as gastrointestinal illness can be large. For the U.S. in 1985, it was estimated that the annual cost of gastrointestional (GI) illness to society was US$19.5b for cases with no consultation, US$2.75b for those with consultations and US$0.76b for those requiring hospitalisation (Ford and Colwell, 1998). These costs do not include the deaths associated with these illnesses. Waterborne disease makes up a large proportion of the cases of GI illness, so from this assessment alone, the potential scale of costs can be ascertained. The EPA have estimated that the U.S. annual medical and productivity-loss costs due to waterborne illness ranges from US$3b to US$22b (Gelt, 1998).

The social cost due to disinfection by-products can be gauged from the estimate that over 10,000 cancer contractions (bladder and rectal) may be attributed annually to water supply in the US (Morris *et al.*, 1992). Although there is uncertainty about the carcinogenic capacity of DBPs, and even though the survival rates for these particular cancers are higher than most (Hallenbeck and Cunningham, 1986), the medical and productivity-loss costs alone would be significant.

3.10.2 Cost - Risk Benefit Analysis for Known Risks

When the risks of an activity are known, their "acceptability" in pure economic terms can be ascertained. This economic value is the cost to reduce or control this risk. Any health risk of a chronic nature that does not cost anything to be eliminated is an unacceptable risk.

The economic practice common in risk evaluation is to determine the "dollars spent per statistical life saved", involving the following process (Cox and Ricci, 1989):

1. Determine dollar value per statistical life that those at risk place on their own lives, determinable by survey or insurance studies;
2. Evaluate financial cost of alternative control options, even banning that activity. The opportunity costs (benefits foregone) should be included along with the direct cost of control to provide the total economic cost to society;
3. Implement the control options which have benefits outweighing their costs;
4. A risk is deemed to be acceptable if the costs of further control exceed the benefits for further control.

This technique can be applied to the issues of water quality in a supply as given in Table 3.15.

Table 3.15 Application of Cost-Risk-Benefit Analysis for Water Supply

Process	Application to Water Supply
1	Determine value that the consumers of the water supply have on their health and safety.
2	Examine the tradeoff between the different costs attributable to water quality eg. What level of disinfection and infrastructure improvement provides the preferred total consumer cost.
3	Implement the methods that present the minimum total cost.
4	The risks associated with this water level are deemed acceptable if the costs to supply a better quality prevent the quantity of supply to allow emergency storage for issues like fire fighting.

The determination of the value that a person will place on their own health forms their willingness to pay for a method to reduce the risks of adverse health.

3.10.3 Willingness to Pay

Willingness to pay is the principle equating the costs of illness and death by determining the amount people would be prepared to pay to avoid the inconvenience of getting sick or dying. This value changes from person to person, but the value represents the importance placed on their life.

Avoiding problems found in data collection, such as people saying they would be prepared to give everything to stay healthy, is imperative for surveys. The approach taken by many data samplers is to ask how much the person would be willing to pay to reduce the chance of dying in the next year by 0.1%. Then the person is asked how much this value would alter if the reduction were 0.2% and so on.

Using regression analysis, a value is determined using "willingness to pay" for 100% chance of avoiding sickness or death. Shepard and Zeckhauser (1982) describe the process of costing willingness to pay in detail. By using this method, values of around (1980 US) $800,000 were obtained for females to $1,500,000 for males. The discrepancy was explained in the study as a function of earning capacity in 1980.

The amount of damage potential a community is willing to accept is an alternative method for measuring "willingness to pay". Greene (1978) discusses the maximum tolerable risks. This examines what level of a risk a community will tolerate, in number of deaths/100,000 population. Enteric and diarrhoeal disease commonly caused by poorly treated water supply systems was deemed tolerable at a level of 2.7/100,000 at the time of his analysis. Greene (1978) argued that communities could look towards a considerably lower level of 0.3/100,000 with alterations to treatment and public education.

The nuclear industry has studied the value of health in detail (Roberts, 1984). Based on 1980 monetary values, Table 3.16 lists the expenditure (1980$US) to save a statistical life in a number of different situations.

Table 3.16 Expenditure to save a life

Situation	1980$US
Food for Third World starvation relief	11,200
Intensive care	24,000
Screening for lung cancer	128,000
Heart isotope pacemaker	128,000
Prevention of traffic accident	320,000
Smoke alarms in houses	480,000
Steel industry accident prevention	960,000
Chemical industry accident prevention	1,600,000

These numbers can be adjusted to current $US by applying a multiplication factor of 2.15 (extrapolated from Dandy and Warner, 1989). The water industry is faced with the issue of the level of expenditure they are prepared to pay per life saved, or illness averted.

3.10.4 Costs of Potential Illness from Drinking Water Consumption

Determining the cost of illness is a similar process to the cost of death. It can be evaluated in terms of lost income while the person is sick, medical costs of the illness or the willingness to pay to prevent poor health. In the United States, the assumed value placed on illness due to waterborne disease was placed at US$1994 $3,000 (Craun *et al*, 1994(b)). This value is backed up by the Federal Register of July 29, 1994

Levin and Harrington (1995) provide different values for illness. It was determined that for mild to severe waterborne illness, US$2,680 is acceptable based on medical and lost productivity costs. For extremely mild cases, a level of US$272 was produced which did not consider the medical or lost productivity costs incurred.

For cancer contraction, the contingent value method places every cancer as US$230,000 (Levin and Harrington, 1995). The EPA values for cancer cases are US$2,000,000, which is a considerably higher value. These represent high and low bound options.

The likelihood of contraction of cancer in a lifetime from 1985 is 36% to 37% with death due to cancer related illness (not necessarily the same cancer) of 20% to 23% in an American study (Hallenbeck and Cunningham, 1986).

Costs for the different levels of illness for both biological and chemical problems (Levin and Harrington, 1995) are listed in Table 3.17.

Table 3.17 Contraction costs for water related illness

Illness Category	
Waterborne Disease	
Moderate to severe	$2,680
Mild to Moderate – high	$2,680
Mild to Moderate - low	$272
Disinfection By-Products	
Death*	$2,000,000
Occurrence*	$230,000

* Includes occurrences of bladder or rectal cancer

The costs involved reflect the vast differences in assessing waterborne disease and cancer. By taking these values for these separate illnesses, they can act as the costs of illness in the model.

3.11 Assessment of Need for Disinfectant Residuals: Balancing the Risks

Considering the previous discussion for this chapter, disinfection is a necessary step to ensure against waterborne disease. Unfortunately, concerns about both the aesthetic and health characteristics of disinfection remain. The current discussion raised by parts of the water industry is whether disinfection is required in water supply. With advancements in treatment and system maintenance, many of the water quality concerns are lessened and even in some cases removed.

As discussed, maintaining disinfectant residuals throughout a distribution network is used to prevent microbial regrowth. Due to the concern expressed at the chemical nature of these residuals both for aesthetic and health reasons, the need for residual provision in some systems is questioned by some researchers.

If the system is constructed and maintained perfectly with a pure initial supply, there would be no opportunity for regrowth hence no need for a disinfection residual. An examination of a network in Holland that used groundwater as the primary source (van der Kooij *et al.*, 1999) found that operational changes in water treatment in replacing post-chlorination with ozone treatment and GAC (Granulated Activated Carbon) filtration greatly improved the taste and odour of the system. Complaints dropped by about 80% as a result of the residual removal. This method of reaching quality objectives is recently in favour in a number of European urban centres (Bull, 1992).

In a different overview, (van der Kooij, 1999) the issue of cost is mentioned. Van der Kooij (1999) discussed that in many countries, though increased treatment methods will raise the delivery cost of the water, the final product is an excellent standard of quality at a considerably lower price than bottled water.

In economically disadvantaged countries, chlorination represents the only affordable method of sanitation against waterborne disease transmission. The choice in this situation is water with high waterborne disease risk, or microbiologically safe water with potentially harmful levels of DBPs. Faced with this decision, Reiff (1995) states that the microbial safety of the water is to take precedence based on significantly greater risks attributed to waterborne disease. Reiff (1995) compared other means available to developing countries for sanitation, from boiling (US$50/year) to chlorination (US$0.2/year to US$0.8/year). This study clearly shows the short-term economic advantage in using chlorine to achieve adequate sanitation.

The priority on waterborne disease prevention is ratified by Craun *et al.* (1994(a)) who also mention that the comparison of different disinfectants to determine the least by-product risk is premature as additional research is required. Trussell (1999) concurs stating that for the present, those who can continue to use free chlorine are probably wise to do so, pending further research into the potential side effects of alternative disinfectants.

The World Health Organisation in the latest edition of 'Guidelines for Drinking Water Quality', provides a clear position on the issue disinfection:

'It is cautioned that where local circumstances require that a choice be made between meeting microbiological guidelines or guidelines for disinfection by-products such as chloroform, the microbiological quality must always take precedence. Efficient disinfection must never *be compromised.'*

World Health Organisation, 1998.

The aim of a system manager is to balance the risks to produce the cleanest and safest water attainable to the consumers while remaining economically viable. If, as is the case in Europe, technology and finances allow for limited or no required residual without compromising quality, then that is a well operated system. If the network is in a developing country, or rural area with reduced resources, the decision process becomes more difficult and balancing the risks to provide sanitised water will make a well managed system.

The modelling ability of water quality in distribution systems is an area of technological advancement that has the potential to reduce costs for water managers globally. By analysing what could potentially occur, the balancing act required by operators can be made considerably less difficult.

The determination of the system's residual requirements can be attained through computer simulation, allowing different disinfectants and regimes to be examined. Although a model is only as good as the data feeding it, modelling promises to be of significant value in providing a balanced water quality profile for systems in the future.

3.12 Modelling of Water Quality in Distribution Systems

A method to assist in reduction of water quality risks is to use computer modelling to determine outcomes of a particular system design or chlorine dosing arrangement, or even modelling which determines the optimum design of the system. One of the major recommendations put forward to improve the microbiology of the drinking water in Adelaide, South Australia was that water quality modelling of distribution systems should be adopted as a means to assess the proposed options for water quality improvement (SA Water, 1995). It was also required to complement the water quality monitoring program for operational control.

If modelling for water quality in a distribution system, it is important to have a well-calibrated hydraulic model, preferably based on a tracer study (Vasconcelos *et al.*, 1997). The solver used for modelling in this study is the EPANET package (Rossman, 1994), which has been hydraulically calibrated against both hydraulic data and quality data, as well as compared to previous models relevant to the systems studied.

EPANET has also been used by several other researchers as part of their quality analysis (Bocelli *et al.*, 1998; Goldman and Mays, 1999; Tryby and Uber, 1999; Sakarya and Mays, 1999).

3.12.1 Modelling Disinfection Residual

A basic understanding of disinfection kinetics is necessary in designing a rational and economical disinfection process (Selleck *et al.*, 1978). As the drinking water quality is highly dependent on the interaction between the disinfectant and the supply, modelling the decay of disinfectants in water is an important initial step in understanding their role in system quality.

Control of microorganisms in drinking water requires use of effective disinfectants along with acceptable disinfection design criteria to ensure public health protection as well as risk minimisation from chronic exposure to disinfection by-products (Gyurek and Finch, 1998). The aim is to allow sufficient residual, however, not provide too high a residual or allow the formation of too great a level of disinfection by-products.

EPANET uses a first order decay model seen in Equation 3.15 when examining disinfection kinetics in water systems.

$$\theta(Cn) = -k_1 Cn \qquad \textbf{(3.15)}$$

$\theta(Cn)$ = rate of decrease of disinfectant, k_1 = first order decay rate and Cn = substrate concentration (mass/volume).

The integration of Equation 3.15 with respect to time gives the first order formulation seen in Equation 3.7 from Section 3.6.3.1.1.

3.12.1.1 Modelling Residuals in Pipes

Modelling only considering bulk disinfectant decay in a pipe is not as accurate as including the decay due to the pipe wall into the analysis. Decay due to the pipe wall can have significant effects on the water quality in the system (Biswas *et al.*, 1993).

Although much of the previous modelling has used first-order disinfectant decay, the second order decay model has been found to provide more accurate results when applied to both field and laboratory data (Clark, 1998; Barker *et al.*, 1998). A new edition of EPANET is currently being tested. This model uses n^{th} order bulk decay and 1^{st} and 0^{th} order wall decay giving it greater flexibility (EPANET2, 1999).

If the level of hydraulic data and quality information regarding the system is good, the ability to model for the wall and bulk decay is very achievable. The hydraulic information is important as pumping rates, tank operation and pipe arrangement will all have an effect on the water quality data.

If adequate water quality data for the system is available but the hydraulic data is insufficient, the approximate decay rate (K) encompassing both bulk and pipe wall decay can be determined. These rates can be seasonally based and suffice for any modelling specific to this system.

If the knowledge of the system quality was unavailable, due either to poor monitoring or a non-existent system but the hydraulic information is acceptable, the methodology for pipe wall and bulk decay analysis can be followed.

If the water quality and the hydraulic data are poor, both elements need to be enhanced before decay rate determination of reasonable accuracy can occur.

3.12.1.2 Modelling Residuals in Water Storages

In general, knowledge of the effect of distribution systems on water quality is limited. Water storages are the most visible component of the infrastructure and are usually designed on hydraulic considerations. Mau *et al.* (1996) has determined that the need to attain a thorough understanding of the transport and mixing processes in storage facilities has been prompted by a shift in the focus for water quality in systems from the point of treatment to the point of consumption.

Mau *et al.* (1995) found that mathematical modelling of storages proved a valuable method for the prediction of system response and the fate and migration of water quality contaminants. The level of field data attained that the model is based upon is a dictating factor in the relevance of the results for short term to long term use. Modelling a water tank is usually done in two ways:

- Development of a physical scale model; and
- Development of a mathematical representation of the system.

Clark *et al.* (1996) developed a model that separated the tank into compartments to analyse for mixing in tanks. This method separates the water in the tank into different mixing zones, rather than assuming the tank as one body of water. In considering 1, 2 and 3 compartment models, it was concluded that in some cases compartment models provide a better representation of mixing and residence times in tanks.

The emergence of computers as a modelling tool have popularised mathematical modelling and through improved data collection and model controls, strengthened the capabilities of scale modelling (Grayman *et al.*, 1996).

3.12.1.3 Modelling Residuals in a Distribution System

To model residuals in a distribution system, EPANET (Rossman, 1994) tracks the path of a dissolved substance through the system over time. Using flows obtained from the hydraulic simulation (run prior to the quality analysis), it solves the conservation of mass (Equation 3.16) for each link between nodes *i* and *j*.

$$\frac{\partial Cn_{ij}}{\partial t} = \frac{Q_{ij}}{A_{ij}} \frac{\partial Cn_{ij}}{\partial x_{ij}} + \theta\left(Cn_{ij}\right) \qquad \textbf{(3.16)}$$

Where Cn_{ij} = concentration of substance in link *i,j* as a function of distance and time (mass/ft^3), x_{ij} = distance along link *i,j* (ft), Q_{ij} = flow in link *i,j* at time *t* (cfs), A_{ij} = cross-sectional area of link *i,j* (ft^2) and $\theta(Cn_{ij})$ = rate of reaction of constituent in link *i,j* (mass/ft^2/day).

The rate of reaction $\theta(Cn_{ij})$ is solved separately for each link and tank using Equation 3.17.

$$\theta(Cn) = -\left[k_b + \frac{k_w k_f}{r_h\left(k_w + k_f\right)}\right] \times Cn \qquad \textbf{(3.17)}$$

Where k_b = bulk decay of chemical (sec^{-1}), k_w = wall decay of chemical (sec^{-1}), k_f = mass transfer coefficient (ft/sec), r_h = hydraulic radius (feet) and Cn = substance concentration (mass/ft^3).

If the change in water quality is being analysed for a tank, the decay due to the wall is assumed to be zero, hence the bulk decay (k_b) is the only factor considered.

3.12.2 Optimising Disinfection in Distribution Systems

Optimisation modelling of distribution systems for water quality has not been widely researched, less so the inclusion of hydraulic and operational factors. However, there have been a few models produced recently that consider the optimisation of disinfection in water supply systems. These studies examine the provision of adequate, but not excessive disinfectant residuals throughout a pipe network.

Artificial neural networks (ANN) have been used to predict residual patterns, hence provide a water quality management profile for the system (Rodriguez *et al.*, 1997). The ANN method mimic the operator's knowledge in water chlorination by using historical data and characterising the different decisions made by the operator for different conditions. The process aims to be able to be integrated into operation of water utilities to assist in routine decision making. Though not necessarily an optimisation tool in itself, it aims to allow prediction to the extent that the operator can use it to provide efficient decisions for system quality.

Mixed integer linear programming has been developed to provide locations and operating data for booster disinfection stations in a water network (Tryby and Uber, 1999). This optimal scheduling and location model aims to minimise the total mass of disinfectant added to the system at a set of booster injection stations. This model interfaced with the EPANET package for its quality analysis. This study concluded that optimisation of disinfection should assist designers in booster station selection, and stressed the importance of the role booster disinfection takes in system quality.

This study followed on from one on optimisation of booster disinfection undertaken by Boccelli *et al.* (1998). This study concluded that while trial and error techniques may have difficulty in determining feasible solutions to the minimisation of disinfection, optimisation approaches such as the linear programming formulation from this study allows a comparatively quick generation of alternative booster disinfection schedules.

Sakarya and Mays (1999) have applied a non-linear approach towards optimal operation in supply systems. This study examines the optimal operation of pumping in the system to minimise the deviations of residuals from desired levels as well as to minimise pump operation times and total energy cost. This analysis uses EPANET as the solver. The model applies a penalty factor to the pressures, concentrations and water storage heights in the system as shown by the bracket penalty function method in Equation 3.18.

$$PB_j\left(V_{j,i},R_j\right)=R_j\sum_i\left[\min\left(0,V_{j,i}\right)\right]^2 \qquad \textbf{(3.18)}$$

PB_j = penalty function used, $V_{j,i}$ = the violation of the bound constraint j, R_j = penalty factor, i = represents the pressure penalty term, concentration penalty term or the storage bound penalty term depending on which aspect is being considered.

These penalty factors should be increased if any of these values move away from desired levels, which forces the solution space to an efficient range. The squared aspect penalises the solutions violating the constraints more severely the further they are from the desired levels. This study mentioned its dependence on the selection of good values for the penalty factors in determining the global optimum.

A similar study using simulated annealing to optimise system operation is in progress by Goldman and Mays (1999). Simulated annealing allows a variety of objective functions to be optimised. This model was applied to the North Marin Water District, Novato, California, to determine optimal operation to minimise power costs while satisfying hydraulic and water quality constraints. The objective function used for this model is given in Equation 3.19.

$$Minimise\sum_{p=1}^{P}\sum_{t=1}^{T}\frac{UC_t 0.746PP_{pt}}{EEF_{pt}}D_{pt}+\sum_{n=1}^{N}\sum_{t=1}^{T}\left[Penalties\right]_{nt} \qquad \textbf{(3.19)}$$

UC_i = unit energy cost of pumping during time period t (\$/kw-hr), PP_{pt} = power of pump p during time period t (hp), D_{pt} = length of time pump p operates during time period t (hr), EEF_{pt} = efficiency of pump p during period t and $[Penalties]_{nt}$ = the penalty factors from inadequate hydraulic performance.

The study concluded that for water quality analysis, simulated annealing linked in well with EPANET. By altering the dosage at each pump to fit in with the desired residual pattern an over view of system quality is provided.

3.12.3 Complete Optimisation of Distribution Systems

The application in this study takes the level of combining the hydraulic and quality aspects one step further. The research examines the sizing and location of the pipes, pumps, and tanks as part of the water quality optimisation. How infrastructure can affect the residual levels in the system is examined and optimised together with the provision of a seasonal dosing regime at optional dosing stations.

The approach has been adapted to EPANET as with the other methods, and due to the success shown with the application of other studies, the solver seems suitable. The penalty method used by Sakarya and Mays (1999) for residual levels has been applied as a means of quantifying water quality, allowing modelling to occur.

To take up the point raised that the selection of these penalty factors has a large bearing on the search for the global optimum, another method of quantification will allow cross-checking of the results determined from the penalty analysis. A social cost analysis, based on determining cost structures for the different facets of water quality that may occur at a consumer level has been developed. This method examines the shift in water quality focus mentioned by Mau *et al.* (1996). These structures are based as closely as closely as possible on researched data.

The method used to optimise this combined approach is genetic algorithms, which has been successfully linked with EPANET to provide optimal solutions for hydraulics in previous problems. The optimisation approach used in this study considers three aspects new to research:

- GA optimisation of water quality in pipe systems;
- Optimisation of sizes and locations of infrastructure as well as dosing as part of the water quality analysis for a system; and
- Application of a quantification approach that is based on realistic estimates of water quality costs, to assist in checking the effectiveness of the quality penalty approach used by previous researches to determine the global optimum.

3.13 Summary

The post-treatment water quality structure in a distribution system is dependent on a combination of the sizing and condition of the system infrastructure, initial water quality, and the distribution disinfection regime. These facets determine the residence time, decay rates and potential heath concerns that could be attributed to the supply.

The areas of concern that arise to the consumer from a supply are the risk of illness from waterborne disease in a poorly managed system, potential health concerns from high disinfection by-product levels, and the aesthetic appeal of the supply. These aspects are inter-related and need to be balanced by the system operator.

Water quality modelling has been shown to be a useful tool in assisting water system operators to provide a good level of water quality throughout the system. Not only have advancements in modelling of quality allowed a more developed understanding of disinfection residual profiles in pipes and tanks, but also they have allowed mechanisms of optimising residual levels to within specified guidelines.

Some researchers have used application of system hydraulics, with EPANET proving to be a successful integrating package for water quality analysis. To date, pump and tank operation has been the hydraulic focus in hand with the development of a dosing station regime. This study takes the research into this area further by including water quality analysis in the full system design, pipes, pumps, tanks, dosing stations and dosing regime. It also examines using different means of assessing water quality in a system, from the method used by other researchers of applying a quality penalty, to the application of a social cost framework encompassing the different consumer issues with water quality mentioned earlier.

By adapting the information from this water quality summary to specific systems, a more complete analysis of distribution system design optimisation is achieved.

Chapter 4 Genetic Algorithms

4.1 History of Genetic Algorithms

Holland (1975) first proposed genetic algorithms (GAs) as a search engine for complex solution spaces. Goldberg (1989) extended this research to encompass practical applications of GAs to engineering problems. GAs have been successfully applied to pipe network optimisation (Simpson *et al.*, 1994; Dandy *et al.*, 1996; Savic and Walters, 1997), water resources planning (Connarty, 1995), and structural optimisation (Sved *et al.*, 1991).

In this study they are applied to optimising hydraulics and water quality in water distribution networks.

Genetic algorithms work by analogy to population genetics and, as such, contain processes based on Darwin's Theory of Natural Selection.

4.1.1 Darwinian Theory of Natural Selection

In 1859, Charles Darwin produced the theory of natural selection (Begon *et al.,* 1990). Darwin's research rested on the following propositions:

- Individuals making up a species population are not identical, variance (even slight) occurs in areas such as size, temperature response and development rate;
- Characteristics of an individual are due in part to genetic make-up, meaning that offspring have a tendency to share characteristics with their parents;
- All populations have the potential to achieve domination of their environment;
- Different individuals leave differing numbers of descendants, this is based on number of offspring, survival probability of offspring and reproduction capabilities of the offspring; and

- Number of descendants left by an individual depends crucially on the interaction between the characteristics of the individual and their environs.

As the environment will dictate to a crucial degree which individuals will have greater success pass on their genetic material to the next generation, a selection process is formed which causes the population to adapt to the environment in which it lives. The longer a population survives in a non-changeable environment, the better suited it will become towards it.

4.2 Methodology of Genetic Algorithms

The application of genetic algorithms pursued in this study is to optimise water distribution systems for both hydraulic and quality concerns. With this application of GAs, an individual represents a design for the complete supply network. A population is made up of many network designs. The environment, within which the algorithm is optimising, is the overall cost of establishing a system that treats the water to an acceptable level, and guarantees a supply with acceptable pressures throughout. A comparison of this application to that discussed in Darwin's theory based on biological systems is illustrated in Figure 4.1.

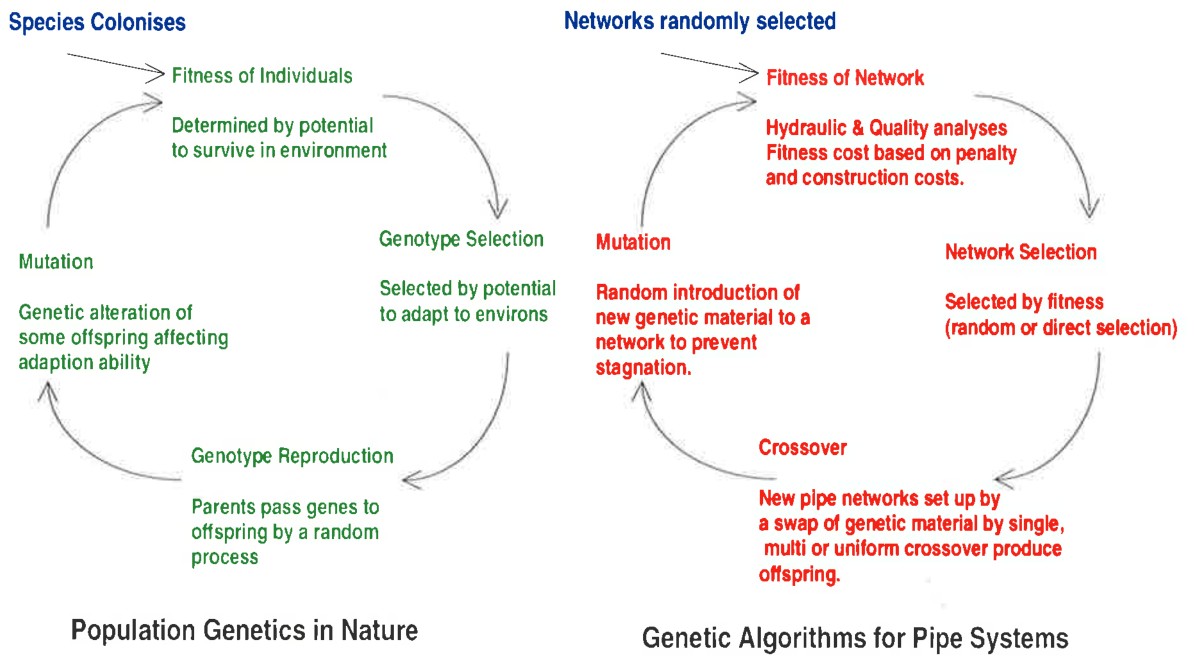

Figure 4.1 Comparison of ecological evolution with supply system optimisation

Figure 4.1 highlights how an biological population interacts over numerous generations with a new environment and gives an outline of how this format can be adapted to distribution network optimisation. One of the vital components that need to be determined is the make-up of the environment that the individual has to conform to. This environment is established by the provision of constraints.

To provide a background of how evolutionary theory can assist in the determination of efficient design pipe systems, a small example of how the methodology operates will be provided throughout this section. An illustration of the worked example is given in Appendix C.

4.2.1 Constraints

In any environment, there are always limitations that once passed will have negative impacts on the individuals. In a human population, this is often climate related. If the temperature moves out of the comfortable range for an extended period, it has detrimental effects on individuals not suited to the new level. The greater the shift out of the acceptable temperature range, the more significant the ramifications for the individuals exposed.

In an engineering problem, there are acceptable results and those deemed inferior. To properly establish the environs, boundary conditions are set to provide a benchmark for the solution space. If the constraints are violated, the solution is infeasible (Goldberg, 1989).

In the case of water distribution systems the constraints take the following forms:

- The total volume of water supplied must equal the total demand;
- The pressure of the water throughout the supply network must be at a level deemed acceptable by the relevant bodies;
- Velocities in pipes throughout the system must be within acceptable ranges;
- Any component (pumps, valves, tanks) must not be used beyond its capacity; and
- The water quality should be at a level deemed acceptable by the relevant bodies.

If all of these constraints on the system are met, the network is working to the satisfaction of the supplier and is an acceptable solution. If one or more of these factors are neglected, the system is not meeting the requirements and is an infeasible option.

If a system does not meet the constraints, the solution can be either disregarded, or a penalty is applied to the result, which is weighted so that the next generation of solutions are encouraged to give more emphasis on the constraint that was unsatisfied.

4.2.1.1 Penalty Method

Genetic algorithms work most effectively on unconstrained problems. A penalty is placed on an infeasible solution to encourage other solutions that satisfy the constraints into the population. The use of pressure penalties includes the constraints into the overall fitness function and allows the GA to make the decisions to find the optimal feasible solution.

It should be noted that the GA model should eventually achieve zero values for the penalty costs. This makes the intermediate iterations preceding the optimal solutions irrelevant, as they are a means to the optimum, rather than solutions in themselves (Taha, 1982).

4.2.2 Decision Variables

The decision process indicates the options available for selection. In genetics, one decision variable could be a measure of intelligence, with several stages of intelligence determining the ease of conforming to the current environ. In pipe systems, the decision variables can be the different diameters available for a pipe.

For this thesis, the following decision variables have been implemented for analysis of different networks:

- Diameter of the each of the new or duplicate pipes in the network;
- Size of all the pumping stations;
- Sizing of all new tanks; and
- The level of dosing of disinfectant at each dosing station.

These four factors can produce many decision variables depending on the number of pipes, pumps etc. requiring analysis in the network. The more options available for a decision, the greater the total search space becomes as indicated by Equation 4.1:

$$\text{Size of Search Space} = \prod_{i}^{n} N_{o_i}^{\;N_{d_i}} \qquad \textbf{(4.1)}$$

N_{di} = number of decisions of type i, N_{oi} = number of options of type i.

The worked example in Appendix C has four decision variables which have two options each. The total number of possibilities is 2^4 or 16 different solutions. If there were five options for each decision, the search space would have 5^4 or 625 possibilities. In the study, the search space is extended to over 10^{64} possible solutions.

4.2.2.1 Coded String

Each individual (distribution system) is represented by a coded string, which is made up of all of the decision variables. Each decision can have its options represented in integer, real, binary (Murphy *et al*, 1993) or Gray (Mulligan and Brown, 1998) form. In this study, integer coding was used. Higher order alphabets also work without loss of efficiency and power (Grant, 1995). The type of coding scheme depends on the problem.

4.2.3 Fitness

In evolution, the fittest individuals leave the greatest number of descendants (Begon, 1990). They tend to be the individuals that are best suited to their environs and have the capacity to propagate effectively.

The application of this scenario to search spaces involves assigning probabilities to each individual of their success as a parent based on their interaction with the environment. The measure of this interaction is classified as the individual's fitness, and the measure of this level is termed the fitness function. The concept of a fitness function is used to evaluate how well the individual's make-up solves the problem (Grant, 1995).

For the optimisation of pipe networks, fitness is measured by the total cost of the project. The lower the total cost, the higher the fitness and, hence, the greater the likelihood of that individual being a successful parent for the next generation.

When examining pipe systems, the fitness of a network is measured by its ability to provide the best infrastructure and treatment processes to guarantee an acceptable level of supply at the lowest cost to the supplier. When constraints are violated and penalty costs are imposed, the total cost is given by Equation 4.2.

$$\text{Fitness} = \frac{1}{\text{Total Cost}} \qquad \textbf{(4.2)}$$
$$\text{Total Cost} = C_I + C_{pen}$$

C_I = present value of the capital and operational cost of infrastructure, and C_{pen} = total penalty cost.

4.2.4 Initialisation

Initialisation is the establishment of an initial population of individuals in the specified environment. It can be equated to a species of individuals moving into a new environment, such as colonisation of an island. The climate, food groups and predators are different, hence, alternative skills and abilities are required in order to thrive under the different conditions.

In modelling, initialisation is achieved by randomly establishing a series of coded strings. The number of strings generated is the population size. The population is important for the effectiveness of the genetic algorithm as it provides the total initial genetic material that the GA has to work with. The larger the initial population, the greater the probability of an optimal solution being identified. However, the number of evaluations required to achieve convergence may be significantly greater as shown by Simpson and Goldberg (1994).

4.2.5 Selection

The selection process determines which individuals are most likely to pass on genetic material to the next generation. The higher the fitness of an individual, the greater the probability is of passing the genetic material to the next generation. Selection determines reproduction as it follows Charles Darwin's 'survival of the fittest' theory (Murphy and Simpson, 1992)

Several techniques can be implemented to decide which individuals would attain this. Tournament selection is a powerful method that has proven to be quite effective (Jiaping and Chee-Kiong. , 1997) Tournament selection is described below.

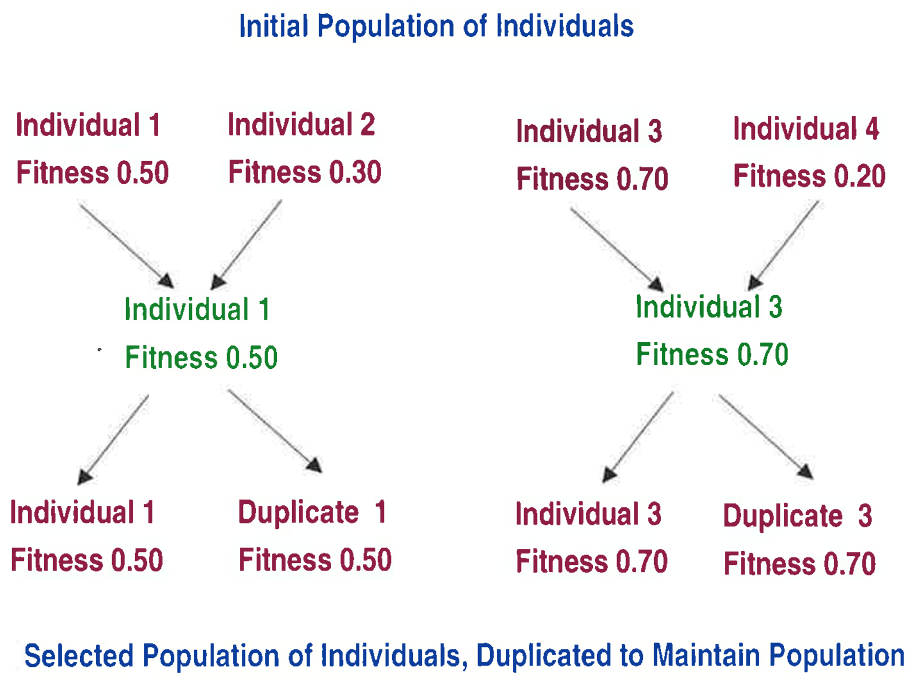

Figure 4.2 Tournament Selection

Individual 1 is selected ahead of Individual 2 as it has a higher level of fitness. A similar process is calculated for Individuals 3 and 4. Figure 4.2 shows what would occur four a population of four individuals, with a tournament selection in groups of two.

The number of individuals in each tournament group can also differ. The lower the number, the more direct the process is. The larger the number in a group, in effect a roulette selection process is occurring in each group.

Proportional selection is a more simplistic, but less powerful method of determination, it is also a method that can be undertaken as the sole selection process. This works on the same principles as a roulette wheel in a casino. Each individual has a portion of the wheel allocated to it in proportion to its fitness. Hence, the probability of selection of an individual is proportional to its fitness. An example of a proportional analysis is illustrated in Figure 4.3.

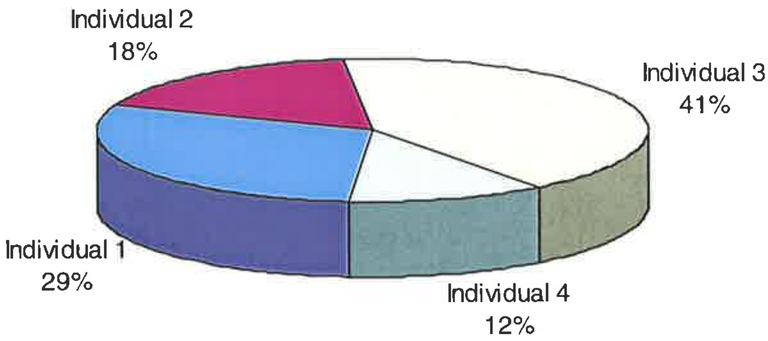

Figure 4.3 Roulette Selection

By this method, the wheel is spun a number of times equal to the population and the individuals selected from these rotations. These individuals become the parents for the new population.

4.2.6 Elitism

Elitism is a method of guaranteeing that the fittest individuals survive the selection process (as there is a probability they will be knocked out). It replaces the worst solutions with the best solutions and carries them through to the propagation phase of the genetic algorithm.

In some cases they can come back into the population. In this case there is an individual being replaced by one of the fittest solutions from previous generations.

4.2.7 Crossover

Crossover involves mixing of the genetic material in the strings. It involves the breakage, partial exchange and reunion of a pair of strings (individuals). There are several different forms that this crossover can take. Three of the main techniques are:

- Single point crossover;
- Multi-point crossover; and
- Uniform crossover.

For analysis of this section, two integer strings will be considered, each string has ten decision variables, with five (0-4) options for each decision. The two strings are shown in Figure 4.4:

0	3	1	4	0	2	4	3	4	1

and

2	4	4	1	0	0	3	2	1	0

Figure 4.4 Example strings for crossover explanation

4.2.7.1 Single-Point Crossover

In this case, there is a specified probability of crossover for each of the individuals. If crossover occurs, a position in the strings is chosen randomly. The genetic material to the left of this position is exchanged between the two strings. The process is illustrated in Figure 4.5. The crossover point is located four bits (decisions) into the string. The material is then transferred for so that the offspring have a mix of both parents' genetics.

0	3	1	4	0	2	4	3	4	1
					+				
2	4	4	1	0	0	3	2	1	0

becomes

2	4	4	1	0	2	4	3	4	1
					+				
0	3	1	4	0	0	3	2	1	0

Figure 4.5 Application of single-point crossover

The crossover point is randomly located four bits (decisions) into the string. The material is then transferred so that the two offspring have a mixture of both parents' genes.

4.2.7.2 Multi-Point Crossover

Multi-point crossover (Figure 4.6) is repeated single point crossover. The same principles are followed, except, each string is dissected in a number (n) locations. The positions of the n cuts are randomly chosen. The genes between cut points are swapped.

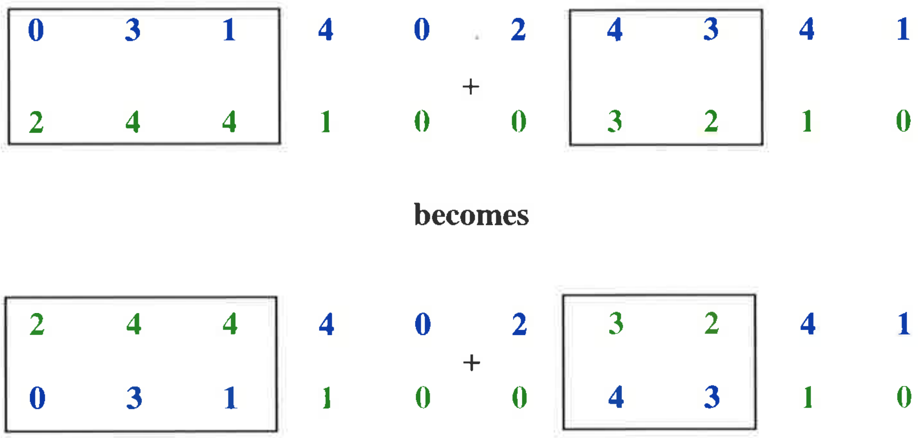

Figure 4.6 Application of multi-point crossover

Figure 4.6 illustrates how multi-point crossover functions. This method is more complex than single-point and a greater mixture of genetic material results.

This scenario results in a much more direct path to the optimal solutions as the search space is examined more thoroughly.

4.2.7.3 Uniform Crossover

Uniform provides potential crossover at each point in a string. A probability of crossover is set and a random number generated for each pair of numbers. If crossover occurs, the genetic material at that position is switched between strings.

Figure 4.7 illustrates how uniform crossover operates. The boxes represent the regions, which are experiencing a transfer of material.

0	3	1	4	0	2	4	3	4	1
2	4	4	1	0	0	3	2	1	0

becomes

0	4	1	4	0	2	3	3	1	0
2	3	4	1	0	0	4	2	4	1

Figure 4.7 Application of uniform crossover

This method allows a rapid examination of the available search space. When combined with a powerful selection process, rapid convergence towards the optimal solution occurs.

4.2.8 Analysis of Selection and Crossover Options

To assist in the determination of which crossover, and selection processes to use, a test optimisation was implemented based on the New York Tunnels Problem. The New York System has 21 pipes with 16 size options for each pipe. This equates to 16^{21} different solutions, or 1.9×10^{25} systems.

There were no constraints placed on this examination, so the minimisation should converge towards a solution with zero new pipes and zero cost. Eight different runs were then made with variations to selection type (proportional or tournament), crossover (single-point or uniform) and population size (200 or 50). The results for a population of 50 are shown in Figure 4.8.

The probability of crossover for these runs was 1.0. The probability of switching for uniform crossover was 0.5 and there was no mutation or elitism in this analysis.

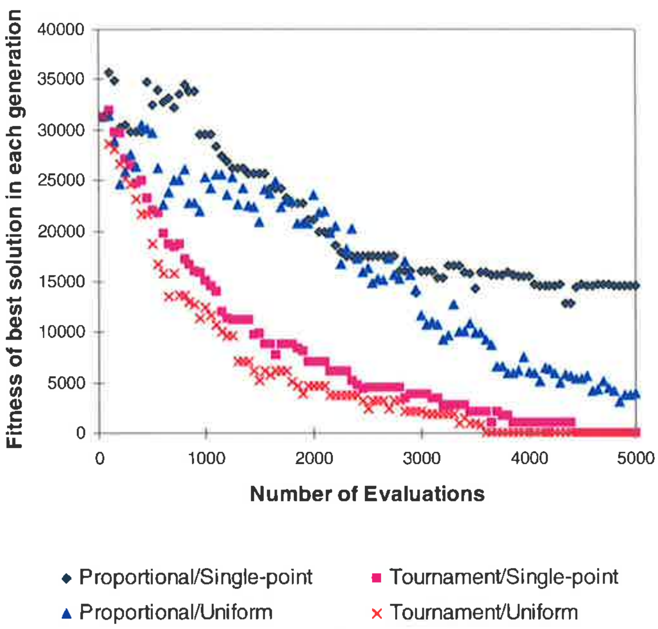

Figure 4.8 Comparison of GA elements for a small population (50)

It is very clear from Figure 4.8 that the use of tournament selection markedly increases convergence. The application of uniform crossover instead of single-point crossover also contributes to a faster result.

The solution series for the proportional selection is not as successful. The combination of selection and single-point crossover converges prematurely, whereas the uniform crossover and proportional selection combination does not converge in 5000 evaluations.

By increasing the size of the population, there are greater options for the selection process, hence, there is a lower probability of the poor solutions being chosen ahead of the good ones. This makes the crossover process more important as it is applied to a more consistent population. Figure 4.9 illustrates how a larger population can affect the selection/crossover process.

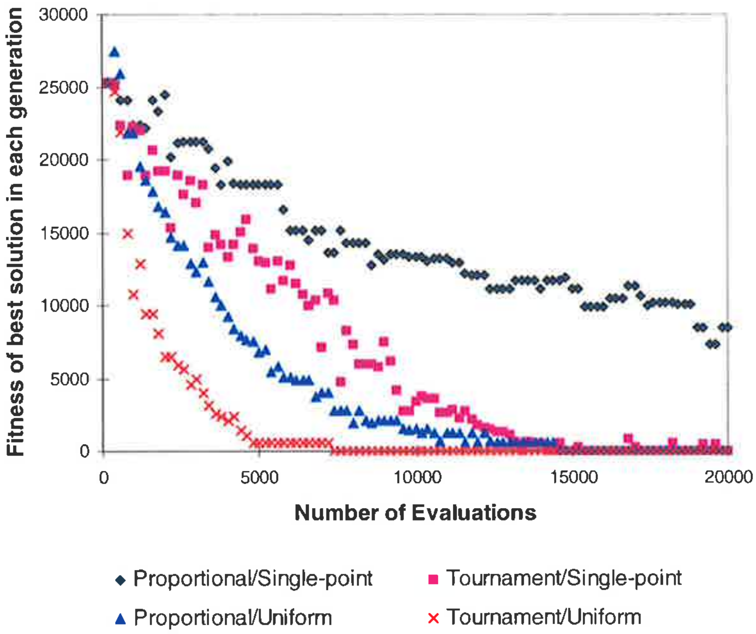

Figure 4.9 Comparison of GA elements for a larger population (200)

In this example, the combination of the tournament selection and uniform crossover is still clearly the most powerful. The second ranked method is the proportional selection and uniform crossover option, which differs from the small population simulation.

So the analysis indicates that uniform crossover and tournament selection are the preferred options for GA simulation in this study.

4.2.8 Mutation

The genetic make-up of offspring is not always fully determined by the genetics of the parents. Indirect genetic effects lead to a more complex form of gene inheritance, where environmental sources of variation can be transferred across generations, contributing to genetic change (Wolf *et al.*, 1998).

This is applied to genetic algorithms by mutation: the occasional random alteration of a gene (decision). Mutation acts as insurance against the loss of genetic material that is potentially useful.

Each gene (decision) is subjected to a probability of mutation. This probability is recommended to be low, as if it becomes too high, convergence will not occur, due to the continuous influx of random material. A mutation probability of 0.01 or less is a commonly used level (Goldberg and Kuo, 1987). If mutation occurs, the mutatiing gene is replaced with a randomly chosen value. Figure 4.10 illustrates the operation of mutation, with the boxed region being the section, which has undergone alteration.

$$0 \quad 3 \quad 1 \quad 4 \quad 0 \quad 2 \quad 4 \quad \boxed{3} \quad 4 \quad 1$$

becomes

$$0 \quad 3 \quad 1 \quad 4 \quad 0 \quad 2 \quad 4 \quad \boxed{0} \quad 4 \quad 1$$

Figure 4.10 Application of mutation to a GA string

If the mutation probability is set too low, there is a real possibility of the GA converging too soon at a non-optimal level. If the mutation probability is too high, convergence will not occur due to the constant influx of fresh material.

4.2.8.1 Creep Mutation

Creep (adjacency) mutation is an altered process of mutation as it concentrates on only changing the decision by one option either way. Suppose a decision has 10 options, and is currently set at option 7, if mutation occurs, the new level will become 6 or 8. This method is excellent if the list of options is an ordered data set (such as ascending pipe diameters). If the data set is less structured (pump sizes), even a shift of 1 up or down can drastically alter the system make-up. Adjacency mutation allows fine adjustment of the solution space.

4.2.9 Convergence

After the strings (individuals) have been exposed to elitism, selection, crossover and mutation, a new generation has formed. This new generation is then assessed for fitness, and the process begins again. This constant cycle continues until the solution set converges.

The position of convergence is very important to the final solution. The aim is to achieve convergence to near-optimal solutions as quickly as possible. There are often local optima, which can affect the pattern of convergence. A point that is a local maximum or minimum is called a local, or relative, extremum (Winston, 1991). These are illustrated in Figure 4.11.

Points A, and C are local optima, while point B is also the global optimum to the function.

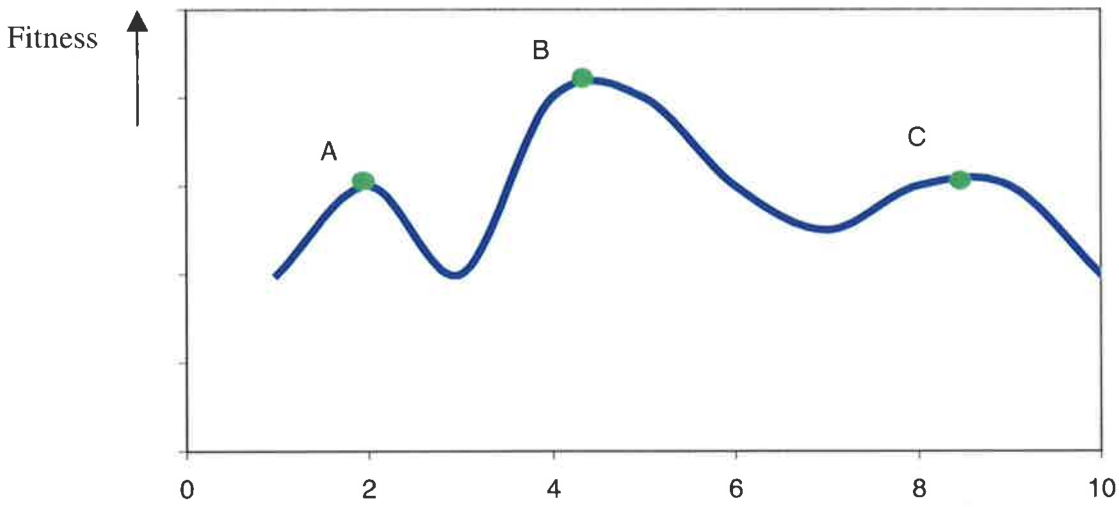

Figure 4.11 Local Optima

4.3 Performance in Applied Problems

Genetic algorithms have been applied to many different types of problems. Some of the different applications include:

- Structural design (Sved *et al.*, 1991);
- Irrigation systems (Hassanli and Dandy, 1996);
- Calibration of water quality models (Mulligan and Brown, 1998);
- Pipe Network Calibration (Savic and Walters, 1995; Vitkovsky and Simpson, 1997)

4.3.1 Application to Pipe Systems

Pipe networks are very well suited to the application of genetic algorithms. Selection of each of the infrastructure components whether they are pipes, pumps, tanks or valves involves a decision process in relation to sizing or whether they will be included in the overall system design.

As there are several options for each component, such as pipe diameter, pump capacity, and tank volume, a solution space exists. The cost function is readily applicable as installation of infrastructure can be priced, and the system has several constraints, which define the problem. Appendix C provides a step by step examination of the GA application to distribution networks.

A need exists for a powerful and robust optimisation package to find efficient cost solutions to pipe systems, as large sums of money are invested in water distribution system projects worldwide. Whether they are applied to the mass installation of urban supply water systems in the developing world, modifications to urban networks, or the establishment of rural irrigation schemes. Genetic algorithms have proven themselves a successful option regardless of scale, from large urban systems (Murphy *et al.*,1996) to small irrigation projects (Hassanli and Dandy, 1996).

4.3.2 Comparison with other Optimisation Methods

The application of genetic algorithms to pipe system optimisation compares very well against other minimisation techniques.

Goldberg (1989) describes the robustness of GA optimisation when compared to traditional methods. Three main optimisation types are examined:

1. Calculus based;
2. Enumerative; and
3. Random.

Calculus methods are limited in that they have a local scope and may miss the global optimum region (Goldberg, 1989). This technique is also dependent on the existence of derivatives. Both these points make the calculus-based methodology unsuitable for a complex system.

The concept of enumerative optimisation involves examining the objective functions at every point in space so that the optimum values can be recorded. Unfortunately, in wider search spaces, the sheer size of the potential option is such that this method becomes inefficient (Dandy *et al.*, 1993).

Random searches also lack efficiency. A pure random walk can expect to achieve levels no better than enumerative searching in the long run, as they are both grabbing a solution and recording the objective function values search. Genetic algorithms use random choice assist in directing the search process.

Genetic algorithms also compare well against linear programming as evident in an analysis of the Loubser and Gessler Network (1990).

Dandy *et al.*, 1993 carried out a comparison of different optimisation methods for a water distribution network described by Loubser and Gessler (1990). This consisted of two reservoirs, 10 supply nodes, 13 existing pipes, 2 possible new pipes, and 8 potential duplicate pipes. The layout of the system is shown in Figure 4.12.

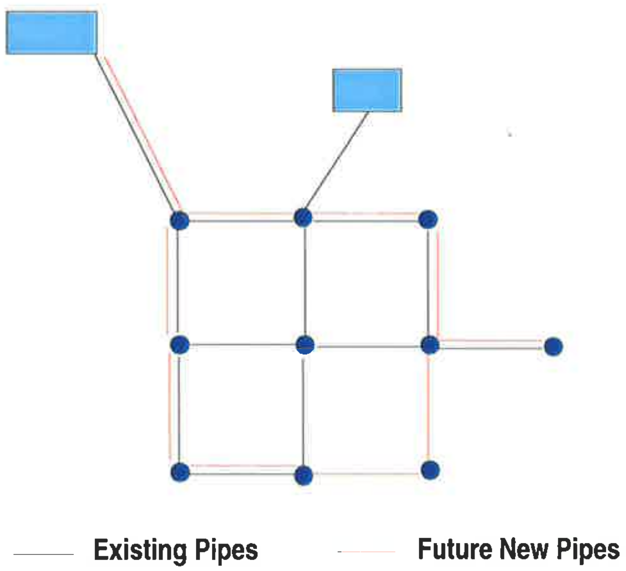

Figure 4.12 The Loubser and Gessler problem

The system shown in Figure 4.12 was optimised using the following methods:

- Partial enumeration;
- Non-linear programming – continuous;

- Non-linear programming – rounded;
- Linear programming;
- Linear programming – rounded; and
- Genetic algorithms.

Partial enumeration, non-linear programming, linear programming were discussed in Chapter 2.

The Loubser and Gessler problem is a South African study, so all of the costing is in Rand (R1=AUS$0.5). The system constraints were that the network must meet demand with a minimum of 20m head a each supply node. The comparison of the different techniques and their respective costs are shown in Table 4.1.

Table 4.1 Solutions of different methods for the Loubser and Gessler problem

Optimisation Technique	Cost (R million)	Min. Press. Head (m)
Partial Enumeration	2.6597	20.4
Non-linear Programming (Continuous)	2.5253	20.00
Non-linear Programming (Rounded) #1	2.4884	20.15
Non-linear Programming (Rounded) #2	2.4116	19.94
Linear Programming	3.1156	20.00
Linear Programming (Rounded)	3.1097	20.26
Genetic Algorithm #1	2.3951	20.11
Genetic Algorithm #2	2.3547	19.96

The advantage that the GA method had over the others is that no rounding of the final solution or cost function approximation is required. Although, being a stochastic optimisation method, it cannot guarantee the mathematically optimum solution to the problem is obtained, it usually obtains as good or better solutions than any other technique (Dandy *et al.*, 1993).

4.4 Expanded Analysis of Genetic Algorithms

Improvement of GA effectiveness is a constant aim of many researchers. Different modifications to the basic format allow for advancement in rate of convergence and result quality. Some of different angles that have been recently applied to genetic algorithms include:

- Fast messy genetic algorithms (Wu and Simpson, 1997); and

- Multi-objective analysis (Horn and Nafpliotis, 1993; Engelhardt, 1999).

Engelhhardt (1999) demonstrated the application of genetic algorithms to the optimal rehabilitation of water distribution systems. This was achieved in two steps, firstly for a single objective economic formulation and then an economic reliability multi-objective formulation.

4.5 Summary

Genetic algorithms have proven themselves to be a robust method of problem optimisation. Furthermore, they have been adapted to the determination of cost efficient water distribution systems with considerable success.

The application of GAs to the New York Tunnels problem produced the current optimal solution for that system. As part of this study involves analysis of the New York system, application of the current best-performing optimisation method will hopefully provide cost efficient solutions for the application of the water quality aspects of system design in conjunction with its proven record in solving for network hydraulics.

Chapter 5 Program Formulation

The formulation phase of a model is an essential guide to the thought process behind a series of results and conclusions.

Identification of the intended purpose of the model is a critical step in the pursuit of a good analysis (Ormsbee and Lingireddy, 1997). This study's focus is to provide a package that optimises water system design for both water quality and hydraulic concerns.

5.1 Concept of a Framework

In an era of economic rationalism, optimisation of major works is essential for project viability. Water utilities are looking towards achieving efficiency of treatment and supply as a possible means of cost reduction. To achieve this improvement, optimisation of chemical dosing levels and pipe sizes to meet water quality and hydraulic requirements in a distribution network is desirable.

To quote Lewis Rossman (1998) in relation to water system analysis:

'*A new framework for performing integrated assessment of the multiple dimensions of risks associated with water disinfection seems called for.*'

5.2 Cost Structure of System Infrastructure

Probably the simplest component of the framework to cost is the network infrastructure, as there are well-established principles to draw upon. There are a number that have carefully laid out cost structures. These can be directly followed in the analysis. The New York Tunnels System, which is one of the case studies discussed in chapters 6 and 7 of in this thesis, follows along this vein with all technical aspects applied.

When considering a more localised study, as with the major case study of this thesis - Yorke Peninsula (see Chapters 8 and 9), it is important to gather relevant data to determine the infrastructure costs. These costs have been based on a combination of local knowledge (SA Water, 1994), cost precedents from a national perspective of the construction industry (Rawlinsons, 1995), and data obtained from international reports (Gumerman *et al.*, 1992).

Appendix D contains the complete discussion on how the infrastructure cost set was determined. The following sub-sections summarise this Appendix.

5.2.1 Issues of Cost

When comparing costs from different studies that stem from different years and countries in order to achieve a fair comparison, the costs must be corrected for the effects of inflation and exchange rates.

The life of the system components is also important as this provides the term over which the costing is viable. There is little point doing a 100-year study unless allowance has been made to replace the majority of the system infrastructure during this time.

5.2.1.1 Rates of Inflation and Exchange

When documenting the costs for this study, the following assumptions were made about costs since 1990:

- The rate of exchange from Australian Dollars to US dollars was assumed to be 0.72, which is approximated for the period of 1990 to 1999. Figure 5.1 (ABS, 1999) illustrates that this is a reasonable value for this period.
- An average annual rate of inflation for Australia since 1990 of 2.5%. The Consumer Price Index (CPI) according to one report (egoli website, 1999) has grown on an average rate of just 1.9% per year since 1990. Another site indicates levels since 1993 ranging between 0.3% and 4.5% (ABS c\- ABC website, 1998) thus a 2.5% level is a reasonable estimate.

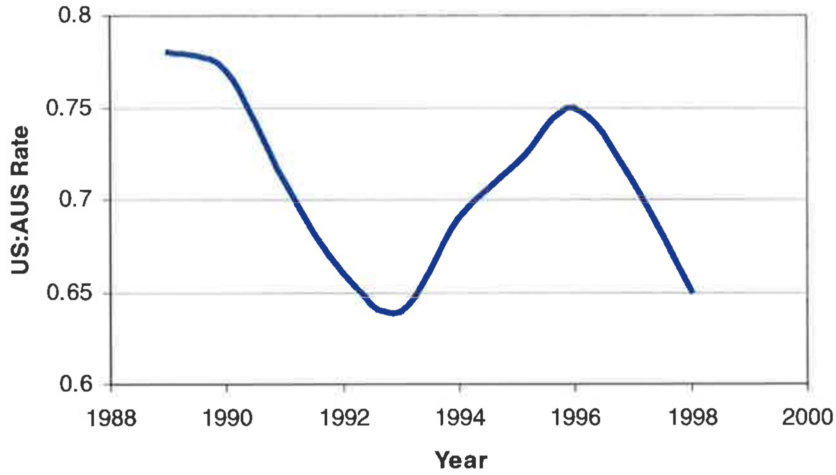

Figure 5.1 Exchange Rate of $AUS to $US

5.2.1.2 Life of Components

When considering adapting an existing network, the issue of replacement becomes an important cost factor. Nelson (1995) used a life range of 50 years for most components in his study, but for pumps and pump accessories, 35 years was the estimated life.

SA Water (EWS, 1986 (a)) has their own structure as to the life of various network components. The data from this study has a range of different infrastructure lives. Table 5.1 lists some of these values.

Table 5.1 Economic Life of System Components

Water Asset Type	Approx. Economic Life (Years)
Metropolitan Water Mains	80
Country Water Mains	80
Water Filtration Plants	50
Treatment Stations	50
Bores and Wells	50
Pumping Station - Building	50
Pumping Station - Mechanics	25
Tanks and Storage	80
Dams and Reservoirs	100
Services	30
Connections	80

For the studies examined, a 30-year project life was assumed, as this allows the analysis to be based over a generation. For a 30-year study, using Table 5.1, only the pump mechanics may need replacing, though in reality, they may not be replaced for a considerably longer period.

5.2.2 Pipes

When calculating the total cost of pipelines, the following aspects of the construction require consideration:

- Pipe material cost;
- Cost of trenching and backfilling;
- Cost for traffic and safety issues; and
- Maintenance costs for network.

All of these costs have been considered for the Yorke Peninsula study and the discussion surrounding these costs can be seen in Appendix D. The cost determined using the US Commerce Report (Gumerman *et* al., 1992) was then compared to the local (SA Water, 1994) and national data (Rawlinsons, 1995) with good results as shown in Figure 5.2.

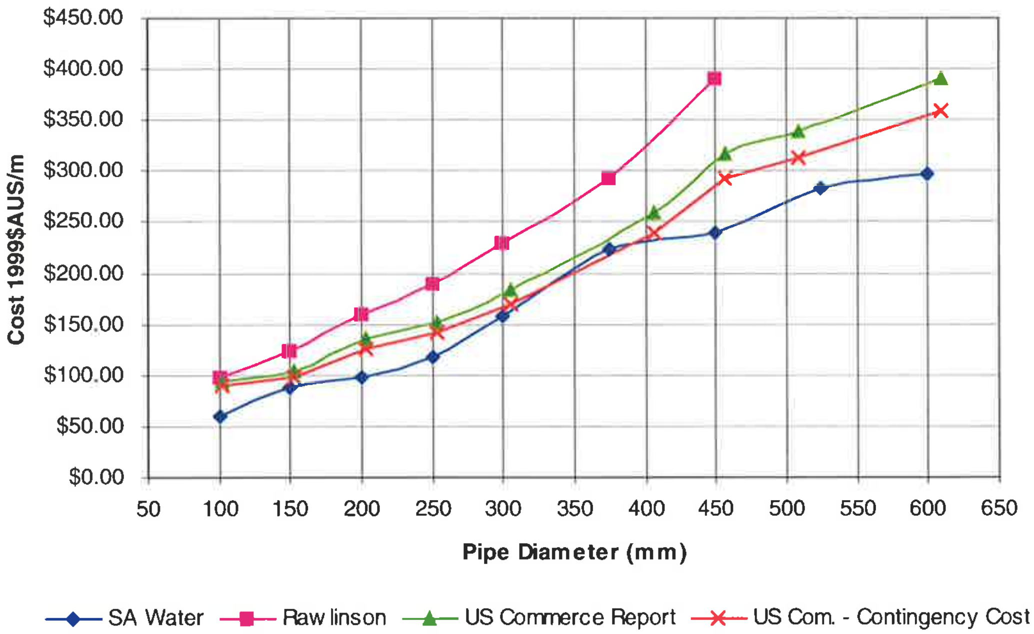

Figure 5.2 Comparison of different cost regimes for pipe costs

The cost estimates for the US Department of Commerce Report fits quite comfortably between the two Australian studies (SA Water, 1994) and (Rawlinsons, 1995). The costs used in this study were those from the US Department of Commerce Report (Gumerman *et* al., 1992).

The formulation of the pipe infrastructure costs in the program is shown in Equation 5.1.

$$C_p = \sum_{i}^{np} C_i l_i \tag{5.1}$$

Where C_p = total pipe cost ($), C_i = cost per unit length of a pipe ($/m), l_i = the length of new pipe i (m) and np = total number of new pipes.

5.2.3 Pumps

Through meetings with personnel from SA Water (Wozniak pers. com., 1998), an approximate relationship between pump power and cost was obtained. This estimate is SA Water's basis for costing pumping stations, with building works, pump cost and all accessories factored in. It has an upper and lower limit to assist in catering for the potential cost situations that could arise. These figures were compared to data obtained from a report from the U.S. Department of Commerce. Figure 5.3 details the different cost regimes.

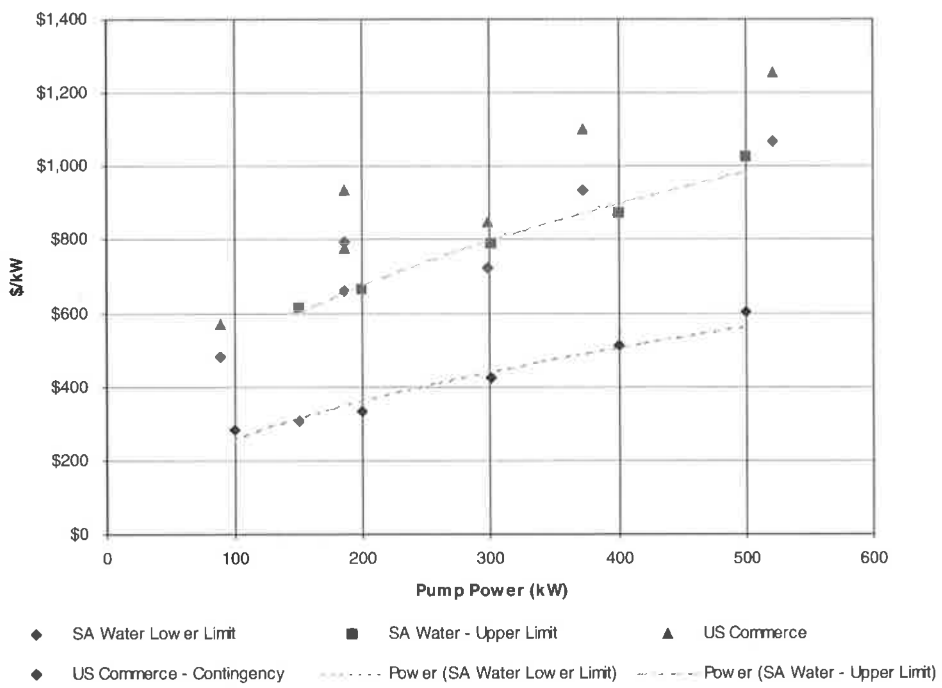

Figure 5.3 Cost regime comparison for pumping stations

When data (which was in a similar price range) from the US Department of Commerce was included as a series of data points on the graph, it was noticed that although in a similar ballpark as the SA Water data, it was consistently above it in cost. The data was then replotted with the exclusion of the contingency costs, and the data points were found to follow a line very similar to the SA Water Upper Limit.

The data used in the study was the US Department of Commerce results. The contingency costs were retained, as the assumed 30-year life of the study is slightly longer than the 25-year life (Table 5.1) for pumping machinery used by SA Water. As it is assumed that the pumps are not to be replaced throughout the study-life, the contingency costs allow for some ongoing maintenance.

The total cost attributed to the construction and maintenance of pumping facilities throughout the system follows the formulation given in Equation 5.2.

$$C_{PM} = \sum_{i}^{nPM} Cp_i W_i \tag{5.2}$$

Where C_{PM} = total pump cost ($), Cp_i = cost per kW for new pump i ($/kW), W_i = power of pump i (kW), and nPM = number of new pumps.

The annual operation and maintenance costs have been applied over a lifetime of 30 years for the project and are given in Appendix D.

5.2.4 Tank Formulation

When comparing the two cost structures over a similar range of tank capacities, it is clearly seen in Figure 5.4 that they correspond reasonably well. The SA Water costs are placed at a consistently higher level, which may be due to the contingency cost issue discussed in the pipe analysis section.

When the contingency cost is removed from the US Commerce Department Report data set, the match with the SA Water data is very close as shown in Figure 5.4.

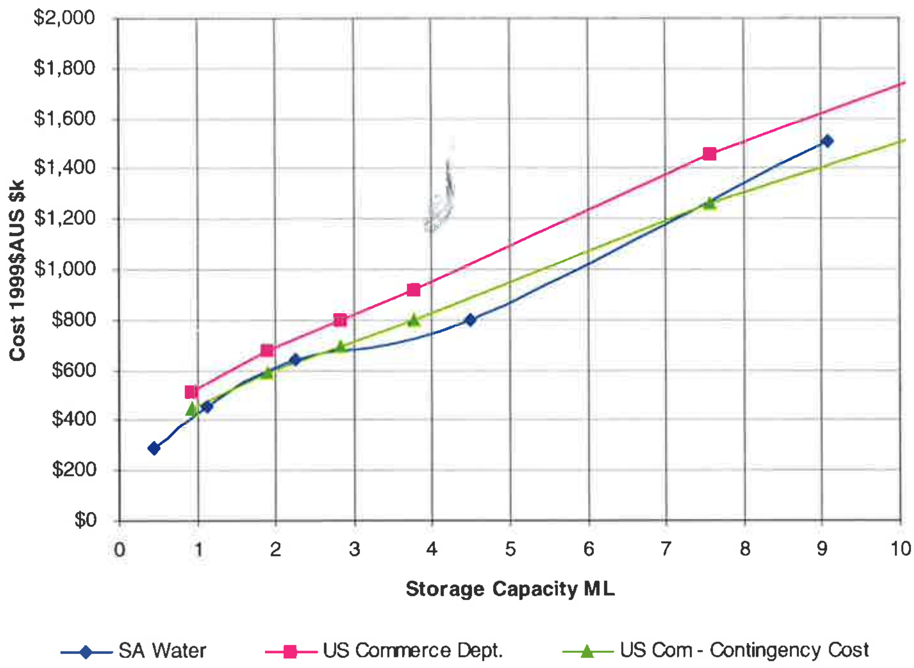

Figure 5.4 Comparison of the cost regimes for concrete tank construction

The acceptability of either cost structure is not a problem, but to remain consistent with the pipe analysis costing, the US Department of Commerce report is the preferred cost structure for this study. So, the cost formulation for tank construction is shown in Equation 5.3.

$$C_T = \sum_{i}^{nT} C_{Ti} \tag{5.3}$$

Where C_T = total tank cost, C_{Ti} = cost of new tank i, nT = number of new tanks.

5.3 Formulation of Water Quality Issues

At a time where there is an increasing focus on quality issues, an examination of one of drinking water quality is exceedingly important in a full study of water supply. Although, considerable research steps are required to obtain accurate relationships for the desired cost functions, the current level of study is sufficient for a model to be established which incorporates a reasonable assumption to complete the analysis.

The formulation of the water quality aspects of the model are less definitive when compared to the infrastructure costs, as they rely heavily on the risk assessment, health costs, and judgement, as previously discussed in Chapter 3. The strength of each aspect depends largely on the amount of previous research carried out in the field. The assumptions and decisions made from the discussion in Chapter 3 are presented in this section.

The four main areas targeted in the quality analysis are:

1. Potential costs of waterborne disease;
2. Risk of illness or death due to disinfection by-products;
3. Consumer concerns with taste and odour; and
4. Establishment costs of an acceptable disinfection facility.

All of these costs are interlinked, with the disinfection station costs determining the initial residual levels entering the system. From this, the consumption of the disinfectant can lead to health concerns from waterborne disease if the disinfectant residual is too low or, illness due to disinfection by-products if these levels are too high.

The other factor, which affects the potential dosing regime, is the consumer reaction to the tastes and odours of the water supply. All of these four interact with each other to provide a cost function for the quality of the system.

The alternative to determining a consumer cost is to apply a penalty structure which restricts the residual chlorine levels (hence water quality) into a certain range. This section also examines this option.

5.3.1 Waterborne Disease Formulation

The formulation of a cost function for waterborne disease is exceedingly difficult, as the level of research that identifies adequate relationships is limited. The steps taken to create an equation for the modelling of this quality aspect are:

- Assumption of an acceptable chlorine residual level which is considered to provide minimal illness concern;
- Estimation of the risk of illness for the acceptable residual level;

- Estimation of a risk of illness under zero residual conditions;
- Determination of discrepancies between residual levels at various nodes in the network and the acceptable residual level;
- Allocation of cost estimates for morbidity or mortality resulting from potential illnesses to consumers; and
- Application of the consumer cost over the entire system population.

The concept of acceptable risk is (as discussed in Chapter 3) comes into the formulation as it is deemed that the risks of waterborne disease can stem from means other than drinking water, including food consumption, water-sports, and washing. Due to this uncertainty, an acceptable annual risk of 1 in 10,000 (0.0001) was formed and is used for risk acceptance in several areas of water research (Gerba *et al.*, 1996; Haas, 1996). It is this level that has been assigned to all residual levels down to the minimum allowable residual level.

The minimum allowable level depends on the system under consideration. For a chlorinated system, this level may be 0.3mg/L to 0.5mg/L (Fang, 1997), whereas a chloraminated system may be 0.5mg/L to 1.5mg/L (Wozniak, 1998). Any level deemed acceptable by the controllers of the system in can be used in this formulation.

In this research, the acceptable residual levels used were:

- 0.3mg/L and 0.5mg/L for a chlorinated urban system; and
- 0.5mg/L for a chloraminated rural system.

The difference in these figures for each system is to allow a comparison of the potential costs of each level of disinfection. For the chlorinated study, 0.3mg/L was at the low end of the observed residuals. A more desirable consistent level is 0.5mg/L, as the water is distributed from each supply node in the system to serve a wider distribution. Due to the residence time in supplying this extension, a higher residual at the system nodes could be required.

Currently, the chloraminated rural system on Yorke Peninsula aims to maintain minimum residual levels of 1.0-1.5mg/L (Wozniak, 1998). The level that this study aims to maintain is

0.5mg/L, the preferred aesthetic level (SA Water, 1995), which would be acceptable in a carefully managed dosing regime.

The associated risk for water containing no residual is the most uncertain area of the formulation. In reality, this level is based on:

- The length of time the unit of water has had no residual level;
- The exposure of the system to potential microbial, protozoic, or viral activity;
- The number of different strains of potential illness in the supply; and
- Any immunity in the consumer population.

Although there has been conjecture about the value of the coliform index in determining waterborne disease risks and prediction of outbreaks (Gleeson and Gray, 1997), it is still widely regarded the most reliable indicator for drinking water, which is supported by its inclusion in most water quality standards (WHO, 1993).

If the system was without a reasonable disinfectant residual for an extended period, the faecal coliform levels experience regrowth to 10^4-10^5 /100ml as seen in Figure 5.5 (Shuval, Cohen and Kolodney, 1973).

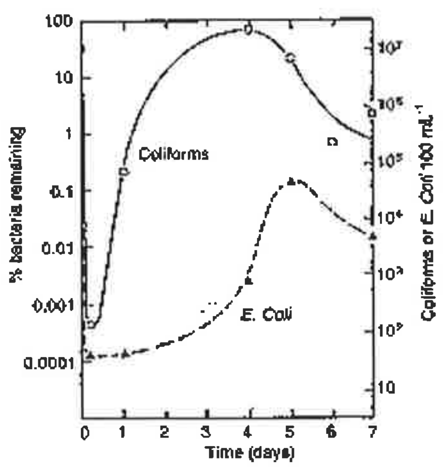

Figure 5.5 Regrowth in water distribution system of coliforms

In this study, regrowth of coliforms after 5mg/L of chlorine exceeded 99.999% coliform removal was analysed. Using a relationship between *E.coli* and risk of illness adapted from

Olson and Nagy (1984), Figure 5.6 shows potential risks of viral illness of an exposure to levels of *E.coli*. Although *E.coli* will not cause a viral infection, the presence of the faecal coliforms indicate that viruses may also be present in the supply.

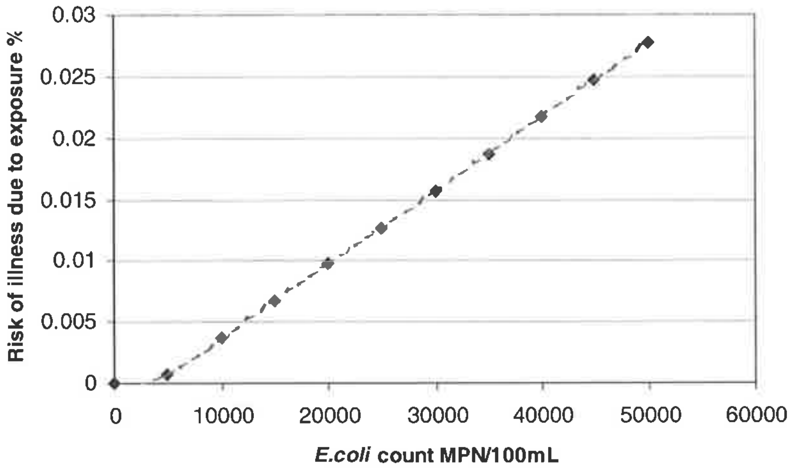

Figure 5.6 Relationship between faecal coliforms and waterborne disease risk

For the range of 10^4-10^5, taking an MPN (most probable number) of 5×10^4, the risk of an exposure is around 0.025%. If the water were continually of this standard then over the season in question, the number of exposures to this would be anywhere from 200-600 depending on how often water is consumed. Assuming that in a season (nearly 100 days), and that four exposures per day occurs, the risk of illness becomes approximately 10% (0.1). Viral infection is one of the risks of illness, protozoa and bacteria are also major contributors to waterborne disease infection. Systems with shorter residence times would have reduced risks, as the regrowth would not be as extensive.

Using the above approach, with inclusion of the discussion in Chapter 3 on this area, the following values were assumed in this study:

- An annual risk level of 0.0001 for water that is considered adequate, as a one in ten-thousand risk is deemed to be an acceptable risk for contact with the water supply (Haas *et al.*, 1996).
- An annual risk level of 0.01 at zero chlorine residual for a urban system due to the low residence times and assuming a higher treatment level of the water entering the system;

- An annual risk level of 0.1 at zero chlorine residual for a rural network with high residence times and assuming no treatment prior to dosing.

The rural system in this research involves water entering the system, which is supplied directly from an open earth reservoir, and has a residence time of up to a month. The urban system has a residence time of one to two days and a much more stringent pre-treatment program explaining the assumption of a significantly lower risk when there is no residual in the system as the incoming water quality is of a higher standard.

The relationship is assumed to be linear for modelling simplicity although extended future research may allow the development of a different curve. The approximate relationship is highlighted in Figure 5.7.

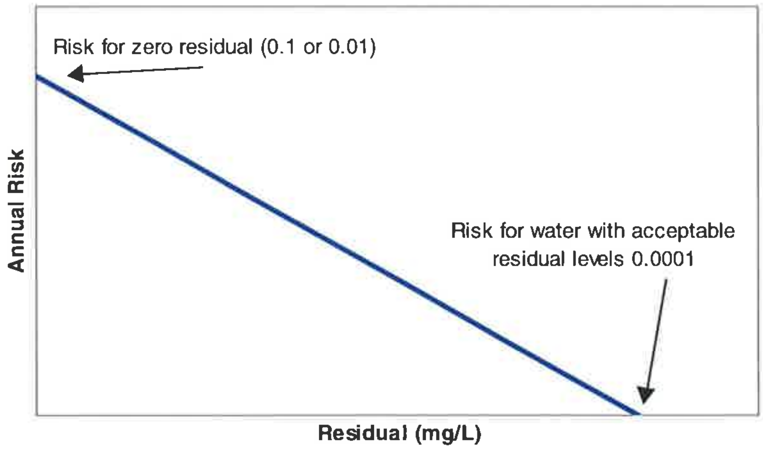

Figure 5.7 Assumed relationship for waterborne disease formulation

Following the calculation of the level of risk the illness potential is costed. Typical costs for waterborne disease illness range from \$272 for mild illness, \$2,680 for moderate illness through to \$2,000,000 if mortality occurs (Levin and Harrington, 1995).

This consumer cost is then applied to the entire system population giving a final relationship as given in Equation 5.4.

$$C_{WD(annual)} = \sum_{i=1}^{tnn}\left(P_i \times \left(rk_{low} + (rk_{no} - rk_{low}) \times \max.\left[\frac{(Cn_{low} - Cn_i)}{Cn_{low}}, 0\right]\right) \times (C_{ill} + C_{mort} \times P_{mort})\right) \qquad \textbf{(5.4)}$$

Where $C_{WD(annual)}$ = expected annual cost of waterborne disease for the population P_i, at *node i*, Cn_i = the chlorine residual level at node i, Cn_{low} = minimum acceptable chlorine residual level, rk_{low} is the risk of illness associated with adequately chlorinated water, rk_{no} = risk associated with water with no chlorine residual, C_{ill} = cost of contracting an illness, C_{mort} = cost of mortality, P_{mort} = probability of mortality upon contraction of waterborne disease, and *tnn* = total number of nodes.

As can be seen from Equation 5.4, the population is an essential factor in assessing the cost of waterborne disease as fewer consumers means a lower priority of water quality based on the social cost formulation.

5.3.2 Formulation for Disinfection By-Products

When formulating the model to consider the potential health concerns due to disinfection by-products, notably THMs, the following process can be used:

- Determination of the disinfectant consumption up to the location in question;
- Estimation of the rate of THM formation for this level of disinfectant consumption;
- Adjustment to allow for water temperature considerations in the THM formation;
- Allocation of the potential risk of the THM level for a consumer at the location;
- Estimation of cost using a contraction cost of the potential illness; and
- Distribution of this consumer cost over the population supplied at this location.

The consumption of the disinfectant was calculated by taking the difference between the nodal residual level and the disinfectant measure at the previous dosing point. Although this may have been affected to some degree by pump and tank operation, the main flow pattern will considerably outweigh these effects.

The determination of the THM formation rate is taken from known levels for the current or similar systems, and is taken to be a direct function of the disinfectant consumption. The variation for different temperature levels is also included, as it can add to or reduce the formed THM level.

The risk of contracting an illness is applied to the formulation. This risk is then costed using data examining costs of cancer contraction due to DBPs. Values of \$230,000-\$2,000,000 (1995\$US) for cancer contraction were determined by Levin and Harrington, 1995.

Again, a linear approach to the formulation is used. There is some conjecture among researchers as to risk assessment for low level accumulation of pathogens. The argument is that a threshold may exist in humans for some chemicals whereby no detrimental effects occur below this level (Hrudey, 1999). This response along with the linear approach used for the modelling is illustrated in Figure 5.8.

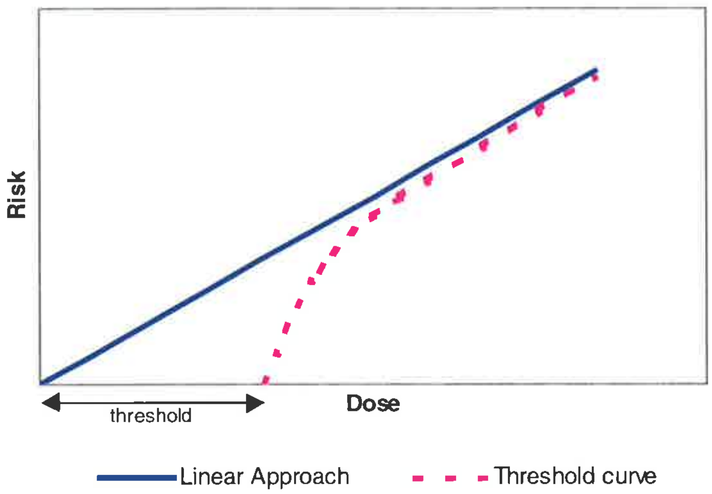

Figure 5.8 Different concepts of low dose risk for carcinogens

As much of the conjecture is questioning the ability to draw a definitive curve through the low dose risk region, a linear approach was used as it is a general approximation. Future research into the area of low dose risk may allow a more accurate curve to be developed, however, this situation is probably some time away.

From the above formulation, the function developed to consider the potential costs, associated with disinfection by-products (DBPs) is given by Equation 5.5.

$$C_{DBP(annual)} = \sum_{i=1}^{nn} (P_i \times (Cn_{dose} - Cn_i) \times (THM_r + T_w \times THM_T) \times rk_{canc} \times C_{canc}) \qquad \textbf{(5.5)}$$

Where $C_{DBP(annual)}$ = annual cost of disinfection by-products for population P_i at node i Cn_{dose} = disinfectant dosing level, THM_r = rate of THM formation per unit of disinfectant consumed, T_w = water temperature, THM_T = THM formation due to temperature effects, rk_{canc} = risk of cancer contraction, and C_{canc} = cost of cancer contraction.

As mentioned in Chapter 3, there is some speculation as to the carcinogenic qualities in humans for THMs. This study has assumed a carcinogenic link, with cost ramifications from cancer contraction, allowing a methodology providing a value function to be used.

5.3.3 Formulation for Taste and Odour

Taste and odour concerns in water distribution are not an issue until consumer complaints occur. This aspect of water quality is based on how the receiving population accepts the taste and smell of the supply.

There are many methods that can be used to quantify for taste and odour problems in supply. Cost determination can take the form of:

- Cost of extended treatment to purify the supply;
- Application of cost of alternative sources (rainwater tank, bottled water); and
- Application of penalty cost to reduce the associated problems.

The third of these approaches (assigning a penalty cost) would work in a similar way to the pressure penalty. As taste and odour are more of a consumer issue, with customer complaint the basis for recognition of a concern, it would be preferable to produce a cost function with a population focus.

The other two methods succeed in doing this. However, they do represent different options for different systems.

5.3.3.1 Approach for large metropolitan cities (New York)

The approaches taken for modelling purposes use these methods. For urban areas in large cities such as New York, London, and Beijing, the option of rainwater tanks may not be applicable although some major cities including Tokyo (Udagawa, 1994) and Singapore (Fong and Nazarudeen, 1996) collect rainwater for secondary use. In the case of attaining acceptable drinking water for large cities, the cost of extended treatment practice to produce higher quality water can be used.

In a recent study (Clark *et al.*, 1998) the costs of additional treatment practice were examined as shown in Figure 5.9 which has been adapted from this research.

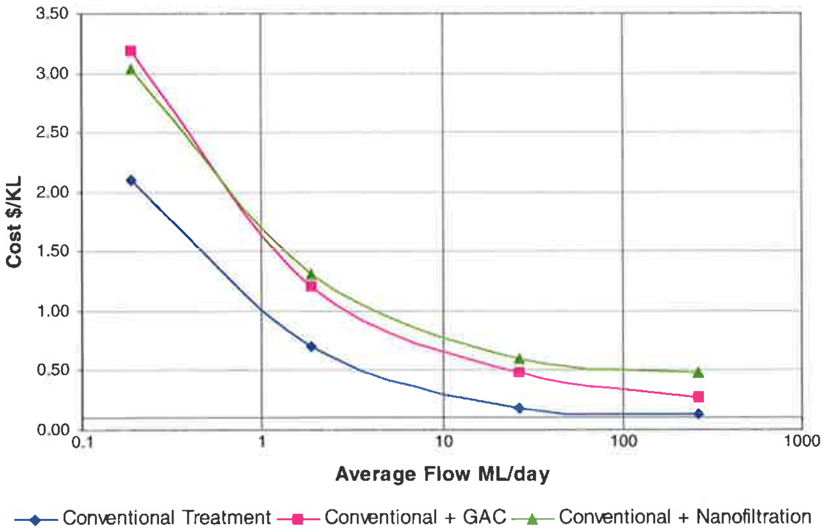

Figure 5.9 Cost of different treatment methods for varying system demands

The New York System had a considerably greater design capacity of 1200MG/day (5000ML/day) than was allowed for in the study by Clark *et al.* (1998). Therefore, it was necessary to adapt this curve to cater for the larger consumption. By fitting curves to the different treatment methods, the relationships and results shown were determined in Table 5.2.

Table 5.2 Extrapolation of data for the New York Tunnels system

Treatment	Cost Equation (Q=ML/day)	R^2 Value	Cost/KL ($)
Conventional	$0.9479Q^{-0.4019}$	0.9667	0.03
Conventional + GAC	$1.6348Q^{-0.3385}$	0.9863	0.10
Conventional + Nanofiltration	$1.7194Q^{-0.2609}$	0.9538	0.20

Before these results can be applied to the New York study, the costs had to be converted back to US$1969. Applying a price index of 4.4 (extrapolated from Dandy and Warner, 1989) over the past thirty years provides the conversion. The breakdown of the costs is shown in Table 5.3.

Table 5.3 Determination of potential costs for New York System

Cost Conversion from 1999$/kL to 1969$/study-life	Conventional Treatment	Conventional + GAC	Conventional + nanofiltration
cost/kilo-litre	0.03	0.10	0.20
cost/kilo-gallon	0.13	0.38	0.76
cost/day (1998$USm)	0.16	0.46	0.91
cost/year (1998$USm)	57	168	332
cost/year (1969$USm)	13	38	75
cost 30-year life (1969$USm)	224	660	1305

From Table 5.3, the annual cost is between $13 - $75 million, depending on treatment. To attain a level which has no taste or odour concerns, $38 million for GAC treatment is the required financial levy. This cost then has to be applied on a population level. Adding GAC or nanofiltration to an existing conventional treatment process may cost less than constructing a plant that does both. However, the assumed costs are for a complete construction as adding a new section in could produce other cost factors such as land restriction, planning and demolition costs.

To apply this analysis to a consumer level, the cost for the total system is broken up over the population at each node. If the additional treatment processes were included, all of the supply would be treated. The formulation for this section determines what population would be willing to pay for the additional treatment, which examines the issue at a spatial and consumer-based level. The formula used to apply this method is shown by Equation 5.6.

$$C_{TO} = \sum_{i=1}^{nn} \frac{(Cn_i - Cn_{all})}{(Cn_{\max} - Cn_{all})} \times \frac{P_i}{\sum_{i=1}^{nn} P_i} \times C_{AT} \qquad \textbf{(5.6)}$$

Where C_{TO} = cost attributable to taste and odour ($m/year), C_{AT} = annual treatment cost, Cn_{all} = acceptable residual, Cn_{max} = maximum residual level where 100% consumer dissatisfaction occurs, P_i = population at node i and tnn = total number of nodes.

Equation 5.6 forms a linear approximation of the willingness to pay for the additional water treatment required to attain an acceptable taste and odour standard. It states that at a residual level, 30% of the way between the allowable residual level and the maximum level (assuming no decay possible), 30% of the population at that location in the system will be willing to pay for the system improvements, whether it be GAC addition or nanofiltration. When this percentile is applied to the expected annual costs of further treatment, a cost function for taste and odour analysis in large urban systems is attained.

5.3.3.2 Approach for smaller regional systems (Yorke Peninsula)

The approach of increasing the level of treatment may not be the preferred option in rural networks with low populations and large residence times. The cost of the treatment and maintenance may be prohibitive. In order to allow options for a better quality supply, other methods of analysis need to be considered.

Bottled water is an alternative to the distribution supply (Harremoes, 1999). As a solution to low population rural systems, it would not be a viable option as the price of supplementing bottled water for 2L a day for a 30-year life would be very extensive. If an entire rural population of 25,000 used bottled water at a cost of $1/person, an annual cost of approximately $9million would result which would correspond to approximately $130million in current finance for the system.

The option, which has been selected for the modelling phase of Yorke Peninsula (discussed in Chapters 8 and 9), is the implementation of rainwater tanks as a replacement option for the system. The methodology is that the system is currently implementing rainwater tanks in some locations (Van der Wel and McIntosh, 1998) due to preferable water quality and to supplement supply during the systems peak flow conditions.

It is acceptable to assume that every house will be affected by this method of willingness to pay, as even those properties currently with rainwater as a supply option will need to replace their tank before the end of the 30-year study life. The life of a South Australian rural rainwater tank system is around 20 years (Van der Wel and McIntosh, 1998). Using this assumption, the cost structure is applicable to all of the population, not just those currently without rainwater tanks connected. The approximate cost of the installation and maintenance becomes the critical factor in the cost of this method. Table 5.4 lists the findings of a recent study (Van der Wel and McIntosh, 1998).

From this study, the total cost of installation for a fully installed tank (irrespective of size) is assumed to be approximately \$3,000 (1999\$AUS) as a new tank is required after 20 years, and 10 years of maintenance is required.

Table 5.4 Distribution of costs for rainwater tank installation

Item	\$ for 9.1kL Capacity	\$ for 18.2kL Capacity
Supply and erect tank	686	945
Tank stand (optional)	360	500
Connect guttering	60	60
Supply and install pump	300	300
In-house plumbing	700	700
Gutter screens	100	100
Total cost for 20 year life	1846 (2206 with tank stand)	2105 (2605 with tank stand)

Although some people with existing connections would experience some reduction in cost as they do not require connection costs, they have been assumed to have the same cost structure. This is applied to the population of the system using Equation 5.7.

$$C_{TO} = \sum_{i=1}^{tnn} \frac{(Cn_i - Cn_{all})}{(Cn_{\max} - Cn_{all})} \times P_i \times \frac{C_{rwt}}{N_d} \tag{5.7}$$

Where C_{TO} = taste and odour cost, C_{rwt} = rainwater tank cost, Cn_{all} = acceptable residual, Cn_{max} = maximum residual level where 100% consumer dissatisfaction occurs, N_d = average number of people per dwelling for that location, P_i = population at node i and tnn = total number of nodes.

The Van der Wel and McIntosh study provided information for Ardrossan (town on the Yorke Peninsula) that illustrated the population growth and the number of dwellings that exist there.

This part of the study is represented in Figure 5.10. This indicates an average number of people per dwelling of around 2.0 in 1990. In the farming towns this number is higher.

Based on approximately 2.5 people to a dwelling (which also accounts for tourist use) for a rural population of 25,000, the maximum cost attributable to taste and odour is for 15,000 tanks at \$3,000 equating to \$30million, which is less than a quarter of the cost of bottled water option.

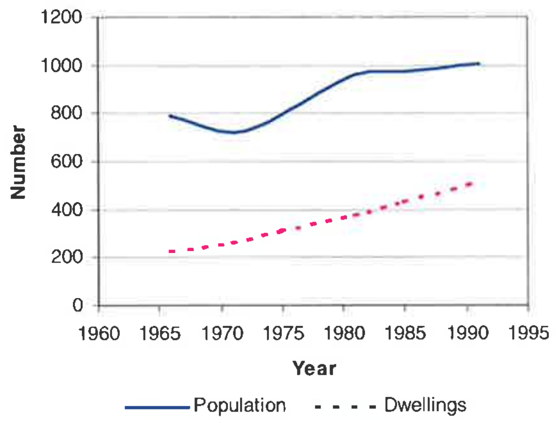

Figure 5.10 Population and dwelling statistics

To make this cost closer to the actual system, demand nodes with an estimated population of less than 50 (farming properties) are assumed to have rainwater tanks regardless of aesthetic concerns. Because of this, only locations with an estimated population of greater than 50 are included in the taste and odour structure. This would not effect the water quality profile throughout the system, as nearly all of the country lands (farming properties) are scattered between the towns with the larger populations, so if each town is of acceptable standard, the supply points between the towns will be as well. As with the treatment option for larger urban supply, this method relies greatly on the willingness for the consumer to get a rainwater tank installed for quality and supply benefits.

5.3.3.3 Approximations in Willingness to Pay Analysis

It is recognised that the willingness to pay relationship is unlikely to be linear and more likely to take the shape of the curve shown in Figure 5.11. The expected curve is merely a representation of what is a more likely scenario, and the discussion generated from this figure is really demonstrating how a population may react to a degradation in quality of supply.

In the expected curve, which is a representation of a cumulative normal distribution of peoples sensitivity to taste and odour, the initial reaction to the water quality worsening (0%-20%) tends to illustrate a lack of consumer response. Few people are really upset enough to be willing to explore options of quality improvement.

Then as the taste and odour problems become greater (20%-40%), the population recognises the degradation and begins to show movement towards system development. Still, the majority of the population chooses to tolerate the water quality and accept the supply.

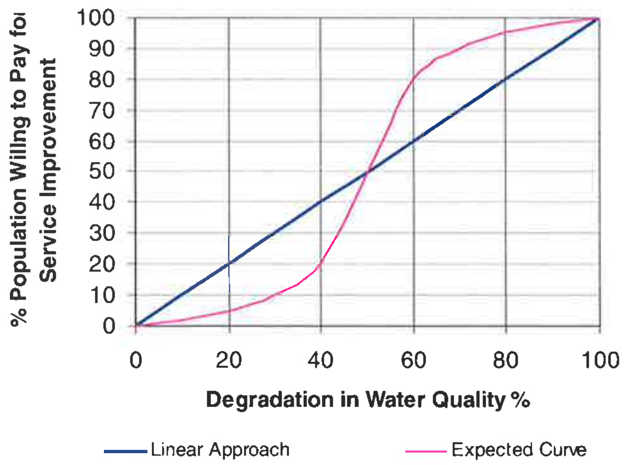

Figure 5.11 Approximate and expected curves for willingness to pay

The next stage (40%-60%) shows a massive move in the population shift with the percentage of population wanting better quality moving from 20% up to 80%. This is the point where major change becomes a notable consumer demand.

The fourth stage (60% to 80%) shows a dieback in the curve as the majority of the population has made their intentions clear. Now, the more resilient consumers who either have high tolerance or are unable to afford the new treatment may decide that the quality is getting unpalatable, and some may become willing to pay.

The final stage (80% to 100%) of degradation has water, which is as bad as the system can allow for. There may still be a threshold with people simply unable to budget for additional treatment costs, but the remaining population who want to change position do so.

Whether this willingness to pay is for new treatment facilities or a rainwater tank, the underlying principle remains the same. It takes a certain quality level before people are willing to spend money and some people will never purchase the better option no matter how poor the supply quality is. The problem is that extensive analysis is required in terms of surveys to determine the exact form of this curve. To attain a framework for this component, a linear approximation was used in the model. Further development of the model may include analysis on the format of the different willingness to pay curves used.

5.3.4 Disinfection Cost

Of the four areas examined in formulating the water quality component of the overall framework, the disinfection cost has the most physical basis. This is dependent on the type of disinfectant, size of the dosing station and the operation and maintenance costs accrued.

In a study examining the installation of chloramination stations (Thomas, 1990), the costs of existing chlorination stations were compared against the costs of chloramination. The adapted (1999$AUS) results of this study for the Yorke Peninsula region are listed in Table 5.5. These costs are approximated as they depend on demands and dosing rates.

The annual costs of the chemicals involved in the dosing is approximately $1.25/kg based on the purchase price of $1,150 for a 920kg drum (Hogben, 1999). It has been assumed that the cost of ammonia is a similar price based on the 1989 level that was approximately $0.8/kg which was adapted from the 1990 study (Thomas, 1990). When factored into present value using a 4% annual sidcount rate, the cost becomes approximately $1.25 as well. It is from these values that the cost of disinfectant is formed.

Table 5.5 Approximate annual disinfection costs for various locations (1999$)

Cost facet	Paskeville p.a.	Maitland p.a.	Mount Rat p.a.
Chlorine (approximate)	30000	15000	15000
Ammonia (approximate)	12000	-	-
Operation & Maintenance	30000	30000	30000
Total	62000	45000	45000

Equation 5.8 gives the total disinfectant cost. This is the sum of the:

- The operation and maintenance costs; and
- The chemical cost of disinfectant.

This is used for all of the dosing points throughout the network.

$$C_{Dis(annual)} = \sum_{i=1}^{tnd} \left(D_{d_i} \times Cn_{d_i} \times C_{dn} + OC_i \right) \qquad \textbf{(5.8)}$$

Where $C_{Dis(annual)}$ = annual disinfectant cost, D_{di} = annual demand (ML/year) at dosing station *i*, Cn_{di} = disinfectant concentration (kg/ML) at dosing station *i*, C_{dn} = unit cost of the disinfectant ($/kg), and OC_i = annual operation and maintenance costs ($) at dosing station *i*, *tnd* = total number of dosing stations.

Many of these costs would differ from rural to urban networks, as the level of infrastructure and labour intensity for a rural network is lower than that of a metropolitan system.

For the New York Tunnels study, as there is only one dosing point, the operation and maintenance costs are assumed irrelative, as they would hold regardless. The price of the disinfectant is the measure for this situation. The formulation is the same as Equation 5.8, with a zero cost applicable to operation and maintenance.

5.3.5 Quality Penalty Cost

The alternative to applying social costs at a consumer level in the system, is to apply a cost function that penalises any residual level that is higher or lower than wanted by the water manager. The aim of this concept is to reduce the cost to zero, which is achieved by making this range the best effective system.

The formulation for this angle in water quality analysis is relatively simplistic and follows the structure seen in Equation 5.9. The further the residual level is away from the desired range, the greater the cost.

$$C_{QP} = \sum_{i=1}^{nn} Pen_{low} \times \max\left[\left(Cn_{low} - Cn_i\right), 0\right] + \sum_{i=1}^{nn} Pen_{high} \times \max\left[\left(Cn_i - Cn_{high}\right), 0\right] \quad \textbf{(5.9)}$$

C_{QP} = quality penalty cost, Pen_{low} = cost for low residual levels, Pen_{high} = cost for high residual levels, Cn_{low} = lowest acceptable residual level, and Cn_{high} = highest acceptable residual level.

The advantages of this technique are its ability to contain the residual levels into a preferred regime. This is often at the expense of other system attributes as the formulation can converge prematurely if the penalty level is set too high. If the level is too small, the residual levels will fail to conform to the requirements. This technique can become population based, rather than node-based by applying the nodal populations to Equation 5.9. Using a similar approach, Sakarya and Mays (1999) discussed penalising to attain desired water quality levels. This method used the sum of the squared differences as discussed in Section 3.11 rather than a linear approach.

As with all penalty costing, to become a viable technique, sensitivity analysis and estimation are essential to guarantee that the formulation takes note of the constraints and converges at the appropriate position.

5.4 Determination of Penalty Costs

The ideal of optimisation for system design is to end up with cost efficient solutions that fit within the system requirements. Hydraulically, these criteria include:

- Each node has satisfactory pressure levels;
- Tanks do not drop below minimum levels; and
- Pumping rate does not exceed pump capacity.

The supply at each node is automatically satisfied by EPANET, what suffers to achieve this supply are the pressures and pumping ability.

The methodology to determination of the penalty costs uses a linear approach as discussed by Murphy *et al.* (1993) as this worked well for GA application. Other methods (Sakarya and

Mays, 1999; Goldman and Mays, 1999) use a sum-of-the-squares approach discussed in Section 3.12.3.

5.4.1 Pressure Penalty Cost

Application of a pressure penalty cost is essential to guarantee a good pressure is supplied to each demand node. It is one thing to get water to that location, but the consumer must be able to water higher parts of the garden, wash the roof, supply an upstairs bathroom, or even put out a fire in their yard. If the water at the node has no pressure, these options become void.

To prevent this, and guarantee that the user has options available to them in their water use, a pressure requirement is set, which is ensured by implementation of a pressure penalty cost, formulated in Equation 5.10.

$$C_{pp} = p_{pen}\left(\sum_{i}^{tnn}\max\left[\left(p_{req} - p_i\right), 0\right]\right) \quad \textbf{(5.10)}$$

C_{pp} = total pressure penalty cost, p_{pen} = pressure penalty, p_{req} = minimum required pressure, p_i = pressure at node i, tnn = total number of nodes.

A typical acceptable pressure in many water systems can be considered as a level of 20m head at each node (EWS, 1986(b)). Drops during peak times down to 17m are accepted, but not desired, hence 20m will be selected as the base level in this study.

In previous research undertaken in pressure penalty determination (Murphy *et al.*, 1993), levels of \$5million/ft to \$10million/ft (1969\$US) on the maximum pressure violation were used for the 1969 New York system.

The values used for the Yorke Peninsula system have been extrapolated from Dandy and Warner (1989) and the exchange rate applied to give a level, which is in current Australian dollars. This process is listed in Table 5.6.

Table 5.6 Conversion process of 1969$US to 1999$AUS

Year	US Index	Converted to $AUS
1970	100.0	142.9
1982	248.6	355.1
1986	283.8	405.4
1999	440.3	629.0

So, from this formulation (Exchange rate assumed at a level of about 0.7) the approximate difference is 6.29 times. So if the New York study were analysed in current Australian dollars, the pressure penalty would range between $30million/ft and $90million/ft.

To adapt this penalty to other systems, making the penalty population dependant is a way of doing it. This is vital, as if the penalty is far too high, the solution space converges too high above the optimal range. If a system has a population of 30,000, and a $30million/ft penalty, a 1ft-(≈0.3m) discrepancy at one node could equate to costs greater than the total infrastructure costs. So by being population dependent, it makes the process more acceptable for general use.

The other conversion is to convert to metric units. The 1969 New York study based on a metric analysis would have pressure penalties of between $100million/m and $300million/m. This is for an approximate population of 8million to 10million people.

By graphing this on a population scale, a pressure penalty can be determined for a system of any size. Figure 5.12 is based on the 1969 level of $5million/ft 1969$US penalty. The curves in Figure 5.12 were produced by converting the pressure penalty used for the New York Tunnels study to a consumer level giving a penalty per person. This personal penalty factor was then applied to different system populations to produce a curve. From this curve, approximate levels of pressure penalties can be determined.

For a system of 30,000, which is the approximate size of the Yorke Peninsula region, a pressure penalty range of between $300,000/m to $500,000/m could be acceptable. Some sensitivity analysis to deter premature convergence is an added measure to ensure a good level has been used.

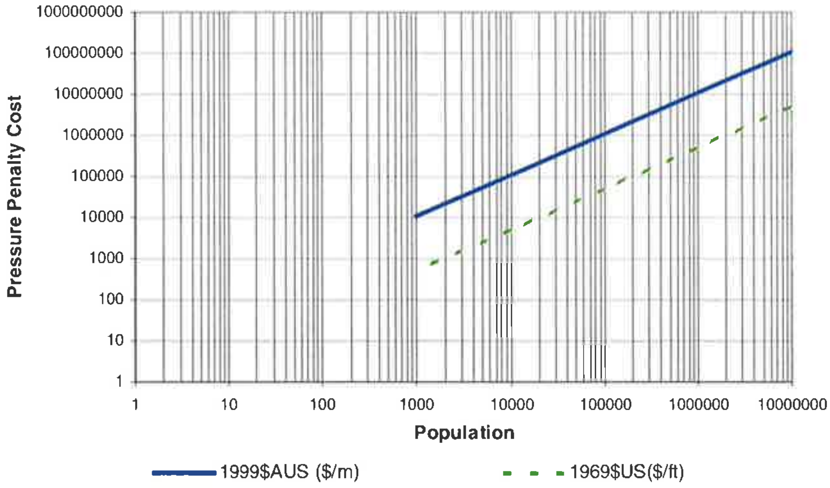

Figure 5.12 Conversion of pressure penalty for different populations

5.4.2 Tank Penalty Cost

In water distribution, it is common practice for a tank to have a minimum storage which is set as being able to supply the town or region for a given period. This extra storage allows room for upstream maintenance to occur, supply of unexpected peak use, and potential fire fighting water.

This emergency storage is designed for rural South Australian systems (EWS, 1986(b)) to allow for 4 hours of flow on the peak day, which usually suffices for a full winter's day.

There are different operation levels (illustrated in Figure 5.13). The lowest is the base elevation level, which is the tank floor. The minimum storage level, which is the height at which sufficient water remains to provide the allocated emergency storage for that system location follows. The next level is the desired minimum level, where the upstream pumps begin operation to start filling the tank. The pumps continue operation until the tank level reaches capacity, which turns off the upstream pumps. The pump operation levels may differ dependent on operating requirements.

The capacity is usually a bit below the top of the tank. The reason for the free space at the top is so the tank does not overfill causing pressure and structural problems.

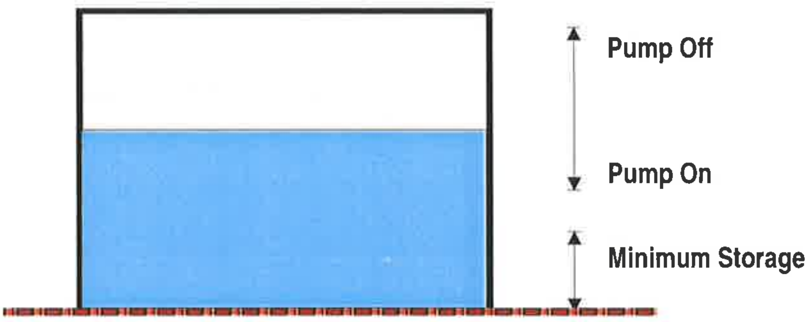

Figure 5.13 Typical tank operation in a water distribution system

The system is only considered to be not working properly if the water level drops into the minimum storage zone. To avoid this occurring in the series of simulations undertaken by the genetic algorithm, a penalty is applied to any tank in the system, which drops below the minimum level.

This penalty cost discourages solutions that do not supply the sufficient flows to parts of the system. The response of the system is to increase the size of the upstream pumps and/or capacity of the larger distributing tanks. So, the basic formula for the implementation of tank penalty becomes Equation 5.11.

$$C_{Tp} = T_{pen}\left(\sum_{t}^{tt}\sum_{i}^{nT}\delta\left(h_{\min} - h_{wi}\right)\right) \tag{5.11}$$

C_{Tp} = tank penalty cost for violating tanks, $\delta(.)$ = the Dirac delta function, h_{wi} = height of water for tank i, h_{min} = minimum acceptable water height, nT = number of tanks, T_{pen} = tank penalty, t-tt = simulation time from time t to total time tt.

Each indiscretion is penalised so that the system is not only spatially optimised, but different flow conditions during a time sequence are also examined.

5.4.3 Pumping Penalty Cost

The pump selection in a water supply system is essential to ensure that the demand objectives are met. If the pumps lack the capacity to cater for the flow or head requirements of the network, parts of the system will not get acceptable supply. It is also noted that the pumped supply systems should not be operational for greater than 16 hours a day, as this allows for plant maintenance, reductions in plant performance and power failures (EWS, 1986(b)).

A pumping penalty has been applied to the study to prevent this occurrence in the optimisation process from the GA.

This penalty places a cost on the violation of the head or flow requirements by the pump. EPANET provides warnings if the pump is experiencing difficulty, by applying a cost for each time the warning appears, the GA forces selection of larger pumps with a greater capacity to meet the system demands. The formulation process for the pumping penalty can be seen in Equation 5.12.

$$If\left(EPANET_{warning}\right) \Rightarrow C_{PMp} = PM_{pen}\left(\sum_{t}^{tt}\sum_{i}^{nPM} PMv_i\right) \qquad \textbf{(5.12)}$$

$EPANET_{warning}$ = the EPANET solver indicating a problem with pump flow or head capacity, C_{PMp} = tank penalty cost, PMv_i = violating pumps, nPM = total number of pumps, PM_{pen} = pump penalty cost, t-tt = simulation time to total time tt.

By the implementation of this process, coupled with the large cost factors associated with upgrading pump capacity, an optimal pump selection process is ensured. Pumps able to cater for the network requirements, while being cost efficient are selected.

5.5 Total Cost Structure

All of the factors mentioned in this chapter have been computed to provide a full framework of the potential costs of the overall system design. Though the different cost components should all be minimised by the GA, traditionally, an order of relative importance exists due to the logic needs of a system.

The first requirement is to have a supply system in place that has the ability to distribute water to demand nodes. Secondly, this supply should be able to be relied upon to cater for different flow conditions so that it can be used to the will of the consumer. Lastly, the water, once at a good supply standard, must not present a health risk to the user population.

The reason for this order is that you cannot provide adequate water quality, until you have a functioning supply system. For this reason, it is traditionally the case that the treatment process be adapted to fit the designed system.

In the method adopted in this study, all of the system components integrate simultaneously so as to present a final solution incorporating each of these three issues on a determined basis. The costs come in three parts:

- Infrastructure costs;
- Penalty costs; and
- Quality costs.

5.5.1 Infrastructure Costs

The total infrastructure cost is the total of the pipe costs, tank costs and pumping costs. Full discussion into these costs can be found in Appendix D, the summary formulation is shown in Equation 5.13.

$$C_I = C_p + C_T + C_{PM} \qquad \textbf{(5.13)}$$

C_I = total infrastucture cost, C_p = pipe cost, C_T = tank cost, C_{PM} = pumping costs.

5.5.2 Penalty Costs

The total penalty cost is the sum of all of the penalty costs applied to the system. The formulation for this is seen in Equation 5.14.

$$C_{pen} = C_{pp} + C_{Tp} + C_{PMp} \qquad \textbf{(5.14)}$$

C_{pen} = total penalty costs, C_{pp} = pressure penalty costs, C_{Tp} = tank penalty costs, C_{PMp} = pumping penalty costs.

5.5.3 Quality Costs

The total quality costs consider costs due to taste and odour, disinfection by-products, waterborne disease, and disinfectant cost as given in Equation 5.15. The taste and odour costs based on a rainwater tank preference (smaller systems) represents lifetime costs for the consumers.

If the annual additional treatment option (large urban supply) is the taste and odour control measure, this has to be adjusted to fit in with the other costs, which are all annual costs, which must be converted to a lifetime cost (Dandy and Warner, 1989).

$$C_Q = C_{TO} + \left(C_{Dis(annual)} + C_{DBP(annual)} + C_{WD(annual)}\right) \times \left(\frac{1-(1+rate)^{-n}}{rate}\right) \qquad \textbf{(5.15)}$$

C_Q = total quality costs, C_{TO} = taste and odour costs, $C_{Dis(annual)}$ = annual disinfection costs, $C_{DBP(annual)}$ = annual disinfection by-product costs, $C_{WD(annual)}$ = annual waterborne disease costs, n = life of study (years) and $rate$ = discount rate over life of study.

5.5.4 Complete Framework

The framework takes into consideration each of the cost components. As Equation 5.16 indicates, the costs are considered jointly.

$$C_{tot} = C_I + C_{pen} + C_Q \qquad \textbf{(5.16)}$$

C_{tot} = total system cost based on addition of other costs.

It is this total cost value that the GA optimisation program minimises, finding the best combinations of pipes, pumps, tanks and dosing regime to determine optimal solutions.

5.6 Summary

The identification of an overall framework that caters for all of the differing facets of system design is an important stepping stone in network analysis. Although some of the parameters require further research to enable the achievement of a higher level of accuracy, the application of all the costs as a unified package presents a comprehensive solution set to the problem. Appendix E presents the different constants used in the model and gives a description, units and comment on their accuracy.

The combined cost structure allows an examination of many of the different aspects that could provide a cost, both direct and indirect. The model compares solutions for all of these costs and presents an option best suited to the entire spectrum.

Chapter 6 New York Tunnels Study

To supply a growing population on Manhattan Island with a steady supply of potable water has represented a challenge for water supply engineers since the 18th Century.

New York City has faced problems with water supply since Manhattan Island was settled. Saline estuaries surrounded Manhattan Island, leaving groundwater wells as the only potable supply. Overuse of these wells resulted in the deterioration of the quality and volume of water available. The population tripled between 1790 and 1810 placing an over demand on the water system (Weidner, 1974).

This was evident when a yellow fever outbreak in 1798 took 2000 lives, wiping out 4% of the population. This number of fatalities can be attributed largely to a sub-standard water supply. The fires of 1776, 1828 and 1835, could not be controlled partly because of the lack of water for fire-fighting purposes, which resulted in large property loss. Since then, the New York distribution system has evolved as the city expanded.

Initially, the city officialdom constructed a series of 12 reservoirs using water obtained from the nearby Croton River (Figure 6.1) and relied on water conservation to maintain supply. Despite these measures, it became obvious that new sources of water would be required.

In 1906, engineers reviewed the New York supply options, and turned their attention towards the Catskill Mountains as a source of water (Figure 6.1). In January 1914, the last segment of tunnel to connect the city to this water source was completed.

The water system underwent review again, and in 1930 it became apparent that further upgrade was necessary. The water source that was examined to boost supply came from the Delaware River (Figure 6.1). This supply became operational in 1937.

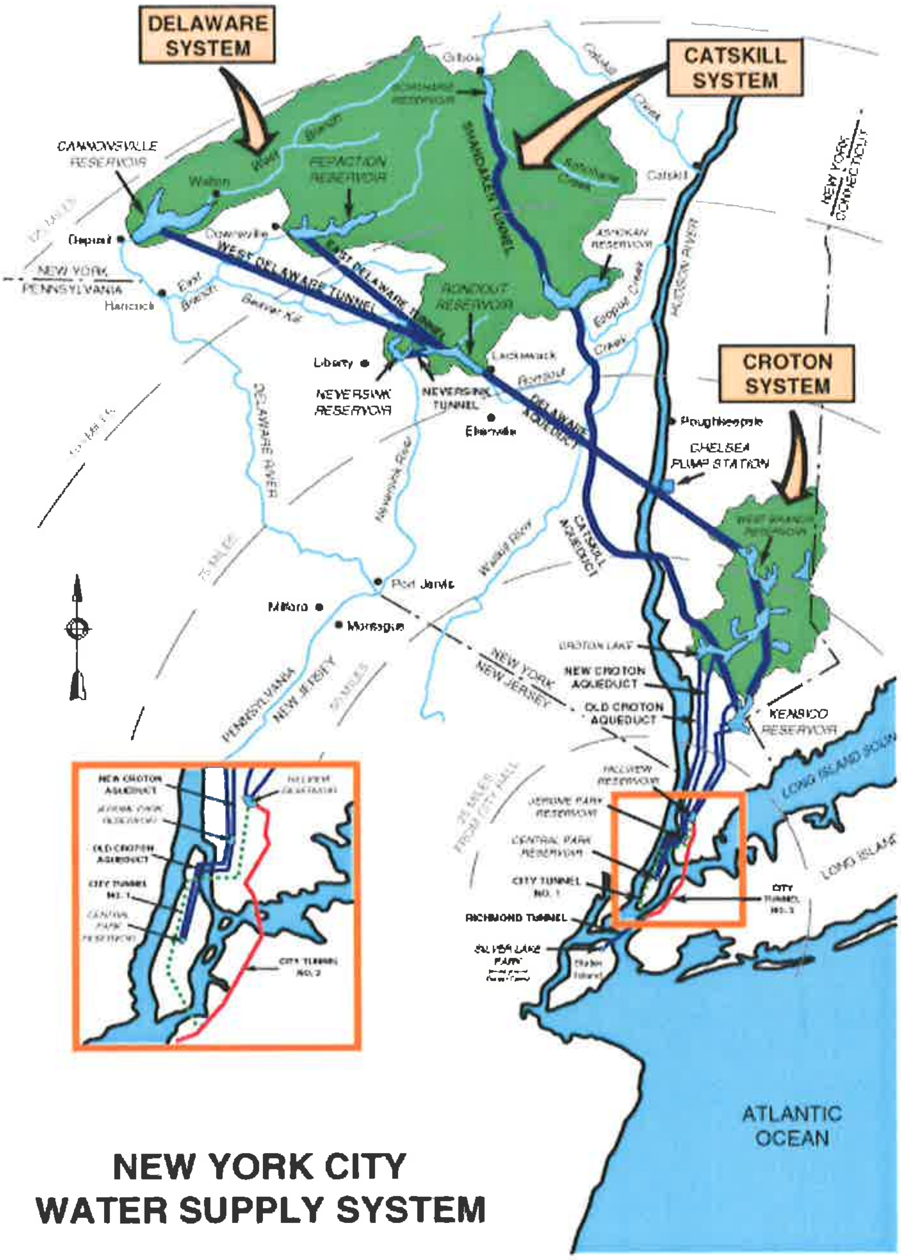

Figure 6.1 New York Water Supply System

Currently the capacity of the New York City Water System is over 550 billion gallons, with 10% from the Croton System and the remaining 90% from the Catskill and Delaware Systems (Marcus, 1999). The structure of the headworks is illustrated in Figure 6.1 (NYCDEP (a), 1999).

The network was constructed as a series of deep rock tunnels with diameters of up to 204 inches (Figure 6.2). In 1969, New York's population was still growing, and it became evident, once more, that the system was insufficient. More tunnels would therefore be required to cater for future demand.

Interest built when the studies by Shaake and Lai (1969) led to the development of an optimisation technique using linear programming to produce the most cost efficient design for the additions to the New York system. All additions to the network involve duplicating one or more of the existing tunnels.

Since this work, other researchers have applied their models to the problem to reduce the cost figure from the Shaake and Lai level of $78.09m down to $38.8m in a study by Dandy *et al.* (1996). This method was based on the use of genetic algorithms and was used as a test case. The studies to date have all focused on hydraulics alone and did not consider water quality. The current network is considerably different to the one shown as several modifications have been made to it since 1969. However, as the research on hydraulics has been based on the 1969 system, this study will concentrate on the problem researched by Shaake and Lai (1969).

6.1 The 1969 Study

The New York Tunnels System in 1969 consisted primarily of two tunnels from Hill View reservoir extending down to Staten Island (Richmond). The study established by Shaake and Lai considered a 21 pipe, gravity fed system with 20 nodes and storage at Hill View reservoir. The system used by Shaake and Lai is shown in Figure 6.2.

The demands at each of the nodes as well as the required pressures for the study are shown in Table 6.1 (Murphy *et al.*, 1993). Imperial units are used for the hydraulics in order to match the original United States study.

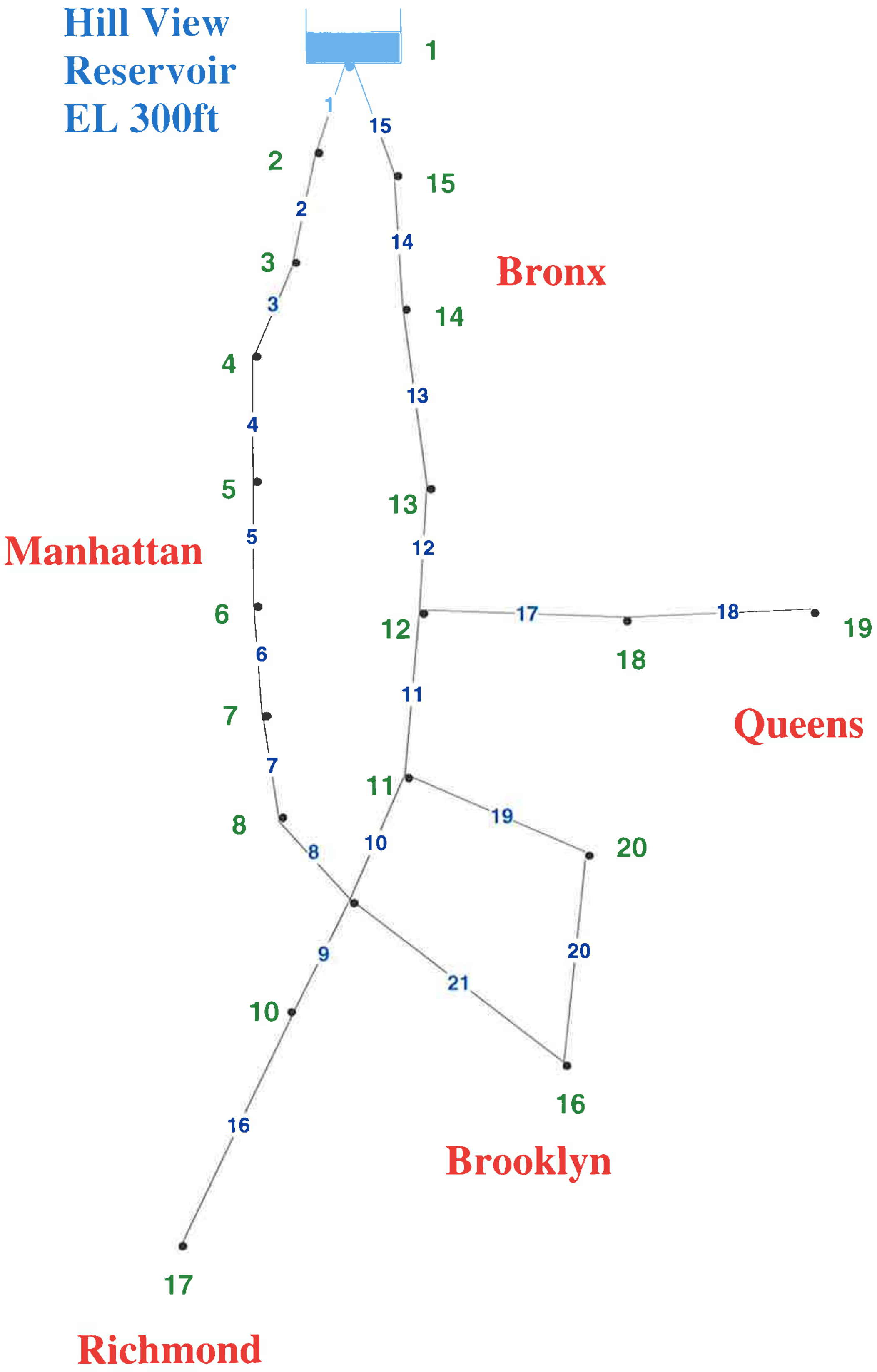

Figure 6.2 1969 New York Tunnels Study

Table 6.1 1969 Study Node Information

Node	Demand (cfs)	Min. HGL (ft)
1	Reservoir	300.0
2	92.4	255.0
3	92.4	255.0
4	88.2	255.0
5	88.2	255.0
6	88.2	255.0
7	88.2	255.0
8	88.2	255.0
9	170.0	255.0
10	1.0	255.0
11	170.0	255.0
12	117.1	255.0
13	117.1	255.0
14	92.4	255.0
15	92.4	255.0
16	170.0	260.0
17	57.5	272.8
18	117.1	255.0
19	117.1	255.0
20	170.0	255.0

The nodal pressures are the hydraulic requirements that must be met by the system. As the network is gravity fed (no pumping), the infrastructure that determines the nodal heads consists of the pipes throughout the system. As discussed in Chapter 2, the larger the diameter, the lower the head loss.

The pipes in the New York study are a system of tunnels through rock. The diameters range up to 204 inches. All of the pipe information is given in Table 6.2 (Murphy *et al.*, 1993), which provides the diameters and the lengths of the existing tunnels.

Table 6.2 1969 Study Pipe Information

Pipe No.	Start Node	End Node	Length (ft)	Diameter (in)
1	1	2	11600	180
2	2	3	19800	180
3	3	4	7300	180
4	4	5	8300	180
5	5	6	8600	180
6	6	7	19100	180
7	7	8	9600	132
8	8	9	12500	132
9	9	10	9600	180
10	11	9	11200	204
11	12	11	14500	204
12	13	12	12200	204
13	14	13	24100	204
14	15	14	21100	204
15	1	15	15500	204
16	10	17	26400	72
17	12	18	31200	72
18	18	19	24000	60
19	11	20	14400	60
20	20	16	38400	60
21	9	16	26400	72

The cost function for the different pipe diameters in the New York study is shown in Equation 6.1 (Murphy *et al.*, 1993).

$$CP_i = 1.1 D_i^{1.24} L_i \qquad \textbf{(6.1)}$$

CP_i = pipe material cost (1969\$US), D_i = diameter of pipe i (in), and L_i = length of pipe i (ft).

6.2 The Current System

The existing New York City Tunnel System for water supply includes the City Tunnels #1 and #2. These tunnels are interconnected to the Hill View reservoir and reconnect at Brooklyn. Construction is occurring on a third tunnel which with the existing tunnels is depicted in Figure 6.3 (Fang, 1997).

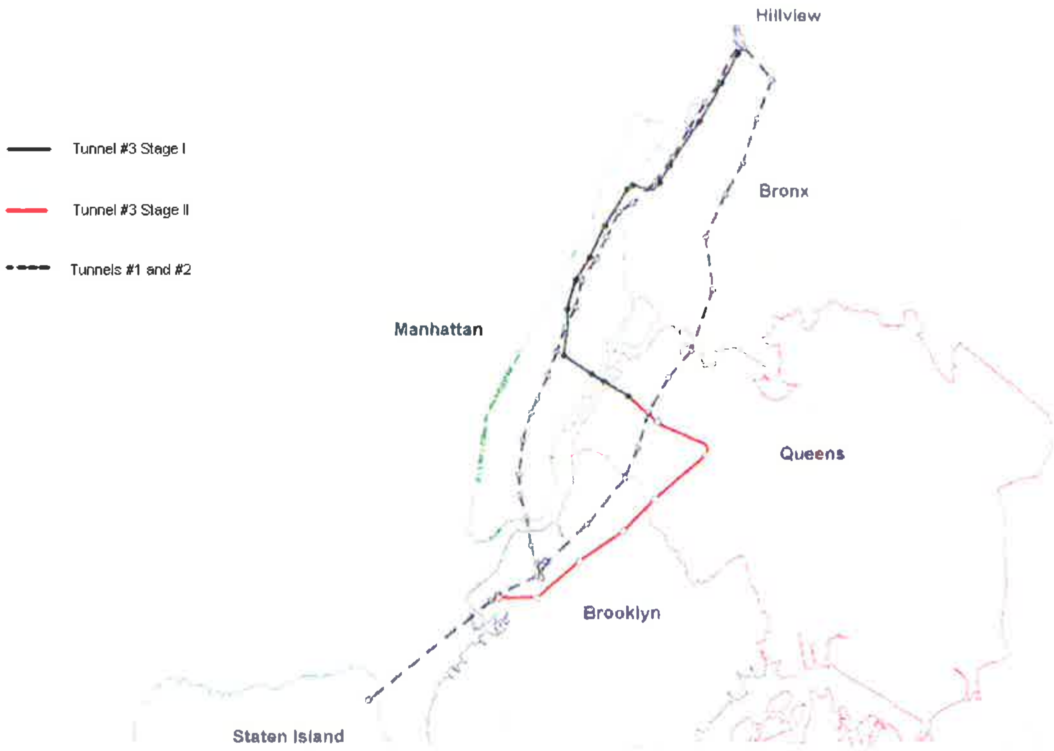

Figure 6.3 Current New York Tunnels System

The current disinfection practices include the addition of gaseous chlorine after Hill View reservoir at typical dosing levels of 1.1mg/L. The residual at the Staten Island (Richmond) end of the system is typically around 0.7mg/L.

Operators aim to achieve relatively constant chlorine residual at the entries of both tunnels. This is difficult to achieve due to changes in water temperature and turbidity as well as changes in flow (Fang, 1997).

These figures were ratified and expanded upon through correspondence with Glaser (1999). Glaser provided typical residuals for different city regions, as shown in Table 6.3.

The water is re-chlorinated when the water reaches Staten Island (Richmond) to a target level of 1.0mg/L, however, the water reaching this node is usually around 0.3mg/L. The target levels throughout the system are 0.6mg/L to 1mg/L (Glaser, 1999).

Table 6.3 Typical Residuals for New York City

Location	Typical Residual (mg/L)
Hill View Reservoir	1.0 – 1.4
Bronx	0.8
Manhattan	0.6
Queens	0.6
Brooklyn	0.5
Staten Island (Richmond)	0.3-1.0

Both the dosing and decay rates vary with season. This is due to the temperature effects as discussed earlier Chapter 3. To determine the decay rates, the change in residual and the residence time for a unit of water to progress through the system are sufficient to identify a first order decay rate for each season.

As decay is to a large extent based on temperature, the other seasonal regimes can be inferred using seasonal climate information. New York experiences sub-zero winters and mild summers with the seasonal air temperature range shown in Figure 6.4 (Glaser, 1999).

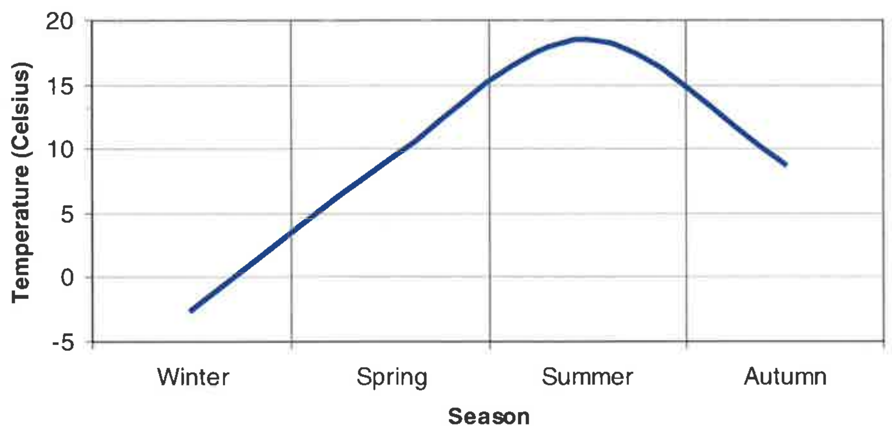

Figure 6.4 Seasonal Temperatures for New York

Water does not experience the same temperature changes as air, as heat transfer occurs more slowly. The temperature range of the source water for the system varies between 35°F (1.7°C) in winter and 80°F (26.6°C) during summer (NYCDEP (a), 1999).

The residence times are required for the determination of a decay rate. The demands throughout the year for New York City provide this information. As seen in Figure 6.5 (Glaser, 1999) autumn (March-May) experiences similar consumption to the winter months (December-November) so the residence time throughout the system would be similar.

As far as modelling the consumption, the pattern that is considered in this study, is the average for each season. Throughout a 24-hour cycle, the demand for water changes markedly based on the different activities accompanying the everyday life cycle, such as sleeping, working, eating or washing.

The average household in New York City uses approximately 100,000 gallons of water each year (NYCDEP (b), 1999). The consumer population in New York City is nearly 8,000,000 (Walker and Stedinger, 1999).

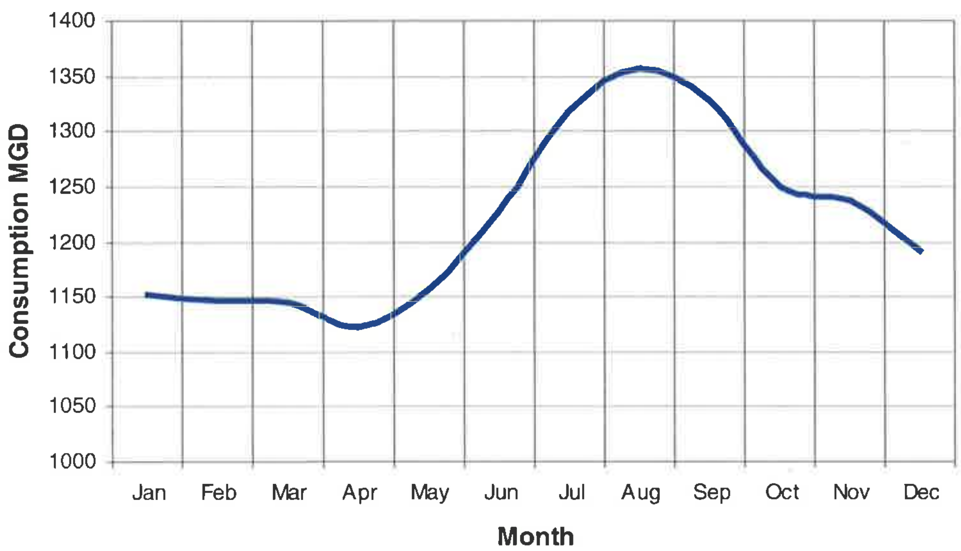

Figure 6.5 New York City Water Consumption

As the New York study looks at supplying the water to a number of distribution points (Bronx, Brooklyn, Manhattan etc.) the water is stored at each location for distribution to the extended system. The storage would buffer variations throughout the 24-hour cycle, hence average demand (consumption) levels have been used for the trunk mains.

The water consumption affects the residence time of the system, and, is thus, a major factor in the calculation of the decay rates of the disinfectant. By knowing the difference in the disinfectant residuals between two locations, and the travel between these locations, the decay rates can be determined using the first-order decay equation (Equation 3.7) discussed in Chapter 3. The decay rate analysis for the different seasons are listed in Table 6.4.

Table 6.4 Decay Rate Formulation (Adapted from Glaser, 1999)

Season	Dose Rate (mg/L)	Consumption (mg/L)	Fraction Decay	Res. Time (days)	Decay Rate (/day)
Winter	0.7	0.4	0.57	1	0.85
Autumn	0.9	0.6	0.67	1	1.18
Spring	0.8	0.5	0.625	0.8	1.10
Summer	1.2	0.9	0.75	0.75	1.85

The decay rates for the system can be represented graphically as shown in Figure 6.6.

Figure 6.6 Decay rates throughout the different seasons

The deduced decay rates will be checked using an EPANET simulation in order to compare them to observed seasonal residual levels. As discussed in Section 3.12, EPANET has been successfully adapted to optimising for water quality in distribution systems.

6.3 EPANET simulation for verification against observed quality data

The comparison of simulated data against the observed data serves to allow calibration and verification of both the decay rates used in the EPANET model and of the EPANET model itself. In Table 6.5, the comparison of the seasonal data as well as the average values from a series of simulations against typical observed residual levels are listed. Although only five of the twenty nodes are represented in the table, it provides an adequate representation of not only the extremities of the network, but of the main branches as well.

Table 6.5 Comparison of EPANET results against observed residual levels.

	Observed	Winter	Spring	Summer	Autumn	Average
Bronx	0.8	0.68	0.76	1.05	0.86	0.84
Manhattan	0.6	0.58	0.62	0.75	0.71	0.67
Queens	0.6	0.58	0.62	0.75	0.71	0.67
Brooklyn	0.5	0.48	0.47	0.55	0.56	0.52
Staten Island	0.3	0.3	0.26	0.3	0.35	0.30

This is illustrated graphically in Figure 6.7, which shows the performance of the selected decay levels to the available data.

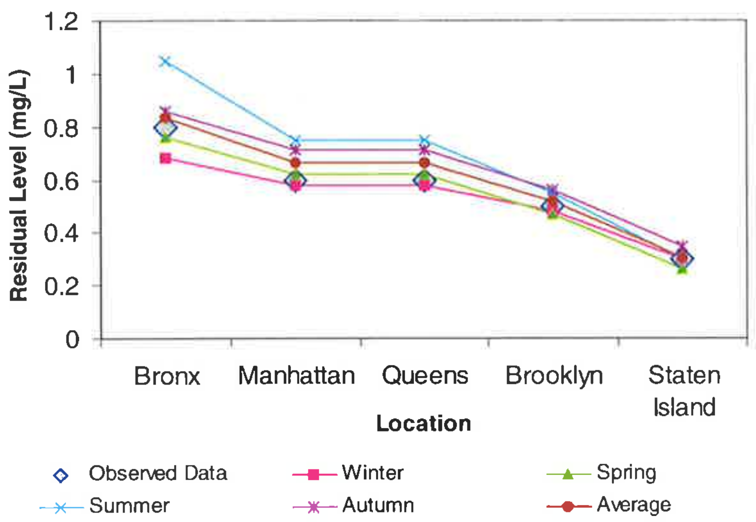

Figure 6.7 Comparison of EPANET quality simulations with observed data

The levels shown in Table 6.5 from the simulated data are usually slightly higher than the typically observed values as the current system has duplicate tunnels, which would increase the residence time and slightly increase the chlorine consumed. They are acceptable, as the simulated results are for five of the distribution nodes, whereas the observed data is collected from much smaller sub-systems from these nodes supplying the surrounding population.

6.4 Water Quality for New York City

A system specific analysis of disinfection by-products, waterborne disease and taste and odour is required to provide an overview of the water quality situation for New York.

Although re-chlorination does occur at the Staten Island end of the system, the level reaching this node should maintain a level above 0.3mg/L as this is the current low level for this part of the system (Glaser, 1999).

6.4.1 Trihalomethane Formation

The quality of the source water entering the system has an effect on the formation rate of trihalomethanes (THMs), which form an important facet in the social cost component of the study. The water temperature range of 35-80^{o}F or 1.7-26.6^{o}C with an average of around 12^{o}C is also important as temperature impacts on THM formation, as discussed in Section 3.8.1.

The recorded range of THMs entering the system had an average of 29µg/L for the Catskill/Delaware system and 47µg/L from the Croton system (NYCDEP, 1999). To determine the THM formation rate, analysis of the water quality of the New York Source water shown in Table 6.6 (NYCDEP, 1999) has to be compared to a similar supply.

Table 6.6 Water Quality of New York Source Water

Facet	Catskill/Delaware	Croton	Combined
pH	6-7.8	5.8-8.2	6-7.9
Colour (Hazen Units)*	3-15 (6)	4-34 (5)	6
TOC (mg/L)*	0.8-2.7 (1.7)	1.4-3.0 (2.38)	1.8

* Average is shown in brackets.

To estimate the THM formation rates data was used from a study of 12 surface waters in Dublin, Ireland (Casey and Chua, 1997). Climate data for Dublin indicates that its annual minimum and maximum average of 43^{o}F and 55^{o}F (Online Weather, 1999) respectively, compares favourably with New York's, which has an annual average of 49.4^{o}F (NRCC, 1999).

The quality of the surface water used for the Dublin study is also important. Values are listed in Table 6.7 (Casey and Chua, 1997) for both the raw and treated water states.

Table 6.7 Water Quality for Dublin, Ireland Study

Facet	Raw Water, Dublin	Treated Water, Dublin	New York City Supply
pH	6.3-8 (7.3)	6-7.7 (6.8)	6-7.9
Colour (Hazen units)*	20-170 (40)	3-15 (8)	6
TOC (mg/L)*	3.5-13.9 (6)	2-4.9 (2.8)	1.8

* Average is shown in brackets.

The treated water from the Dublin study compares reasonably well with the combined supply to New York City. The pH range is similar as is the colour levels. The TOC levels in the Dublin water are somewhat higher than the New York supply, which may cause a faster THM formation rate for the Dublin study.

Although the average air temperatures are similar, the range in New York is greater than for Dublin. The ranges of the two cities are compared graphically in Figure 6.8.

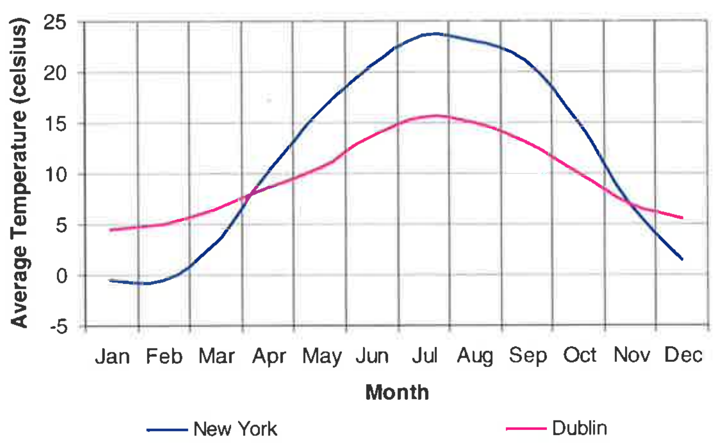

Figure 6.8 Air temperature comparison for New York and Dublin

It is expected that the water temperatures would vary much less than air temperatures. Figure 6.4 considered the seasonal average, whereas, Figure 6.8 examines the monthly average for New York temperatures. The comparative average temperature and the similar level of water quality indicate that the findings from the Dublin study are appropriate to be applied to the New York City study.

The formation rate of trihalomethanes for the Dublin study was approximately 25μg/L for every 1mg/L of chlorine consumed. This level has been applied to the New York Tunnels quality study. The formation rate will vary due to seasonal temperature fluctuation, which has been taken into consideration, as discussed in Chapter 3.

6.4.2 Waterborne disease

The current standard of drinking water contains very low coliform counts as seen in Table 6.8 (NYCDEP (a), 1999).

Table 6.8 Drinking water quality characteristics for NYC in 1997

Month	Total # Samples Analysed	% Containing Total Coliforms	% Containing E.coli
January	899	0.1	0.0
February	823	0.1	0.0
March	937	0.0	0.0
April	913	0.2	0.0
May	944	0.0	0.0
June	913	0.7	0.1
July	972	0.6	0.0
August	930	0.1	0.0
September	924	0.1	0.0
October	955	0.2	0.0
November	910	0.3	0.0
December	949	0.1	0.0
Totals	11069	0.2	0.0

The highest recorded percentages of coliform readings occurred during June and July, which are the warmer summer months. During the summer months, up to 0.7% of the samples tested contain coliforms, with some samples containing faecal coliforms. As there is a higher decay rate during summer, residual levels may drop off at some of the outer locations in the system.

As discussed in Chapter 3, if there is insufficient residual in the system, coliform levels can increase rapidly. The potential annual risk for no residual in the system for the New York

Tunnels study is based on the formulation for a large urban system from Chapter 5, and is assumed to be 0.01/year. The acceptable annual risk of waterborne disease is 0.0001/year (Gerba *et al.*, 1996).

6.4.3 Taste and Odour

As New York is a very large urban centre, the method of application of taste and odour cost is discussed in Chapter 5 as the "willingness to pay" for additional treatment. If the water quality reaches levels, which are considered unacceptable by a consumer, they will pay to have a supply that is treated to a higher standard using GAC treatment (granulated activated carbon). In selecting GAC treatment, the aesthetic standard of the supply will be greatly enhanced.

6.5 Summary

The New York Tunnels study is a well-researched problem, with known optimal solutions for hydraulic conditions. The addition of water quality analysis to the system determines the differences, if any, between the optimal hydraulic solution and the solution satisfying water quality constraints.

The water quality of the New York system has been obtained from current observed records. Many of the water quality parameters have been ascertained from adaptation of this data, combined with quality simulations to provide a computer analysis, which corresponds as closely as possible to the real system. The remaining water quality parameters were gained from examination of a similar system.

By combining all the quality parameters, with the previous research on the hydraulic structure, which was implemented successfully by GA application (Murphy *et al.*, 1993), a complete overview of this traditional study can be examined.

Chapter 7 New York Tunnels Results

This chapter for the New York Tunnels study will provide the following results for discussion:

- A pure hydraulic analysis to compare the programmed GA against other GAs or algorithms used by other modellers;
- A pure hydraulic analysis until convergence, followed by the inclusion of a water quality penalty cost structure (Chapter 5) to see if the optimal solution is consistent;
- A full water quality penalty cost analysis, to examine the convergence path with comparison to the previous runs;
- A pure hydraulic analysis until convergence, followed by the inclusion of a water quality social cost structure (described in Chapter 5) to see if the optimal solution is consistent; and
- A full water quality social cost analysis, to examine the convergence path with comparison to the previous runs.

By examining these different modelling runs, sufficient comparison will be made to see what differences, if any, the pure hydraulic solution has from the observed solution for the combination of water quality and system hydraulics. Unless specified otherwise, each generation of the GA (discussed in Chapter 4) uses 1,000 evaluations, allowing the graphs representing the generation number to be compares by the number of evaluations.

7.1 Hydraulic Analysis

The hydraulic analysis considers the pressure constraints on the system. If the pressures are satisfied, the system is deemed feasible. If there is a discrepancy in the pressure regime, the network solution is deemed infeasible. The cost method to penalise the pressure discrepancies is described in Chapter 5, and adds a direct pressure penalty cost for any offending nodes.

7.1.1 Previous Research

Shaake and Lai (1969) were the first to examine the New York Tunnels problem. This approach involved a linear programming technique. As Murphy *et al.* (1993) indicates in Table 7.1, this involved duplicating nearly all of the pipes, at a total construction cost of $78.09m.

Table 7.1 Different network solutions to the New York Tunnels study

Researchers	Shaake and Lai	Quindry *et al.*	Gessler	Bhave	Morgan and Goulter	Fujiwara and Khang	Murphy *et al.*
Year	1969	1981	1982	1985	1985	1990	1993
Pipe			Diameters of duplicate tunnels (inches)				
1	52.02	0	0	0	0	0	0
2	49.90	0	0	0	0	0	0
3	63.41	0	0	0	0	0	0
4	55.59	0	0	0	0	0	0
5	57.25	0	0	0	0	0	0
6	59.19	0	0	0	0	0	0
7	59.06	0	100	0	144	73.62	0
8	54.95	0	100	0	0	0	0
9	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0
11	116.21	119.02	0	0	0	0	0
12	125.25	134.39	0	0	0	0	0
13	126.87	132.49	0	0	0	0	0
14	133.07	132.87	0	0	0	0	0
15	126.52	131.37	0	136.43	0	0	120
16	19.52	19.26	100	87.37	96	99.01	84
17	91.83	91.71	100	99.23	96	98.75	96
18	72.76	72.76	80	78.17	84	78.97	84
19	72.61	72.64	60	54.40	60	83.82	72
20	0	0	0	0	00	0	0
21	54.82	54.97	80	81.50	84	66.59	72
Total Cost $m	$78.09m	$63.58m	$41.8m	$40.18m	$39.20m	$36.1m	$38.80m
Feasibility	Feasible	Feasible	Feasible	Feasible	Infeasible	Infeasible	Feasible
Method	Continuous	Continuous	Discrete	Continuous	Discrete	Continuous	Discrete

After the Shaake and Lai analysis, researchers have been applying their models to the problem. The current optimum of $38.80m is the solution found using the GA optimisation method (Murphy *et al.*, 1993) which is shown in Figure 7.1. Other researchers are still applying their models to this problem, however, a better feasible solution has not been successfully defended to date. One of the reasons for this is the lack of consistency in the coefficients used in the pipe head loss calculations.

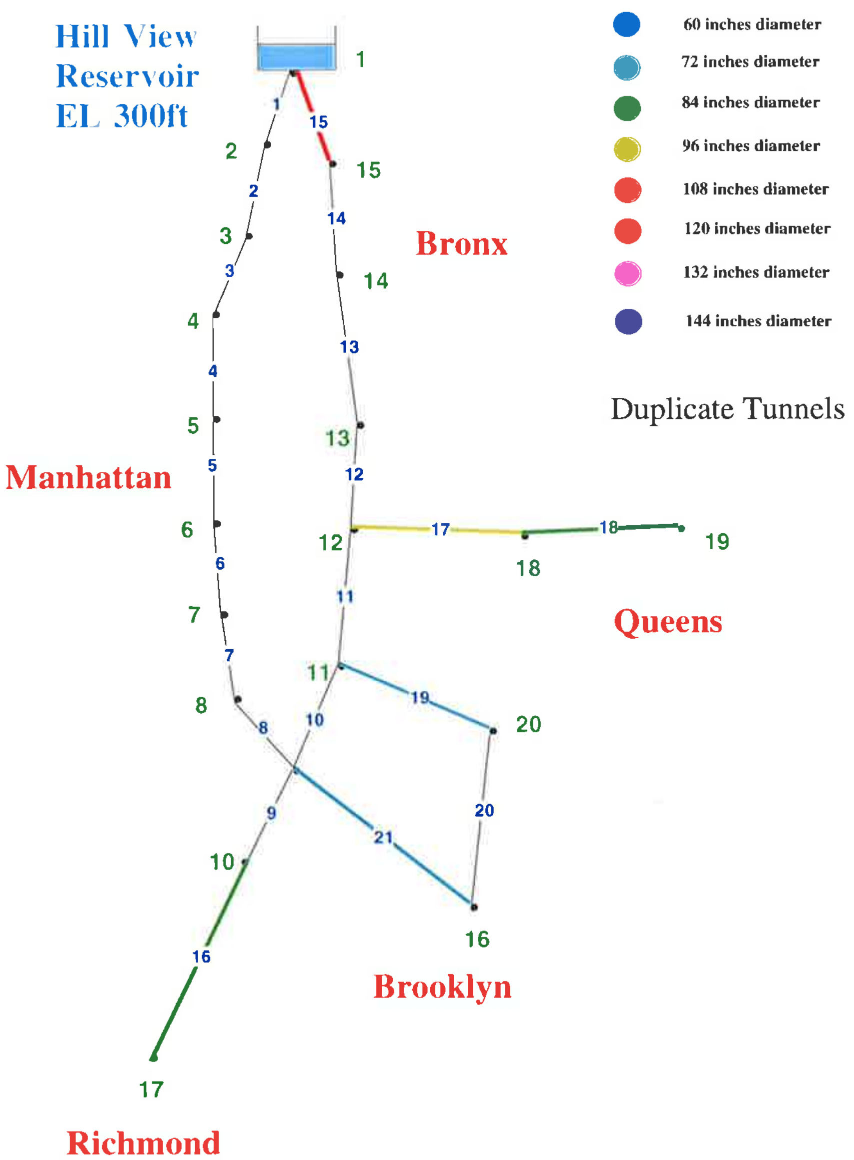

Figure 7.1 Current optimum hydraulic solution of $38.8m (Murphy *et al.*, 1993)

Comparisons of some of the different solutions to the New York Tunnels problem are pictorially represented in Appendix F.1.

7.1.1.1 The importance of using the correct Hazen-Williams information

One of the critical factors in the determination of the feasibility of the solution are the three Hazen-Williams exponents used to determine the head loss due to pipe wall friction. Even marginally different factors can produce remarkably varied results. The Hazen-Williams equation used to determine the head loss due to flow in this analysis for imperial units is given in Equation 7.1 (adapted from Streeter and Wylie, 1983).

$$h_{f_i} = \frac{4.7291 L_i Q_i^{1.852}}{C_i^{1.852} D_i^{4.8704}} \tag{7.1}$$

This formulation is consistent with the previous GA application towards this study by Murphy *et al.* (1993), which allows accurate comparison of the GA formulated for this study to previous studies to occur.

If more approximate Hazen-Williams coefficients are used (1.85, 4.72, and 4.87), which are the default levels for EPANET, the optimal solution identified by the GA used in this study becomes \$38.13m. The comparison of the accurate optimum with the approximate optimum can be found in Appendix F.1, where the different solutions for New York Tunnels are represented pictorially.

7.2 Adaptation of Thesis GA Format to the New York Problem

The genetic algorithm developed for this study was used to carry out a hydraulic optimisation in order to compare it with the work undertaken by Murphy *et al.* (1993). One important factor in the optimisation is the random number seed used. The random seed dictates the solution space used for the optimisation, and different seeds can return different optima.

After analysis of a variety of seeds, the Murphy improved genetic algorithm found the optimal solution to the problem to be \$38.80 million (1969\$US). The genetic algorithm formulated for this study was applied to numerous seeds as well. The \$38.80m solution occurred in nearly half of these runs. Five of the hydraulic runs with similar seeds were compared to the Murphy solution, and the results of these runs are given in Table 7.2.

The population size used was 1000, the probability of mutation was 0.01, and the probability of uniform crossover occurring was 0.5. Elitism was included in these results and tournament selection, with groups of 3 the selection process.

Table 7.2 Comparison of formulated GA runs to optimal solution

Pipe Number	Seeds used for the Run					
	Murphy *et al.*	200	210	220	230	240
	Feasibility of Solution					
	Feasible	Feasible	Feasible	Infeasible	Infeasible	Feasible
	Construction Cost of Solution					
	38.8	38.8	40.07	38.64	38.64	39.86
	Pipe Diameters (inches) for solutions					
1	0	0	108	0	0	0
2	0	0	0	0	0	0
3	0	0	0	0	0	0
4	0	0	0	0	0	0
5	0	0	0	0	0	0
6	0	0	0	0	0	0
7	0	0	108	144	144	108
8	0	0	0	0	0	0
9	0	0	0	0	0	0
10	0	0	0	0	0	0
11	0	0	0	0	0	0
12	0	0	0	0	0	0
13	0	0	0	0	0	0
14	0	0	0	0	0	0
15	120	120	0	0	0	60
16	84	84	84	96	96	96
17	96	96	96	96	96	96
18	84	84	84	84	84	84
19	72	72	72	72	72	72
20	0	0	0	0	0	0
21	72	72	72	72	72	72

As shown, the GA run using the random number seed 200 found the optimal solution. The other runs converged to similar solutions (\$38.64m-\$40.07m). This demonstrates the importance of seed selection in the optimisation process, although with sufficient population, the GA is able to locate the optimal solution frequently regardless of seed.

Figures 7.2 to 7.4 compare the convergences of the different seeds for the hydraulic scenario. Figure 7.2 considers the total cost structure, which is made up of the infrastructure cost (Figure 7.3) and the costs due to pressure penalties (Fig 7.4). This process examines the adaptability of the GA to different initial search spaces, and provides the preferred seeds for the extended quality analysis.

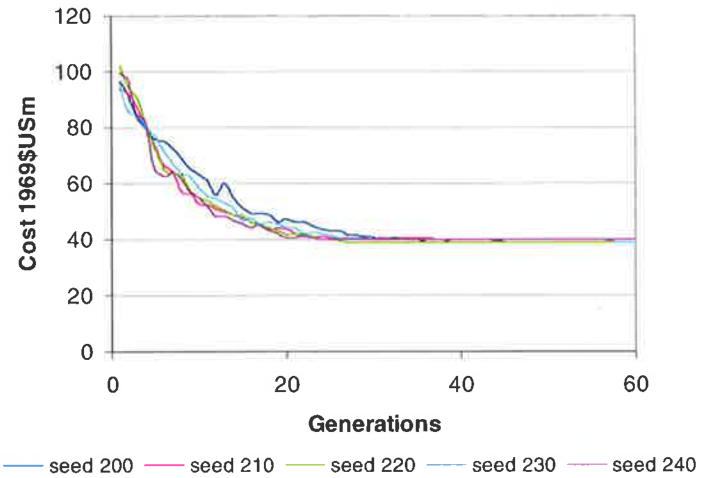

Figure 7.2 Total costs of different seeds for the pure hydraulic optimisation

The GA takes approximately 40,000 evaluations to converge completely, this compares favourably with the research by Murphy, Simpson and Dandy, which took 100,000 generations to achieve convergence (Dandy *et al.*, 1996), (Murphy *et al.*, 1993).

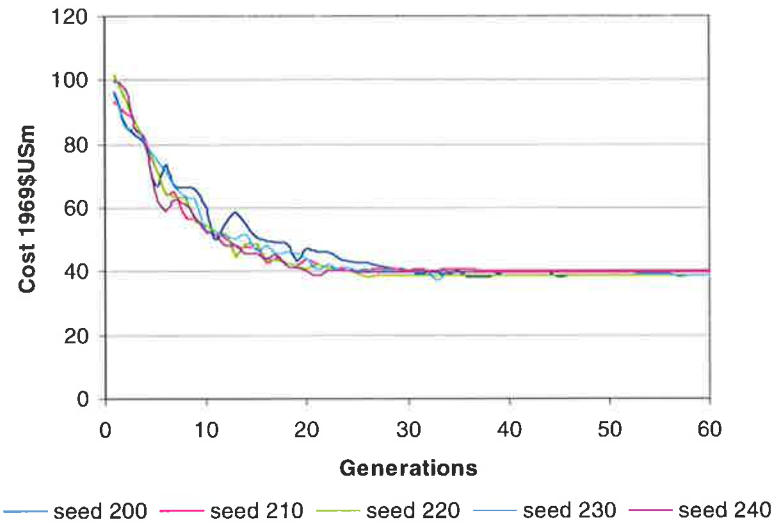

Figure 7.3 Construction cost for the pure hydraulic optimisation

The construction costs (Figure 7.3) drop below the optimum level in some cases, but the pressure penalty (Figure 7.4) forces the costs back to optimal levels.

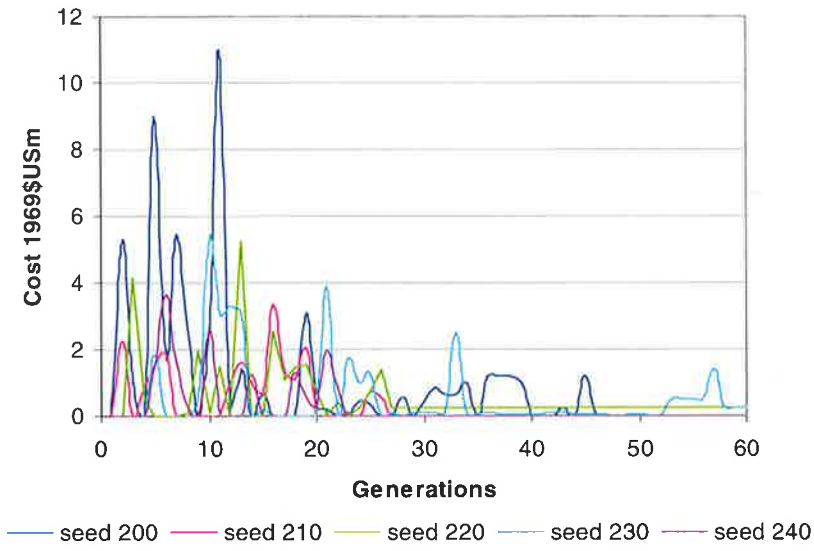

Figure 7.4 Pressure penalty costs for the pure hydraulic optimisation

The pressure penalties ($5,000,000/ft) begin at around $5m, which equate to a 1ft deficit over the system nodes. This level drops to $0m in most cases, however, some of the runs have an optimum which is an infeasible solution. The infeasible solutions of seeds 210, 220, and 230 have a penalty cost of approximately $0.27m, which equates to two thirds of an inch total pressure deficit over all the nodes in the system.

The importance of the production of pure hydraulic results for the New York problem is to calibrate the model used in this study against the previous research in this area. By finding similar results to the genetic algorithm research done on this study, the developed model has shown its capability to find the current optimal solution efficiently. This shows that the hydraulic element of both the analysis and optimisation are correct, so the model can be applied to water quality in the knowledge that it has satisfied the hydraulic research to date for this system.

The different pipes selected from this analysis are shown graphically in Figure 7.5. As can be seen from Figure 7.5, pipes 2-6, 8-14, and 20 are not duplicated in any of the hydraulic solutions. Pipes 16-19, and 21 are duplicated in all of the solutions, whereas, pipes 1, 7, and 15 occur in some of the solutions. Seeds 210, and 220 produced the same optimum, so they follow the same curve on the diagram.

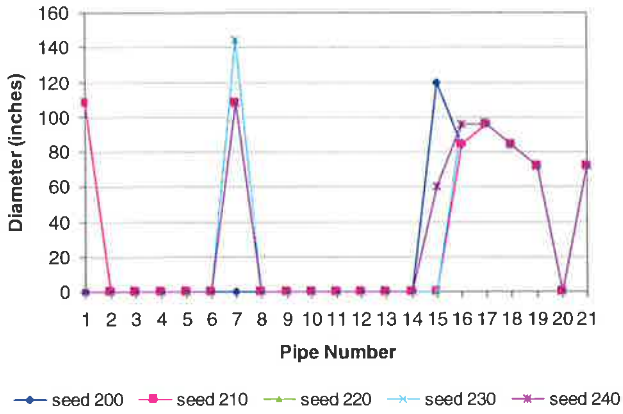

Figure 7.5 Comparison of pipe duplication options for hydraulic solutions

7.2.1 Effect of Pressure Penalty

If the pressure penalty of the system is increased, the chances are greater that the solution will not reach the desired optimum as seen in Figure 7.6 showing the total cost.

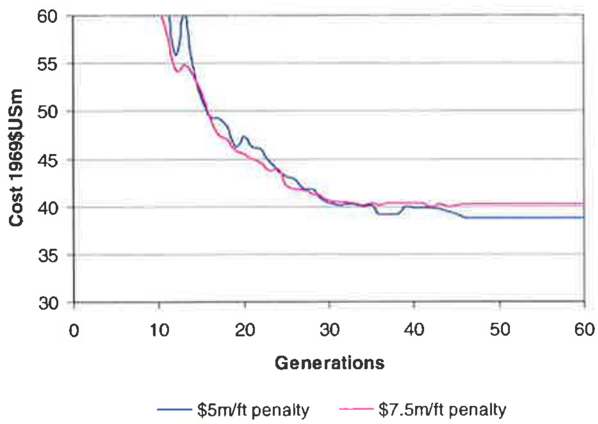

Figure 7.6 Convergence of different pressure penalties using a seed of 200

If the pressure penalty is placed too low, the reverse can occur as an infeasible solution is found, which is not penalised sufficiently to bring it back to the desired pressure levels.

7.2.2 Hydraulic Summary

The importance of the production of pure hydraulic results for the New York problem is to calibrate the produced model against the previous research in this area. By finding similar results to the genetic algorithm research done on this study, the research can move onto the area of water quality analysis with confidence that the optimisation package is likely to find optimal solutions at an efficient rate.

The data obtained from the hydraulic analysis provided the best solutions using a seed of 200, and a pressure penalty of $5,000,000 (1969$US) per foot of deficit. These values will be carried through into the quality study of the system.

7.3 Inclusion of Water Quality into the Analysis

The water quality analysis of the system will consider both the quality penalty approach, and a consumer-based method, where the potential costs of illnesses are combined with the consumer's willingness to pay. Both of these methods are discussed in detail in Chapter 5.

The decision variables for this analysis are:

- The sizes and locations of duplicate pipes in the system;
- Sizing of pumping stations and booster pumping stations;
- Sizing of storage tanks;
- Location of dosing stations; and
- Seasonal dosing rates for treatment throughout network.

7.3.1 Application of Quality Penalty

The quality penalty method (discussed in Chapter 5) works similarly to the pressure penalty method, in that it will apply a cost to chlorine residual levels that outside the target levels. The total cost also includes the capitalised cost of disinfectant. The quality penalties used in this analysis were a maximum of $10m (1969$US) for a breach of either the minimum, or maximum acceptable residuals at any node in the system.

Two different optimisation runs were examined for this section:

- Hydraulic analysis for 50 generations then quality penalty for remaining generations; and
- Pressure and quality penalty applied from the start.

These two runs were each carried out for accepted residual ranges of 0.3 to 1.8mg/L and 0.5 to 1.8mg/L. The 0.3mg/L level is the minimum level acceptable in the current system whereas 0.5mg/L is closer to the desired level (Glaser, 1999). As the system is currently not dosed much above 1.4-1.6mg/L a top level of 1.8mg/L was introduced to assist in the aesthetics of the supply.

7.3.1.1 Application of hydraulic optimisation then water quality penalty analysis

The optimisation of the solution set for the hydraulic aspects followed by a combined quality-hydraulic analysis is a process that has potential benefit. The solution set for the quality analysis starts from a stable hydraulic setting, and in addition, the potential time savings in running time due to the shorter simulation duration required for the hydraulic analysis allows better results to be developed in less time.

Figure 7.7 shows the total cost convergence pattern for this optimisation method. Initially, with only hydraulic costs forming the structure, the solution set converges to the current hydraulic optimum of $38.80m at generation 44 (44,000 evaluations). Once this is achieved, the water quality penalty cost and disinfection costs are included in the analysis as seen by a sharp jump in the costs at 50 generations. From the 50-generation mark, the model optimises the total cost including the new set of constraints of water quality along with the hydraulic aspects, adjusting the latter if need be to provide a preferable solution set.

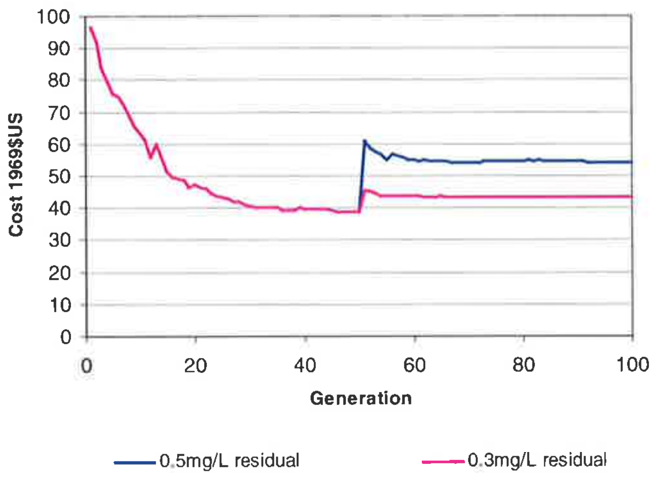

Figure 7.7 Evolution of total cost for the hydraulic then quality penalty case

From Figure 7.7, the two lines represent the two different penalty conditions. The 0.5mg/L minimum is included to simulate for a situation where each of the nodes has holding capacity before redistribution to the wider system. The 0.3mg/L minimum is a less stringent level to see how this effects network design and operation. Figure 7.8 shows the construction costs associated with the system design.

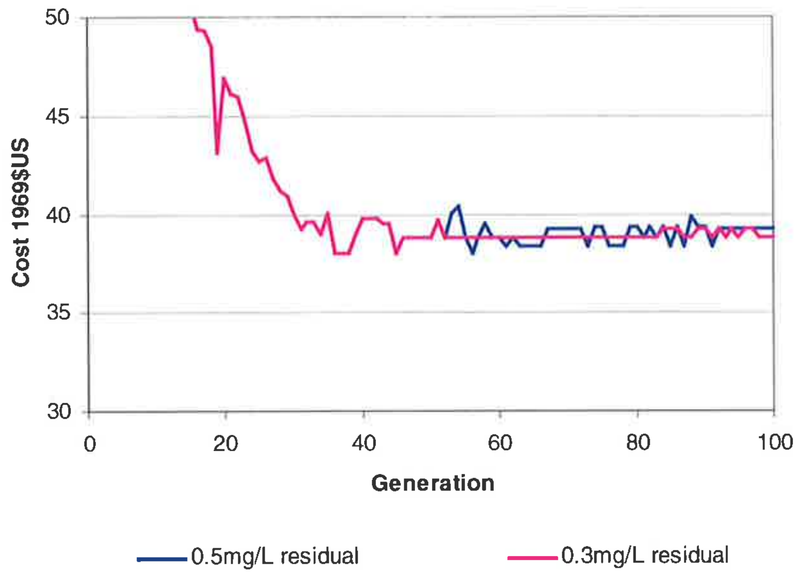

Figure 7.8 Construction costs for the hydraulic then quality penalty case

The construction costs for the two runs show convergence to $38.80m from generation 45-50. Then disturbance from this level occurs with the inclusion of quality costs, until the solution sets start to reconverge after 90 generations at a level of $39.21m.

The 0.3mg/L run does not converge as cleanly as the 0.5mg/L run as its quality costs compared to the hydraulic costs are considerably lower. The pressure penalty costs in Figure 7.9 compared with the quality penalty costs in Figures 7.10 and 7.11 illustrate this point.

Figure 7.9 shows convergence to zero pressure penalty after 45 generations for both runs. As the first 50 generations are purely hydraulic, the 0.3mg/L and 0.5mg/L runs experience exactly the same results during this time hence the curves are interposed. It is only once water quality is included in the formulation that the curves of the two runs differ.

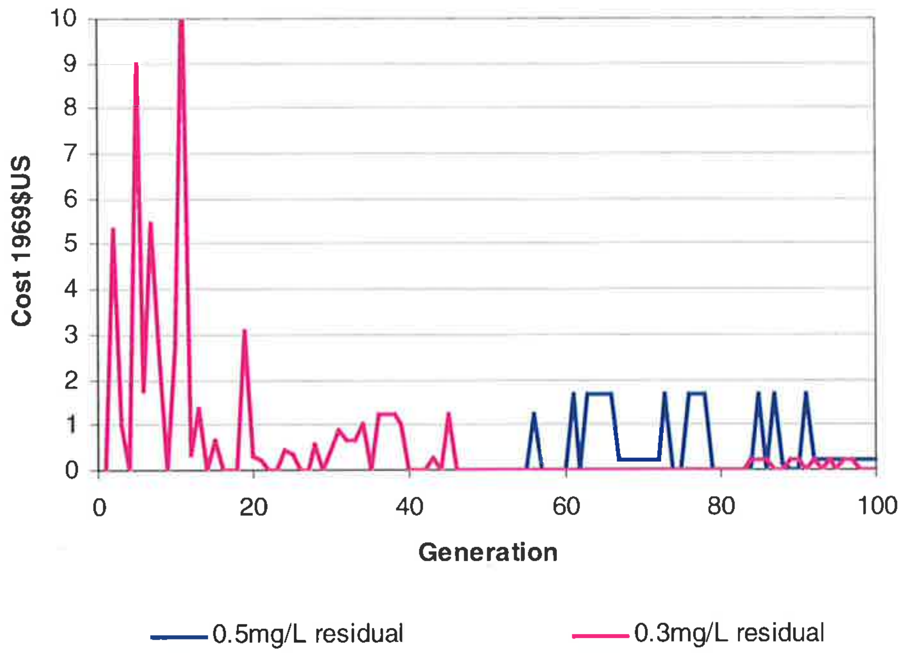

Figure 7.9 Pressure penalty costs for the hydraulic then quality penalty case

There is some large movement away from the zero level in the 0.5mg/L case before convergence to a penalty of $0.20m after 90 generations. The 0.3mg/L case has a higher value placed on hydraulic costs, so the solution set experiments with the possible outcomes attainable without sacrificing pressure levels before moving to the $39.21m solution with a $0.20m pressure penalty.

The reason why the 0.5mg/L solution shows a greater readiness to search for solutions that may attract a pressure penalty is evident in Figure 7.10 showing the respective quality penalties.

Achieving a minimum level of 0.5mg/L under strict dosing guidelines is considerably more difficult to attain than a 0.3mg/L level. If the system finds that due to customer complaint or disinfection costs it can only maintain a level of 0.25mg/L at one of the nodes, the quality penalty for the 0.5mg/L will be considerably greater than that for the 0/3mg/L case. This is due to the relative gap between the attained and desired levels.

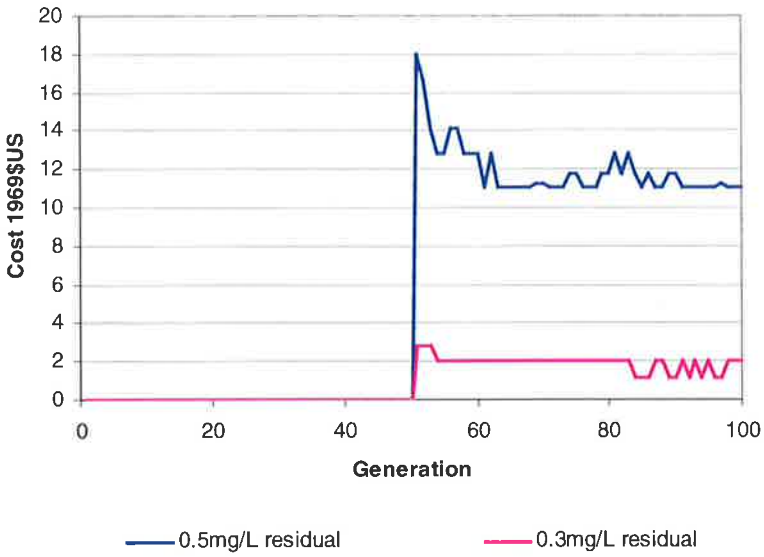

Figure 7.10 Quality penalty costs for the hydraulic then quality penalty case

The optimal hydraulic case of $38.80m attracts nearly twice the penalty of the $39.21m solution for the 0.3mg/L case. The 0.5mg/L run has penalties up to ten times the 0.3mg/L case, which makes the model's desire to determine the solutions that best-satisfy water quality ten times more urgent as the pressure penalty structure for the two runs is identical. Part of this search for the preferred solution set is the disinfection cost analysis shown in Figure 7.11.

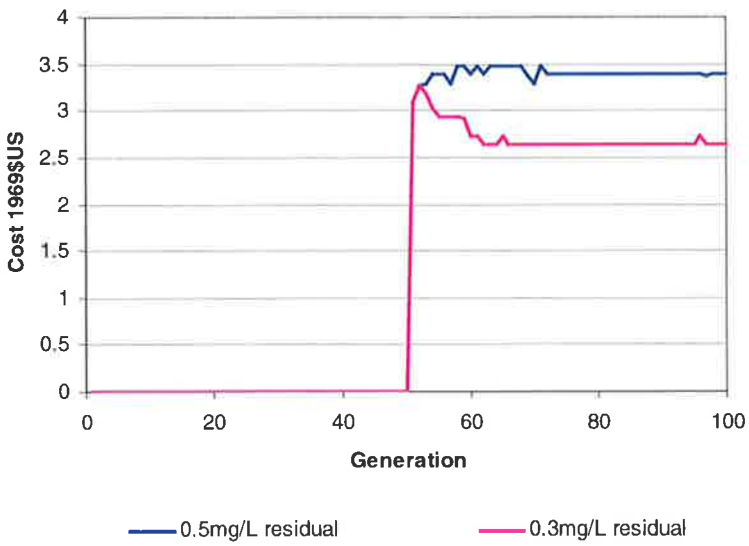

Figure 7.11 Disinfectant costs for the hydraulic then quality penalty case

Due to the lower required residual level, the disinfection costs for the 0.3mg/L run are lower than the 0.5mg/L case.

To check this method to see if it was finding the efficient solution set and not by-passing potentially better solutions, the model was adjusted to combine the quality and hydraulic costs for the complete optimisation.

7.3.1.2 Application of full quality penalty analysis

By determining the solution set with a full analysis, the model spreads its focus amongst the different cost components. This approach is more complex as it has an increased workload to achieve efficient solutions.

Figure 7.12 shows the convergence pattern for the total cost structure using this technique.

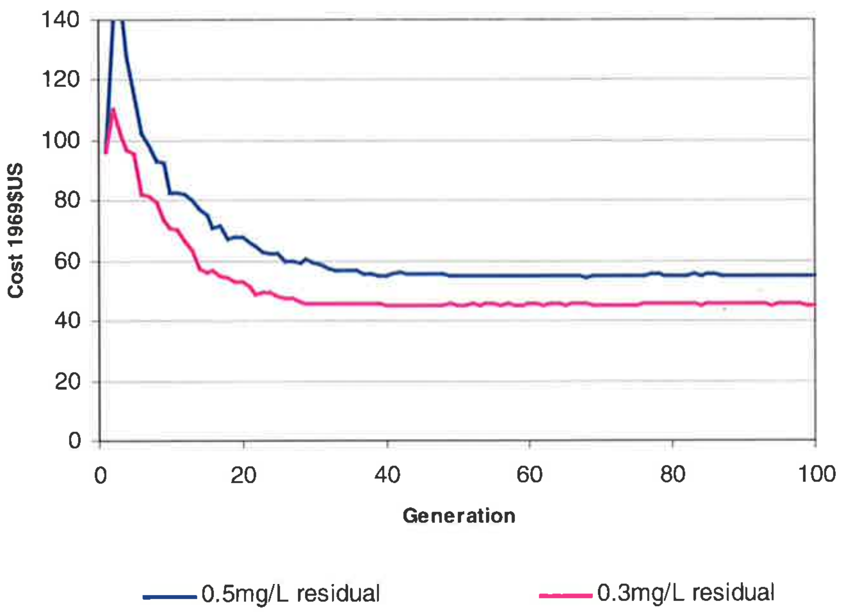

Figure 7.12 Total cost of best solution for the full quality penalty optimisation

The solution set follows the typical convergence pattern down to the efficient solution space. The problem with a residual of 0.5mg/L is it attracts a higher cost than the 0.3mg/L residual due to the additional quality penalties as discussed previously. The construction costs for each of the runs is shown in Figure 7.13.

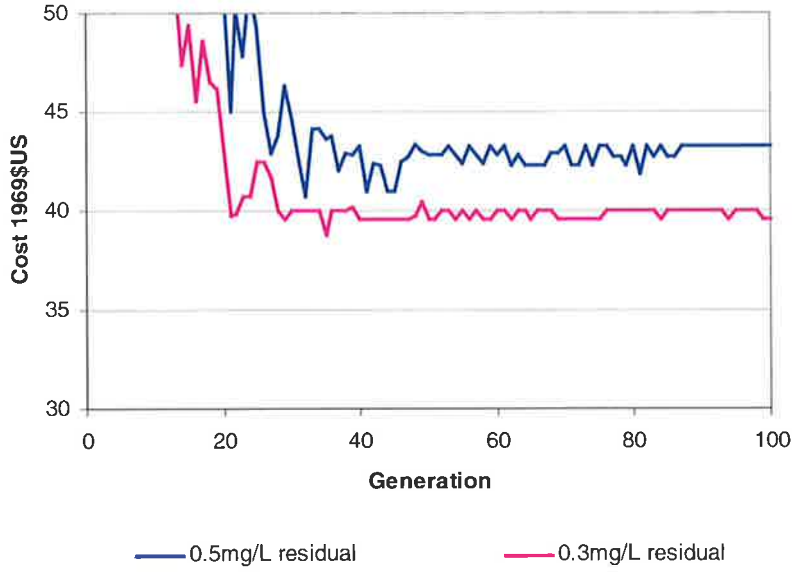

Figure 7.13 Construction cost for the full quality penalty optimisation

The 0.5mg/L solution set never comes close to the optimal hydraulic solution range. The best solution has a construction cost of $42.71m cost. The 0.3mg/L case drops to a construction cost of $39.6m, but fails to drop to the hydraulically efficient range as well. Both of these solutions attract a pressure penalty so are technically infeasible for hydraulics as shown in Figure 7.14.

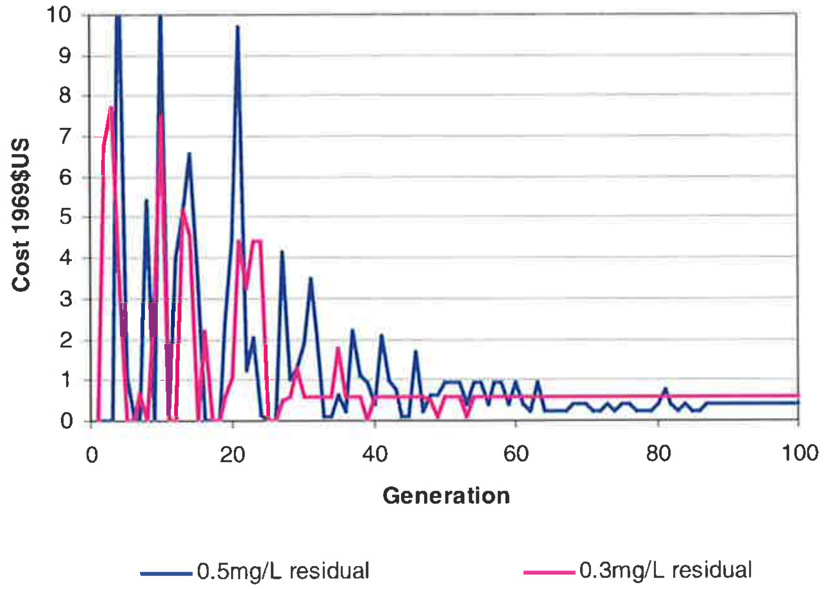

Figure 7.14 Pressure penalty costs for the full quality penalty optimisation

The pressure penalties represented in Figure 7.14 show convergences to better solutions however fail to completely remove the pressure deficit.

The water quality costs are shown in Figures 7.15 and 7.16 for the quality penalties and the disinfection costs respectively. As was represented with the results in Section 7.3.1.1, the 0.5mg/L solution set had a greater penalty and required higher dose levels during different times of the year to try and meet the minimum required residual when compared to the 0.3mg/L run.

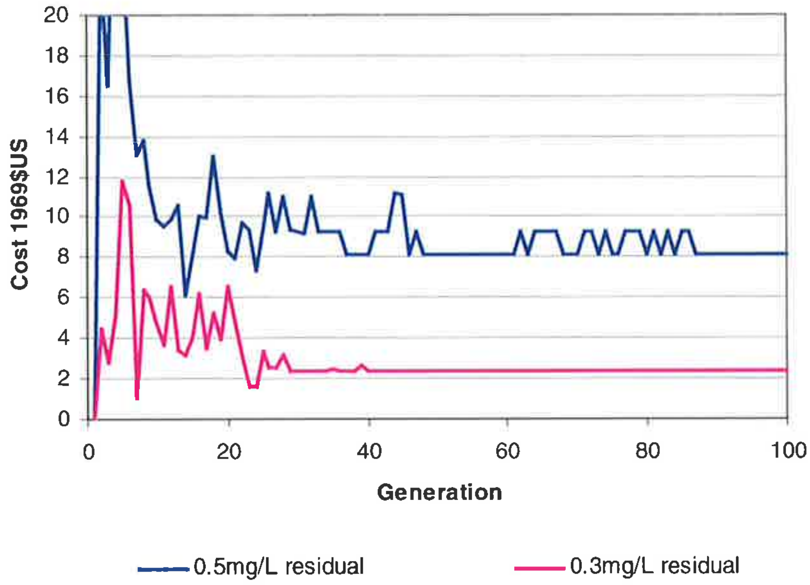

Figure 7.15 Quality penalty costs for the full quality penalty optimisation

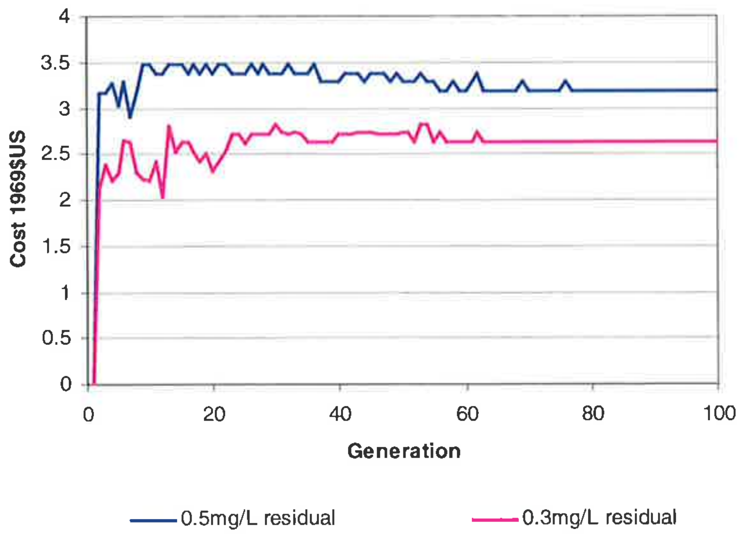

Figure 7.16 Disinfectant costs for the full quality penalty optimisation

Both of these graphs illustrate convergence before the 100 generation (100,000 evaluation) mark. The results from this analysis proved to differ from the previous two-stage process, with the hydraulic then quality method providing better overall solutions for both runs.

7.3.1.3 Discussion of Quality Penalty Results

Comparing the two methods of optimisation, the hydraulic then quality approach seems to provided a better solution set. It also has the advantage of been a potentially faster process as the simulation duration for the hydraulic part of each does not need to have the length of the quality analysis. Table 7.3 lists the different costs for the best solution from each of run.

Table 7.3 Comparison of different costs for quality penalty analysis

Run Case	Construction Cost ($m)	Pressure Penalty ($m)	Quality Penalty ($m)	Disinfection Cost ($m)	Total ($m)
0.5mg/L minimum residual					
Hydraulic Optimal	38.80	0	12.81	3.38	54.98
Hydraulic then Quality	39.21	0.20	11.08	3.38	53.87
Hydraulic and Quality	42.71	0.23	9.28	3.19	55.41
0.3mg/L minimum residual					
Hydraulic Optimal	38.80	0	2.02	2.63	43.45
Hydraulic then Quality	39.21	0.20	1.16	2.63	43.21
Hydraulic and Quality	39.60	0.56	2.39	2.63	45.19

For both of the residual requirements, the hydraulic optimal attracts quality costs which put it behind the more efficient solutions found using the hydraulic then quality analysis. Naturally, if the penalties are altered, the hydraulic solution may be able to become the preferred option.

From Table 7.3, it is clear that for a population of 1000 over 100 generations, the results from the hydraulic and quality analysis are worse than the hydraulic then quality method. This will be discussed further in Section 7.3.1.4.

The pipe diameters for the different runs are listed in Table 7.4. What these results indicate is that the best quality penalty solution for the 0.3mg/L-1.8mg/L range and the 0.5mg/L-1.8mg/L range corresponds to the preferred result of the social cost analysis.

In this solution, pipe 15 is expanded to 144 inches, whereas pipe 16 is decreased to 72 inches with regard to the optimal solution for the pure hydraulic case.

Table 7.4 Solutions for the different quality penalty cost structures

Pipe Number	Hydraulic then Quality 0.5mg/L	Hydraulic and Quality 0.5mg/L	Hydraulic then Quality 0.3mg/L	Hydraulic and Quality 0.3mg/L
Construction Cost	$39.21m	$42.86m	$39.21m	$39.60m
Total Cost	$53.87m	$54.51m	$43.21m	$45.19m
1	0	0	0	108
2	0	0	0	0
3	0	0	0	0
4	0	0	0	0
5	0	0	0	0
6	0	0	0	0
7	0	84	0	96
8	0	0	0	0
9	0	0	0	0
10	0	0	0	0
11	0	0	0	0
12	0	0	0	0
13	0	0	0	0
14	0	0	0	0
15	144	168	144	0
16	72	60	72	84
17	96	96	96	96
18	84	84	84	84
19	72	60	72	72
20	0	0	0	0
21	72	84	72	72

7.3.1.4 Expanded Analysis

There is some concern that the GA examining the quality and hydraulic concerns simultaneously does not encompass a wide enough search space to attain the more cost-efficient solutions determined using the hydraulic optimisation followed by a quality analysis. To examine this, the model was run again with a population size of 2000, which is double the previous population used. The optimisation package considered the quality penalty cost structure for both the 0.5mg/L and the 0.3mg/L minimum residuals.

The results for these runs are represented in Figure 7.17, that shows the total costs, Figure 7.18, that shows the construction costs, Figure 7.19, pressure penalty costs and Figure 7.20, which shows the quality penalty costs.

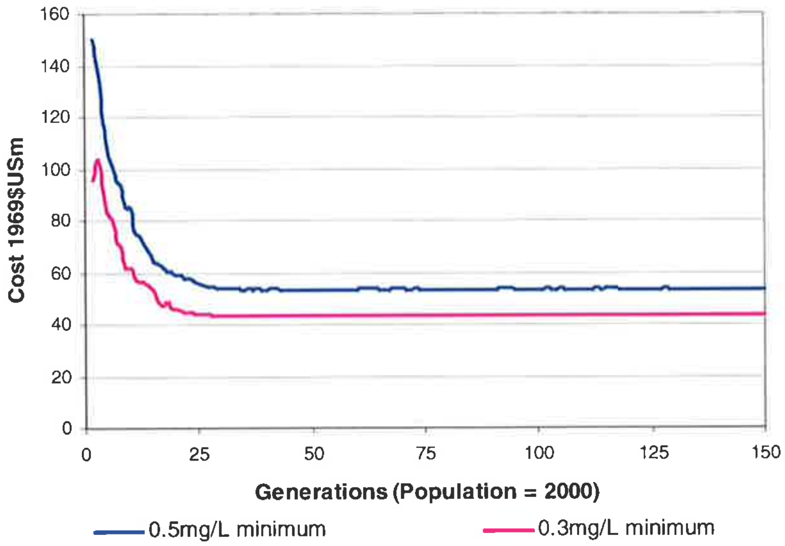

Figure 7.17 Total cost regime for population = 2000

The \$53.21m and \$43.11m solutions for the expanded analysis of the 0.5mg/L set and the 0.3mg/L set are easily preferable values to the \$55.41m and \$45.19m solutions from the previous shorter runs using this method. They also eclipse the values determined using the hydraulic then quality method with costs of \$53.87m and \$43.21m respectively. Table 7.5 compares the different solutions.

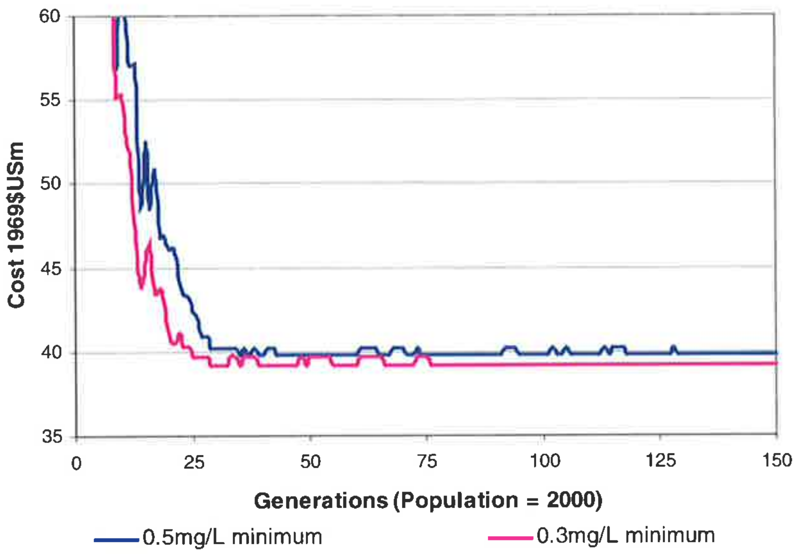

Figure 7.18 Construction costs for population = 2000

The construction costs from Figure 7.18 show that for a 0.5mg/L minimum residual, the construction cost selected was $39.78m (described in Figure 7.21), which with a preferable residence time achieved better quality conditions throughout the network. The 0.3mg/L solution opted for the $39.21m solution, which marginally improved its dosing regime to attain a better result. The 0.3mg/L case did not use the $39.78m solution, as it did not have as heavy a basis on the quality costs. Table 7.5 compares these results.

Table 7.5 Comparison of population sizes 1000 and 2000 for penalty analysis

Cost $m	Population = 1000		Population = 2000	
	0.3mg/L minimum	0.5mg/L minimum	0.3mg/L minimum	0.5mg/L minimum
Total	$45.19m	$55.41m	$43.11m	$53.21m
Construction	$39.60m	$42.71m	$39.21m	$39.78m

The solutions found from the expanded analysis differed dependant on the minimum required residual. The construction cost of the 0.3mg/L residual solution was $39.21m, which matched the previous solutions, however, the 0.5mg/L residual solution had a construction cost of $39.78m, which had a total cost regime which was preferable to any of the previously determined solutions. Figure 7.19 illustrates the pressure penalties for a population of 2000, whereas Figure 7.20 shows the quality penalties for each run.

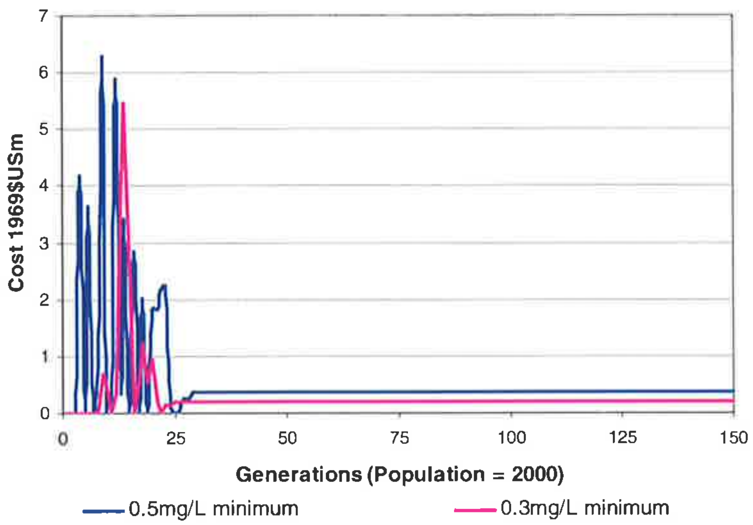

Figure 7.19 Pressure penalties for population = 2000

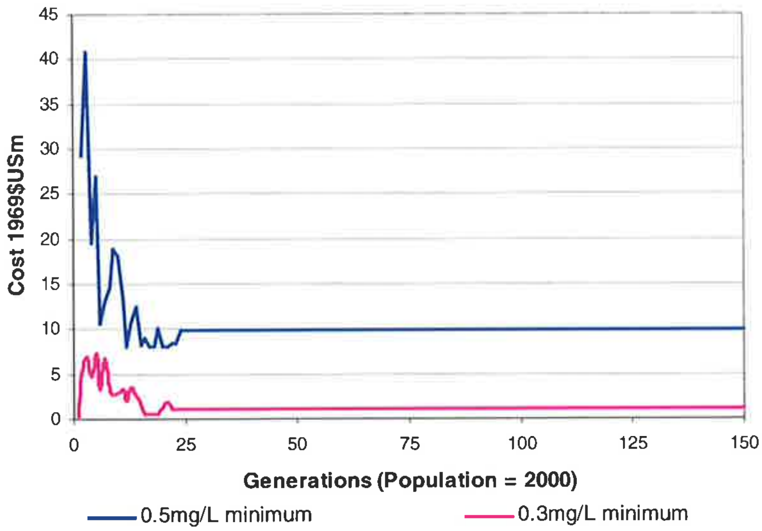

Figure 7.20 Quality penalties for population = 2000

The pipe sizes and layout of the two solutions can be seen in Figure 7.21.

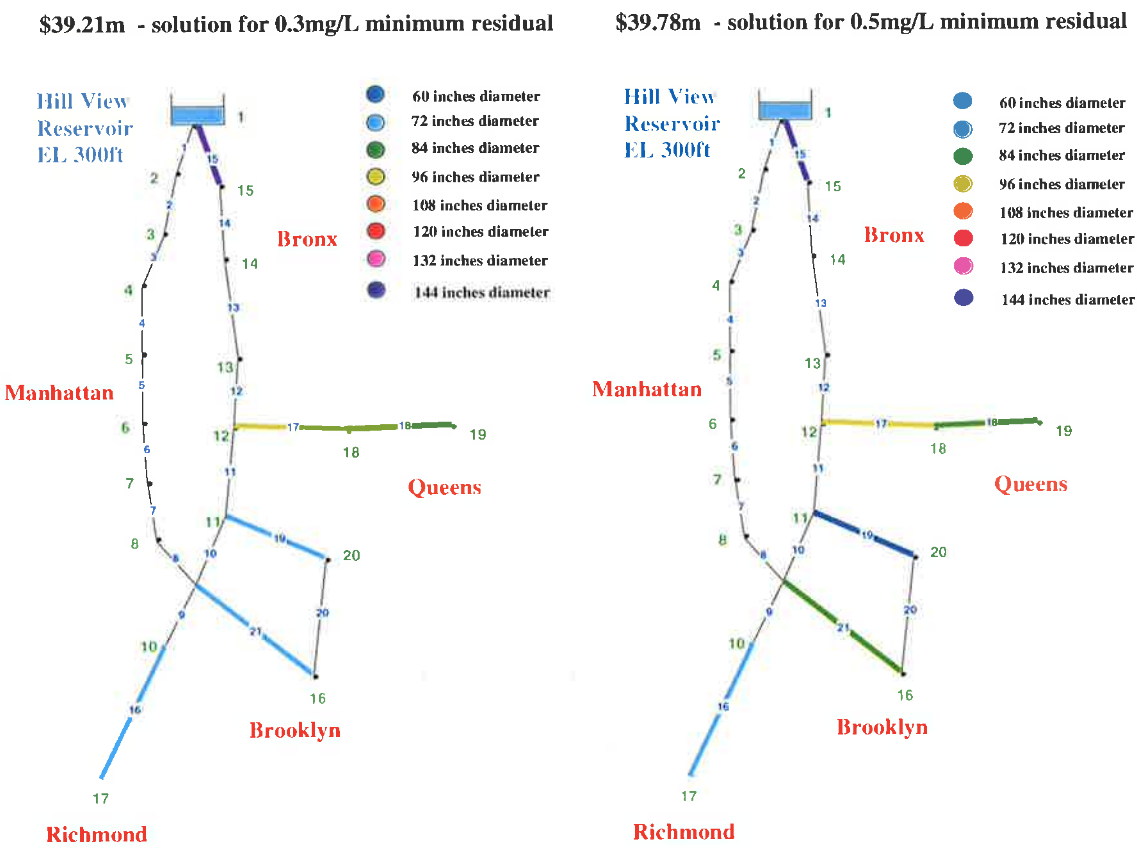

Figure 7.21 Efficient solutions for extended penalty analysis

The difference between the two solutions is seen in pipes 19 and 21. The \$39.21m solution has diameters of 72 inches for both of these pipes, whereas the \$39.78m solution has pipe diameters of 60 inches and 84 inches respectively.

The penalty cost structure provided good convergence on the different cost factors. The penalty cost factors were minimised and the physical make-up of the systems determined was not drastically different from the optimal results of prior researchers. The extended full run found the best result field, however, the combination of hydraulic analysis followed by the combined analysis provided efficient results in considerably shorter run time.

To provide a second option for the analysis of water quality alongside hydraulics, the application of social cost have been tested for the New York Tunnels study as well.

7.3.2 Application of Social Cost Structure

The application of the total social cost structure considers the relationships between the following potential system costs:

- Construction cost;
- Pressure penalty cost;
- Cost of disinfectant;
- Willingness to pay for improvement of taste and odour;
- Potential costs due to waterborne disease risk; and
- Potential costs resulting from exposure to disinfection by-products.

7.3.2.1 Hydraulic Solution followed by Quality Optimisation

This run optimises purely for hydraulics for the first 50 generations (50,000 evaluations), then, includes the water quality costs for the remaining 50 generations. In doing this, the hydraulic optimum of \$38.8m is reached and quality costed, so a better picture of the potential of the solution found in Section 7.3.1 can be determined.

The total cost structure for this method is illustrated in Figure 7.22. As seen, the solution optimises to the \$38.8m level and has converged after 45 generations. Then, with the inclusion of the water quality costs, the solutions fluctuate until they achieve convergence at a level of \$39.21m for the construction costs seen in Figure 7.23.

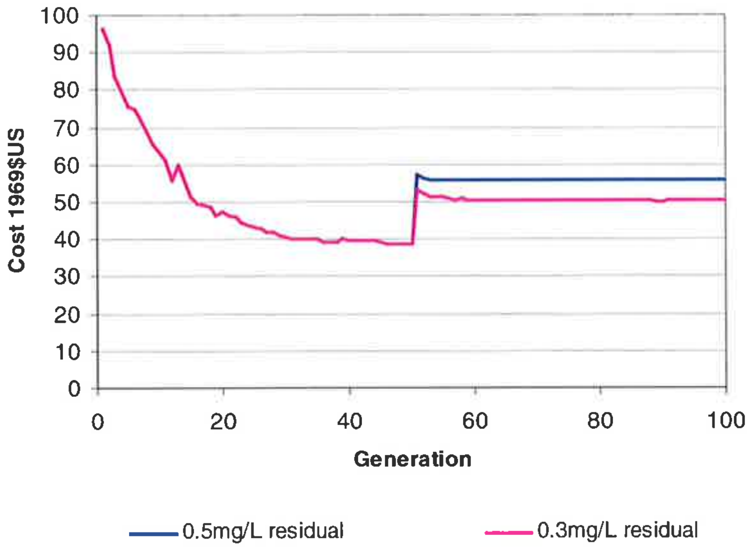

Figure 7.22 Total cost convergence for hydraulic, then quality optimisation

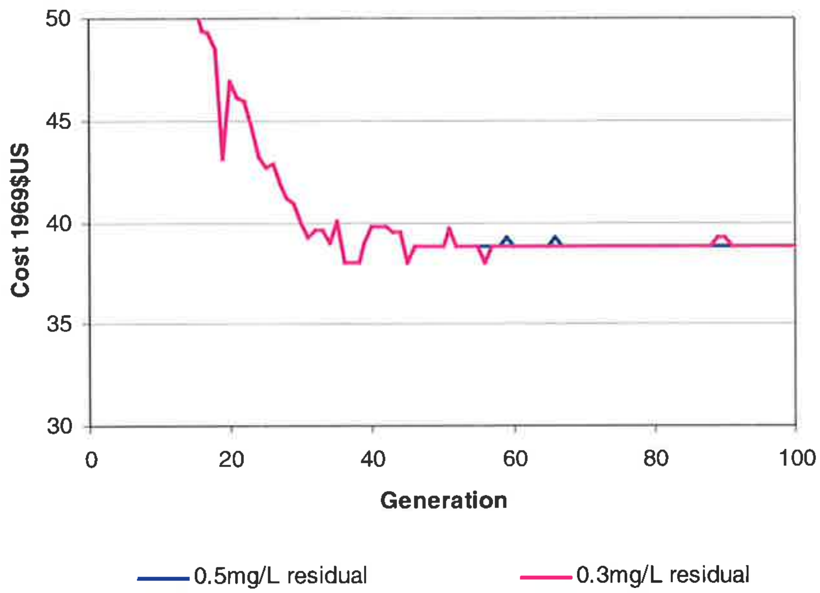

Figure 7.23 Construction costs for hydraulic, then quality optimisation

The convergence of the construction costs (Figure 7.23) shows how the hydraulic optimal solution of \$38.80m is found in the first 50 generations. There is then a series of fluctuations until the \$38.80m solution is reached again. At the 90 generation mark, the construction costs rise to a \$39.21m level which corresponds to the observable dip in the total cost analysis.

The pressure penalty cost analysis (Figure 7.24) also illustrates this deviation from the hydraulic optimum.

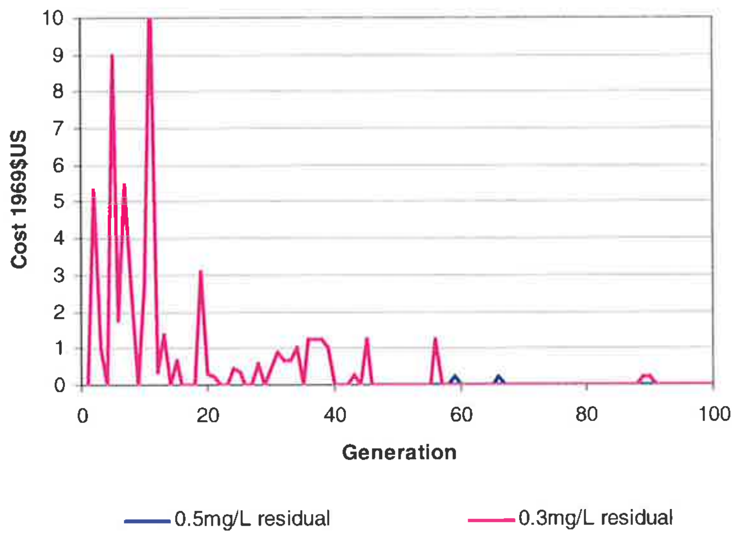

Figure 7.24 Pressure penalty costs for hydraulic optimisation then quality analysis

The different water quality costs are shown in Figures 7.25 to 7.27, depicting the potential waterborne disease costs (Figure 7.25), potential costs due to disinfection by-products (Figure 7.26) and the costs due to the disinfection process (Figure 7.27).

The costs due to taste and odour are not shown on a figure, as the best solution set did not have any costs after the 50-generation mark, so a line on the x-axis could represent it.

What are noticeable from Figures 7.25 to 7.27 are the relationships between the curves. Figures 7.26 and 7.27 representing disinfection by-product cost and disinfection cost follow a very similar pattern, whereas the waterborne disease cost curve from Figure 7.5 tends to follow a different line.

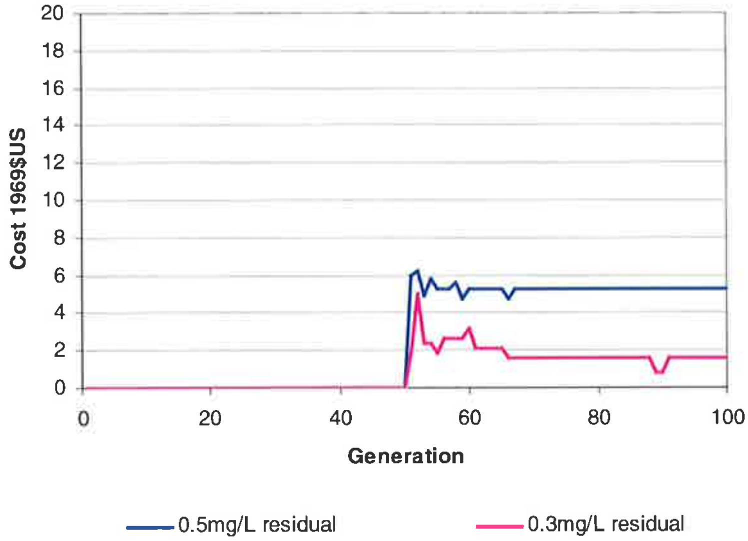

Figure 7.25 Waterborne disease costs for hydraulic then quality analysis

Figure 7.25 shows the waterborne disease costs for these runs. The 0.5mg/L run has higher costs than the 0.3mg/L run as the potential shortfall in residual level can be a lot more significant.

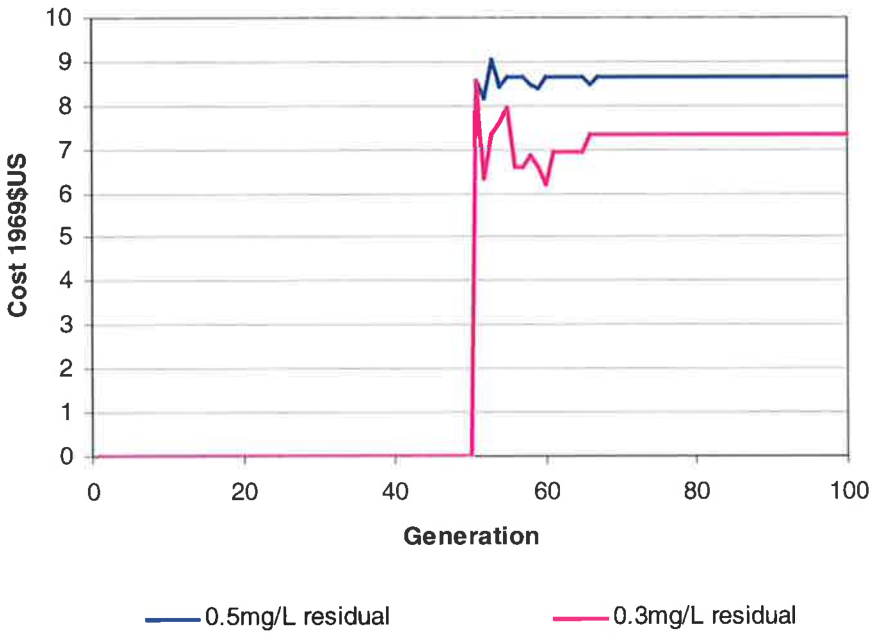

Figure 7.26 Disinfection by-product costs for hydraulic then quality analysis

The potential disinfection by-product costs (Figure 7.26) jump around after the 50-generation mark, but settle out soon afterward. The disinfection costs (Figure 7.27) follow a similar curve.

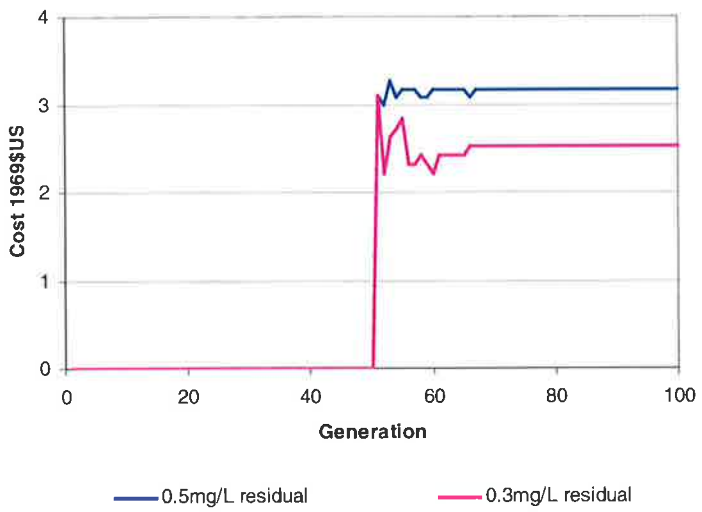

Figure 7.27 Disinfection costs for hydraulic then quality analysis

Both of the social-cost regimes using the hydraulic then combined technique produced a result with a construction cost of $39.21m. This result was discussed in Section 7.3.1. To provide a different angle on these results, the social cost method has been adjusted to run over the entire simulation in conjunction with the hydraulic analysis.

7.3.2.2 Combined social cost and hydraulic analysis

The combined cost regime produces an optimisation run based on the inclusion of all of the social costs and the hydraulic costs simultaneously from the first generation. The total cost analysis for all of these factors can be seen in Figure 7.28.

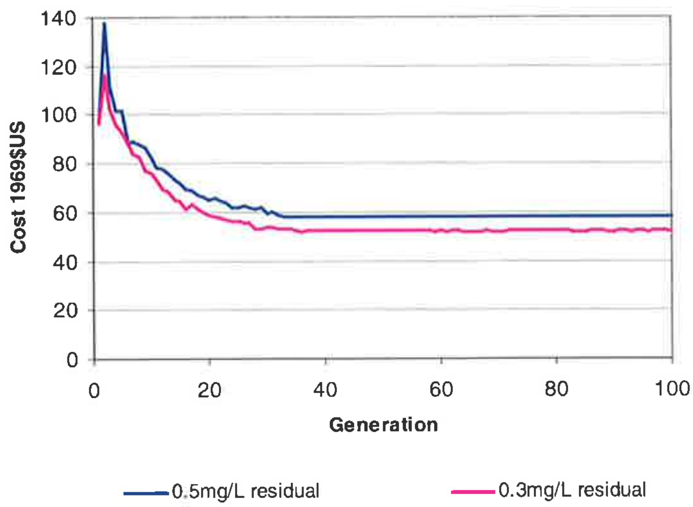

Figure 7.28 Total cost regime for quality social cost and hydraulic optimisation

Evident from Figure 7.28 is the convergence of the total cost regime. The 0.3mg/L minimum residual example has a more variant convergence pattern due to the less severe cost regime imposed when compared to the 0.5mg/L example. The 0.5mg/L case aims to restrict residual levels even further so the costs applied are greater, hence can force a shallow convergence which is evident in Figure 7.29 examining the construction costs.

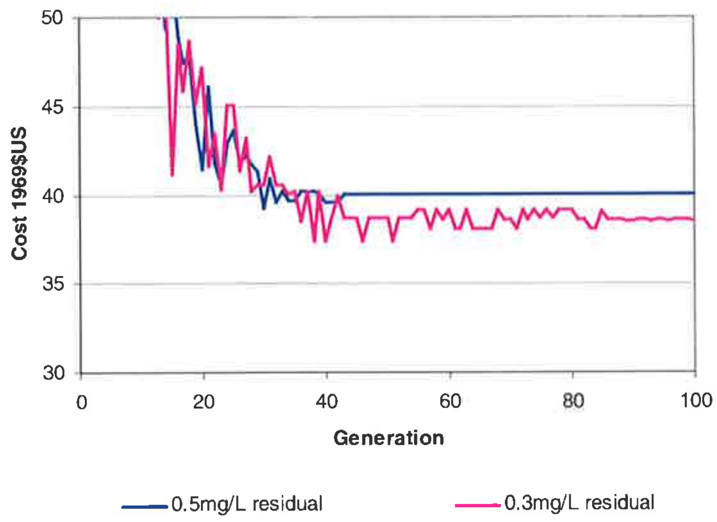

Figure 7.29 Construction costs for full optimisation

The 0.5mg/L example converges to a construction cost of \$40.07m, which is considerably higher than the \$38.64m observed for the \$0.3mg/L solution. The \$40.07m result satisfies the pressure demands of the system as shown in Figure 7.30, whereas the \$38.64m result suffers a \$0.27m pressure penalty (less than one inch head) which is seen for generations 69-71 on Figure 7.30.

As with the quality analysis, the full optimisation produced results that are less efficient than the hydraulic run followed by the quality and hydraulic examination. However, the comparative results for this type of run between the social cost formulation and the quality penalty method indicate the potential of the social cost method.

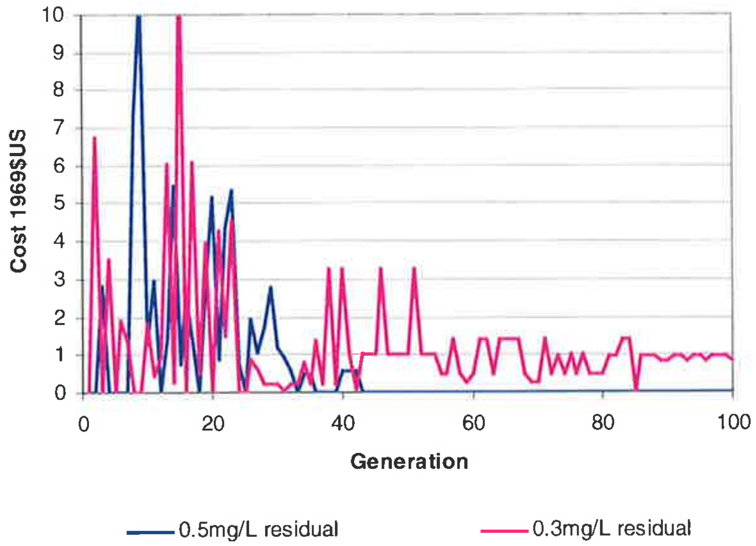

Figure 7.30 Pressure penalty costs for full optimisation

The water quality costs for the two cases are shown in Figures 7.31 to 7.34. Figure 7.31 shows the potential waterborne disease costs attributable to the solutions presented over 100 generations. The waterborne disease costs for the 0.3mg/L solutions tend to be considerably lower than for the 0.5mg/L solutions. This is because a residual level of 0.2mg/L for example would suffer a much more severe cost if 0.5mg/L were required than 0.3mg/L. Chapter 5 formulates the cost structure for waterborne disease.

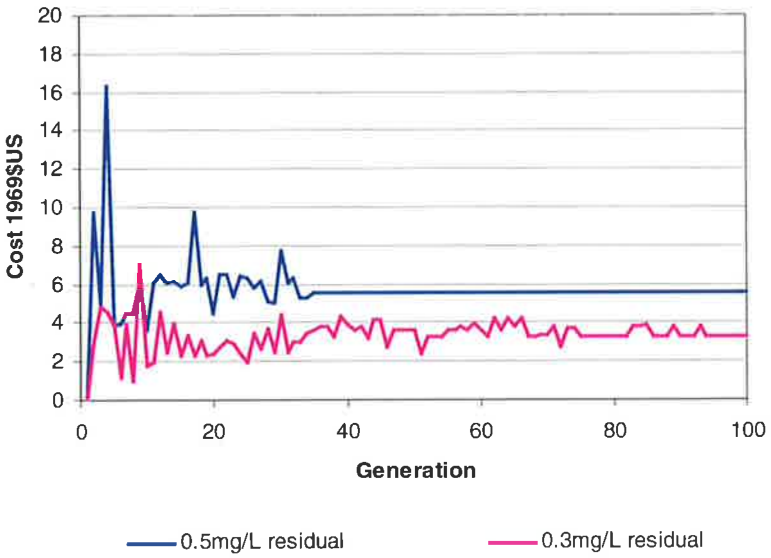

Figure 7.31 Waterborne disease costs for complete optimisation

Figure 7.32 examines the potential costs due to excessive disinfection by-product levels. Again the 0.5mg/L solutions are in a higher cost bracket, as a greater level of chlorine dosing occurs to aim for the higher residual levels. The disinfection by-product costs follow a similar curve to the disinfection costs seen in Figure 7.33, as the production of disinfection by-products is directly related to the amount of disinfection consumption.

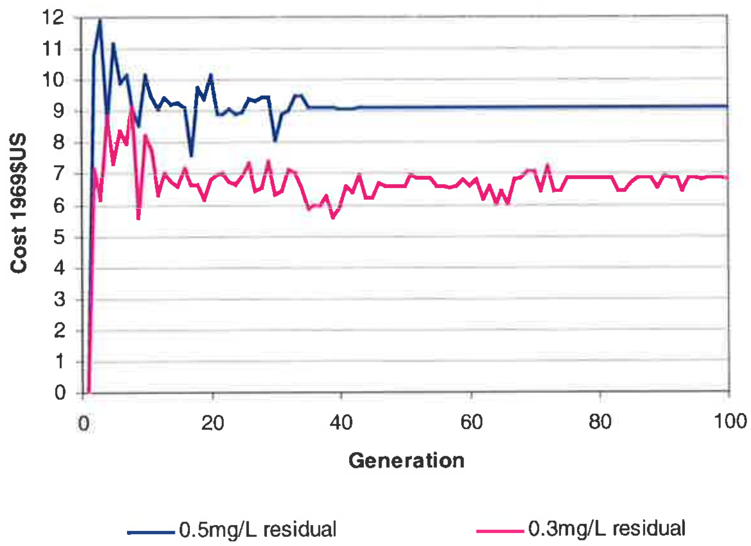

Figure 7.32 Potential disinfection by-product costs for full optimisation

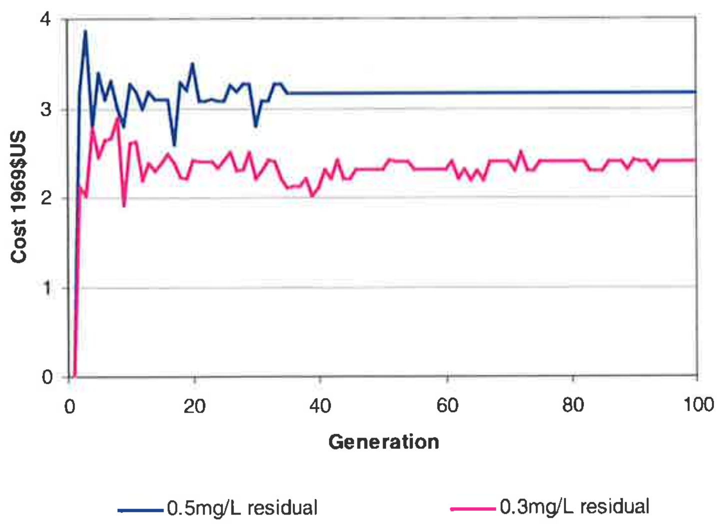

Figure 7.33 Potential disinfection costs for full optimisation

The costs due to taste and odour are eliminated very quickly from the analysis. The level of taste and odour was set at 1.8mg/L as this produces levels of 1.5-1.7mg/L in the upper system, which could cause complaint. Figure 7.34 shows how the solutions that gain a cost for taste and odour are quickly dispatched from the optimisation.

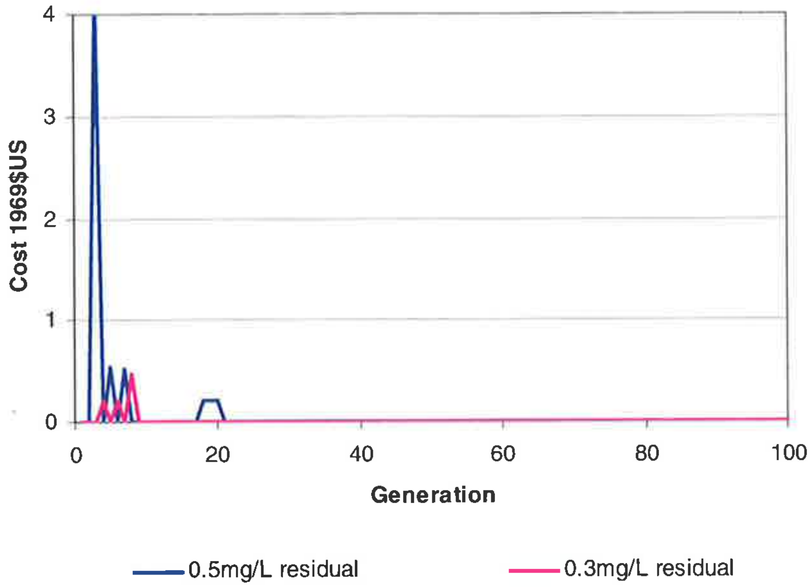

Figure 7.34 Taste and odour costs for full optimisation

The potential costs due to waterborne disease and disinfection by-products have an inverse relationship with each other. If a small amount of disinfectant is added to the supply, the DBP costs will be low as there is not much disinfectant to form the by-products, whereas the waterborne disease costs could be high, as there is little or no residual left in the distribution system.

If a large amount of disinfectant is added, sanitation of potential waterborne disease is achieved, but considerable DBPs are formed posing a potential health risk.

The analysis of the social cost components of the total cost structure provided an insight into some of the relationships between the water quality factors. These relationships are listed in Table 7.6.

Table 7.6 Relationships of different cost factors in social cost structure

	Construction Cost	Pressure Penalty	Taste and Odour	Disinfectant	Disinfection By-products	Waterborne Disease
Construction Cost		Inverse Relationship	Direct Relationship	Direct Relationship	Direct Relationship	Direct Relationship
Pressure Penalty	Inverse Relationship		Inverse Relationship	Inverse Relationship	Inverse Relationship	Inverse Relationship
Taste and Odour	Direct Relationship	Inverse Relationship		Direct Relationship	Direct Relationship	Inverse Relationship
Disinfectant	Direct Relationship	Inverse Relationship	Direct Relationship		Direct Relationship	Inverse Relationship
Disinfection By-products	Direct Relationship	Inverse Relationship	Direct Relationship	Direct Relationship		Inverse Relationship
Waterborne Disease	Direct Relationship	Inverse Relationship	Inverse Relationship	Inverse Relationship	Inverse Relationship	

The pressure penalty cost decreases when the construction cost increases, as larger diameter pipes result in a lower head loss. Due to the increased construction costs, the residence time in the network is increased, which results in more disinfectant being required to sanitise the system, which results in a greater potential cost due to higher DBP levels. If the disinfectant level remains constant, the risk of waterborne disease will escalate, as less residual is in the system, which results in increased costs. DBP costs will also rise as the increased residence time creates greater consumption of disinfectant. Taste and odour cost will probably be better, as the water has less disinfection to give an odour to the consumer.

The waterborne disease costs are inversely related to the disinfection by-product potential costs, as the greater the level of disinfectant results in lower waterborne disease costs, but higher DBP costs. If little or no disinfectant is added, DBP potential costs will be marginal, if existing, whereas, the waterborne disease risk of the supply will escalate due to the lack of a residual.

A summary of the solutions for the different optimisation paths is listed in Table 7.7.

Table 7.7 Solutions for the different social cost structures

Pipe Number	Hydraulic then Quality 0.5mg/L	Hydraulic and Quality 0.5mg/L	Hydraulic then Quality 0.3mg/L	Hydraulic and Quality 0.3mg/L
Construction Cost $m	$39.21	$40.07	$39.21	$38.64
Total Cost $m	$55.64	$57.88	$50.07	$51.73
1	0	108	0	0
2	0	0	0	0
3	0	0	0	0
4	0	0	0	0
5	0	0	0	0
6	0	0	0	0
7	0	108	0	144
8	0	0	0	0
9	0	0	0	0
10	0	0	0	0
11	0	0	0	0
12	0	0	0	0
13	0	0	0	0
14	0	0	0	0
15	144	0	144	0
16	72	84	72	96
17	96	96	96	96
18	84	84	84	84
19	72	72	72	72
20	0	0	0	0
21	72	72	72	72

The solutions for the hydraulic analysis followed by the combined optimisation found better results to the combined approach from the first generation. The combined approach failed to reach the optimal levels, as was the case with the penalty analysis (Section 7.3.1), so the search space was expanded to a population of 2000 and 200 generations (four times the search space). The resultant solutions from this expanded analysis are summarised in Table 7.8.

Table 7.8 Comparison of population sizes 1000 and 2000 for social cost analysis

Cost $m	Population = 1000		Population = 2000	
	0.3mg/L minimum	0.5mg/L minimum	0.3mg/L minimum	0.5mg/L minimum
Total	$51.73	$57.88	$50.28m	$57.54m
Construction	$38.64	$40.07	$39.21m	$38.55m

The different components of the results for the expanded analysis are seen in Figure 7.35 to Figure 7.38. Figure 7.35 shows the total cost convergence for the two runs, Figure 7.36 shows the construction cost convergence, Figure 7.37 shows the pressure penalty analysis of the two solution searches, and Figure 7.38 shows the total social cost convergence path.

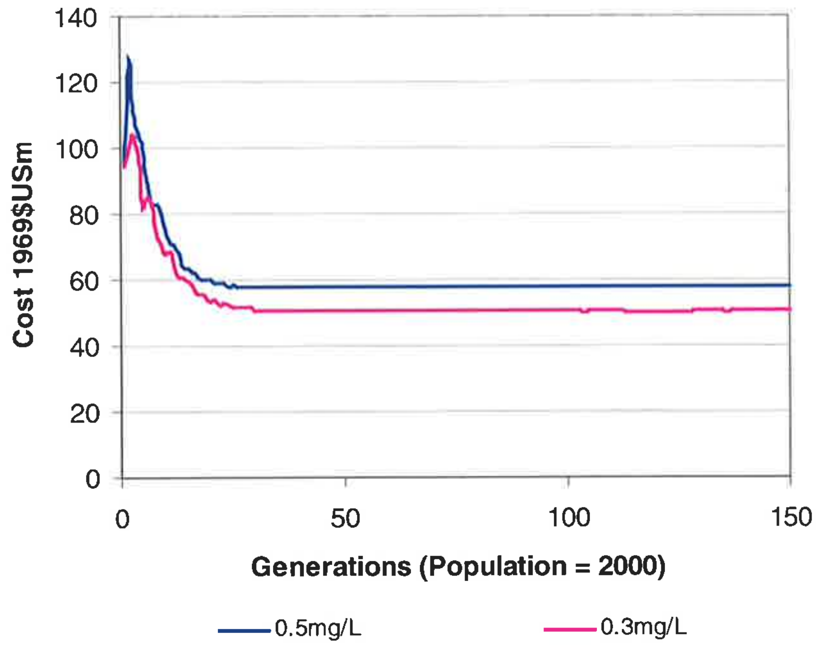

Figure 7.35 Total cost convergence for population = 2000

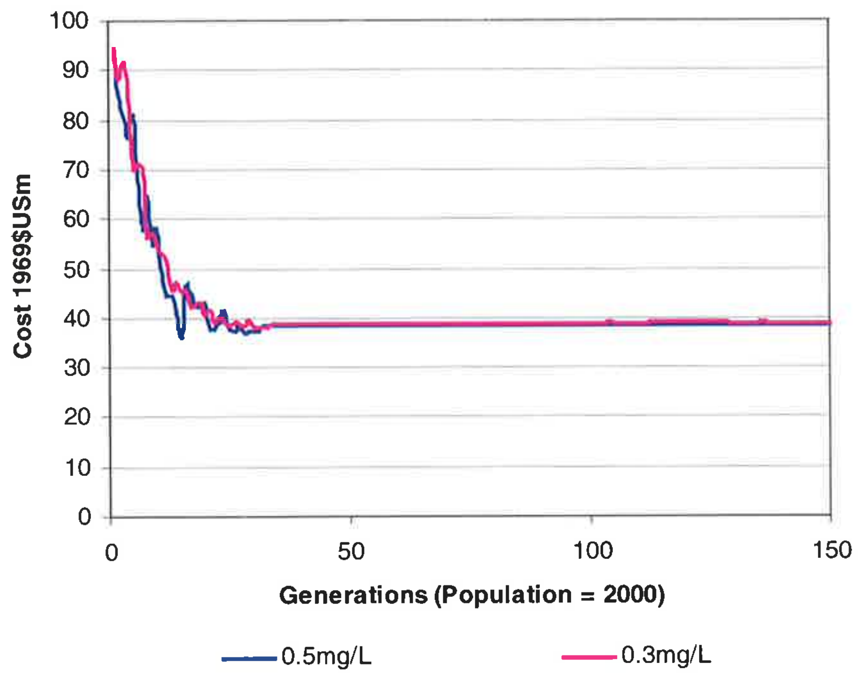

Figure 7.36 Construction cost convergence for population = 2000

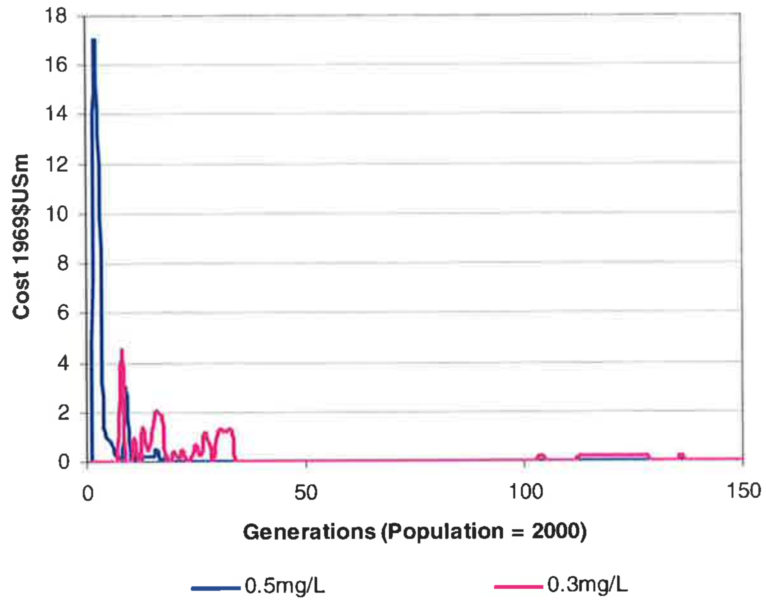

Figure 7.37 Pressure penalty convergence for population = 2000

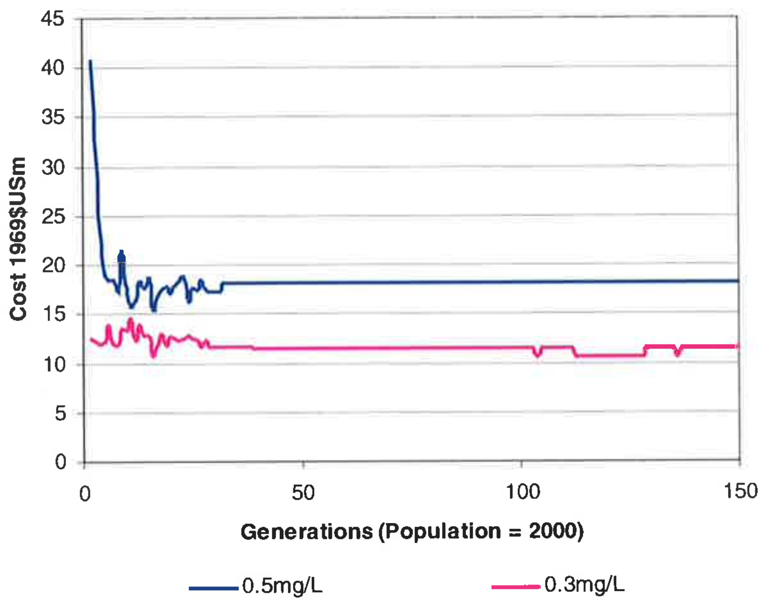

Figure 7.38 Combined social cost convergence for population = 2000

The expanded analysis determined the \$39.21m solution for the 0.3mg/L minimum residual, this lends support to this solution being the preferred layout for this quality level. The 0.5mg/L minimum residual analysis introduces a construction cost of \$38.55m, the total cost of \$57.54m is below the $57.88m for the similar run with population = 1000, but well above the hydraulic then social cost run with a total cost of $55.64m and the layout of the $39.21m solution.

To assist in attaining a better concept of the preferred solution, comparison of the social cost and penalty cost methods is implemented.

7.4 Comparison of Social Cost to Penalty Cost Framework

The social cost method and the quality penalty approach are different methods of quantifying the water quality in a system. For a minimum desired chlorine level of 0.5mg/L, the results of the different runs are listed in Table 7.9.

Table 7.9 Quality penalty, social cost and pure hydraulic solutions

	Format of Run					
	Murphy et al. solution	Hydraulic optimum	Hydraulic then quality penalty	Hydraulic then social cost analysis	Expanded Q. penalty & hydraulic	Expanded social cost & hydraulic
Total Cost $	$38.80m	$38.80m	$53.87m	$55.64m	$53.21m	$57.54m
Construction $	$38.80m	$38.80m	$39.21m	$39.21m	$39.78m	$38.55m
Pipe Number	Pipe Diameters (inches) for solutions					
1	0	0	0	0	0	0
2	0	0	0	0	0	0
3	0	0	0	0	0	0
4	0	0	0	0	0	0
5	0	0	0	0	0	0
6	0	0	0	0	0	0
7	0	0	0	0	0	132
8	0	0	0	0	0	0
9	0	0	0	0	0	0
10	0	0	0	0	0	0
11	0	0	0	0	0	0
12	0	0	0	0	0	0
13	0	0	0	0	0	0
14	0	0	0	0	0	0
15	120	120	144	144	144	0
16	84	84	72	72	72	96
17	96	96	96	96	96	108
18	84	84	84	84	84	72
19	72	72	72	72	60	72
20	0	0	0	0	0	0
21	72	72	72	72	84	72

The preferred result found in 2 out of 4 runs was the $39.21m solution. As part of these optimisation runs, the chlorine dosing regime was optimised as well as the pipe system design, to find the balance of all of these quality costs. The seasonal dosing patterns are listed in Table 7.10.

Table 7.10 Recommended dosing regime to attain a 0.5mg/L residual

Season	Recommended Dose (mg/L)
Winter	1.6
Spring	1.8
Autumn	1.8
Summer	1.8

The residual levels experienced at some of the different locations in the system are shown in Figure 7.39. As winter has the lowest decay rate, and the demand during this period is not significantly lower than the rest of the year, the dosing levels for this season are lower than autumn, spring and summer.

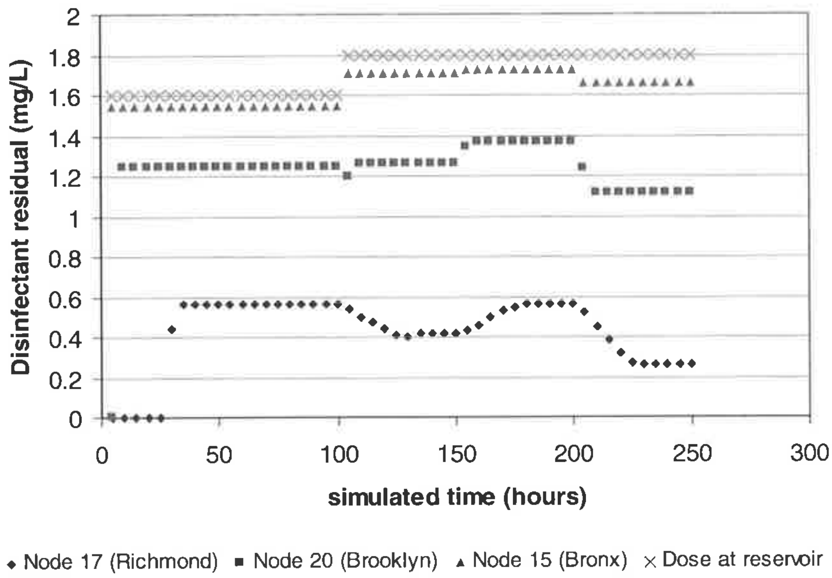

Figure 7.39 Residuals at different New York locations for 0.5mg/L residual

The costs were considered at the end of each season, which were 0-100 simulated hours for winter, 100-150 for spring, 150-200 for autumn, and 200-250 for summer. The reason for this unusual order is that winter is one extreme with the highest residence time and the lowest decay rate, whereas summer has the highest decay rate and lowest residence time. Spring and autumn are usually in between.

If the 0.3mg/L residual is the requirement, then the dosing regime is listed in Table 7.11. There is no need for a table comparing the four different methods for this level, as the $39.21m solution was selected in each case.

Table 7.11 Recommended dosing regime to attain a 0.3mg/L residual

Season	Recommended Dose (mg/L)
Winter	1.0
Spring	1.4
Autumn	1.0
Summer	1.8

The residual pattern throughout the system from the application of this dosing regime is shown in Figure 7.40.

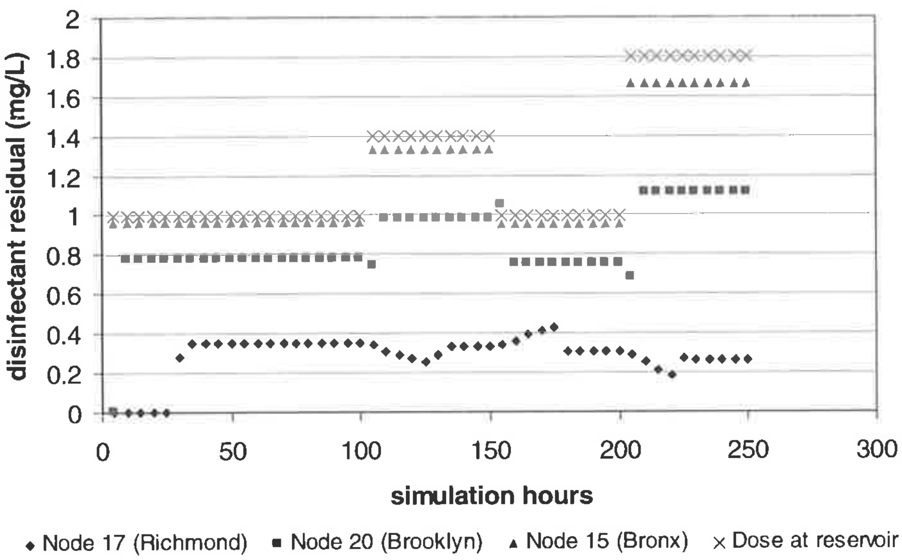

Figure 7.40 Residuals at different New York locations for 0.3mg/L residual

From Figure, there is some fluctuation for Richmond (Node 17) during simulation hours 0-30, 100-125, 150-175, and 200-220. This variation prior to convergence for each season is due to the residence time effect. The simulation experiences a change in decay rate and demand, which leads to different water quality from the dosing station as not only is a new level of disinfectant added, the consumption rate for this disinfectant is altered.

These dosing regimes are seasonal estimates to provide an approximation of the most efficient means of satisfying the water quality situation using the selected pipe system. By combining the dosing and infrastructure aspects, a complete overview is achieved.

7.5 Discussion of New York Tunnels Results

The results obtained for the New York Tunnels study show that the best solution for the combined hydraulic and water quality analysis differs from the optimal solution for hydraulic objective only.

As seen in Figure 7.41, the optimal combined solution of $39.21m construction cost increases the size of pipe 15 and decreases the size of pipe 16, compared to the optimal hydraulic solution of $38.80m. The reason for these alterations is due to the fact that node 17 at Richmond (Staten Island), is the critical node for water quality in the system. As node 17 is situated at the network extremity, it experiences the highest residence times, hence the lowest chlorine residuals.

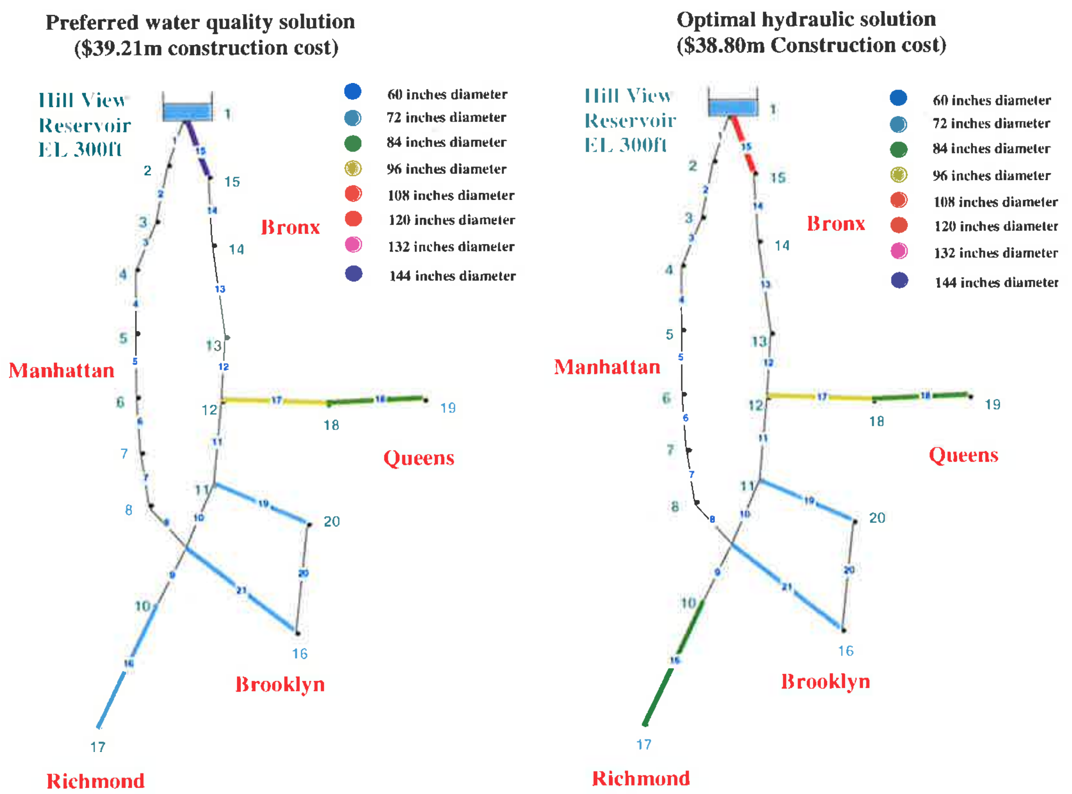

Figure 7.41 Comparison of optimal hydraulic and preferred combined solution

By altering the diameters of two of the pipes, the preferred water quality solution of $39.21m significantly reduces the residence time to Node 17 (Richmond/Staten Island) by about one and a half-hours. This saving in retention time corresponds to a reduction in the amount of disinfectant decay. Figure 7.42 depicts the chlorine residual at Node 17 for a simulation where the initial residual throughout the system is 0.0mg/L. The time to reach a steady state value is an indication of the residence time in the network up to this point. The hydraulic optimal against the solution determined to best fit the quality and hydraulic conditions on the system based on a summer simulation of the two solutions.

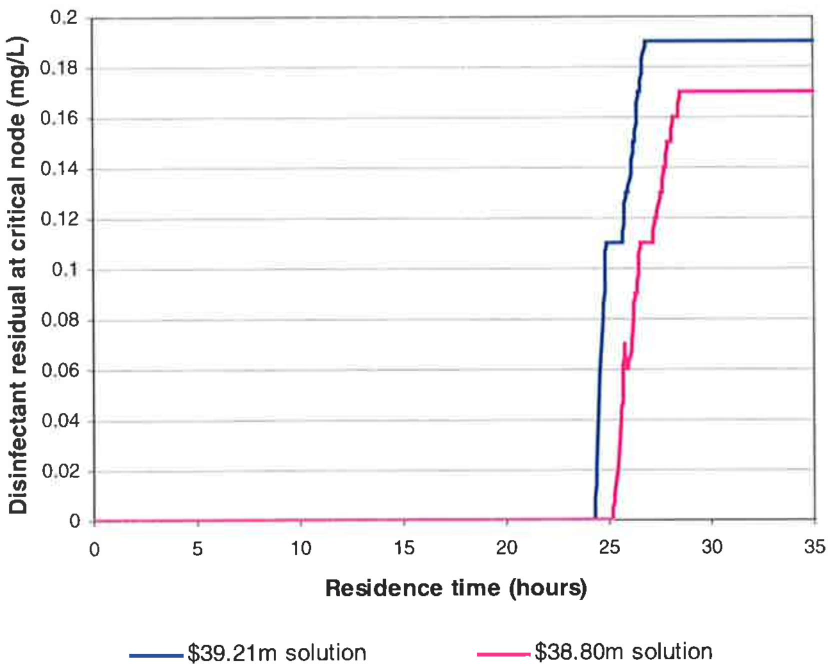

Figure 7.42 Comparison of residence time and residual for critical node (Node 17)

The residence-time savings are amplified during the cooler months due to the greater residence times experienced as consumption is reduced. The $38.80m solution also has lower residual levels at the critical node with 0.17mg/L instead of 0.19mg/L found using the $39.21m solution. Although the savings are slight, it does represent a more efficient solution for the same system constraints, and a 10% difference in residual level at a critical node could be the difference between residual reaching an extremity of the distribution system beyond this point or not.

The residual levels converge to a horizontal line, this is due to the assumption of a constant daily supply level. If the supply nodes were end user points rather than storage locations for further distribution, the convergence pattern would follow the daily water consumption pattern to a degree. However, for this study, the nodes are locations of redistribution, which justifies the assumption of a constant daily supply quota.

The EPANET solution set for the flows through the different duplicate pipes from this simulation have been listed in Appendix F.2. Table 7.12 lists the relevant data for pipe 16 connecting the critical node (node 17), which is the major contributor to the difference in residence times between the two solutions of \$38.80m and \$39.21m.

Table 7.12 Use of duplicate pipes for the different solutions

Hydraulic optimal solution of \$38.80m						Preferred quality and hydraulic solution of \$39.21m					
Pipe	Diam	Flow	Vel	HL	Cl	Pipe	Diam	Flow	Vel	HL/m	Cl
	in	gpm	ft/s	ft/kft	mg/L		Inches	gpm	ft/s	ft/kft	mg/L
D16	84	14715.6	0.85	0.05	0.21	D16	72	12261.4	0.97	0.07	0.26
16	72	9807.2	0.77	0.05	0.2	16	72	12261.4	0.97	0.07	0.26

The bulk of the residence time saving between the two solutions is a result of the increased velocity through both pipe 16 and duplicate pipe 16. The difference in velocity is 0.97ft/sec for both pipes in the \$39.21m solution as compared to 0.77ft/sec and 0.85ft/sec for the pipe and duplicate pipes respectively in the \$38.80m solution. This represents a potential time saving in water age over the 26,400 feet of the system of 1.47 hours for the existing pipe and 0.88 hours for the duplicate pipe.

This also explains the staggered convergence of the residual level. Figure 7.42 illustrates this with a plateau of approximately 1-hour for both solutions halfway up the movement from 0mg/L to the fully converged level. This is due to the water reaching from one of the pipes, but yet to reach from the other pipe.

7.5.1 Analysis of Importance of Residence Times

To analyse the concept that residence times dictate the potential of the solution set, the model was re-programmed to optimise the residence time of the solution as well as the hydraulics. The water quality analysis was disregarded for this study.

The residence times were penalised with a penalty of \$1m (1969\$US) for every hour the water required to get from the reservoir at the head of the system to the critical node. Convergence of the water quality had to be achieved before the residence times were recorded. Pressure penalties applied to any solutions failing to meet the hydraulic constraints. Figure 7.39 shows the construction cost convergence and their respective residence times.

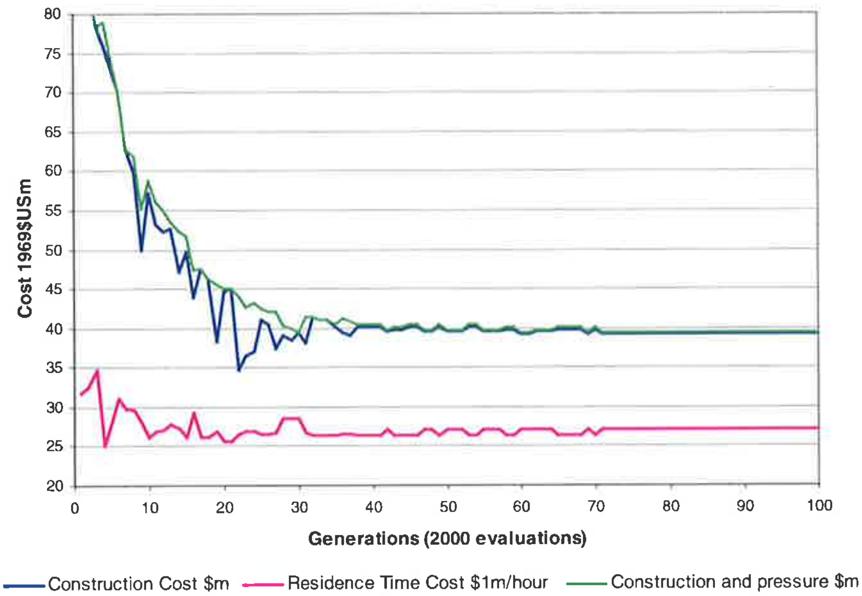

Figure 7.43 Residence time optimisation

The optimisation moves strongly into the inferior pipe design range to attempt to minimise residence time, with construction costs below \$35m after 20 generations. The large pressure penalties that coincide with these results push the convergence graph towards solutions with a much lower or no pressure deficit.

This method was first tested using a starting population of 1000, however, the starting solution set was too small achieve convergence to the more efficient solution sets.

The best three overall solutions from this simulation are listed in Table 7.13, and it is clearly seen that the preferred water quality solutions of \$39.21m and \$39.78m were also the preferable solutions for the optimisation of residence time.

Table 7.13 Pipe duplicate sizes for optimal residence time solutions

	GA Result #1	GA Result #2	GA Result #3	GA Hydraulic
	Construction Cost (1969\$Usm)			
	\$39.21m	\$39.78m	\$39.63m	\$38.80m
	Residence Time (hours)			
	27.0 hours	26.4 hours	27.0 hours	28.9 hours
Pipe No.	Pipe diameter (inches)			
1	0	0	0	0
2	0	0	0	0
3	0	0	0	0
4	0	0	0	0
5	0	0	0	0
6	0	0	0	0
7	0	0	0	0
8	0	0	0	0
9	0	0	0	0
10	0	0	0	0
11	0	0	0	0
12	0	0	0	0
13	0	0	0	0
14	0	0	0	0
15	144	144	144	120
16	72	72	72	84
17	96	96	108	96
18	84	84	72	84
19	72	60	72	72
20	0	0	0	0
21	72	84	72	72

Both the \$39.21m (GA#1) solution and the \$39.78m (GA#2) solution were preferable to the \$38.80m solution, which had a residence time of nearly 29 hours, 2 hours greater than the \$39.21m solution, and 2.5 hours greater than the \$39.78m solution.

Naturally, the choice of the solution would be dependent on the selection of penalties, pressure, residence time or quality. The solution which would best fit out of the three is the \$39.21m solution as it has less than half an inch pressure deficit compared to the more expensive \$39.78m solution with approximately three quarters of an inch deficit. The two-hour saving in residence time over the \$38.80m solution allows a potential reduction in disinfection costs as well as improved water quality conditions.

7.5.2 Potential of Using Residence Time to Optimise for Water Quality

As the results from the residence time analysis matched well with the runs based on quality cost structures, there is potential for this method to be a technique in its own right for such problems.

Another method to determine the residence time using EPANET is to analyse the water age instead of the disinfectant. This process takes just as long as tracing a disinfectant as it is also connected to the quality section of the solver. As you cannot predict the water age before starting a run though, testing for convergence has to occur which makes it marginally less effective than testing for a chemical which can be valued by a switch once the chemical concentration reaches the desired value.

7.5.3 Analysis of the Computational Ability of the Different Methods

One of the important aspects in GA optimisation is to minimise the computer run time. The work from this study was done on a Sun Ultra 1 running at 167 Mhz O/S Solaris 2.5 (128 MB Ram).

The power of the computer has a strong bearing on the speed of the run. The use of compilers also affects the run time.

From a programming perspective, the factors contributing towards program run-time are:

- Population of the search space;
- Number of generations (iterations) that the model runs over; and
- Number of time-steps that require data analysis in the simulation.

The formulation for approximate running time determination can be seen in Equation 7.2.

$$RT = GA_{pop} \times GA_{gen} \times T_{sim} \tag{7.2}$$

RT = total running time, GA_{pop} = population size of genetic algorithm, GA_{gen} = number of generations specified in genetic algorithm, and T_{sim} = number of timesteps in each simulation.

For the different methods to determine the preferred solution sets for the combined hydraulic and quality analysis, the components from Equation 7.2 are listed in Table 7.14 to produce an idea of the relative running times of the different methods.

Table 7.14 Approximate run times to determination of cost-efficient solutions

Method	Pop. size	Generations (Approx.)	Time-steps	Run Time (Approx. Hours)
Hydraulic Only*	1000	50	1	0.3
Hydraulic then Quality Penalty	1000	90	200	12
Hydraulic combined with Quality Penalty	2000	35	200	9
Hydraulic then Social Cost Analysis	1000	70	200	9
Hydraulic combined with Social Cost	2000	100	200	24
Residence Time Analysis	2000	60	500	36

*The optimal solution considered for the hydraulic only run was $38.80m.

Due to the pure hydraulic analysis being a steady state run, there was little computation time required. The quality runs used a 200-hour duration with time-steps of 1 hour, this duration encompassed the four seasons analysed. The residence time optimisation had a 50-hour duration with 10 time-steps every hour.

The runs that incorporate a hydraulic analysis initially can undergo severe reductions in computation time by altering the duration of the simulation from the EPANET input file. This can represent nearly a 50% saving in run-time, but requires the operator to alter the duration from the steady state duration (0 hours) to the required duration just prior to the inclusion of the water quality analysis, achieving convergence of the chlorine residuals for an annual analysis.

With further analysis of the different penalties for each technique, a more complete understanding of the preferred method could be provided.

7.5.4 Value of the Different Optimisation Techniques

The different approaches to the same problem enabled analysis of the similarities and differences in the solutions and pathway to the determination of the solutions. Each of the different methods provided some value in the determination of the resultant solutions.

The use of a quality penalty provided a glance at a rigid analysis that used arbitrary penalty values to force the residual levels to be within set confines. Of the two quality analysis, this was the simpler and showed that the best hydraulic solution was not necessarily the best solution for water quality in a gravity fed system. Naturally, with penalty adjustments for both the quality aspects and hydraulic components, the weighting could provide these solutions, however, even with reasonable weighting, the efficient solutions differed marginally.

The second form of analysis was to apply a social cost to the system. This examined the provision of costs based on research into potential health risks, aesthetic appeal and alternative treatment methods. By placing these costs on different quality levels, a more realistic cost approach was used with numbers that were researched specifically for the system rather than arbitrarily selected as a constraining parameter. With the results of this analysis corresponding very well with the quality penalty optimisations, the solutions determined from both of these methods gained impetus.

The reason for the focus on analysis of residence time is that the quality penalty and social cost optimisations pointed towards residence time being the critical factor in gravity fed systems. This was verified with the results obtained from the residence time optimisation run.

The combination of the three different methods produced a solution set that had residence time as its critical factor in determining the best system make-up. The social cost and quality penalty analysis did more than provide a system layout, they fitted a seasonal dosing regime to the selected network providing another design aspect to the system.

7.6 Design Recommendations from Analysis of the New York Tunnels Problem

The solutions provided from a water quality perspective indicate that a construction cost of \$39.21m result would be the best suited to the overall analysis. This would involve duplicating the pipes as listed in Table 7.15, along with the lengths and costs respective to each pipe.

Table 7.15 Recommended pipe duplication and respective costs

Pipe Number	Diameter (inches)	Cost/ft (1969$US/ft)	Length (ft)	Cost/pipe (1969$US)
15	144	522	15500	8,091,000
16	72	221	26400	5,834,400
17	96	316	31200	9,859,200
18	84	267	24000	6,408,000
19	72	221	14400	3,182,400
21	72	221	26400	5,834,400

Although there is a slight pressure deficit at node 17 (less than half an inch) with this solution, the residence time saving of nearly two hours during the critical summer season is seen to offset this. Improving the water quality two hours faster has the potential to create a large cost benefit. As well as a faster response to any water quality concerns, the amount of disinfectant required is also reduced.

The dosing regime recommended for the system to supply a minimum residual of 0.3mg/L throughout the system, which is the minimum level currently desired (Glaser, 1999) is shown, with predicted system residuals back in Figure 7.36. The chlorine dosing levels for each season are:

- 1.0mg/L for autumn;
- 1.0mg/L for winter;
- 1.4mg/L for spring; and
- 1.8mg/L for summer.

Summer is the critical dosing period due to the increased chlorine decay rate.

7.7 Conclusions

There are several conclusions able to be examined from the results of the New York Tunnels analysis. Some of these conclusions provide an expansive outlook on optimisation for both water quality and hydraulics. Other conclusions relate to the methodology of water quality optimisation.

The conclusions that are relevant to hydraulic and quality optimisation are:

- The most cost efficient solution for both hydraulics and water quality considerations is not necessarily the same solution as the purely hydraulic optimum; and
- The determination of the more efficient water quality solutions is highly dependent on the residence time to the critical node at the critical season.

The New York Tunnels study is well researched, and the current optimal levels were solved for in this application and found that under the previous penalty regime were not the optimal solutions once water quality was included in the optimisation. The hydraulic optimum of \$38.80m did provide a good solution, but due to its higher residence time was not able to provide the same level of quality result as the \$39.21m. For the critical season at the critical node, the residence time was two hours longer than the most efficient solution, and the residual level at the critical node was 10% lower than the \$39.21m solution.

The conclusions more specific to the type of quality analysis are:

- Determination of hydraulic optimal values before quality analysis narrows the search space for a gravity fed system;
- The quality penalty approach and the social cost approach both provide a good structure to gain desired system quality; and
- Results obtained are subject to a degree on the value of the different penalties placed on different quality levels.

As the hydraulic optimal solutions did provide reasonable quality solutions as well, by optimising purely for hydraulics initially, not only was the effective search space favourably narrowed, but the hydraulic optimum was included in the solution set to see whether it was preferred for water quality as well. The convergence achieved using this solution was generally faster than applying a full analysis on both quality and hydraulics from the start. There is also the potential to achieve significant savings in computer running time using this method.

The convergence to optimal values in both the social cost and quality penalty applications indicated their ability to provide efficient solutions to this optimisation. The efficient solutions are be subject to fluctuation as different weights are applied to the hydraulic and quality aspects, however, this can be controlled by the designer.

All of the quality-modelling techniques determine the optimal solution to be a construction cost of $39.21m as the tradeoff between saving in residence time and pressure deficit reduction is minimised.

Chapter 8 Yorke Peninsula Distribution System

The Yorke Peninsula is located 50km west of Adelaide, South Australia. A map showing the location, scale and layout of the region can be found in Appendix I, in a pocket in the back cover of the thesis.

The current water supply for the peninsula is predominantly pumped from the Murray River via the Swan Reach Stockwell Pipeline, and stored in an open reservoir at Upper Paskeville. The water is then transported through a series of pumps, valves, tanks and pipes to a number of seaside towns and farming communities.

The current system is at or near its capacity and future growth has been predicted for the region. Investigations into development of the current system in order to cater for future growth represents a challenge facing the South Australian water industry.

The water quality of the supply is another issue of concern. The network experiences residence times of greater than 15 days (Thomas, 1990), so changes in the water quality throughout the distribution system become pivotal in the quality of the water received by the consumer.

Current levels of dosing using chloramines are high to maintain adequate residuals downstream. The task of balancing the future hydraulic needs of the system as well as providing acceptable water quality creates an interesting study that is relevant both theoretically and in real terms.

8.1 Physical, Spatial and Temporal Issues

The Yorke Peninsula network has design criteria associated with spatial and temporal conditions. Water consumption rates vary with the seasonal fluctuations and the related tourist trade.

8.1.1 Spatial Arrangement of Population on Yorke Peninsula

The Yorke Peninsula consists of several towns with populations up to 2000 interspersed with farming properties. Table 8.1 below shows the population distribution on the peninsula.

Table 8.1 Distribution of Services on the Yorke Peninsula

Water District	Number of Services, 1985	Predicted Services, 2011	Est. Population, 2011
Inland Water Districts			
Arthurton	57	80	200
Maitland	522	750	1875
South Kilkerran	22	20	50
Curramulka	83	100	250
Minlaton	395	550	1375
Yorketown	378	500	1250
Subtotal	**1458**	**2000**	**5000**
Coastal Water Districts			
Port Clinton	255	500	1250
Price	99	150	375
Ardrossan	710	1200	2800
Port Victoria	266	650	1625
Pine Point	107	150	375
Port Vincent	435	800	2000
Stansbury	342	700	1750
Wool Bay	106	250	625
Coobowie	170	300	750
Edithburgh	328	700	1750
Subtotal	**2818**	**5400**	**13500**
Country Lands	2271	2600	6500
OVERALL TOTAL	**6547**	**10000**	**25000**

Appendix I contains a map showing the locations of the towns.

For estimation of the population receiving the supply, a figure of two and a half people per service was used. In some of the coastal towns such as Ardrossan, the people per service can be as few as two (Van der Wel and McIntosh, 1998) as the coastal locations are more popular with retirees and people on holidays. The farming towns down the spine of the peninsula are more likely to have more residents per service.

The fairly even distribution of people throughout the peninsula ensures that all regions are effected in a similar way when quality concerns become a factor. In general, as potential costs accrued from poor water quality are based on the number of consumers, a large population in one part of the network will take priority over a small population at another extent as the potential heath costs are so different between the two regions. This is not an issue for the Yorke Peninsula as the spatial arrangement of the consumer base covers all boundaries of the system.

8.1.2 Temperature

The temperature of the water in the system helps to dictate disinfectant consumption. The Yorke Peninsula has a range of water temperatures from 11°C in winter through to 27°C in summer. The monthly water temperature pattern is shown in Figure 8.1.

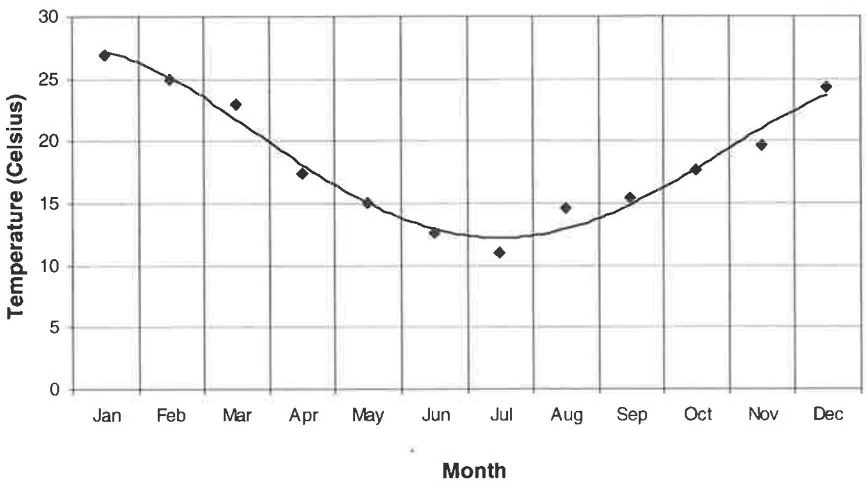

Figure 8.1 1998 Monthly Water Temperatures for Yorke Peninsula

8.1.3 Dosing Rate of Disinfectant

Even though temperature, pH and DOC all affect decay rate, the dosing rate dictates the amount of disinfectant able to be consumed, hence creating a quality boundary condition. The average monthly dosing rates used during 1998 at both Maitland and Upper Paskeville dosing stations is represented in Figure 8.2.

For the majority of the year, the levels hover around 7-8mg/L of chlorine at Paskeville with an ammonia complement of approximately 2.5mg/L. At Maitland, where the booster dose is currently applied, chlorine dose levels of 2.5mg/L are usually observed. During August to October of 1998, a decrease in the Paskeville levels to 5mg/L occurred. This was compensated for by a sharp rise in the Maitland dosing levels up to 5mg/L.

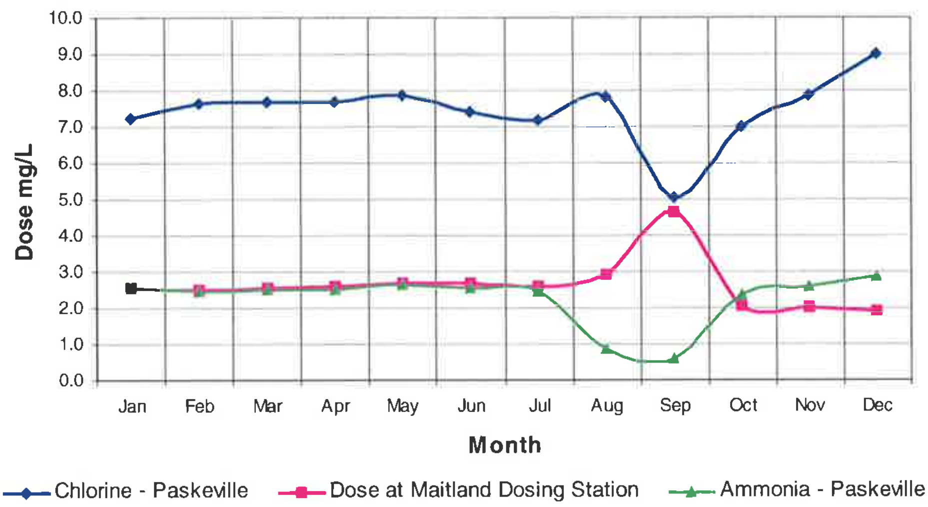

Figure 8.2 Monthly Dosing Rates at the Yorke Peninsula for 1998

8.1.4 Water consumption for the Yorke Peninsula

While the doses of disinfectant added are known, the amount of water the dose is applied to needs to be ascertained – i.e. consumption rates. These are shown in Figure 8.3.

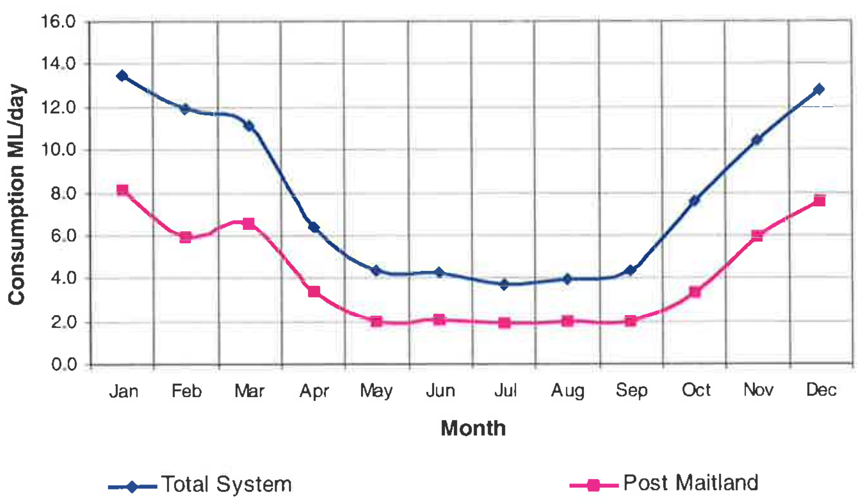

Figure 8.3 1998 Monthly Consumption Data

The reasons for requiring consumption data are fourfold:

1. By knowing the amount of water at each station, the total amount of disinfectant used is known hence enabling costing to occur;
2. The residence times for the system can be predicted which enables the amount of disinfectant consumption to be calculated;
3. Affects the system hydraulics as peak demand pattern must be catered for; and
4. Illustrates the variability of the system demand both on a daily and annual basis.

The regional centres of Maitland, Edithburg and Port Vincent were examined in a study by the EWS on supply systems (EWS, 1986(b)),. The average annual supply for each service in each of these regions is shown in Table 8.2.

Table 8.2 Approximate supply for each service on the Yorke Peninsula

Town	Average Rainfall (mm)	Average annual supply KL/service/year
Edithburg	416	220
Maitland	507	325
Port Vincent	360	260

The consumption changes throughout the day. To simulate the changing rates of consumption, curves from WATSYS, a modelling package for Australian were used. These curves differ depending on time of year, as garden watering, shower frequency and even consumption vary throughout the annual cycle. The 24 hour winter and summer curves is shown in Figure 8.4 (WATSYS, 1985).

The consumption patterns throughout the year are based on the daily WATSYS curves (Figure 8.4) and the annual consumption curve (Figure 8.3). Assuming an average multiplication factor of 1 for average flow, the following factors were applied for each season:

- 0.5 for winter;
- 0.7 for autumn;
- 1.0 for spring; and
- 2.1 for summer.

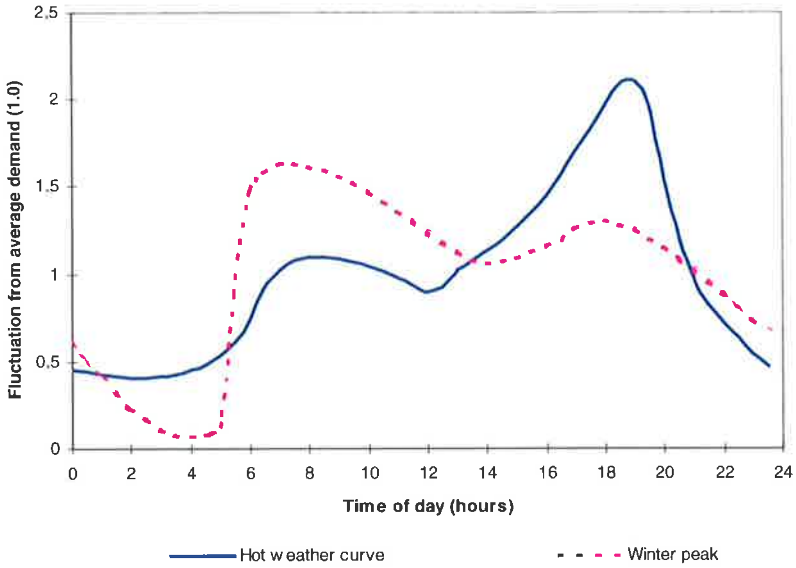

Figure 8.4 WATSYS curves for daily fluctuation in demand

The multiplication factor for summer is higher than the recommended value of 1.8 (SA Water, 1994). This is because it is the peak long-term demand that is desired for testing. This attracts a multiplication factor of 2.1, which is the peak month demand (SA Water, 1994). Using peak month demand to represent seasonal demand allows for summer tourism specific to this system. Knowing the consumption variation allows the computation of residence times and required dosing rates using EPANET. This allows the amount of disinfectant used to be determined.

8.1.5 Current Disinfectant Use for the Yorke Peninsula System

The current levels of disinfectant use are shown in Figure 8.5 in kg/day for each month. Figure 8.5 allows current daily disinfectant costs to be calculated based on $1.25/kg (Hogben, 1999).

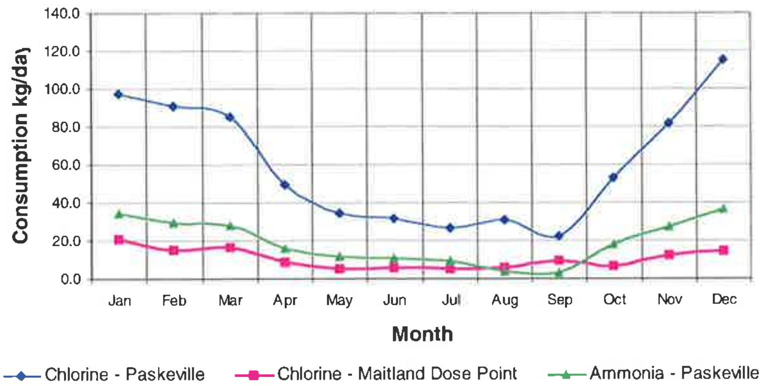

Figure 8.5 Current Disinfectant Use for Yorke Peninsula

8.1.6 Determination of Residence Times for the Yorke Peninsula

Secondly, residence times are needed to determine the water quality and are vital to the entire quality make-up of the network. Figure 8.6 shows the calculated average residence times for winter and summer.

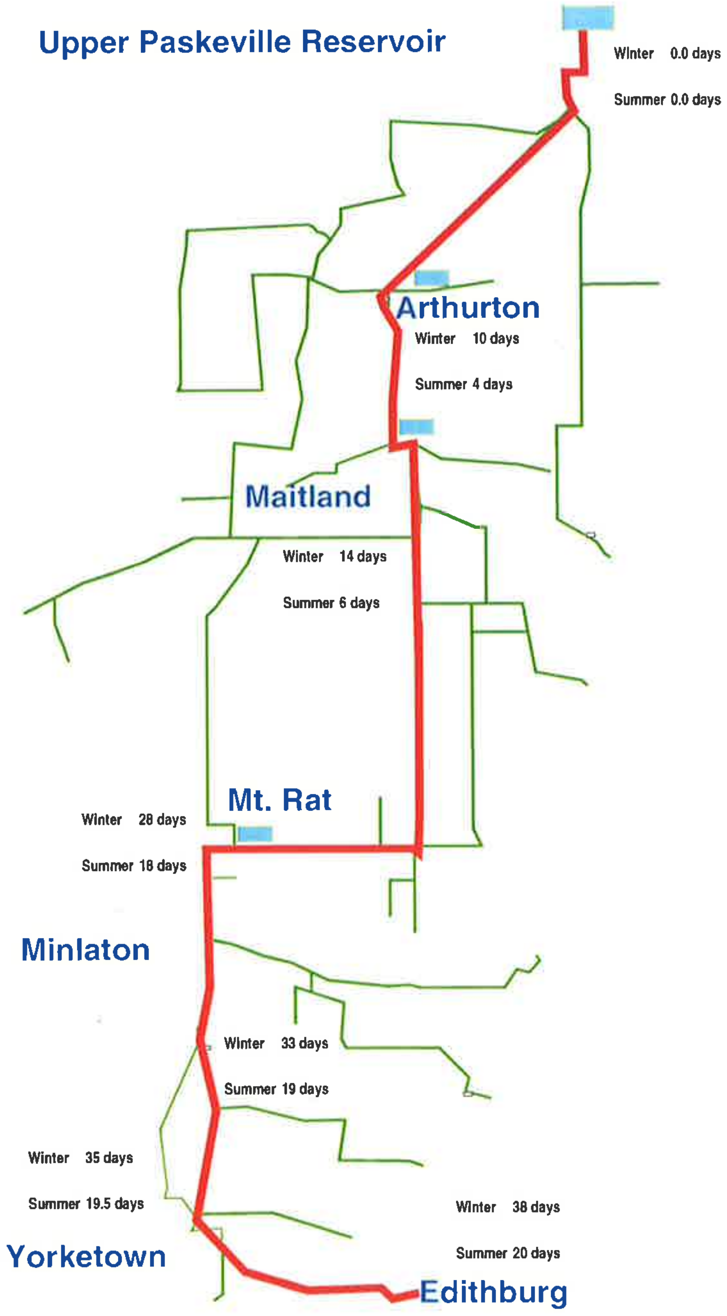

Figure 8.6 Residence Times for the Yorke Peninsula Network

These residence times were calculated by mapping the known flow of the supply through the trunk main and major tanks in the system. By dividing the volume of the route into the flow, the residence time is found. It must be noted that these are not the peak conditions, i.e. highest demand period in summer and lowest demand period in winter for the year. The summer peak would realise less than half the summer duration and the highest residence time in winter would be larger than the stated average value.

The residence times were calculated for the Yorke Peninsula Trunk Main, which runs down the spine of the peninsula as shown in Figure 8.6. The residence times assist in the determination of decay rates for the system. A full analysis of residence times is given in Appendix G.1

The other factor completing the provision of decay rates is the spread of residual levels throughout the system as shown in Table 8.3 (SA Water, 1999). The dosing rates and flows assumed to assist in these calculations were all adapted from field data supplied by SA Water (Hogben, 1999). Table 8.3 is a seasonal summary of the recorded residual levels throughout the year. These levels have been averaged over the season. As the quality data was compiled on a weekly basis, a seasonal average considered enough points to allow a reasonable approximation to take place.

Table 8.3 Average Chlorine Residual Levels from Recorded Data

	Summer	Autumn	Winter	Spring
Paskeville	6.5	6.1	5.6	6.0
Arthurton	4.2	4.3	3.3	3.5
Pre Maitland Tanks	3.7	4.1	2.9	3.0
Post Maitland Tanks	5.2	4.4	4.6	5.0
Post Mount Rat	3.5	3.5	4.3	2.8
Minlacowie Tank	3.2	3.4	3.7	2.3
Yorketown	3.1	2.6	2.7	2.7
Edithburg	2.5	2.4	2.5	2.4

Once residence times and residual data were determined, calculation of the seasonal decay rates could take place.

8.2 Decay Rate Calibration of the Yorke Peninsula System

The determination of decay rates is vital to the quality aspects of the system design. By taking the observed data from the system and comparing it to EPANET simulations using decay rate options, the preferred parameters can be determined.

The decay rates have been examined for each of the seasons and compared against the typically observed data. It should be noted that the comparison has only been taken from the southern side of the Maitland dosing point. This is because the system between Upper Paskeville Reservoir and Maitland is controlled alternately between the pumping stations located at Upper Paskeville and Kainton Corner, and the gravity fed Arthurton tanks as represented in Figure 8.7

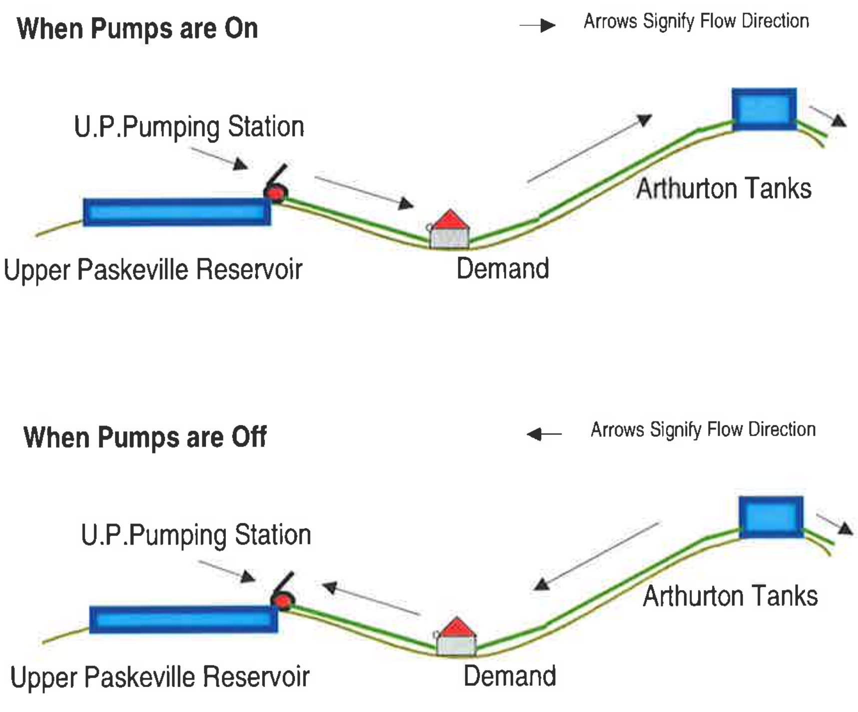

Figure 8.7 Pump and Tank Operation in the Upper Section of the Network

There are large discrepancies between the residual levels from water pumped fresh from a dosing station, and water that has spent time in a large storage facility, as shown in Table 8.4. Fitting simulated data is difficult, as the pump operation times are variable.

Table 8.4 Variability of the Chlorine Residuals Prior to Maitland Tanks

Season	Pre Arthurton Tanks		Pre Maitland Tanks	
	Min (mg/L)	Max (mg/L)	Min (mg/L)	Max (mg/L)
Summer	2.8	5.9	2.5	4.8
Autumn	3.6	5.7	3.1	4.9
Winter	0.9	5.4	0.5	5.2
Spring	1.0	6.6	0.9	5.0

As can be seen from the observed data ranges (in Table 8.4), there is a great variation between the minimum and maximum levels. This is most notable during spring and winter for both pre-Arthurton, and pre-Maitland tanks. The quality data from the system is collated weekly at some locations, monthly at others, which makes the results for a region with such contradiction in supply process difficult to replicate through simulation. As the majority of the values tend to closely group themselves in between the maximum and minimum levels, taking the mean levels allows for analysis of the data.

8.2.1 Simulation Results for Maitland to Edithburg

The decay rates for each of the seasons were plotted against the recorded levels. By applying different decay rates to the EPANET solver, the preferable rate for that period was ascertained. Summer usually experiences the highest decay rates caused by the higher water temperature providing conditions for microorganism growth. The decay rates selected for comparison in Figure 8.8 were 0.025/day, 0.035/day, and 0.045/day. The Maitland dose point is situated approximately 45km along the Yorke Peninsula Trunk Main.

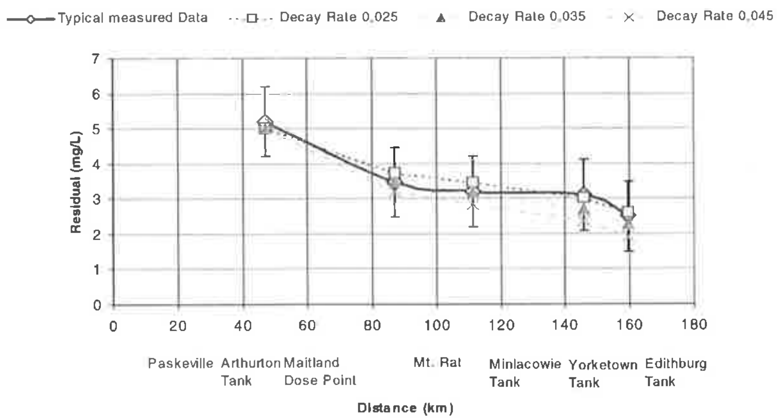

Figure 8.8 Chlorine Residual Comparison for Summer

Figure 8.8 shows the decay rate of 0.035 is the best fit of the measured curve. The measured data at Yorketown tank is higher than this curve, however, as this is the location that is measured on a monthly basis, the collated samples from this location have the greatest uncertainty.

A similar approach was taken for autumn. The only difference was that decay rates used were 0.015/day, 0.025/day and 0.035/day as shown in Figure 8.9. Figure 8.9 illustrates that the decay rate of 0.025/day fits the measured curve better than the other rates.

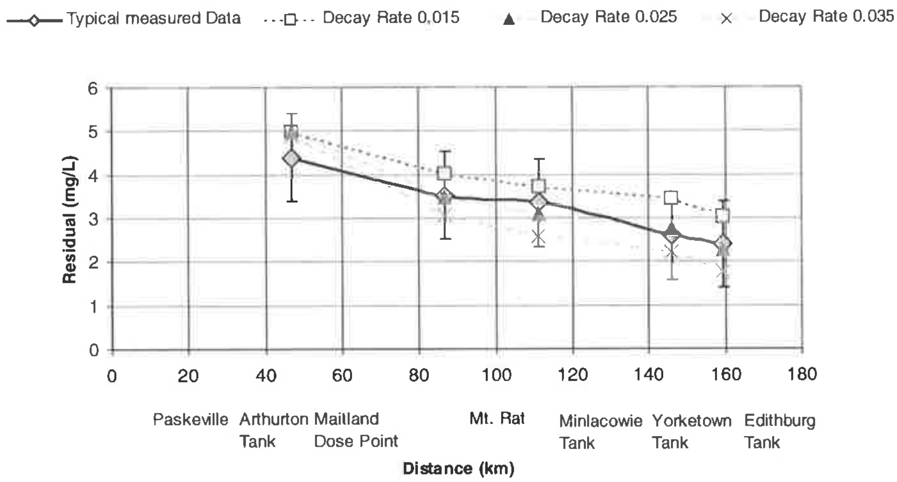

Figure 8.9 Chlorine Residual Comparison for Autumn

As the temperature and demand for spring is similar to autumn, it was expected that spring would follow a similar pattern. However, due to tank operation (alternating tanks), the measured data shows considerable variation, hence, the fitted curves did not correlate as well as autumn. This is shown in Figure 8.10.

The tank rotation allows the water from Mount Rat Storage to be added to the supply. This storage may not have been used during the winter period, so the residual levels would be low in comparison to the water entering the tanks from Maitland.

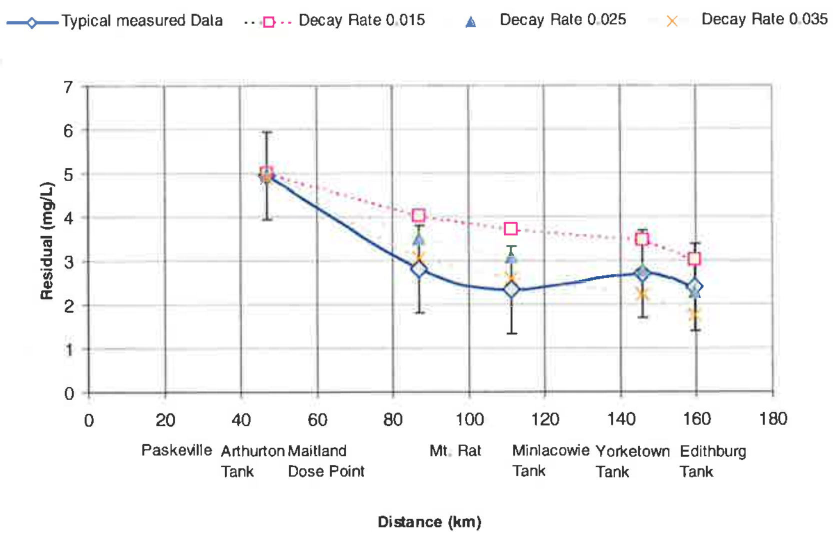

Figure 8.10 Chlorine Residual Comparison for Spring

From Figure 8.10, the decay of 0.025/day was selected, not only in response to the autumn period, but also due to the Yorketown tank through to Edithburg tank section of pipeline where the rate fits well. These two tanks experienced much less of the quality differences observed upstream.

The final season for data comparison was the winter season. Again, a drop in residual levels was expected, so the decay rates shown in Figure 8.11 are 0.005/day, 0.010/day, and 0.015/day.

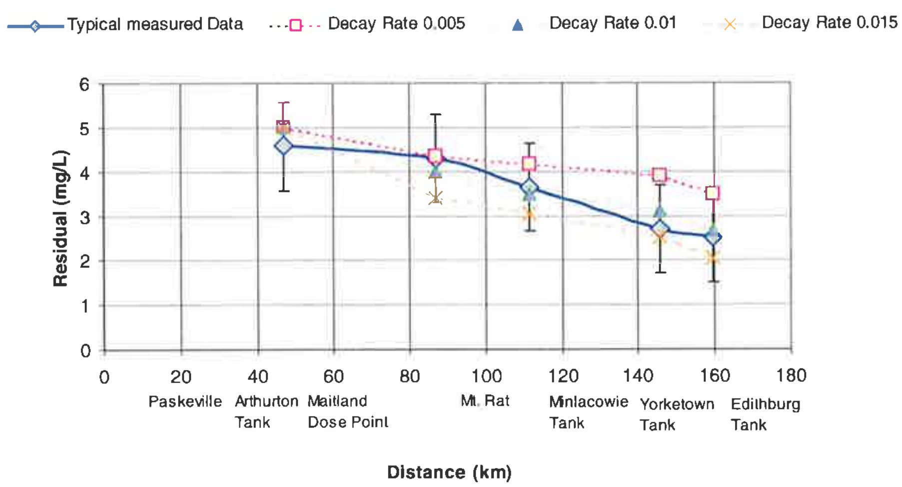

Figure 8.11 Chlorine Residual Comparison for Winter

Figure 8.11 shows the decay rate of 0.010/day was the best fit to the recorded data. This decay rate was therefore selected for the extended simulations using EPANET.

Ideally, a more detailed calibration would be carried out for a more frequent basis. However, as the residual information was measured weekly, to gain a sufficient data set, a seasonal analysis was deemed the preferable approach.

8.2.2 Issue of First Order against Second Order Decay Rates

As discussed in Chapter 3, the decay relationship that best resembles real-life conditions is the second-order decay model. As EPANET uses the first order decay model (also discussed in Chapter 3), the decay rates for the lower parts of the system have been assumed on a first-order basis.

To produce a model representing the observed situation, some modification of methodology was instigated to allow for an adapted approach to decay rates. Second order decay has a relatively faster rate initially compared to the rate after a certain time lapse. By assuming the part of the network between Paskeville and Maitland to have a higher decay rate, a more applicable decay situation can be developed.

As Arthurton tanks and the Upper-Paskeville to Arthurton trunk main represent about 80% of the residence time for the major branch of the upper system. By assigning a separate decay rate to this section, the resultant decay for the overall system will be more realistic than applying the lower system decay rate to the entire network.

8.2.3 Factoring a Separate Decay Rate for Arthurton Tank

As mentioned previously, the upper extent of the system experiences considerable variability based on the control mechanism at the time. The supply from the upstream pumping stations has different quality issues than the supply from Arthurton tanks (Arthurton tanks are shown in Figure 8.12).

Figure 8.12 View of Arthurton Tanks

To provide data that can adequately determine a first-order decay rate for the Arthurton tanks, hence the upper portion of the system, selective data based on knowledge of residence time and operation (as given in Table 8.5) must be considered.

Table 8.5 Approximate Decay Rate Determination for Arthurton Tank

	Autumn	Winter	Spring	Summer
Paskeville Dose (mg/L)	7.5	7.5	7.5	7.5
Pre Maitland (mg/L)	4.2	4.5	4	3.5
Residence Time (days)	9	11.2	7.5	4.7
Decay Rate (/day)	0.064	0.046	0.084	0.162
Adapted Rate (/day)	0.08	0.06	0.11	0.20

The Paskeville levels represent the chlorine-dosing rate at the chloramination station. During the year, when pumping occurs from Upper Paskeville, the measured data just after this station produces levels of around 7-7.8mg/L. Over the year, the usual recorded levels are between 5.5mg/L and 7.0mg/L as during other parts of the year, this collection point registers much lower levels as it obtains its supply from the Arthurton tanks.

To simplify this process for modelling purposes, this decay rate can be applied to the Arthurton tanks and the trunk main from Upper Paskeville to Arthurton, as they cater for over 80% of the residence time at that part of the system. By adapting the residence times so the extra decay occurs at the tanks and initial trunk main only, a reasonable approximation of results is shown in Figure 8.13.

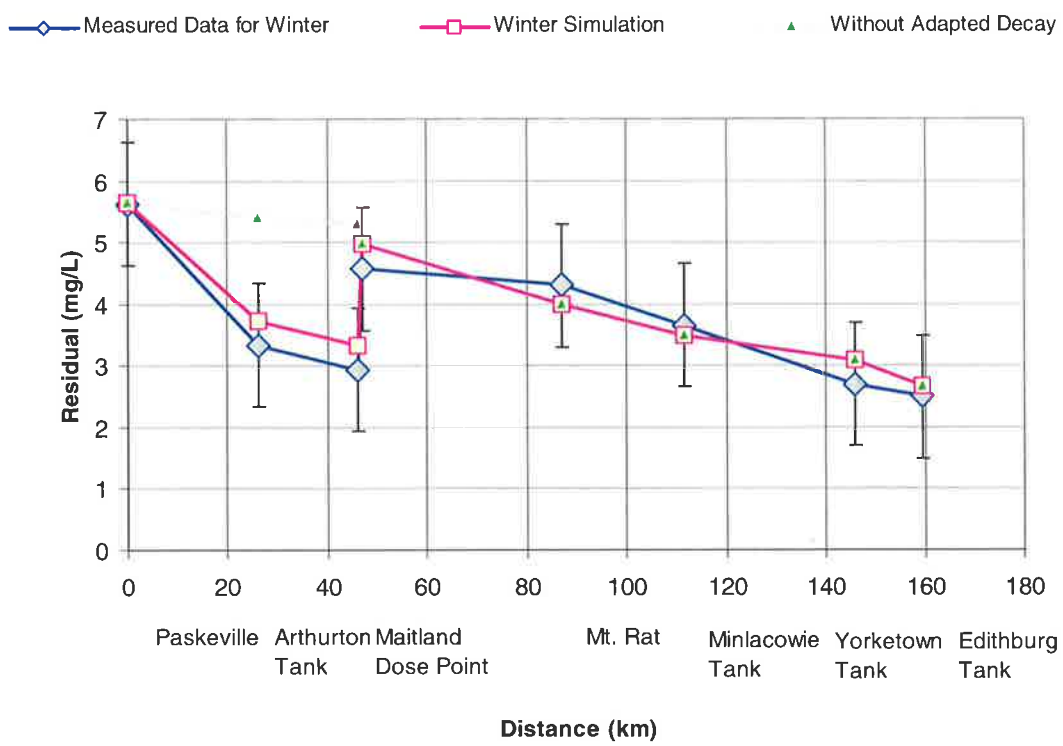

Figure 8.13 Adapting decay rate simulation for winter

The application of adapted decay rates for the other seasons further enhances the accuracy of the simulated results. By using these new rates, a better concept of the quality issues for the network can be determined. The simulation results for the remaining seasons are given in Appendix G.2.

8.2.4 Summary of Decay Rate Determination

The decay rates that were included as part of the optimisation for the network are listed in Table 8.6.

Table 8.6 Chloramine Decay Rates for the Yorke Peninsula System Simulation

Season	Distribution Decay Rate	Adapted Rate for Arthurton Tank/Upper System
Autumn	0.025/day	0.08/day
Winter	0.010/day	0.06/day
Spring	0.025/day	0.11/day
Summer	0.035/day	0.20/day

The combination of the two decay rates presented a more realistic quality picture of the system quality than a single decay rate. The decay rates represent one of the major factors in the determination of the dosing regime. Along with demand and infrastructure setup, it forms an integral component of the quality analysis.

8.3 Infrastructure Options for Yorke Peninsula

The consumption levels are a factor in determining the residence time, which has been estimated in Section 8.1.5. The tank, pump and pipe options for development of the system also affect the residence times.

As formulated in Appendix D, the cost structure for pipes, tanks and pumping stations provides the different options available for implementation into the system. The resulting penalty costs and quality decisions resulting from these choices provide the full cost analysis.

The layout of the system was provided by SA Water (Wozniak, 1998) in a WATSYS file. This data included all the pipes, tanks, major pumping stations and all valves (check, control and pressure-reducing valves). The distribution of the supply throughout the nodes was also supplied in this information. By knowing the locations and the relative demands at these locations, the system performance can be determined.

Additional information on the system was determined through field analysis and local sources (Kobelt, 1998; Sorrell, 1998; Hogben, 1999)

8.3.1 Pipe Selection

It was assumed that all pipe duplications for Yorke Peninsula will be ductile iron, concrete lined (DICL) pipes. Although cheaper pipe materials exist, especially for the lower diameters (PVC), DICL was chosen as it matches with existing infrastructure. The different pipe diameter options, as well as their costs/m are listed in Table 8.7 (adapted from Gumerman *et al.*, 1992).

The following series of assumptions were made to achieve the total costs for different pipe diameters:

- Moderate-Low traffic conditions;
- Sandy soil or light soil;
- 1.2m (4ft) of cover for trenches;
- A F.S. (contingency) cost is included to cater for any breaks or mishaps; and
- Native soil used as backfill material.

Table 8.7 Pipe options for the Yorke Peninsula system

Nominal Diameter (mm)	Actual Diameter (mm)	Construction Cost ($/m)	In-ground Pipe Costs ($/m)
0	0.0	$0	$0
100	101.6	$56	$94
150	152.4	$64	$104
200	203.2	$93	$134
250	254	$115	$152
300	304.8	$145	$183
400	406.4	$216	$258
450	457.2	$272	$316
500	508	$296	$339
600	609.6	$343	$389

The duplication of many of the pipes is an important consideration in the Yorke Peninsula as the holiday periods generate high peak flows and insufficient pressures for many locations. The potential pipe duplications are discussed in Chapter 9. Appendix H.3 contains the EPANET input file that includes the infrastructure data (lengths, diameters and roughness's) used throughout the network.

8.3.2 Tank Selection

Tanks provide a balancing storage, which allows towns to have a continuous supply at constant pressure. The locations of the new tanks were based on previous studies by SA Water. Wool Bay, Coobowie, Petersville and Port Victoria have all recently had tanks installed, although not all were on-line at the time of this study.

The costs attributed to the different options for tank sizes are listed in Table 8.8.

Table 8.8 Tank options for Yorke Peninsula

Storage Capacity (ML)	Cost ($)	Diameter (m)
0.0	$0.00	0
0.9	$513.000	15.1
1.9	$681,000	22.0
2.8	$800,000	22.8
3.8	$920,000	31.2
7.6	$1,454,000	44.0
15.1	$2,296,000	62.0
22.7	$3,035,000	76.0

Allowances were made for possible new tanks at the three main storages of Arthurton, Maitland and Mount Rat. This potential storage can provide a more robust system if required during the peak flow conditions. Figure 8.14 shows the potential new tank locations.

8.3.3 Pump Selection

Also shown in Figure 8.14 are the potential locations of pumping. The four main pumping stations that transfer water down the trunk main are to be replaced. These stations are:

- Upper Paskeville Pumping Station;
- Kainton Corner Pumping Station;
- Maitland Pumping Station; and
- Mount Rat Pumping Station.

To provide a more efficient supply to the system extremities, booster-pumping stations have been implemented as part of the design model. The selection process for location of these stations was provided in a report by SA Water (EWS, 1987), which listed some of the future infrastructure aims of the system. Booster locations included in the model are:

- Booster to supply Ardrossan;
- Booster to supply Port Victoria.
- Booster to supply Minlaton Tank;
- Minlaton Tank Booster, which supplies Port Vincent and Stansbury; and
- Booster to supply water to Yorketown;

All of the existing and potential tank and pump locations are shown in Figure 8.14.

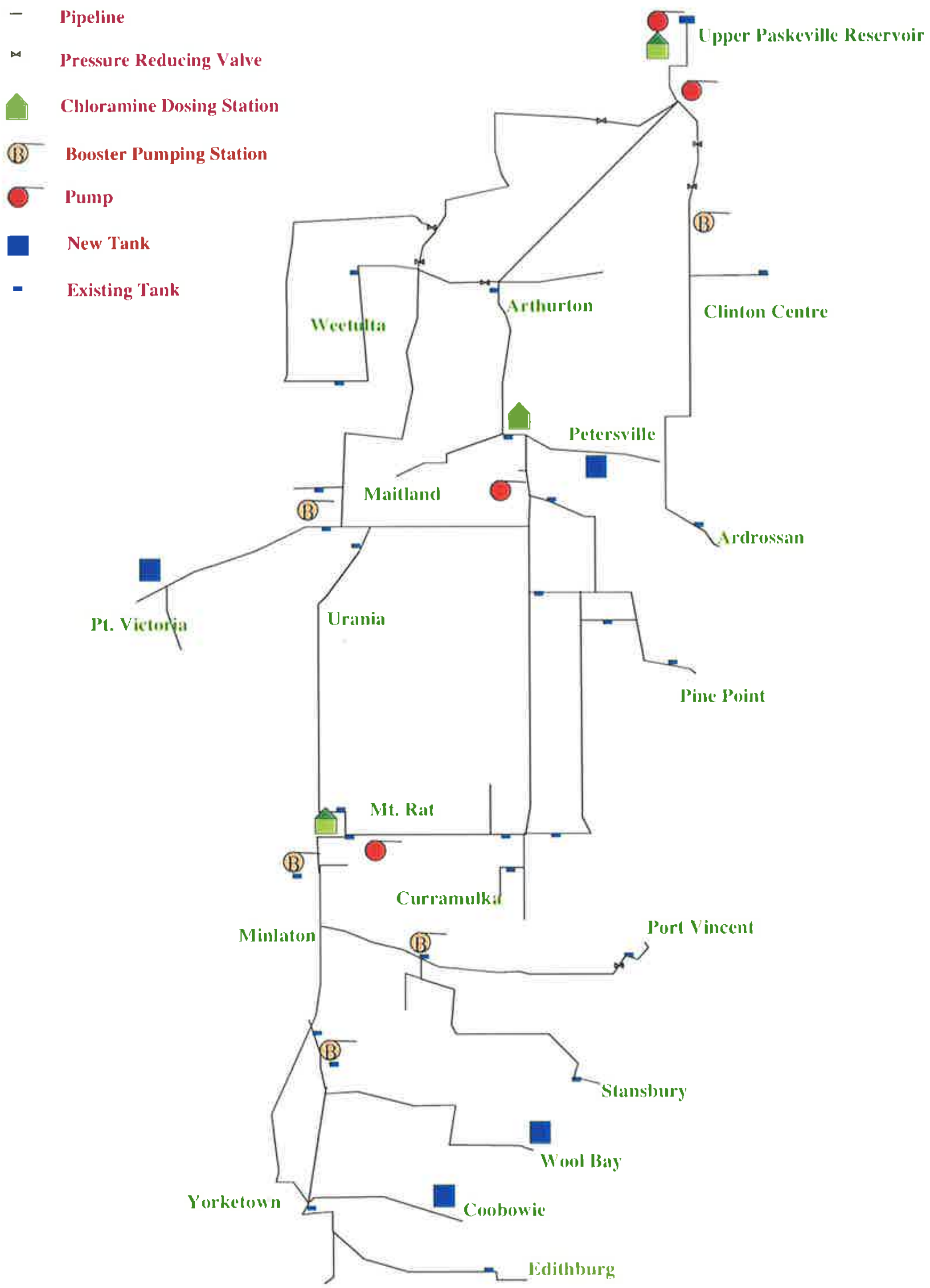

Figure 8.14 Pump, Tank and Dosing locations for the Yorke Peninsula study

The total pumping costs for different size pumps assuming a full running time (with a few hours gap for maintenance purposes) are listed in Table 8.9.

Table 8.9 Pump options for Yorke Peninsula over a 30-year life

Design Capacity ML/day - L/s	30.42m TDH	60.84m TDH	91.26m TDH
0.0 – 0	$0K	$0K	$0K
1.9 – 11	$776K	$952,K	$1,200K
3.8 – 22	$993K	$1,396K	$1,827K
18.9 – 109	$2,743K	$4,853K	$6,372K
37.9 – 219	$5,144K	$8,483K	$11,957K
94.6 – 547	$10,686K	$19,906K	$28,634K

8.3.4 Dosing Stations

Three dosing stations have been included in the design (seen in Figure 8.14). They are:

1. Upper Paskeville dosing station;
2. Maitland dosing station; and
3. Mount Rat dosing station.

Both Upper Paskeville and Maitland are existing stations. Mount Rat was taken off line after the system changed from chlorination to chloramination. Mount Rat dosing station has been included in this analysis to see if three dosing points allow a more consistent residual pattern at the system extremities, than two. If three are not required, the genetic algorithm should remove Mount Rat from the solution.

Each of the stations has a possible dosing level for each season. If one of the stations was not required in any one season, it accrued no maintenance or labour costs for that period. The structure and operation of the dosing stations is dependent on the EPANET water quality simulations.

8.4 Water Quality Information

Upper Paskeville reservoir, the main storage for the Yorke Peninsula system receives the majority of its supply from Swan Reach, on the Murray River from where it is pumped. Some of the supply during the peak periods comes from the Bundaleer Trunk Main, which receives water from the nearby Barunga Reservoir.

Both the Upper Paskeville and Barunga Reservoirs are earthen and open roofed. All the other storages throughout the system, notably the major storages at Arthurton, Maitland and Mount Rat are closed concrete storages. The water in Upper Paskeville Reservoir has been chlorinated prior to it entering the storage.

The water is chloraminated at Upper Paskeville after leaving the reservoir, and is re-chlorinated after the Maitland Tanks.

8.4.1 Typical Microbial Quality of Current Supply

The change to chloramine provided an improvement in the water quality of the supply. Table 8.10 lists data from monitoring of the supply throughout Yorke Peninsula during the chlorination period with three dosing stations and the current situation with two dosing stations and chloramine (Thomas, 1990).

Table 8.10 Microbiological quality of the Yorke Peninsula supply

	Chlorination			Chloramination		
	Chlorine mg/L	Total Coliforms /100mL	E.coli /100mL	Chlorine mg/L	Total Coliforms /100mL	E.coli /100mL
No. Samples	1904	1904	1904	2094	2094	2094
No. Positive	-	404	202	-	22	10
% Positive	-	21.2	10.6	-	1.1	0.5
Maximum	3.8	210	210	4.6	26	9
Minimum	<0.1	0	0	0.1	0	0
Median	0.1	0	0	2.6	0	0

As seen in Table 8.10, the introduction of chloramine allowed a residual to be maintained throughout the system (2.6mg/L median instead of 0.1mg/L median). The isolation of *E.coli* was also reduced by about 20 fold.

The drinking water conditions for Yorke Peninsula compare to those from the New York study with total coliform levels of 1.1% positive samples comparing to 0.2% for New York. The *E.coli* levels were 0.5% positive samples comparing to less than 0.1% for the New York system (NYCDEP, 1999). Keeping this in mind, the potential waterborne disease risk range was adapted from 0.0001-0.01/year risk for the New York study to 0.0001-0.1/year for Yorke

Peninsula. The lower risk level is for a chlorine residuals above the minimum allowable residual and the upper level is for zero chlorine residual. This formulation is described in Chapter 5.

8.4.2 Assumed TTHM Formation for the Yorke Peninsula System

The formation of disinfection by-products for the Yorke Peninsula system has been based on a similar system in South Australia. In 1989, field studies on the Tailem Bend to Keith pipeline system were carried out. They produced data on the trihalomethane formation for various points on the system with respect to disinfection consumption. Figure 8.15 illustrates the combined chlorine consumption for the chloraminated supply to Keith and the resultant TTHM formation from this consumption.

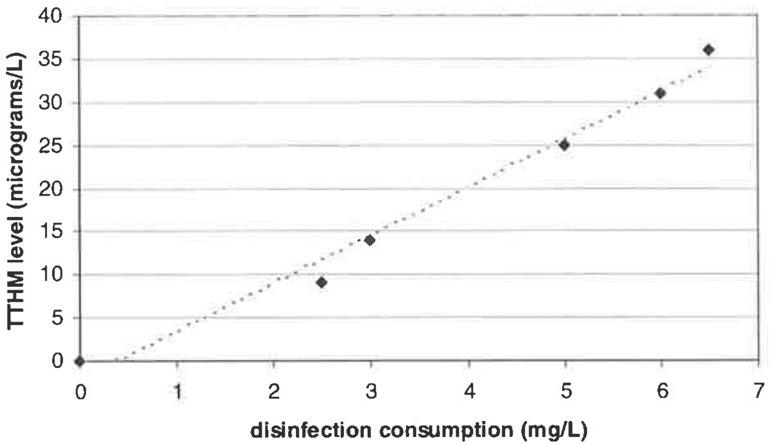

Figure 8.15 TTHM Formation due to Disinfectant Consumption for a Rural Chloraminated System with Long Residence Times

From Figure 8.15, a level of 5μg/L of TTHM formation for 1mg/L of disinfectant consumption was assumed for use in this study. These levels are subject to some fluctuation due to the effects of temperature as discussed in Chapter 3 examining the formation of disinfection by-products.

Figure 8.15 has been adapted from the fieldwork presented by Thomas (1990). The Tailem Bend to Keith system has a long retention time (>15 days), which is an important similar to the Yorke Peninsula system. The Tailem Bend to Keith system also takes its supply from the River Murray, so the quality of the source water is similar.

8.4.3 Taste and odour

The taste and odour analysis for the system uses the consumer's willingness to pay for implementation of a rainwater tank system. The formulation for this process is outlined in Chapter 5.

The residual that is considered the maximum allowable level has been set at 4.0mg/L. Although the aesthetic level is 0.5mg/L (SA Water, 1995), researchers have reported that taste and odour are not going to be of huge concern up to residual levels of 3.0mg/L (Krasner and Barrett, 1984) or 5.0mg/L (White, 1980).

8.5 Summary

With adequate knowledge of the supply and quality situation in the current system, combined with the determination and calibration of suitable seasonal decay rates, the system can be simulated for both hydraulic and quality concerns.

By applying set values for infrastructure items and knowing the constraints governing the water quality issues, a total cost regime can be determined. This cost regime can be optimised to provide efficient solutions for this system, and allow relationships to be drawn for different facets of a complex system

Chapter 9 Results for Yorke Peninsula

The relationships and process information determined in Chapter 7 relating to application of water quality to traditional network design are applied to the more complex Yorke Peninsula system. This system was introduced in Chapter 8, with a specific formulation enabling a realistic appraisal of the network to be achieved.

This chapter examines the production of a system design that caters for a 30-year period from 1999 until 2029. This analysis includes the optimisation of duplicate and new pipes, tank extensions and new tanks, pumps, and the dosing regime for disinfectant. The dosing regime for this network is spread over 3 potential dosing points and the dosing rate is adjusted seasonally.

This chapter provides the following results for discussion:

1. A cost analysis of the recommended changes by the Engineering and Water Supply Department, EWS (1987) for system extension until 2012;
2. A hydraulic optimisation of the system that finds an efficient combination of pipes, tanks and pumps to satisfy demand and pressure constraints; and
3. A hydraulic and water quality optimisation based on the social cost structure discussed in Chapter 5.

Due to the large amount of computer time that optimisation of a system the size of Yorke Peninsula represents, the results determined are cost-efficient solutions rather than optimal solutions. The social cost method for water quality quantification was used based on its success from the New York tunnels study (discussed in Chapter 7). Some of the costs in this analysis have been adapted to more realistic levels for a rural system as discussed in Chapters 5 and 8.

9.1 Previous Examination of Design Options

To introduce the potential structure the results may take, a report issued by the EWS (1987) examining design changes to carry the system through to 2012 has been analysed and applied to the same cost structure as the potential model results. Although approximate, these levels will give a background to the potential problem spots in the system, and how the GA performs in relation to cost and in providing a system that will cater for the needs through to 2029.

9.1.1 Design Recommendations by EWS for 2011 study

In 1987, a report issued by EWS highlighted the alterations to the Yorke Peninsula system to cater for the needs of the network through to 2012. Although this thesis examines the 30-year period between 1999 and 2029, the alterations recommended in this 1987 report make an interesting comparison. In fact, the 1987 report (EWS, 1987) constitutes a major resource in the locating of duplicate pipes and system controls.

To provide a comparison for the potential costs of the solutions, the EWS (1987) works have been valued using the same cost structure as the optimisation model. The recommendations that are already in the network data provided by SA Water (Wozniak, 1998) on the system layout for modelling purposes have not been included in the cost analysis. All other recommendations from the EWS (1987) report have been costed. Table 9.1 lists the locations where pipe duplication has occurred, and the lengths and costs for construction of these pipes.

The total estimated pipe costs are $8.8m. Some of these pipe costs are replacement, however, the majority are duplicate or new pipes. The pumping arrangement for the system is also addressed as an area where upgrading of existing stations and the addition of booster stations forms the capital cost structure. The operation and maintenance costs are ongoing and depend on the frequency of use and size.

Table 9.1 Duplicate pipe locations and costs

Location Start – Finish	Diameter (mm)	Length (km)	Cost 1999 AUS$K
Arthurton-Ardrossan Tank	250	15	2280
Inlet to Port Clinton Tank	150	2.9	302
Inlet to South Kilkerran Tank	150	2.6	270
Port Victoria Tank to Port Victoria Township #1	250	0.55	84
Port Victoria Township #1 to Port Victoria Township #2	150	0.8	83
1.3km upstream of Edithburg Tank to Coobowie Tank	200	3.7	496
Port Clinton Tank to Port Clinton	150	4.2	437
1km downstream of Minlaton Tank to Stansbury #1	300	10	1830
Stansbury #1 to Stansbury #2	250	6.5	988
Stansbury #2 to Stansbury #3	200	2.5	335
Stansbury #3 to Stansbury Township	150	5	520
1km downstream of Minlaton Tank to Port Vincent	200	8.5	1139
Total			8764

The pump locations and desired sizes are listed in Table 9.2. Also shown are the closest pump options available to the optimisation model, the construction costs and the estimated operation and maintenance costs based on the formulation in Appendix D.

Table 9.2 Pump recommendations for upgrade of Yorke Peninsula System

	Actual Levels		Program Level		Cost 1999 AUS$K for 30-year life		
	Head m	Cap L/s	Head m	Cap L/s	Const	Energy	Total
Minlaton	60	150	60	219	1000	933	1933
Pt Victoria	60	26	60	22	500	192	692
Minlacowie	60	72	60	109	750	556	1306
Cunningham	30	60	30	109	500	167	667
Port Vincent	60	48	60	109	750	417	1167
Muloowurtie	75	150	100	219	1000	1167	2167
Cobowie	33	60	30	109	750	83	833
Upper Pask*	60	500	60	547	2000	1800	3800
Kainton Corner*	60	500	60	547	2000	1800	3800
Maitland*	60	170	60	219	1000	1029	2029
Mount Rat*	60	150	60	219	No Upgrade	933	933
Total					$10,250K	$9,077K	$19,327K

* indicates existing major pumping station (not booster)

The estimated capital and electricity costs for pumping for a 30-year life based on the formulation used in this study is $19.3m. This brings the total comparable costs to $31.4m.

The WATSYS file that presented the system layout had 25 tanks (Wozniak, 1998), the report examine upgrade options highlighted several locations where extra storage was deemed necessary. These locations and approximate storage size and total costs are shown in Table 9.3. These costs include small boosters to supply the town from the tank.

Table 9.3 Recommended new tank locations for the Yorke Peninsula

Location	Storage (kL)	Approximate Cost Based on Cost Structure from Chapter 5 1999$K (AUS)
Pt Victoria	270	500
Coobowie	500	500
Wool Bay	410	500
Petersville*	500	500
Price	320	500
Minlaton Tank	2500	800
Total		$3300K

* Tank has been constructed but was not in initial report

The total costs for the tanks equals $3.3m. The tank costs are based on the values discussed in Chapter 8. The tank at Petersville (between Arthurton and Ardrossan) was not included in the 1987 report, but has been constructed.

9.1.2 Water Quality Physical Costs

The dosing stations attract costs due to labour and maintenance as discussed in Chapter 5. The other cost with dosing is the actual cost of the chemical. As the current Yorke Peninsula system has two dosing stations in operation, at Upper Paskeville and Maitland, the costs involved are the ongoing costs plus the costs of disinfectant. The typical dosing levels for the system are 2.5mg/L of chlorine at Maitland, and 7.5mg/L of chlorine and 2.5mg/L ammonia at Upper Paskeville (Hogben, 1999).

A potential cost analysis for this dosing arrangement over a 30-year life is listed in Table 9.4. These costs are based on the analysis from Thomas (1990) and use a cost of $1.25/kg of chemical added (chlorine or ammonia). Any differences when compared to Table 5.5 are due to the fact that the costing regime used for chemical costs follows the costs described by Hogben (1999) rather than the predicted costs from the Thomas (1990) study.

Table 9.4 Current cost profile for dosing stations

Location	Chemical dose	Flow ML/year	Chemical Cost 1999$K (AUS)	Maintenance 1999$K (AUS)	Total 1999$K (AUS)
Upper Paskeville	10.0mg/L	3750	37.5	30	67.5
Maitland	2.5mg/L	2050	5.1	30	35.1
Total annual cost	-	-	42.6	60	102.6
For 30-year life	-	-	737	1038	1774

The costs of disinfection are approximately $1.8m bringing the total expected cost up to a level of $33.2m.

Although these costs are applied to a different cost regime than the 1987 EWS values, they are consistent with the study structure. The pump and tank costs required some assumption as the values were either unlisted or rounded to fit with the options available. The recommendations of this 1987 report are shown in Figure 9.1.

The implementation of the EWS strategy is to occur in four stages. The first stage requires the booster pump to Port Vincent/Stansbury to be constructed, along with the Ardrossan pipeline. Storages at Wool Bay, Coobowie, Price and Port Victoria are also in the first phase.

The medium-term requirements are to lay new mains to Port Vincent, Coobowie, Price and Port Victoria. The third phase requires new mains to Stansbury and Pine Point as well as a booster pump to assist in supplying Port Victoria.

The final stage is to upgrade the major pumping stations, construct a second tank at the Minlaton tank site and install new booster pumping stations near Curramulka, Minlacowie and Cunningham (near off-take of new Ardrossan main).

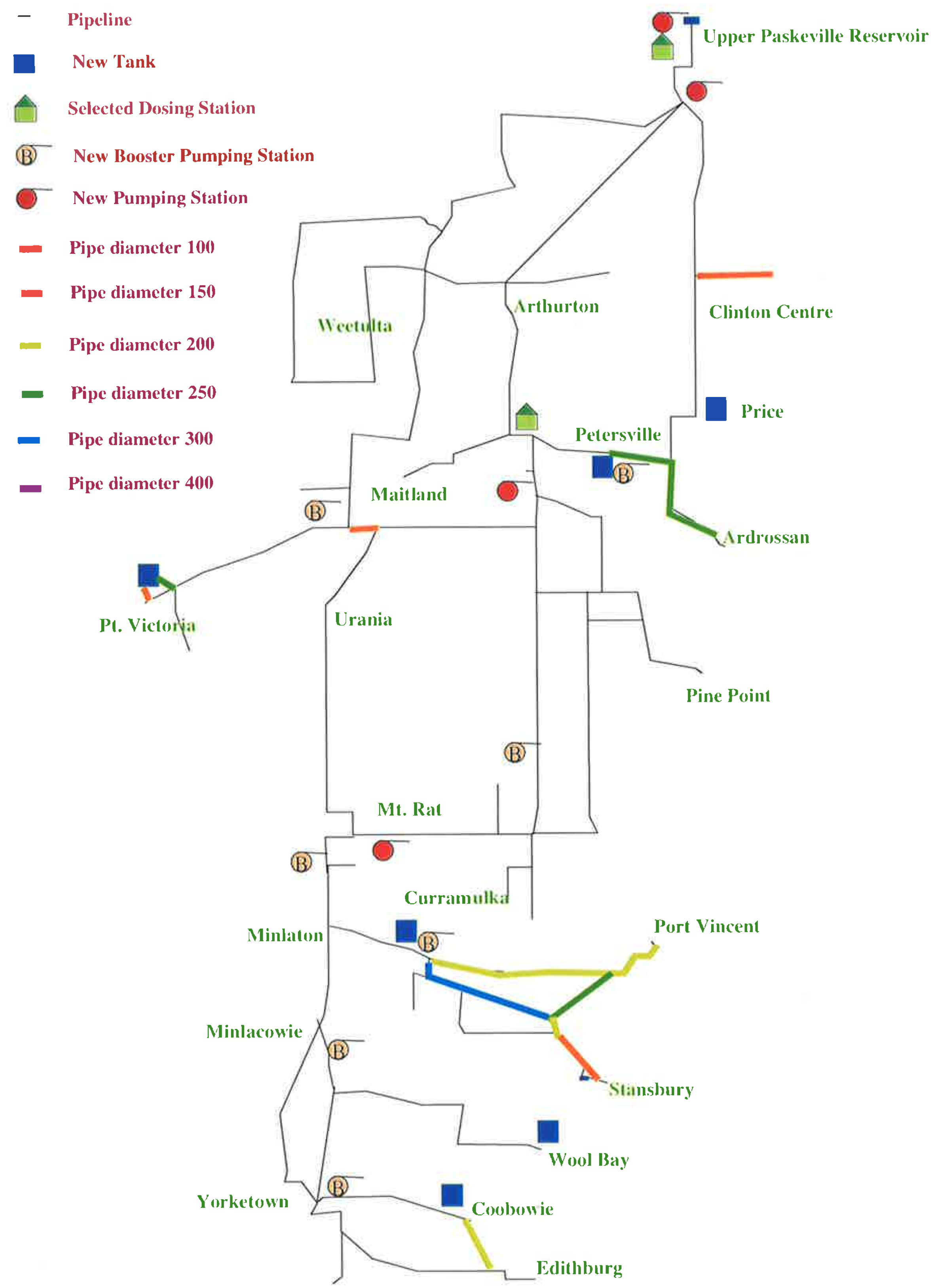

Figure 9.1 Layout of EWS 1987 recommendations

9.2 Hydraulic Optimisation of the Yorke Peninsula System

As discussed in Chapter 2, modelling of complex systems can require extensive computation time. By reducing the total search space, the speed of the optimisation is increased. In doing this, care is required to ensure that the potential global optimal solution is not removed as an option.

With the Yorke Peninsula study, there are 206 potential new pipes each with 10 possible diameters, 7 potential storages with 8 possible volumes, and 9 potential pumping stations with 14 possible pump curves. Water quality aside, the search space is $10^{206}\times8^{7}\times14^{9}$ solutions, which is approximately equal to 4.3×10^{222} solutions. To assist in providing a more rapid convergence, this search space can be reduced in two ways. The first is to reduce the number of options available. The second is to reduce the number of infrastructure components.

The main factor affecting the search space is the number of pipes in the system (1×10^{206}). Computer runs examining the system hydraulics found that the maximum diameter for the new pipes was 400mm. Allowing for an increase to 450mm, this reduces the total number of options from ten to eight. The search space for the pipes is reduced to 1.1×10^{186} and the total search space becomes 1.8×10^{204} possible solutions.

The major reduction in search space comes from only duplicating the pipes that are important to the hydraulic makeup. Of the 206 potential new pipes, the majority of them are not required to satisfy the hydraulic constraints of the system. To determine which pipes are to be selected for sizing, the following methods are used:

- Most pipes from the EWS (1987) report are to be included as options;
- A GA optimisation for pressure to determine the pipes required when the system has no pumps operating, and is supplied only from tanks;
- A GA optimisation for the capacity (maintaining supply) as well as pressure to determine the pipes required to allow complete supply; and
- Any other pipe that may be deemed worthy of further examination.

The GA parameters used for the hydraulic analysis (excluding population size) are:

- The probability that uniform crossover occurs is 1.0, the probability of a switch is 0.5;
- Tournament (direct) selection with groups of three is the selection method;
- The probability of mutation for pipes, pumps and tanks is 0.005; and
- The probability of adjacency (creep) mutation is 0.005 for pipes, and 0.02 for pumps and tanks.

9.2.1 Pressure Analysis

The pressure analysis assumed that the demand was 7.5 times the average demand level. This represents the instantaneous peak flow, which is used for pressure analysis as it tests out the system when it is most strained. The normal multiplication factor for instantaneous peak flow for South Australian studies is 5.0 (SA Water, 1994), however, due to the tourism loading on the services for Yorke Peninsula during the summer period, this is increased to 7.5. The required minimum pressure at every demand node is 20m (EWS, 1986(b)), as discussed in Chapter 5.

The cost structure for the pressure analysis is made up of the pipe costs and the pressure penalty costs. The pumping costs are irrelevant as it is assumed that the system is gravity fed for the steady state analysis. The total cost regime for the pressure analysis is shown in Figure 9.2.

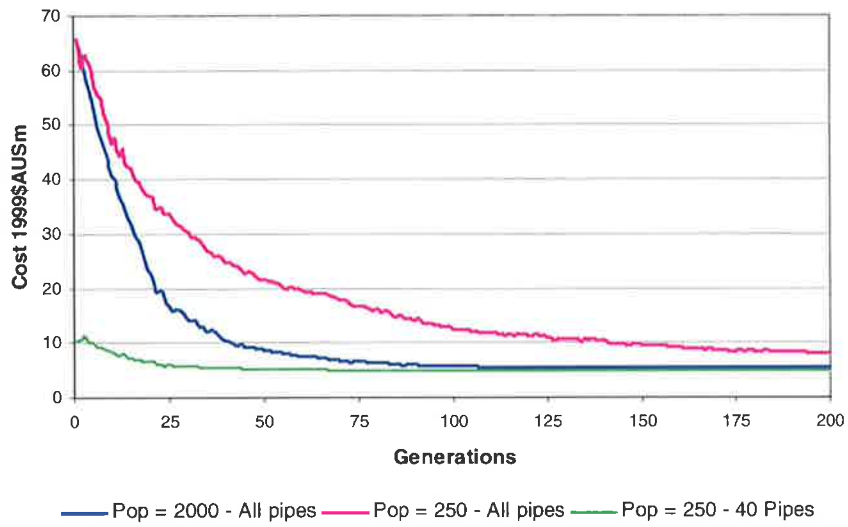

Figure 9.2 Best cost convergence for pressure analysis

In this analysis, three runs have been highlighted. The first is an analysis with all of the pipes being available for duplication with a high population of 2000 to counter the large search space.

The second also analyses all the pipes, however, has a reduced population of 250, which due to computational requirements is a more realistic parameter for the extended hydraulic and quality analysis. The final run examines a population of 250 analysing a reduced search space of 40 selected pipes. These pipes were selected using an iterative process, which is discussed further, in Section 9.2.2.

The pipe selections are listed in Table H.1, Appendix H.1. Table H.1 shows that the run of population size 250, with a selected number of pipes duplicates significantly fewer pipes (12 duplicates) than the run for all of the pipes with a population size of 250 (50 duplicates). The run with the larger population (2000), optimising for all the pipes converged to a solution with 16 duplicate pipes.

The pressure penalty profiles attached to the different runs are shown in Figure 9.3. From this it is clearly shown that the reduction in pipe options offered in the 40-pipe run hastens the removal of pressure deficiencies.

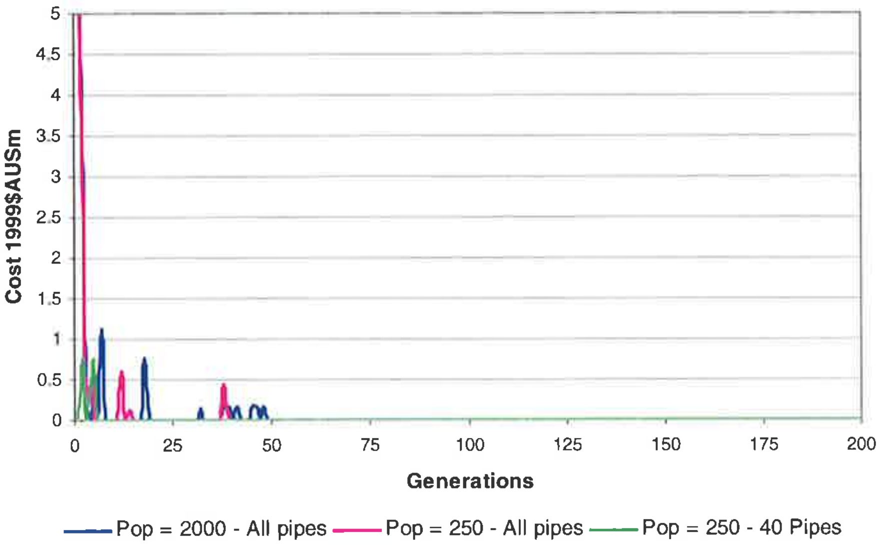

Figure 9.3 Pressure penalty profiles for pressure analysis

The results from the pressure analysis are part of the pipe selection process. These results, combined with the EWS recommendations, and the analysis of the extended period simulation, will form the basis for the location of pipe duplications.

9.2.2 Extended Period Simulation of Yorke Peninsula System

To determine from capacity optimisation which pipes are required for sizing, an iterative process is used. The pipes that were selected for sizing by EWS (1987) as well as the three-iteration process are listed in Appendix H.2.

The capacity testing was implemented using a series of 4 peak days. It is unlikely that the system will have to cater for this, however, the ability to meet demand under such duress makes for a robust design. Figure 9.4 compares the curve for the peak day against the average day demand curve.

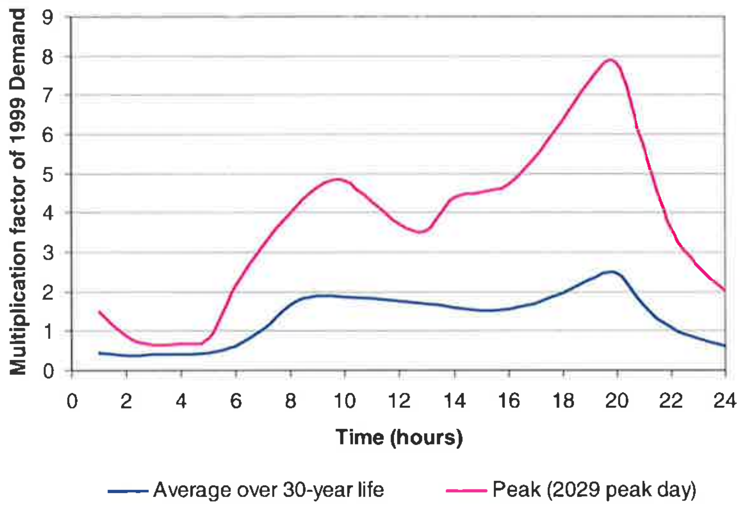

Figure 9.4 Comparison of peak day to average day

The peak day is calculated using the multiplication factor of 2.7 recommended by SA Water (1994). This is applied to the maximum average annual demand over the life of the system, which due to the 2% population growth rate (EWS, 1987) occurs in 2029 (assuming a project life of 1999 to 2029). The average daily flow is the average flow for the average year, which occurs approximately midway throughout the cycle (2014). The average is assumed for a medium day whereas the peak day is for a hot day (WATSYS, 1985) when demand is likely to be greater. A series of four of these peak days will thoroughly test the system.

The population size used for the selection process was 500, this means that there are 500 evaluations each generation. Initially all of the pipes were included for duplication in the optimisation run. The second run involved only those pipes duplicated from the first run, as well as few selected pipes, the third iteration involved these selected pipes with those duplicated in the second iteration. The first iteration included 206 pipes, the second iteration analysed 79 and the third examined 38 pipes. From this analysis, 40 potential new pipes were selected for the extended analysis. These pipes are listed in Appendix H.2. Figure 9.5 shows the convergence of the best-cost solutions for the three iterations.

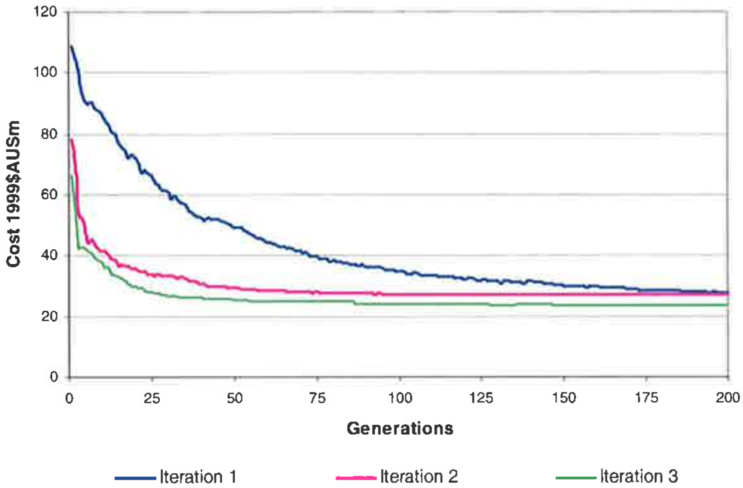

Figure 9.5 Convergence of 3-iteration solutions, total costs

The infrastructure convergence curves have been broken up into pipes (Figure 9.6), tanks (Figure 9.7), and pumping (pump construction and operation) costs (Figure 9.8). The curves experience a faster convergence per iteration.

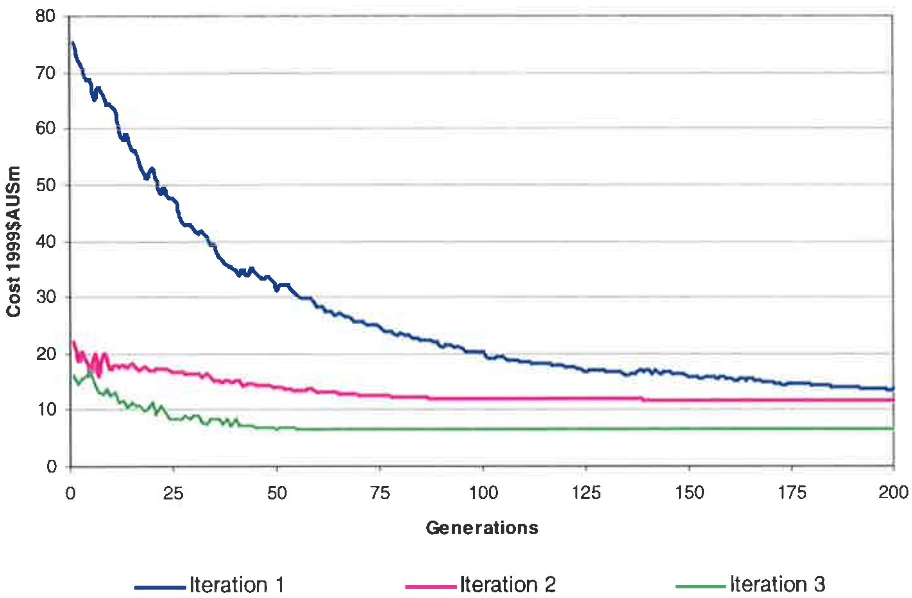

Figure 9.6 Convergence of pipe costs for 3-iteration process

As the search space is reduced by pipe selection, the infrastructure aspects most likely to benefit are pipes, which is evident from Figure 9.6. Tanks are also favoured as the reduced search space allows greater concentration on their sizing.

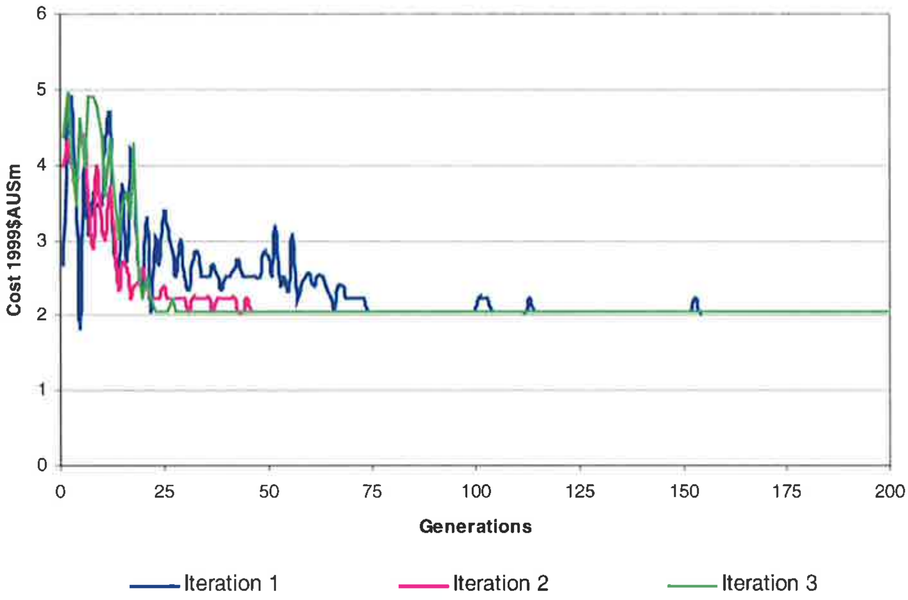

Figure 9.7 Conversion of tank costs for 3-iteration process

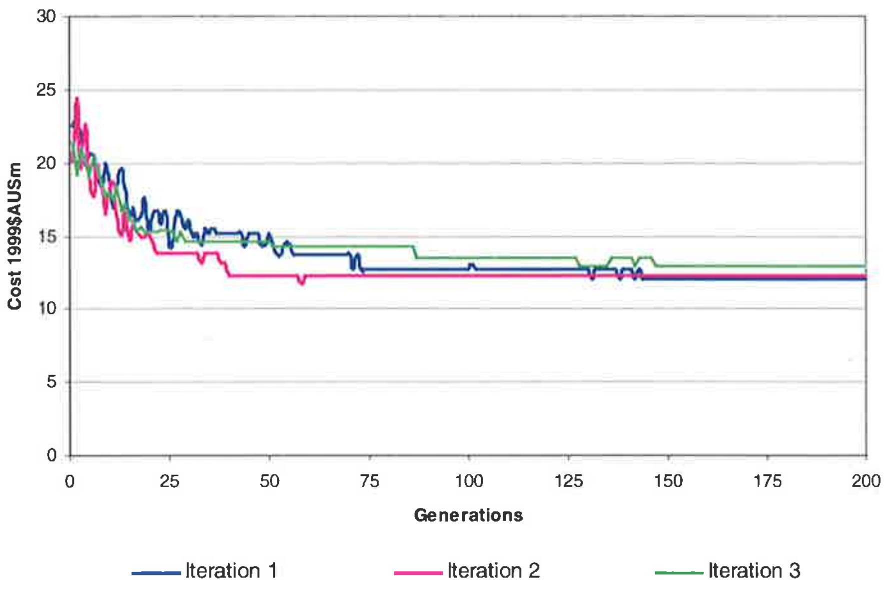

Figure 9.8 Convergence of pump costs for 3-iteration process

The pipe and tank costs clearly improved due to the reduction in search space. The pump costs do experience some continued variance. Reducing the number of pumping options could lessen this. The pump selection varies with the pipe sizes as the pump sizes for a system with fewer pipes to meet capacity and pressure demands would be greater than one with more extensive pipeworks. Overall, the search process is greatly enhanced by this reduction in search space. Reducing the number of infrastructure options also takes computational effort off of the hydraulics package, as less analysis is required.

The total penalty costs made up of the pressure penalty, the pumping penalty and the tank penalty (as discussed in Chapter 5) are shown in Figure 9.9. The first iteration reduces the penalty costs to zero, the second iteration attracts a single tank violation ($1m penalty), whereas the third generation cost has two tank violations ($2m penalty). This illustrates part of the problem in reducing the search space to a partial level. Some of the decisions required to fulfil the constraints set out for the model are removed from the search space disallowing the determination of the global optimum.

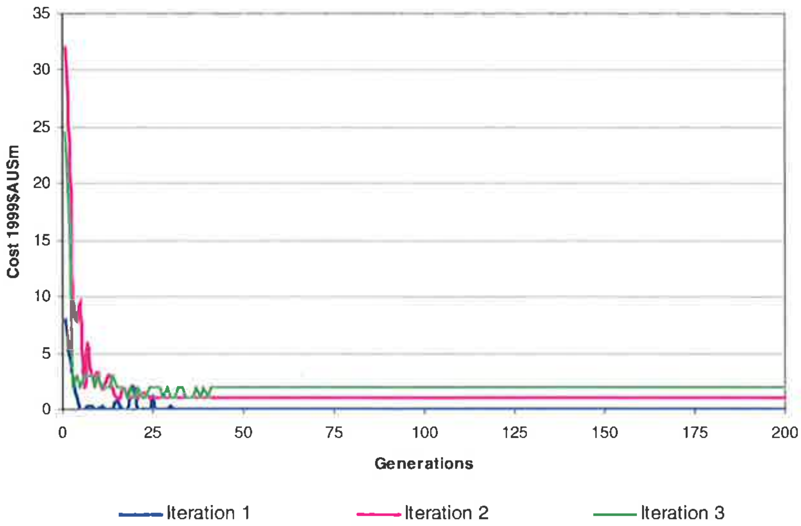

Figure 9.9 Convergence of hydraulic penalty costs for 3-iteration process

By determining the whereabouts of the hydraulic violation, additional pipes can be added to the analysis to counter any penalties that may occur in the extended analysis. In the case with the penalties evident from Figure 9.9, the South Kilkerran Tank was dropping to minimum levels. Inclusion of potential duplicate pipes leading up to the tank on top of the 38 pipes from the third iteration rectified this concern.

The locations of the 40 potential new pipes are shown on Figure 9.10. The pipe ID numbers relate to the EPANET signature. Th EPANET input file for the examination of the Yorke Peninsula System is included in Appendix H.3.

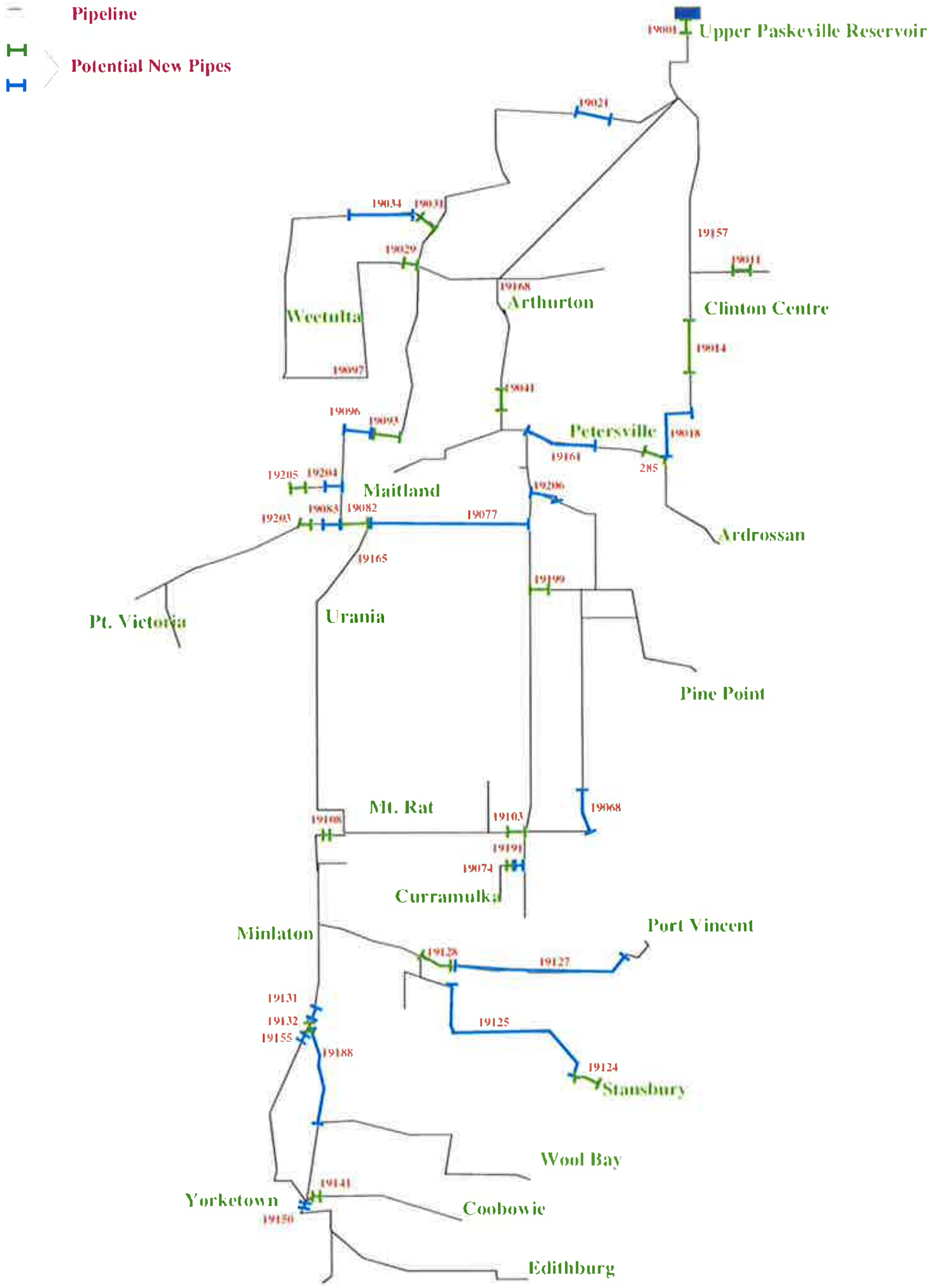

Figure 9.10 Locations of new pipes for the Yorke Peninsula system

The examinations of these potential pipes as well as the tanks and pumping possibilities form the extended analysis. This study looks at the different loading cases and tank operation throughout the different seasons. In doing this, water quality can also be examined.

9.2.3 Extended Hydraulic Analysis

The extended analysis examines the following to provide an idea of how the system will cope with an annual cycle:

- 1 hour of peak instantaneous flow to examine pressures throughout the system;
- 4 days of peak day flows to analyse the system capacity;
- 10 days of summer flow to check pump and tank dynamics;
- 17 days of spring flow;
- 17 days of autumn flow; and
- 38 days of winter flow.

The four days of peak day simulation combined with 10 days of summer simulation provide an adequate overview of the system operation during the high loading months. This allows enough time for the residual levels at the base of the system (Edithburg) to converge adequately for analysis. The combined spring and autumn analysis over a total of 34 days gives an adequate perception of the non-peak seasons.

The 38 days simulated for winter allows for the long residence times for this time of the year to provide acceptable data for this period. It is a long period of simulation, but required achieving the level of convergence for analysis of the water quality.

The results of the extended analysis produced two types of solutions. The first included more pipe duplications and fewer pumping points whereas the second made up for the lack of pipe duplications by more pumping to tanks throughout the system.

9.2.3.1 Hydraulic Solution with Weighting to Pipe Duplication

Figure 9.11 shows the convergence of the highest fitness (lowest total cost) for the first solution, which concentrated more on pipe duplication. Figure 9.12 shows the different infrastructure cost convergence paths for the GA search, and Figure 9.13 shows the hydraulic penalties converging to zero, thus indicating hydraulically feasible solutions.

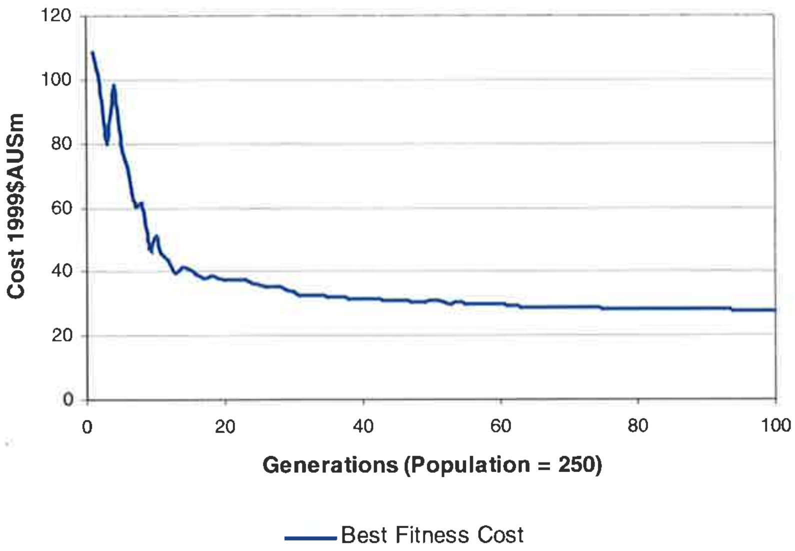

Figure 9.11 Convergence of Highest Fitness for Extended Hydraulic Analysis

The highest fitness converges to a cost of \$27.7m after 100 generations (25,000 evaluations). The infrastructure costs (Figure 9.12) converge at different rates. The tanks converge first (\$2.05m) as they have the smallest search space, the pipes next (\$11.4m) as the cost differences between choices are small, finally the pumps (\$14.22m), where a minor change in pump selection can lead to a significant difference in construction, operation and maintenance costs.

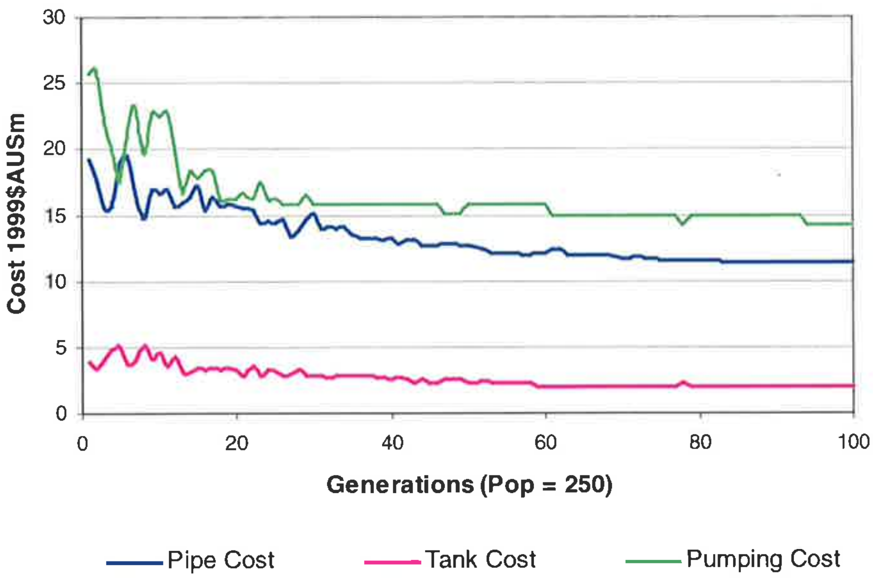

Figure 9.12 Infrastructure Cost Convergence for Extended Hydraulic Analysis

The hydraulic penalty costs (Figure 9.13) become zero after 30 generations (7,500 evaluations). The pressure penalty costs take the greatest time to converge as they are dictated by pump and pipe infrastructure selection, which has a large search space.

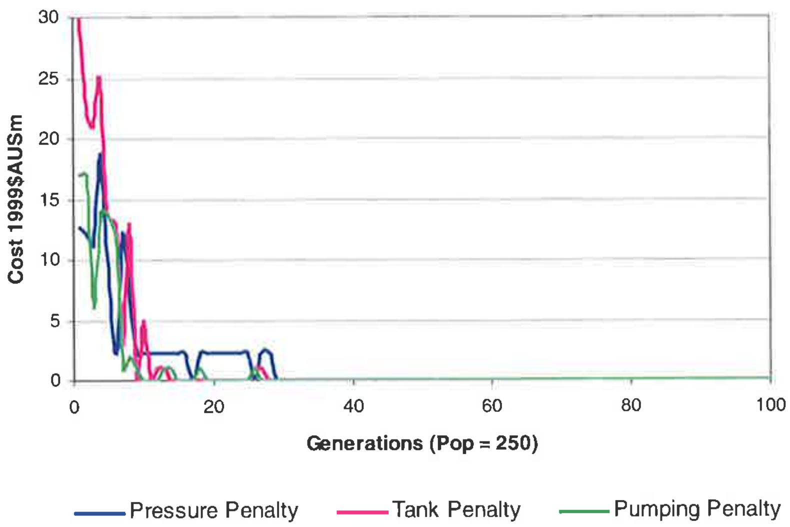

Figure 9.13 Hydraulic Penalty Convergence for Extended Hydraulic Analysis

The penalty cost structures are discussed in Chapter 5, the penalties used in the analysis of the Yorke Peninsula system are presented in Appendix E, which lists the constants used by the model. The layout of the hydraulic solution for this analysis is shown in Figure 9.14.

The water is pushed from Upper Paskeville Reservoir to Arthurton Tanks through two pumps in series at Upper Paskeville and Kainton Corner. These pumps have a combined design head of 130m, which corresponds to a shutoff head of approximately 173m, as EPANET has a factor of 1.333 between the shutoff and design head. The design capacity is 219L/s, which results in a maximum capacity of 437L/s, as EPANET considers the design capacity to be half of the maximum capacity. Arthurton Tanks supply the upper portion of the system and Maitland Tanks. The water from Maitland Tanks can flow to Ardrossan through a loop connected through pipe 285 connecting Petersville with this part of the system. A new tank at Petersville regulates the head.

Maitland Pumping Station Pushes the supply to Curramulka, Pine Point and farming communities, and with Mount Rat Pumping Station, supplies Mount Rat Tanks. The water then supplies the lower end of the system. The pipes connecting Minlaton with Stansbury and Port

Vincent have been duplicated to increase capacity. A booster pumping station supplies Minlaton tank. Pipe duplications occur throughout to alleviate pressure and capacity concerns.

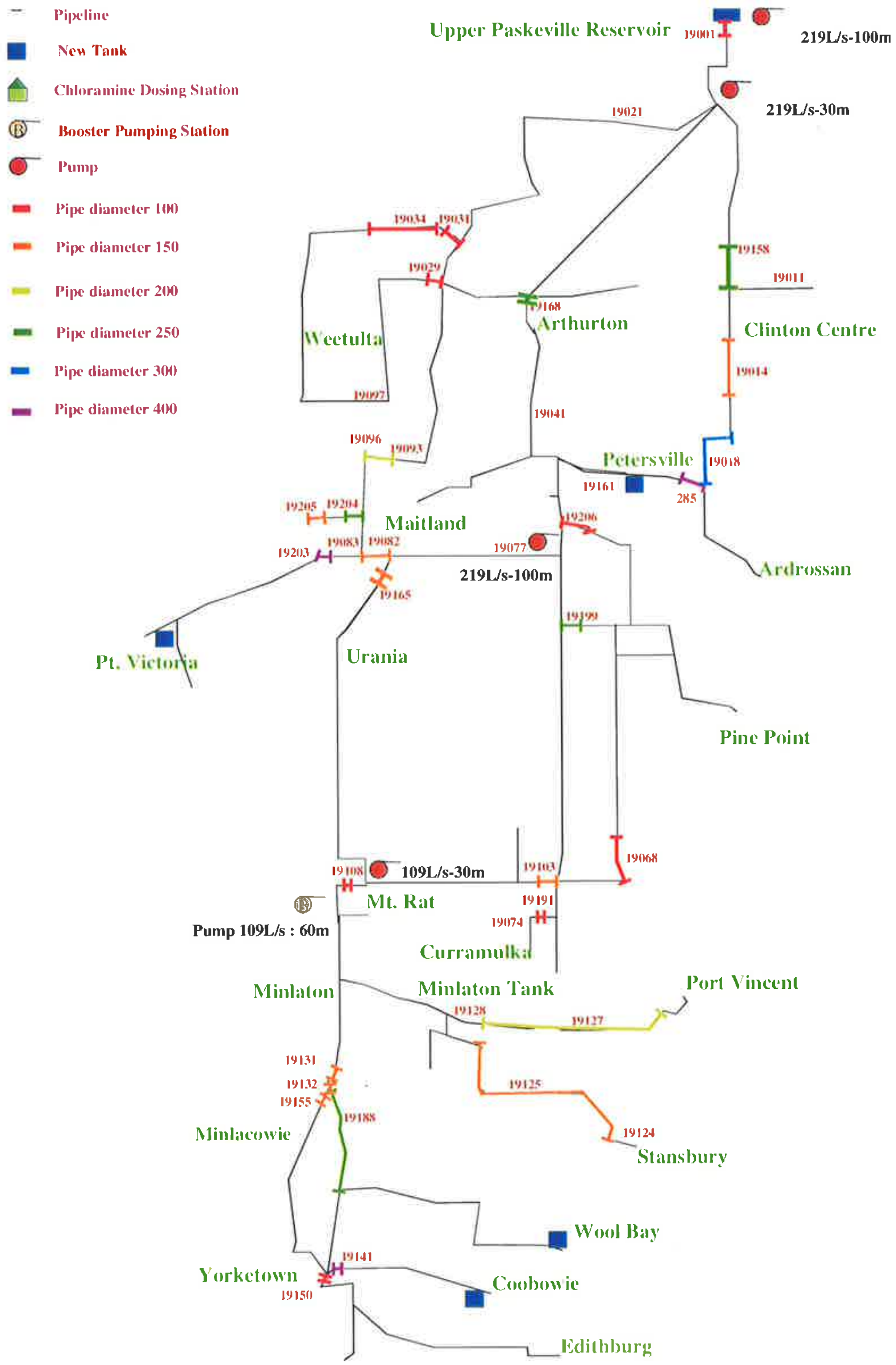

Figure 9.14 Hydraulic Solution for Yorke Peninsula Problem

9.2.3.2 Hydraulic Solution with Simplified Data Set

The second hydraulic option used a simplified data set compared to the first. Reducing the number of tank and pump options and decreasing the number of pipes that may be subject to duplication further reduced the search space. The reduction was feasible, as the pump selections never extended past 11 of the possible 14 options, and the tank selections never extended past 3 of the 6 options. In removing the selections that are unnecessary in determining efficient solutions, the GA has fewer possible solutions to sort through.

The two pipe duplications that attracted the highest cost, 19125 to Stansbury and 19127 to Port Vincent were also removed from this analysis, to test how the GA would respond to this option. These pipes were removed in response to not only their high cost, but also their exclusion from some of the 3-iteration solutions, which are shown in Appendix H.2.

Due to the reduced search space, a smaller initial population (150) was implemented in this analysis. The convergence curves for the lowest cost solution is shown in Figure 9.15. Included in Figure 9.15 is the average cost curve. This curve decreases from a huge cost to a level hovering around \$50m. This cost fluctuates due to the potential penalties for hydraulics that a tank, pump or pipe in the wrong location, or of the wrong size can present.

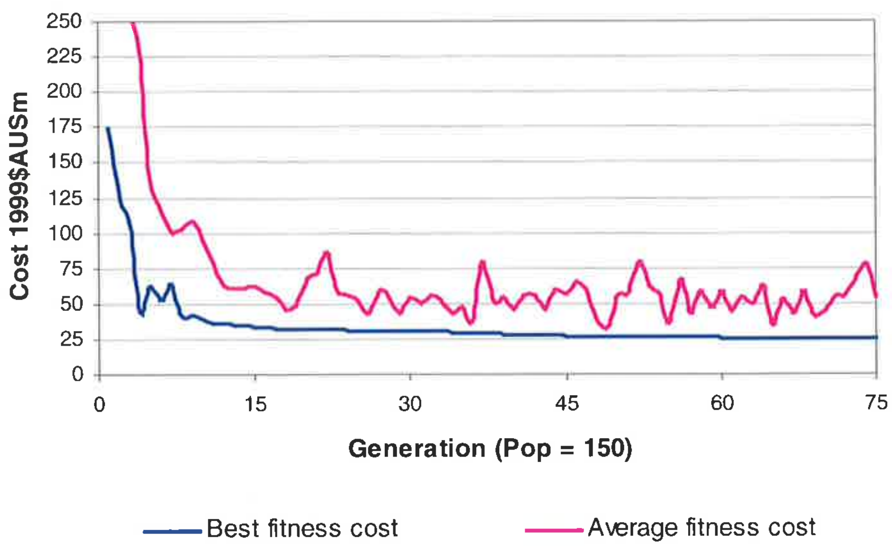

Figure 9.15 Best and Average Fitness Convergence Curves for Simplified Analysis

The best-cost solution moves down to a level of \$24.7m, which is 11% lower than the \$27.7m solution determined in the previous section. The reason for this cost reduction is the removal of the long pipes supplying Stansbury and Port Vincent. The GA did not make the necessary changes in the previous analysis as to attain a productive cost arrangement for this section of the system is difficult.

The capacity to Stansbury and Port Vincent needs to be increased to maintain the tank levels supplying the towns. This is effected by the pipe duplications to the towns, tank volume at Minlaton Tanks and pump selection to supply Minlaton Tank as well as Stansbury and Port Vincent. To produce a cost-effective solution, the GA has to remove both of the pipe duplications, and correctly size a pump to supply Minlaton Tank.

The costs for each of these decisions (Minlaton Tank Supply Pump, Stansbury Pipe and Port Vincent Pipe) are all significant portions (5-10%) of the total infrastructure cost. Because of this, the GA shows a reluctance to make these changes, as unless these are made simultaneously, penalty costs will occur at Port Vincent and Stansbury, as the supply to these locations cannot prevent the minimum tank levels being breached. If the population size were increased, the chances of such a move occurring would be enhanced.

It is clear that the reduction in search space assisted the GA to find more efficient solutions. Further reduction in search space would be risky, as the number of pump, tank, and pipe options available for selection all have the potential to form part of the preferred solution. Further reducing the number of pipes may begin to cause pressure and capacity problems for the system as well. The infrastructure costs of this solution are shown in Figure 9.16.

The tank costs remain unchanged from the previous solution converging to \$2.05m. The pipe costs are significantly lower (\$7.63m) as over 30km of pipe has been removed from the search space, which contributed a cost of at nearly \$4m. The pumping costs converge to a slightly higher level (\$15.05m) accommodating fewer pipe duplications.

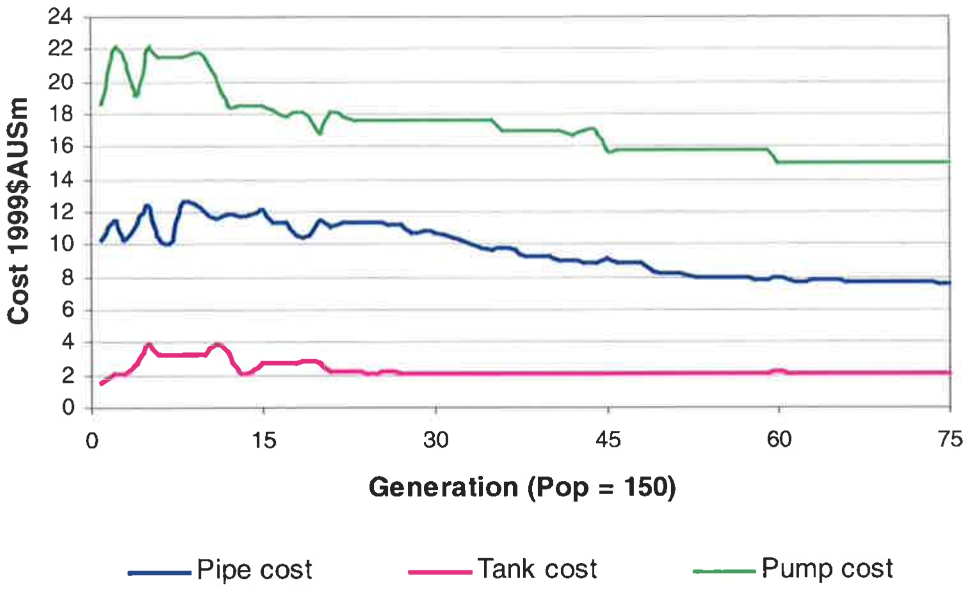

Figure 9.16 Infrastructure Costs for Simplified Analysis

The penalty convergence curves due to the infrastructure selections are shown in Figure 9.17.

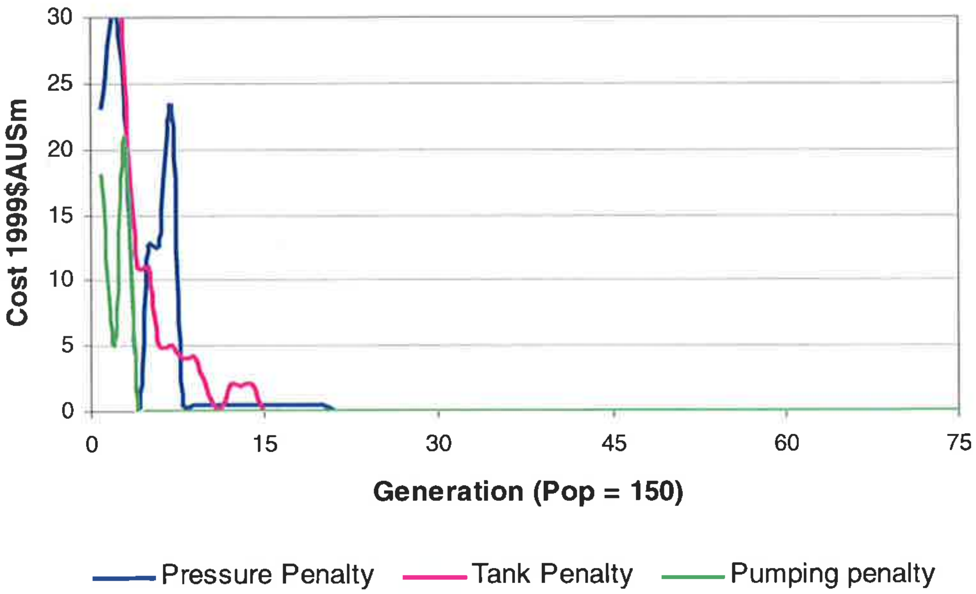

Figure 9.17 Penalty cost convergence for the Simplified Analysis

The layout of this solution is shown in Figure 9.18.

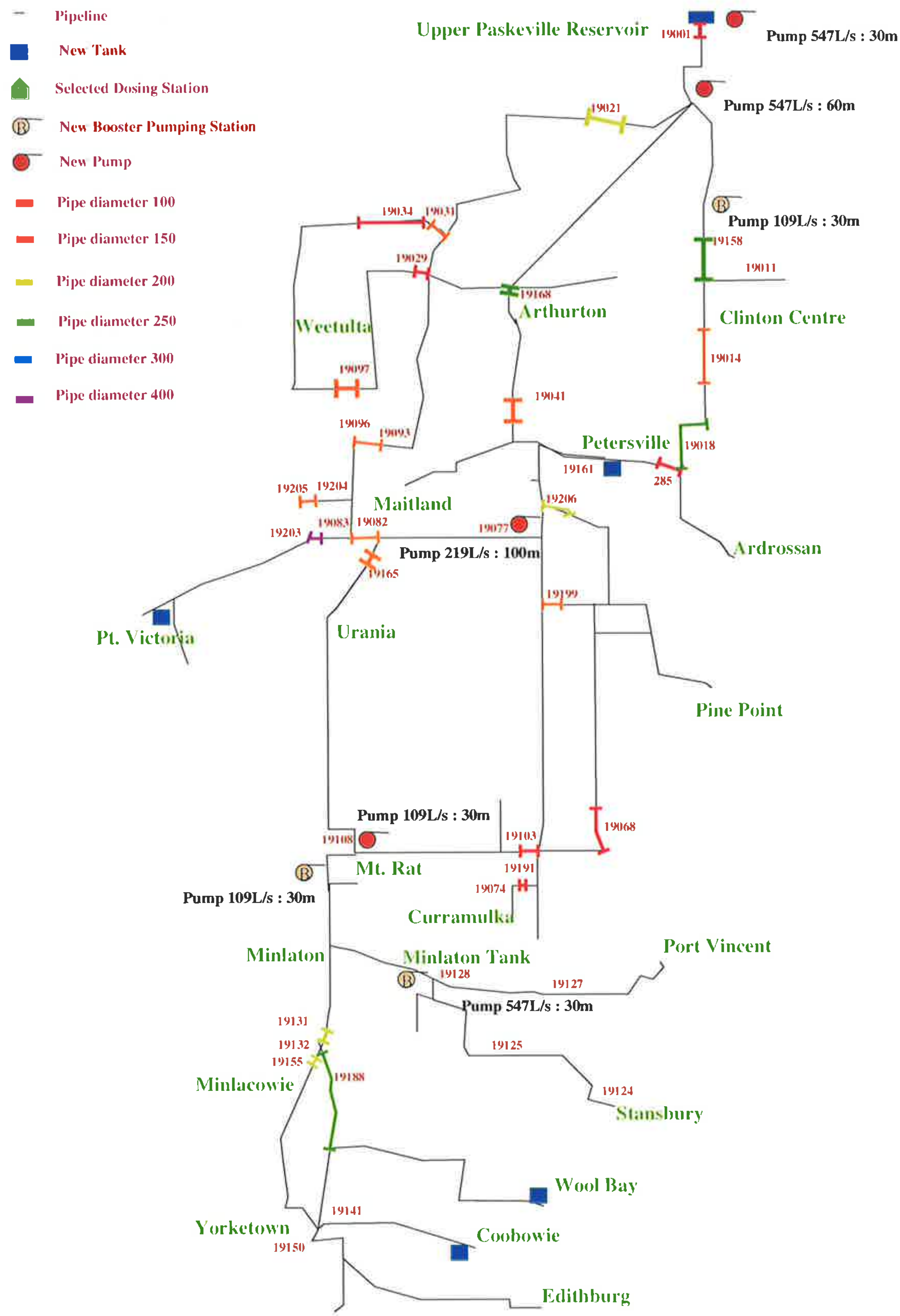

Figure 9.18 Network Additions for Simplified Extended Analysis

The differences between 9.18 and 9.14 are that a booster pumping station is now located just after the Minlaton tank (upstream of pipe 19128) to boost water to Port Vincent and Stansbury, thereby alleviating the need for pipe duplications. The extra pumping ability also removes the need for more minor duplications around Minlaton and Yorketown, as the water supply is more efficient to these locations. In making this decision, the GA showed versatility, which is a necessity in the water quality analysis of the system.

9.3 Water Quality Analysis

The system has undergone analysis of hydraulics resulting in a range of efficient feasible solutions. The solutions are cost-efficient, as they compare well with the previous EWS (1987) examination of this system resulting in a reduction in the hydraulic costs from an estimated \$31.4m to \$24.7m using the simplified search space, and \$27.7m using the larger search space. The efficiency of these solutions is heightened as the constraints that the solution had to match up to were more severe than the EWS (1987) examination, as demands through to 2029 were analysed, which are considerably higher than the 2011 demands required by the EWS (1987) report. Following on from these hydraulic results, the inclusion of water quality into the analysis brings several additional aspects to the design.

The first is the dosing regime for the system. Currently, to achieve residuals of approximately 2mg/L at the end of the system, residuals of around 7mg/L are observed in some sections. Restricting the overall dosing pattern to more acceptable levels is to be achieved.

The variation that this quest for good residual levels will have on the system infrastructure is an aspect that is of interest. With various controls throughout the network by way of tanks and pumps, water quality may become a factor in the final layout.

The social cost framework (as discussed in Chapter 5) will be applied to the system. Not only does this section aim to draw trends in design from the results, but intends to produce a cost effective design for hydraulic and water quality aspects for the Yorke Peninsula system.

Instead of looking to maintain a 2mg/L residual throughout the system, this analysis looks to keep residual levels above 0.5mg/L, which is the recommended aesthetic level (SA Water, 1995), and below 3mg/L, which is the maximum health related guideline level for South Australia (SA Water, 1995). Taste and odour concerns have also been predicted to take place for water with greater than 3mg/L chloramine (Krasner and Barrett, 1984).

The method used to gain the most effective convergence is to run the model for hydraulic analysis only, then, once convergence of the hydraulics has occurred, include the water quality component. This approach was discussed and used effectively in the earlier examination of the New York tunnels problem (Chapter 7).

9.3.1 Social Cost Analysis of Yorke Peninsula System

The social cost framework considers implementation of rainwater tanks for taste and odour concern, potential health risks due to waterborne disease and disinfection by-products (DBPs), and the physical costs of the chloramination process for the system. To illustrate the application of the GA to water quality analysis, a shortened, simplistic optimisation is discussed in this section. The population is 150; the total number of generations is 120, with 60 generations (9000 evaluations) of the hydraulic analysis and 60 generations (9000 evaluations) for the combined analysis. The infrastructure arrangement is as described in the simplified analysis of the hydraulic section (Section 9.2.3.2) and the dosing levels are rounded to the nearest mg/L. The convergence of the total fitness function is shown in Figure 9.19.

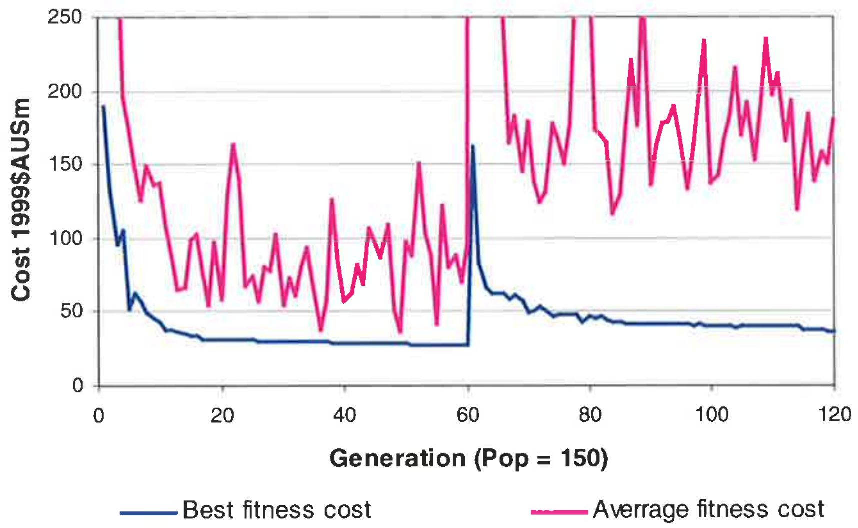

Figure 9.19 Convergence curve for fitness for social cost analysis

The solution initially reduces rapidly as only the hydraulic costs are being considered. After 60 generations (9,000 evaluations) the solution has converged significantly. The water quality aspect of the analysis then is added to the function as can be seen by the significant increase in fitness cost. The GA then optimises for the expanded problem until convergence is achieved for the full framework. The average fitness cost experiences a similar jump. This still fluctuates significantly, as the GA probabilities of creep mutation (Chapter 4) used are quite high for tanks and pumps.

An examination of the pumping costs indicates that the inclusion of water quality forces the expansion of existing stations and construction of new ones. The hydraulic analysis converged to approximately \$16m in pumping costs. This was raised to around \$20m after inclusion of water quality. Figure 9.20 shows the convergence of the pumping costs for the social cost analysis.

A potential reason for this is the saving in residence time that pumping achieves. By decreasing the residence time, lower levels of disinfectant can be added reducing taste and odour, DBP and disinfection costs without increasing waterborne disease costs, as was discussed in Chapter 7 for the New York tunnels problem.

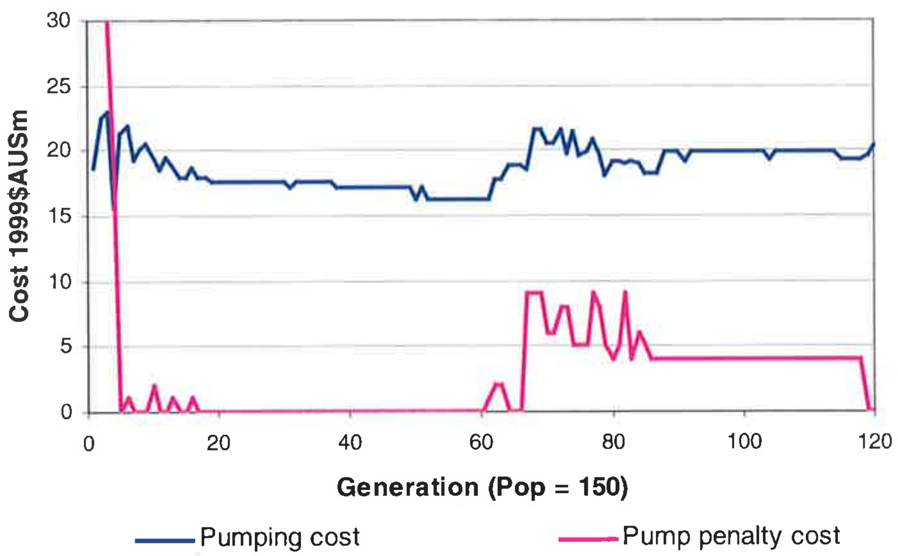

Figure 9.20 Pumping convergence for social cost analysis

The convergences of the new tank costs are shown in Figure 9.21. These costs converge to the same levels as the hydraulic solution after the inclusion of water quality into the analysis. The penalty costs of the existing tanks does alter for the inclusion of water quality costs, as the changes in pumping affect the tank levels throughout the system.

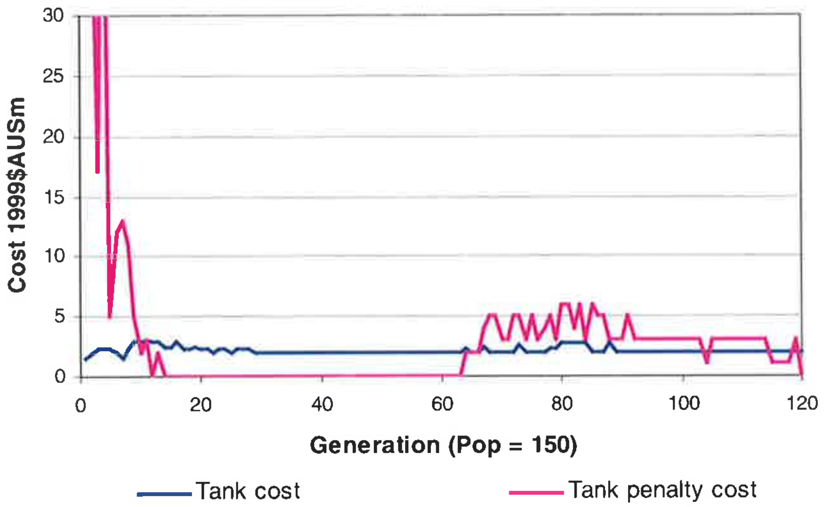

Figure 9.21 Tank convergence for social cost analysis

Figure 9.22 shows the costs of the pipe selections and the pressure penalty costs. Neither is greatly altered by the inclusion of water quality into the analysis. There is still some possible alteration with the hydraulic costs as the pipe and pumping costs are still fluctuating, however, the percentage difference of this alteration is low compared with the overall costs of the system.

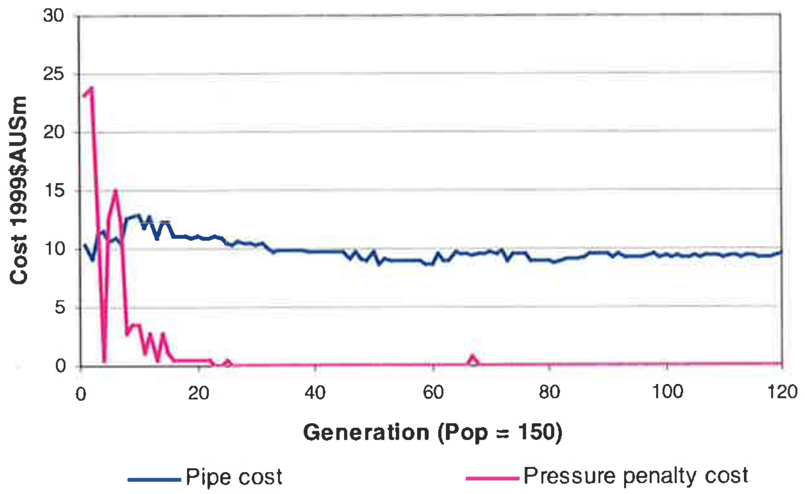

Figure 9.22 Pipe and Pressure Penalty convergence for social cost analysis

The water quality costs for the social cost method show excellent convergence to zero. As Figure 9.23 shows, the waterborne disease costs and the disinfection by-product costs are reduced to relatively minor levels ($0.74m and $0.1m respectively). These are reduced from maximum levels of about $400m and $25m respectively.

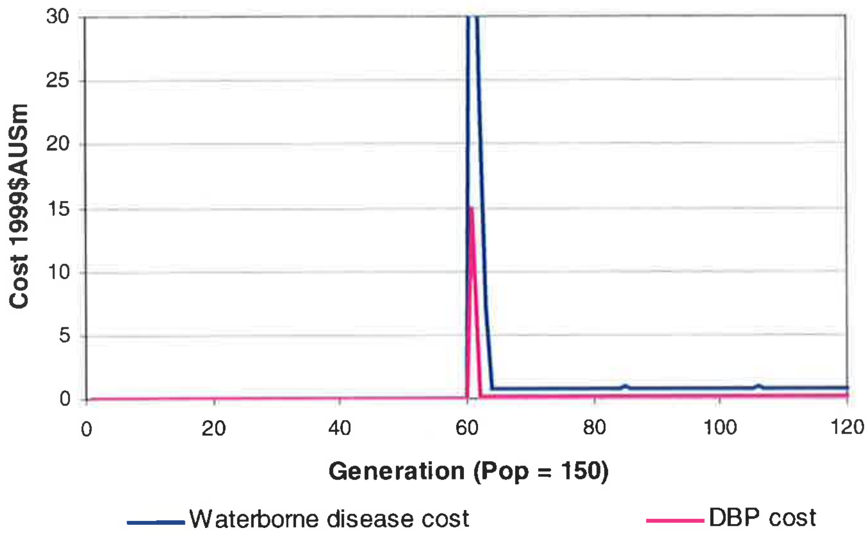

Figure 9.23 Waterborne disease and DBP cost convergence curves

The taste and odour costs and the physical costs of disinfection also converge well as shown in Figure 9.24.

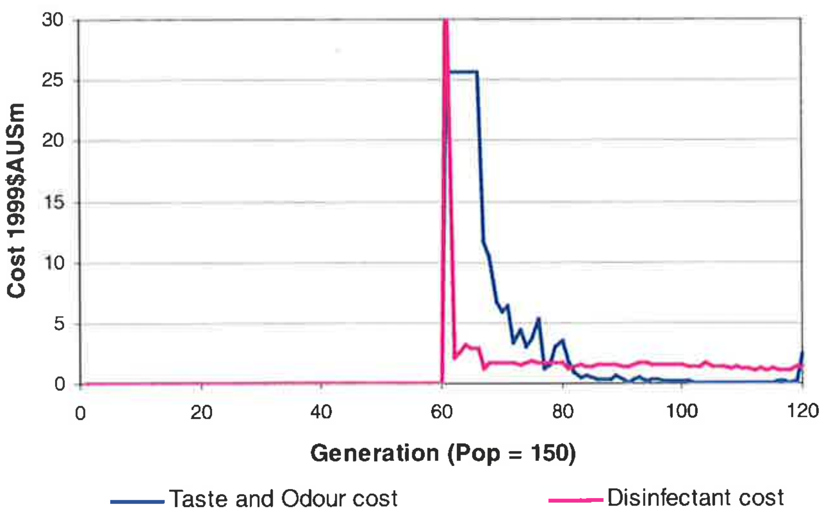

Figure 9.24 Taste and Odour and Disinfectant cost convergence curves

The taste and odour cost converge to zero from the maximum of \$25.56m, which occurs when the entire population living in the settled areas (population>50) of the system decide to invest in rainwater tanks to supplement their supply. The disinfectant cost converges down to about \$1.35m. The chlorine residual levels throughout the system from the solution produced are shown in Figure 9.25.

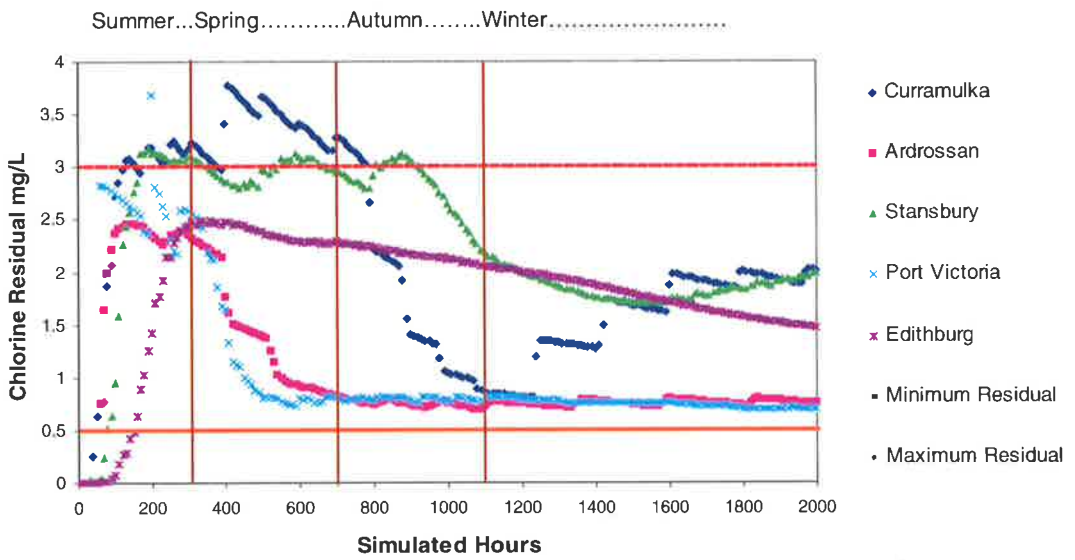

Figure 9.25 Simulated Residuals for Yorke Peninsula Towns for Social Cost Solution

Figure 9.25 looks at the following towns:

- Ardrossan, a fishing town on the east coast at the top end of the peninsula;
- Port Victoria, a fishing town on the west coast towards the middle of the system;
- Curramulka, a farming community right in the middle of the system
- Stansbury, a seaside town on the east coast in the lower end of the system; and
- Edithburg, the shipping port located right at the bottom of the system.

These towns represent a good coverage of the system, with all extremities examined, as can be shown from the Map of the Yorke Peninsula in Appendix I.

In the main, the residual levels for the towns after convergence has occurred for each season are within the 0.5mg/L to 3mg/L boundary specified as the desired levels. The measured portion for the GA cost analysis is at the intersection of the seasonal time boundaries (summer = 300 hours, spring = 300-700 hours, autumn = 700-1100 hours and winter = 1100-2000 hours of simulated time). For this run, only Curramulka, with a residual of 3.1mg/L in spring exceeded the guidelines. To attain this residual, the dosing regime recommended is shown in Table 9.5.

Table 9.5 Dosing regime for a typical short GA run for the Yorke Peninsula Study

Season	Paskeville	Maitland	Mount Rat
Summer	3mg/L	1mg/L	0mg/L
Spring	1mg/L	3mg/L	0mg/L
Autumn	1mg/L	0mg/L	1mg/L
Winter	1mg/L	2mg/L	1mg/L

The dosing level options are very approximate (0mg/L, 1mg/L, 2mg/L, 3mg/L etc), however, they give a good understanding as to how the GA undertakes the task of providing results for water quality concerns. In reality, the level of applied dose is less strict. From these levels, Mount Rat dosing station is not used throughout summer or spring, and Maitland is not used in autumn.

The social cost structure works effectively in producing a cost-effective solution to the Yorke Peninsula system. The resultant hydraulic costs compare well with the EWS study, and the water quality obtained from a combination of the selected infrastructure and the dosing regime produce an effective solution. The solution recommended by the social cost analysis is shown in Figure 9.26.

The total infrastructure costs posed by the best solution are approximately \$31m, with \$1.3m added disinfection costs, bringing the total physical cost approximation (penalties notwithstanding) to \$32.3m. This compares with an estimate of \$33.2m for the EWS costs (Section 9.1), which considers the demands on the system to 2011. The model used in this study considers the system to 2029, which requires the system to cater for considerably greater peak and average demands.

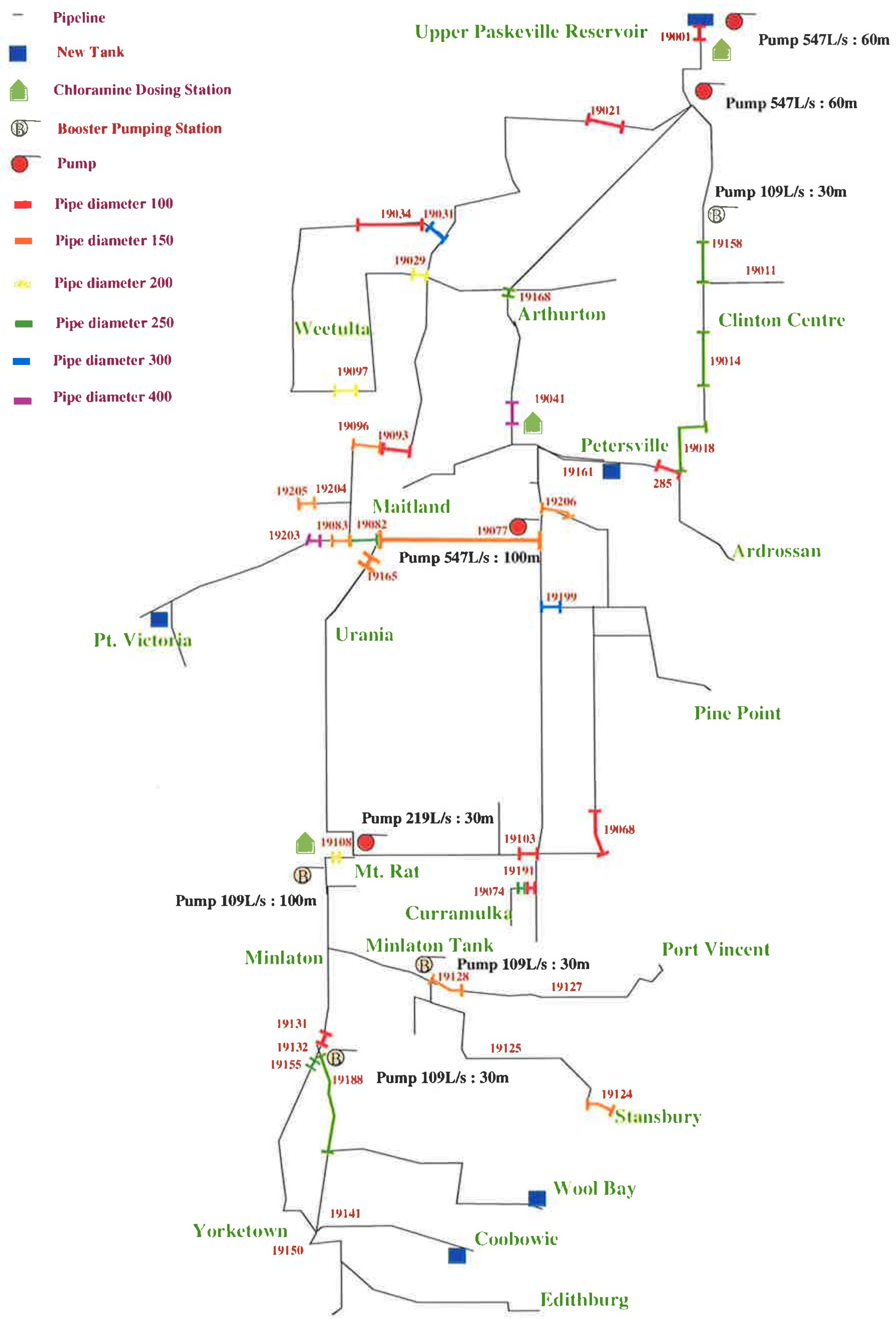

Figure 9.26 Social Cost solution for Yorke Peninsula distribution system

The major differences between the setup for the combined analysis (Figure 9.26) and the hydraulic analysis (Figure 9.18) are the expansion of pumps south of Mount Rat that supply the Minlaton tank site, (upstream of pipe 19128). The pump capacity from has been increased from 109L/s and 30m head to 109L/s and 100m design head, allowing a greater flow if required.

The pumping ability from the pumps in series at Upper Paskeville and Kainton Corner has been increased. The combined design capacity is still 547L/s, however, the design head has increased from a total of 90m, to a total of 120m. This increase in design head allows the pumping of greater capacities, for the same head increase, compared to the former pump regime.

The pump capacity from Maitland pumping station through to the Curramulka region has also been increased from 219L/s and 100m head to 547L/s and 100m head. A booster pump to Yorketown and Edithburg from Minlacowie tank has been installed with a 109L/s design capacity and a 30m design head.

All of these selections decrease the residence time in the system. The only pump that decreased in size was the booster from Minlaton tank to Port Vincent and Stansbury, which didn't require a 547L/s flow with a 30m head, only a 109L/s flow and a 30m head, thus representing a cost saving. This booster may have been reduced by the hydraulic run in the GA if given adequate convergence time.

Some of the other differences in pipe size may be part of the ongoing GA process, and with sufficient initial population and lengthened run-time, may not be selected. The GA also found a good water quality regime for the design.

Given more run time, it is believed that the GA would converge to provide residual levels more consistently around the 0.5mg/L due to cost savings in disinfection practice, the results obtained from the social cost method illustrate a good convergence to the set residual boundaries. The hydraulic aspects would also continue to be integrated at an improved rate with the water quality cost structure.

Despite being a general examination, the above analysis succeeds in showing the capabilities of the GA and the general trends that a system of the nature of the Yorke Peninsula will follow for a full optimisation.

9.4 Discussion of Results for the Yorke Peninsula Study

The Yorke Peninsula system is one of the most difficult systems to examine in South Australia. This is due to a combination of:

- Complexity of the system with considerable numbers of pumps, tanks and system loops;
- Seasonal fluctuation in demand, as due to tourism, the summer and winter peaks are much more pronounced than a typical system;
- Long residence times, making the analysis of water quality an extremely difficult task as the convergence of disinfectant residuals takes up enormous computer run-time; and
- Relatively low, evenly distributed population, forcing an analysis to concentrate reasonably evenly on the entire network rather than supplying to a city that makes up the majority of the population.

From the results shown in this chapter, the areas that require further discussion include the differences and reasons for the differences between the hydraulic results and results of the full analysis. The best process of attaining good results at an efficient rate for a system with the complexity of Yorke Peninsula is also an area for discussion. The third aspect to be developed is the method of penalising for poor water quality for different systems, as a low population means that less focus will be on water quality issues for a social cost framework.

9.4.1 The Difference between the Hydraulic Solutions and Combined Solutions

Unlike the examination of the New York tunnels system (Chapters 6 and 7), the converged hydraulic solutions for the Yorke Peninsula system did not provide as good a starting point for the inclusion of the water quality analysis. The quality cost factors went through a period of instability before the robustness of the GA technique force a reasonable overall solution. The infrastructure component that is the most sensitive to this is the system pumping ability of the system.

The hydraulic solution optimises for pressure and capacity constraints to find the least cost solution that minimises pipe duplication, tank installation and pumping capacity. Unlike a gravity-fed system, where the preferred hydraulic solutions use the fewest pipes with the lowest diameters, the best hydraulic solution for a controlled system does not consider residence because of the pumping effects.

A gravity-fed system, in minimising the pipe sizes, inadvertently reduces the residence time. This aspect (pipe minimisation) as well as tank minimisation do assist in the reduction of residence time, as shown by the small change in levels from the preferred hydraulic solutions to the cost-efficient combined solutions. For a pumped system, the pumping minimisation that occurs in the hydraulic solution does not always reduce residence time as well. The best hydraulic solutions have the lowest pumping capacity and head to maintain the system. These are then the least desirable feasible solutions for residence time, as they are for delicately balanced systems. An increase in pumping ability would allow a greater flow to be pumped, reducing the travel time to tanks, and allowing for quick supply to areas that are experiencing a quality concern. To examine this in greater detail, the results of the social cost analysis using the simplified system structure (Section 9.3.1) are discussed by comparing them with the hydraulic analysis (Section 9.2.3.2) of the same system.

Once the combined approach is included in the optimisation, the pumping levels go through another little optimisation that balances the additional cost of increasing capacity against the quality cost savings in a reduction of the residence time. In the upper end of the system (Paskeville to Maitland) this is even more important as the decay rates for this area are greater than for the lower end of the system.

A difference in the solutions was that the Upper Paskeville pumping station had a level of 30m head for the hydraulic solution, which combined with Kainton Corner (60m head) gave a design head of 90m. With the combined analysis, Upper Paskeville pumping station was increased to 60m head giving a total design head of 120m.

The pump curves for these solutions are shown in Figure 9.27. These curves are based on the EPANET pump curve method, where the actual capacity of the system is twice the design capacity, and the shutoff head for the pump is 1.333 times the design head.

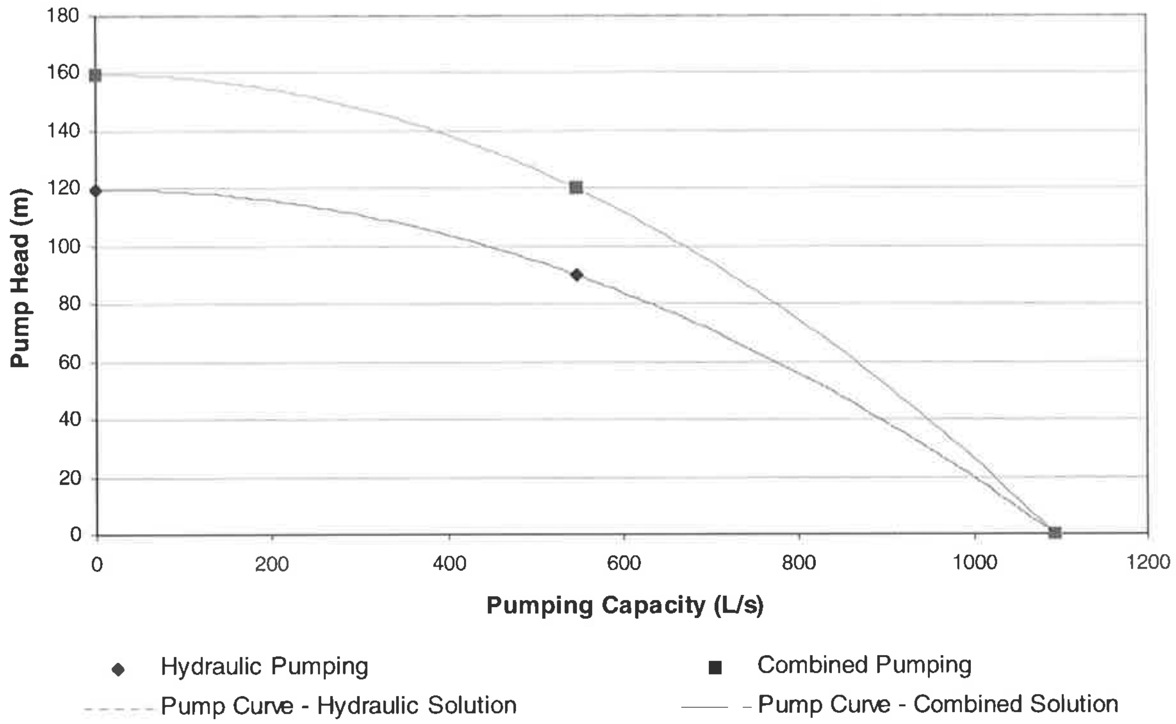

Figure 9.27 Pumping Curve Comparison for Upper Paskeville/Kainton Corner

To guarantee a supply from Upper Paskeville to Arthurton, a head lift of around 115m is required accounting for a 110m difference in elevation, and friction losses. The pumping capacity to manage this for the preferred hydraulic solution is approximately 225L/s, whereas the preferred combined solution can provide this head with a flow of nearly 600L/s. This increased flow reduces the residence time of the water by approximately one day.

As this is at the top end of the system, the decay rates for the water initially are greater than the lower end of the system. This is due to the two first-order decay models used to simulate the second-order decay model for this study. A one-day saving in residence time can result in a 5% saving in chloramine residual during winter (0.06/day decay rate) to a 16% saving in summer (0.20/day decay rate) as shown in Table 9.6.

As this saving is directly after the initial dosing at Upper Paskeville, the effect that it has on system quality is very significant.

Table 9.6 Water quality improvement by altered pumping structure

season	Decay rate for upper system (day^{-1})	% chloramine remaining for combined pumping solution	% chloramine remaining for hydraulic pumping solution	% chloramine saved by water quality optimisation
Summer	0.2	88.7%	72.6%	16.1%
Spring	0.11	93.6%	83.9%	9.8%
Autumn	0.08	95.3%	88.0%	7.3%
Winter	0.06	96.5%	90.8%	5.6%

The expansion of the pumping station from south of Mount Rat to Minlaton tank also represents residence-time savings for the system. The selected pump has a design capacity of 109L/s and an increase in design head from 30m to 100m allowing an increase in pumping capacity for the same head increase. The booster to Yorketown, and indirectly Edithburg, Wool Bay and Coobowie from Minlacowie tank has the potential to save residence times to these locations in the system by 0.5-2 days dependant on season and location.

Although the additional pumping requirements add to the infrastructure costs of the system, the savings in residual can be quite significant throughout the system. By reducing the residence time, the amount of disinfectant decay is reduced, thus lowering the health, aesthetic, and physical costs of the water quality profile. As found in the earlier New York tunnels study, the residence time of the system is the critical component in achieving a preferred water quality regime. The GA optimises to reduce the residence time based on the water quality costs, while not increasing the system cost too greatly. In a complex controlled system, it alters pump selection to achieve this aim.

9.4.2 The Process of Determining Good Solutions Efficiently

During the analysis of the Yorke Peninsula system, gaining a good solution set occasionally proved to be difficult, or take a long time to achieve. This was due to the complexity of the system and its long residence times. It was found that a trade-off emerged between convergence to solutions of lower cost and computation time. The 3-iteration method of determining a simplified hydraulic structure for the system proved to be important in achieving good results to this problem in a reasonable duration.

The method of solving for hydraulics, then the combined approach as discussed for the New York tunnels problem (Chapter 7) proved useful, yet not as successful due to the pumping aspects of the Yorke Peninsula system discussed in Section 9.4.1. This technique still allowed the determination of better result sets. This approach also allows a significant reduction in run-time compared to other approaches, as the duration of the simulation can be manually altered so that a hydraulic analysis, which requires a 100-hour simulation, can be used to attain hydraulic convergence. This is then followed by a combined analysis using a 2000-hour simulation.

9.5 Conclusions

The results obtained from the analysis of the Yorke Peninsula system indicate that residence time is again the important factor in the improvement of the water quality profile. By increasing the pumping capacity of the system, the GA effectively selected a layout that reduced the residence time, allowing significant savings of the disinfectant residuals.

Using this reduction, the GA succeeded in providing an acceptable residual pattern throughout the system, with nearly all demand nodes in the system being within the water quality guidelines for each season. The GA managed to acquit itself admirably when faced with optimising for water quality, pumps and tanks, which was a considerable jump from optimisation of pipes alone.

Although the results are more general than the New York analysis, they provide a good insight into the relationships formed in a controlled system. The results also provide good solutions to a very difficult distribution system, eclipsing previous studies while providing a good water quality profile. The GA optimisation method can take considerable computer time, as was evident with the Yorke Peninsula study, which limits the convergence towards the optimal solutions with systems of that level of complexity.

As with the New York study, the preferred solutions for the combination of water quality and hydraulics differed from the hydraulic solution alone. The difference in the Yorke Peninsula system was more marked than the gravity-fed New York study due to the alterations in pumping capacity for the system.

Chapter 10 Summary and Conclusions

10.1 Summary of Thesis

This thesis has provided a single framework for the optimisation of water distribution systems considering both hydraulic and water quality considerations.

The framework that was used involved the quantification of both the infrastructure costs and the issues of water quality. By combining these factors into a single fitness function and employing a GA optimisation technique, cost efficient solutions for both these facets can be determined. By linking these parameters with the water simulation package, EPANET, optimisation of water system design was achievable.

To thoroughly test the performance of the different methods of combining the water quality and hydraulic issues, the model was applied to the well-researched New York tunnels problem. This study was successfully optimised by previous researchers using GAs, allowing the model designed for this research to be compared against a recognised design. The model built for this thesis successfully identified the current optimal solution for the New York tunnels problem, matching the work carried out by previous researchers.

The inclusion of the water quality framework in the design process led to a slightly different preferred final design. This solution saved a considerable amount of residence time, which reduced the disinfection costs and the potential health and aesthetic costs. The best method of determining the efficient solution space was also discussed, with the method of optimising for hydraulics prior to the inclusion of the water quality analysis proving the most viable option.

The methodology determined from the New York study was then applied to a system on the Yorke Peninsula, South Australia. This system was a controlled network of pipes, pumps and tanks, with several potential dosing locations. The GA model successfully found an efficient solution able to meet the demands over a 30-year system life, and identified how the inclusion of water quality in system design could alter the preferred solution space.

10.2 Contributions of the Research

This thesis has made the following contributions to the field of water engineering:

1. It has developed a GA model capable of identifying efficient designs and operations for a distribution system based on hydraulic and water quality concerns;
2. It has formulated a framework for quantification of water quality concerns due to waterborne disease, disinfection by-products and taste and odour based on realistic potential costs. This allowed a total quantification of different distribution costs, that were combined in a fitness function to be optimised;
3. It has shown that the hydraulic optimal solution is not necessarily the preferred solution for the inclusion of water quality;
4. It has contributed to the mechanism of modelling long duration simulations by developing methods of reducing the computational time for an optimisation by both a reduction in search space, and lowering the simulation time required; and
5. It has identified relationships between the different water quality issues with each other and the systems hydraulic costs.

10.2.1 The Development of a Model that Can Optimise for Hydraulic and Quality Concerns

Research prior to this thesis concentrated primarily on optimising distribution systems for hydraulics. Some researchers had examined optimisation of dosing station location and dosing regime (Bocelli *et al*, 1998), others went a step further and included pump and tank operation in optimising disinfectant residual (Goldman and Mays, 1999).

This thesis took this work a step further by sizing and locating pipes, tanks and pumps (ie the hydraulics of the system), along with the location and seasonal dosing regimes of the dosing stations. In taking this step, the model was able to base a system layout on an optimised fitness function that considered both hydraulic and water quality costs.

This research also pioneered the inclusion of optimising water quality in distribution systems using genetic algorithms.

10.2.2 The Formulation of a Social Cost Framework

Previous research into optimising for water quality in a system used penalty factors to regulate the disinfectant residual levels throughout the system. This thesis introduced the idea of a social cost framework. This framework quantified the potential costs due to waterborne disease, disinfectant by-products, taste and odour and physical disinfection costs, combining them in a single structure to minimise the total cost of these issues.

The social cost framework was based on the consumer population throughout the system and provided realistic costs based on actual concerns. Other researchers have discussed the need to achieve balance between the risks posed by waterborne disease and disinfection by-products. However, this study balances the costs of potential cancer contraction from DBP exposure and the costs of waterborne disease with the disinfection costs, aesthetic (taste and odour) costs and infrastructure costs of a distribution system.

10.2.3 The Hydraulic Optimum compared with the Hydraulic and Water Quality Optimum

A major contribution of this thesis is the examination of whether the inclusion of water quality costs in the fitness function has a significant bearing on the optimum design. If the answer is no, the hydraulic optimal is the overall optimal, hence removing the need for inclusion of water quality concerns in the design. If the answer is yes, the examination of water quality becomes an important facet to be considered during the distribution system design process.

This study shows that the efficient solutions for combined water quality and hydraulics do not necessarily match the optimum solution for hydraulics alone. Analysis of the New York tunnels problem illustrated that the solution using the combined fitness function produced a preferred solution that had slightly higher infrastructure costs and did not preform as well hydraulically, yet saved a significant amount of residence time in the system. The results from the Yorke Peninsula system showed that the preferred solutions for the inclusion of water quality had higher infrastructure costs due to expanded system pumping, which also saved a considerable amount of residence time.

10.2.4 The Contribution to Mechanism of Modelling for Water Quality in a System

As discussed throughout the thesis, modelling for water quality requires significant run times to allow for convergence of residual levels. When combined with a large search space due to the extensive infrastructure locations and sizing options, the long simulation times require considerable computation time to optimise for a distribution system.

This study suggests the following techniques to reduce the computational time:

- An iterative approach to reduce the search space as used for the Yorke Peninsula study;
- Determining that the most efficient method of obtaining the preferred solutions was to optimise for hydraulics, then include water quality into the analysis once convergence was achieved; and
- By manually altering the duration of the hydraulic analysis, a saving of nearly 50% was achievable.

10.2.5 The Development of Distribution System Relationships

Through examining the different issues using a common fitness function, this study has identified relationships between the different cost components that make up the system cost. The relationships determined, include those between the various water quality factors, as well as those between the hydraulic costs and the water quality costs.

10.3 General Conclusions

The successful application of genetic algorithms to water distribution systems has been well documented. The inclusion of water quality in the GA provides the following conclusions:

- The GA model constructed for this study compared well with previous optimisation models, finding the current best solution for the New York tunnels problem;
- The GA succeeded in increasing the efficiency of system pumping and tank construction for a complex system
- The GA is successful in minimising the water quality costs experienced in the supply;
- The GA is able to be applied to optimisation of dosing levels for different demand conditions; and

- The best convergence path for determination of the complete system solution is to firstly attain convergence for the hydraulic solutions, then include the water quality aspects into the optimisation.

There were three different methods used to determine the best solution for water quality as well as hydraulics. These were:

1. The quality penalty method, which has been adapted from previous studies;
2. The social cost method, which applies realistic costs to distribution issues; and
3. The residence time method, which was included as a possible mechanism of optimising system layout for the New York tunnels problem.

The conclusions that can be made from these methods include:

- Both the quality penalty method and the social cost method have proven to be good mechanisms of restricting the system residuals to desired levels;
- The weightings used in the quality penalty method were a major factor in the response of the system to water quality issues. This was less of a concern in the social cost framework, as these costs were based on system information and made specific to each network; and
- The residence time method showed potential as a possible mechanism to optimise for water quality. The application of this method would be exceedingly difficult for the Yorke Peninsula network, as there are many pumps and tanks that would produce an irregular residence time pattern. For systems with shorter residence times and fewer controls, such as the New York study, this method could be adapted to work very well.

The model provided cost-efficient solutions to different systems. The following conclusions were made in respect to comparing the entire model framework with one based purely on hydraulics:

- The optimal hydraulic solution is not necessarily the preferred solution for the analysis of hydraulics and water quality issues;
- The optimal hydraulic solution tends to provide reasonable water quality efficiency in gravity fed systems;
- The optimisation of residence time is an important factor in determining the preferred solutions for the combination of hydraulics and water quality; and

- In pumped systems, the preferred water quality solutions require greater pumping capacity to reduce residence times than the optimal solution for pure hydraulics.

10.4 Case Study Conclusions

This section presents conclusions that are specific to the individual case studies of the Yorke Peninsula system, and the New York tunnels problem. As the systems studied were significantly different in both layout and population served, some conclusions from the results cannot be generalised to cover both.

10.4.1 Case Study 1 – The New York Tunnels Problem

The New York tunnels problem is a well-researched case study. It is a relatively simple urban supply scheme with low residence times and a high population. It is a gravity-fed system with a relatively short residence time that uses chlorine as a disinfectant. The conclusions from a study with these attributes include:

- Residence time was the controlling factor in determining chlorine residuals in the system; and
- The preferred combined solutions were very similar to the preferred hydraulic solutions.

10.4.2 Case Study 2 – The Yorke Peninsula Distribution System

The Yorke Peninsula distribution system presented a very different case. This rural system used chloramine to disinfect in order to cover the long residence times. The system is controlled with tanks and pumps, and services a relatively small population. The conclusions specific to this study were:

- Combining different first-order decay rates for the upper parts of the system to the middle and lower parts, provided a good approximation of second-order decay, which matched the measured water quality data for the system;
- Tank rotation throughout the year is vital in reducing residence time in the system;
- The GA was able to produce a cost efficient design optimising the tank sizes, pumping ability, pipe sizes and dosing regime; and
- Pumping structure of the system was the major difference between the hydraulic and combined solutions as it had a marked effect on the residence times.

10.5 Recommendations for Further Work

The results of this research indicate that the application of a water quality framework is a successful mechanism of controlling residual levels. By improving the system layout and implementing an efficient dosing regime, the overall quality of supply is enhanced. While the results, discussion and conclusions resulting from this thesis are both interesting and encouraging, further work is required by future researchers to improve this framework.

10.5.1 Social Cost Research

More work needs to be done to improve the estimates of waterborne disease costs and disinfection by-product costs. The components that are required to advance the quantification of waterborne disease include:

- A better understanding of what microorganisms have an effect on public health;
- A more accurate method of estimating the levels of pathogens in water supply than the current coliform index;
- The effect that different disinfectants have in reducing different microorganisms;
- Further research into the risks associated with different pathogens in the water supply; and
- A more extensive analysis of the 'willingness to pay' for consumers to prevent waterborne diseases.

This work would require bacteriologists, health scientists and economists to provide a greater accuracy for waterborne disease quantification.

The aspects that are required to enhance the knowledge of the effect of disinfection by-products on the consumers include:

- Extensive epidemiological analysis of different systems to examine the suspected link with chlorinated water and cancer;
- Analysis of different methods to assess risks for low doses than the linear multi-stage model currently used;
- Further research into the health effects of factors other than THMs, such as haloacetic acid HAAs in chlorinated supplies; and
- Improved methodology of testing for low doses using rodents.

This work would require toxicologists, epidemiologists and data analysts to quantify for disinfection by-products to a higher accuracy.

The costs for the water quality issues are based on linear structures, where they would be more likely to follow a curve. An example of this was the "willingness to pay" concept raised for quantification of taste and odour concerns. The actual graph would look different from the linear approach used, but further study would be required to determine the true nature of these curves. Other curves unlikely to be of a linear nature are the examination of DBPs and the costing for waterborne disease. The risk levels for the potential health effects of drinking water also require further research, as they are based on a great deal of assumption at present.

10.5.2 Model Extension

As computational ability increases, the complexity of the system and the size of the search space can be increased without impinging on running time. Further research is required on testing out the conclusions from this study on different systems using larger populations and longer simulation times. Mechanisms for further reduction of run-time for real time simulation of distribution systems is also required. With faster running times, greater analysis of the GA process in terms of mutation, crossover and selection could be applied to different systems.

Further research into the optimisation of residence time as a mechanism for efficient design may prove rewarding in the determination of cost-effective solutions. This process was discussed in relation to the New York tunnels problem, and used with success in a basic form. With further analysis, it could prove a robust method for solving water quality issues.

10.5.3 Extensions into Water Quality Modelling Practice

The current version of EPANET uses a first-order model for disinfectant decay. As discussed in this study, second-order decay fits the measured values in a system more accurately. The new version of EPANET, which is currently going through a testing phase, implements n^{th} order bulk decay and 0^{th} and 1^{st} order wall decay. Adapting the research in this thesis to a package that considers the improved decay curves, can only enhance the accuracy of the model.

10.6 Closing Notes

The use of water quality modelling as a tool in distribution system design has only recently become possible due to improvements in computational ability. At the time of submitting this thesis, this approach was possible for detailed analysis of simple systems, or basic analysis of complicated networks. As technology progresses, the speed of computation time will be enhanced, allowing for more detailed examinations of complex systems to occur. This greater ability for analysis will make the use of the techniques discussed in this thesis commercially viable.

In the early to mid 1990s, the use of optimisation techniques such as genetic algorithms required large amounts of computer run-time, which made the majority of the work in this area theoretical in nature. Due to the increase in computer speed, and development of more efficient modelling practice, the use of such methods is now available commercially for water authorities to contract. In a relatively short time, a similar situation for water quality modelling, using packages similar to GAs will eventuate.

The discussion, frameworks, and results from this thesis provide a good analysis of the advantages present by including water quality in the design process. It is hoped that actions will be taken, similar to those discussed in this thesis to make the examination of drinking water quality a pro-active, rather than reactive component of distribution system design.

Appendix A Glossary of Terms

Acronym	Full term
ANN	Artificial Neural Network
ARMCANZ	Agricultural and Resource Management Council of Australia and New Zealand
AUS$	Australian Dollars
CPI	Consumer Price Index
D/DBP Rule	Disinfection/Disinfection By-product Rule
DBP	Disinfection By-Products
DICL pipe	Ductile Iron, Concrete Lined pipe
DOC	Dissolved Organic Carbon
DP	Dynamic Programming
EPA	Environmental Protection Agency (South Australia)
EPANET	Water Modelling Package from the USEPA
EWS	Engineering and Water Supply Department (South Australia)
GA	Genetic Algorithm
GAC	Granulated Activated Carbon
GI	Gastro-Intestinal Illness/Cancer
HAA	Haloacetic Acid
IARC	International Agency for Research on Cancer
LMS model	Linear Multi-stage model
LP	Linear Programming
MCL	Maximum Contaminant Level
MG	Mega-Gallon
ML	Mega-Litre
MRDL	Maximum residual disinfectant level
MTD	Maximum Tolerated Dose
NHMRC	National Health and Medical Research Council

NLP	Non-linear Programming
NYC	New York City
NYCDEP	New York City Department of Environmental Planning
NYT	New York Tunnels
RR ratio	Relative Risk ratio
SA Water	South Australian Water Authority (formerly EWS)
THM	Trihalomethane
TOC	Total Organic Carbon
TTHM	Total Trihalomethane
UK	United Kingdom
US$	United States Dollars
USEPA	United States Environmental Protection Agency
UT cancer	Urinary Tract Cancer
WATSYS	Water Modelling Package for Water Systems
WHO	World Health Organisation

Chemistry

Al	Aluminium
C	Carbon
CH_3Cl	Chloroform
CH_3X	Trihalomethane
Cl	Chlorine
Cl_2NH	Dichloramine
Cl_2O	Chlorine dioxide
Cl_3N	Trichloramine
$ClNH_2$	Monochloramine
Fe	Iron
H	Hydrogen
HOCl	Hypochlorus Acid
N	Nitrogen
O	Oxygen
O_3	Ozone

Appendix B Nomenclature

Abbreviation	Explanation
μ	Dynamic viscosity
ρ	Density of a fluid
ν	Kinematic viscosity
γ	Unit gravity force
v_s	Specific volume
A	Area
C	Hazen-Williams Roughness Coefficient
C_{AT}	Annual additional treatment cost to remove taste and odour
C_{canc}	Cost of cancer contraction
$C_{DBP(annual)}$	Annual cost of disinfection by-products
$C_{Dis(annual)}$	Annual cost of disinfection
C_{dn}	Unit disinfectant cost
C_I	Infrastructure cost
C_{ill}	Cost of illness contraction due to waterborne disease
C_{jm}	Cost per unit length of size m for pipe j
C_{mort}	Cost of mortality (death cost) ie. value of life
Cn_{all}	Acceptable residual level before taste and odour concerns occur
Cn_{di}	Residual concentration at dosing station i
Cn_{dose}	Concentration of disinfectant residual at dosing station
Cn_i	Residual concentration at node i
Cn_{low}	Minimum residual level for adequate waterborne disease control
Cn_t	Concentration of residual at time t
Cn_w	Residual concentration at the pipe wall
C_p	Cost of pipes

C_{pen}	Total penalty costs
CP_i	Cost of pipe material for pipe i
C_{PM}	Cost of pumping
C_{PMp}	Total pumping penalty cost
C_{pp}	Total pressure penalty cost
C_Q	Total quality costs
C_{rwt}	Cost of installation of a rainwater tank
C_T	Cost of tanks
C_{TO}	Cost of taste and odour
C_{tot}	Total cost structure for framework
C_{Tp}	Total tank penalty cost
$C_{WD(annual)}$	Annual cost of waterborne disease
D	Diameter
D_{di}	Annual demand at dosing station i
D_{pt}	Length of time pump p operates during time t (Goldman and Mays, 1999)
EEF_{pt}	Efficiency of pump p during time t (Goldman and Mays, 1999)
$EPANET_{warning}$	Hydraulic solver indicates problem with pumps
f	Friction factor
g	Gravitational force
GA_{gen}	Number of generations specified in genetic algorithm (GA)
GA_{pop}	Population size of genetic algorithm (GA)
h_f	Head loss in a pipe
h_{min}	Minimum acceptable water height in tanks
h_w	Height of the water in tank
i	Discount rate
k_1	First order decay rate
k_2	Second order decay rate
k_f	Mass transfer coefficient
k_w	Decay rate at pipe wall
$k_{w,0}$	0^{th} order decay rate at the pipe wall
$k_{w,1}$	First order decay rate at the pipe wall
L	Length of pipe

M_f	Momentum flux
n	Life of project (years)
N_d	Number of people per dwelling
N_{di}	Number of decisions of type i (GA)
nF	Number of fixed grade nodes in network
nJ	Number of junctions in network
nL	Number of loops in network
N_{oi}	Number of options of type i (GA)
np	Number of pipes
nP	Number of pipes in network
nPM	Number of pumping stations
nT	Number of tanks
OC_i	Operation and maintenance costs at node i
PB_j	Penalty function used in system operation (Sakarya and Mays, 1999)
Pen_{low}	Water quality penalty cost factor
P_i	Population at node i
p_i	Pressure at node i
P_{mort}	Probability of contraction becoming mortal
PM_{pen}	Pumping penalty factor
PMv_i	Violation occurrence at pump i
PP_{pt}	Power of pump p during time t (Goldman and Mays, 1999)
p_{req}	Required nodal pressure
Q	Flow
Q_e	External inflow or demand at node
Q_{in}	Flow entering node
Q_{out}	Flow exiting node
r_h	Hydraulic radius of a pipe
$risk_{canc}$	Potential risk of cancer contraction due to disinfection by-products
$risk_{poll}$	Assumed risk of waterborne disease from polluted water
$risk_{pris}$	Assumed risk of waterborne disease from pristine water
R_j	Penalty factor (Sakarya and Mays, 1999)
RR	Relative Risk factor in epidemiology

RT	Running time of the model
t	Time-step
THM_r	Rate of THM (trihalomethane) formation
THM_T	Additional THM formation due to temperature fluctuation
tnd	Total number of dosing stations
tnn	Total number of nodes
T_{pen}	Tank penalty factor
T_{sim}	Number of timesteps in each simulation
tt	Total time
Tt_{mix}	Time taken to achieve mixing in a storage
Tv_i	Violation occurrence at tank i
T_w	Water temperature
UC_i	Unit energy cost of pumping (Goldman and Mays, 1999)
v	Velocity
V_i	Initial volume of water in storage
$V_{j,i}$	Violation of bound constraint j (Sakarya and Mays, 1999)
x	Distance along a link
X_{jm}	Length of pipe j of size m

Appendix C Genetic Algorithm Example

C.1 Problem Definition

The problem examined is a very simple distribution system. The process was applied to a water network to allow an understanding of the research methodology in this thesis. Figure C.1 illustrates the system structure.

Figure C.1 Worked example for application of genetic algorithms to pipe systems

This is a system that consists of two pipes, a pump-house, and a treatment facility. It supplies water to two towns and has the following assumptions (for simplicity) made on the system:

- The pipes are assumed to suffer no losses due to friction, thus pipe length is irrelative;
- There are no valves in system; and
- Water to treatment plant is supplied from a constant level reservoir.

C.2 Algorithm Constraints

Constraints on this system are:

- Town demand must be met;
- Water satisfy health and safety for towns; and
- The water must have 20m pressure in towns.

The pressure requirements and location elevations that are established constraints are listed in Table C.1.

Table C.1 Location constraints for the worked example

Location	Elevation (m)	Required HGL (m)
Treatment/Pump-house	50	50
Town 1	20	40
Town 2	25	45

The capacity and quality requirement must be met, as no variance is acceptable.

C.3 Decision Variables

For simplicity, the decision process is made up of two options, yes (1) and no (0). 'Yes' means that the system facet is connected, whereas 'no' indicates that it has not been implemented. The four decision variables for the worked example are listed in Table C.2.

Table C.2 Decisions required to solve worked example

Decision	Yes	No
Include Treatment	1	0
Use Pump	1	0
Connect Pipe 1	1	0
Connect Pipe 2	1	0

From the constraints the best solution becomes 1 0 1 1, as this solution includes treatment and the pipe installation. The pumping station is a luxury item, which is not required as part of the network, and its exclusion will save cost but not contradict the problem goals.

C.4 Coded Strings and Search Spaces

Coded string made up of:

Treatment facility - Pumping station - Pipe 1 - Pipe 2

As there are two possible options for each decision, the 16 possible outcomes making up the search space are:

0 0 0 0	0 0 0 1	0 0 1 0	0 0 1 1
0 1 0 0	0 1 0 1	0 1 1 0	0 1 1 1
1 0 0 0	1 0 0 1	1 0 1 0	1 0 1 1
1 1 0 0	1 1 0 1	1 1 1 0	1 1 1 1

The strings vary from 0 0 0 0, which equates to not installing the pumps, pipes, or treatment facility, to 1 1 1 1, assuming all infrastructure is installed in the system.

C.5 Fitness

The fitness costs for this problem are made up of the infrastructure costs and the penalty costs of not meeting problem criteria.

Table C.3 Infrastructure costs

Decision	Infrastructure cost ($million)
Treatment facility	5
Pumping station	10
Pipe 1	15
Pipe 2	10

The penalty costs are $20million for each town that receives an inadequate hydraulic supply. For quality deficiency, $5million penalty cost.

C.6 Initialisation

Four randomly coded strings have been initialised:

1 0 1 1 0 (no treatment, yes pumping, yes pipe 1, no pipe 2)
2 1 0 1 0 (yes treatment, no pumping, yes pipe 1, no pipe 2)
3 1 0 0 1 (yes treatment, no pumping, no pipe 1, yes pipe 2)
4 0 0 1 1 (no treatment, no pumping, yes pipe 1, no pipe 2)

These strings form the population field for the genetic algorithm.

C.7 Selection

The selection process is based on the probabilities gained from the fitness values in Table C.4.

Table C.4 Fitness of parent population

Individual	String	Infrastructure Cost	Penalty Cost	Fitness Cost
1	0 1 1 0	25m	30m	55m
2	1 0 1 0	20m	20m	40m
3	1 0 0 1	15m	40m	55m
4	0 0 1 1	25m	10m	35m

By implementing roulette selection with 1, 2, 3, and 4, the probability is that the fitter components will survive so that 2 and 4 move through to the crossover phase. These are duplicated so as to maintain the population level.

C.8 Crossover

This is the propagation process of the genetic algorithm. If strings 2 and 4 are the selected individuals, if both sets were to undergo single-point crossover, the process could be as seen in Figure C.2 (the boxes represent crossover position):

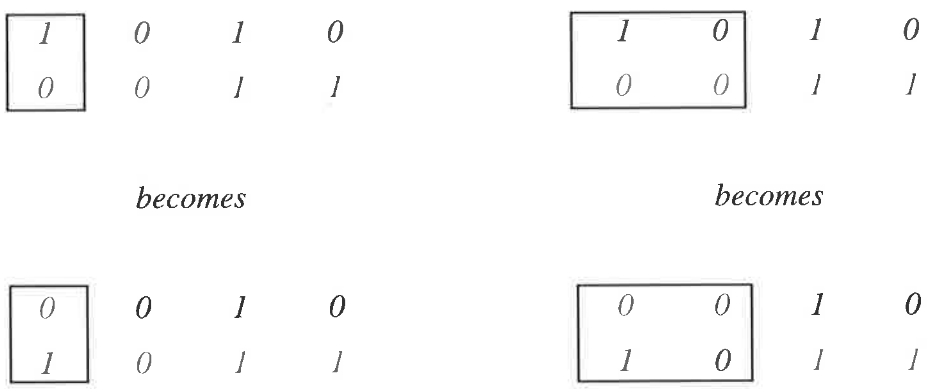

Figure C.2 Crossover of strings

The genetic material has been exchanged, so the two parent sets form four new offspring.

C.9 Mutation

As the probability of mutation is usually pretty low (0.01) there would be only a 16% chance that mutation would occur in any of these four strings. To allow example though, it is assumed that string 2 experiences mutation at point 2 as seen in Figure C.3.

0	0	1	0		0	0	1	0
1	$\boxed{0}$	1	1		1	0	1	1
0	0	1	0		0	0	1	0
1	$\boxed{1}$	1	1		1	0	1	1

Figure C.3 Mutation of strings

C.10 Convergence

This process is repeated continuously until convergence is shown. After each generation, the fitness is ascertained. In the case of the four new strings post mutation, their respective fitness, are seen in Table C.5.

Table C.5 Fitness of new strings

Individual	String	Infrastructure Cost	Penalty Cost	Fitness Cost
1	0 0 1 0	15m	30m	45m
2	1 1 1 1	40m	0m	40m
3	0 0 1 0	15m	30m	45m
4	1 0 1 1	30m	0m	30m

From this table, two of the solutions have no penalty problems, one of the solutions has achieved the lowest cost value. If they were to go through the process again, chances would have Individual 2 and 4 go through crossover where there would probably be two of the fittest solutions in the next generation. Eventually, convergence would dictate that, the problem has been optimised for the particular data set.

Appendix D Costing of Network Items

D.1 Background

The aim of the optimisation program is to determine the most cost-effective method of network design. This makes the cost structure on which it is based pivotal. Unlike the cost structure associated with system health risks that are based on risk analysis and subject to a significant error margin, infrastructure costs have precedents.

In determining the costs of pipes, pumps, tanks and fittings, installation and ongoing maintenance, an overview of the hydraulic components of system design can be provided. The major research tools that have been used to assist in this are threefold:

- Rawlinsons Australian cost handbook (Rawlinsons, 1995);
- SA Water cost data (SA Water, 1994); and
- US Department of Commerce (Gumerman *et al.*, 1992).

Construction companies and consultants use Rawlinsons as a guide to annual prices of infrastructure used in civil works. It includes price indices for different states and regional areas as well as how prices have changed over the last three decades. It tends to provide a conservative estimate on costing and does not consider all of the components of water supply infrastructure.

Rawlinsons is held with such regard by the construction industry that it is recognised in the courts as an authoritative text for cases relating to construction cost. It has rapidly become a benchmark for costing.

The SA Water cost data is an experience-based guide of water distribution costs. It tends to be localised to South Australian knowledge as it considers costs accrued from previous works of a similar nature. It would tend to be fairly approximate if adapted to interstate or overseas problems.

The US Department of Commerce put out a report in 1992 that examined standardised costs for water distribution systems (Gumerman *et al.*, 1992). The report was a republished version of an EPA report. It presents construction and operation-maintenance cost data related to domestic water supply systems. It examines several different makes of pipe as well as tank and pump costs. It also looks at prevention costs relating to corrosion and some water quality concerns.

The costs in the US Commerce Department Report are for September 1989. As all costs will be compared as at January 1999$AUS, these costs have been assumed to apply at January 1990.

D.2 Cost Variations with Time

When looking at reports relating to infrastructure requirements, it is important to allow for price fluctuations over time. This enables all costs irrespective of date or currency to a standard level. To do this, building price indices and inflation rates need to be documented in conjunction with currency exchange rates.

The majority of the data was determined since late 1989. When documenting the costs for this study, the following assumptions were made about costs since 1990:

- The rate of exchange from Australian Dollars to US dollars was assumed to be 0.72, which is an approximate average for the period of 1990 to 1999. Figure D.1 shows variations in the exchange rate over the period 1983-1998.
- An average rate of inflation for Australia since 1990 of 2.5% per year as the Consumer Price Index (CPI) according to one report (egoli website, 1999) has grown on an average rate of just 1.9% per year since 1990. Another site shows levels since 1993 ranging between 0.3% and 4.5% (ABS c\- ABC website, 1998) thus a 2.5% level would be a reasonable estimation.

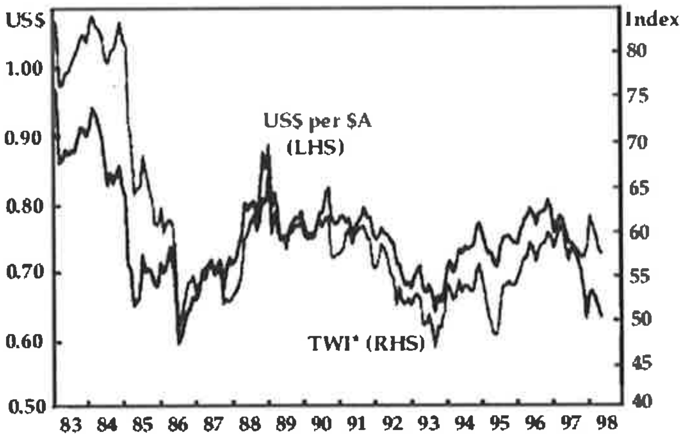

Figure D.1 Exchange Rate of $US to $AUS

www.abc.net.au/money/vault/educate/educate7/invest7/http (1999)

Typical inflation rates for the 1990s are listed in Table D.1 (ABS c\- ABC website, 1998).

Table D.1 Inflation Rates in the 1990s

Year	CPI Headline Rate %	Treasury Underlying Rate %
93-94	1.8	2.1
94-95	4.5	2.5
95-96	4.2	3.2
96-97	1.2	2.0
97-98	0.3	1.5
98-99	2.5	2.5

The combination of these two assumptions provides the following relationship of cost structure over the past decade shown in Table D.2.

Table D.2 Assumed Price Fluctuation in 1990s

Year	Years to 1999	C.P.I.	Conversion to 1999$US
1990	9	1.25	1.73
1991	8	1.22	1.69
1992	7	1.19	1.65
1993	6	1.16	1.61
1994	5	1.13	1.57
1995	4	1.10	1.53
1996	3	1.08	1.50
1997	2	1.05	1.46
1998	1	1.03	1.42
1999	0	1.00	1.39

From this structure all costing is balanced into 1999$AUS so all analysis are performed on a level playing field.

D.3 Issue of Replacement

When considering adapting an existing network, the issue of replacement becomes an important cost factor. The life of system components is a good indicator as to when replacement is required. A system life of 50 years for major system components is usually fairly acceptable (Nelson, 1995).

Nelson used 50-year life range for most components but for pumps and pump accessories, a 35-year life was used.

SA Water (EWS, 1986(a)) also have their structure as to life of network components. The data from this study has a range of different infrastructure lives. Table D.3 lists some of these values.

Table D.3 Economic Life of System Components

Water Asset Type	Approx. Economic Life (Years)
Metropolitan Water Mains	80
Country Water Mains	80
Water Filtration Plants	50
Treatment Stations	50
Bores and Wells	50
Pumping Station - Building	50
Pumping Station - Mechanics	25
Tanks and Storages	80
Dams and Reservoirs	100
Services	30
Connections	80

As the Yorke Peninsula network is about 35 years old, pipes and tanks would not require replacement for this study. Pump mechanics should require urgent replacement and the pumphouse may require replacement during the life of the study.

D.4 Factors that dictate cost

To construct a pipe network, it is not just the component cost that determines the expenditure for the project. Factors such as labour and equipment also require consideration. Maintenance is also a design issue

Other costs relating to contracting fees, land acquisition and insurance costs can also have a bearing on the overall price of the network.

D.5 Pipe Cost Analysis

It is more than just pipe material price that makes up the overall cost structure as seen in Figure D.2. As an example, with the Yorke Peninsula Study, ductile iron concrete lined (DICL) pipes were chosen for costing. The diameter range was from 100mm to 750mm as no pipes from this system currently exceed this level.

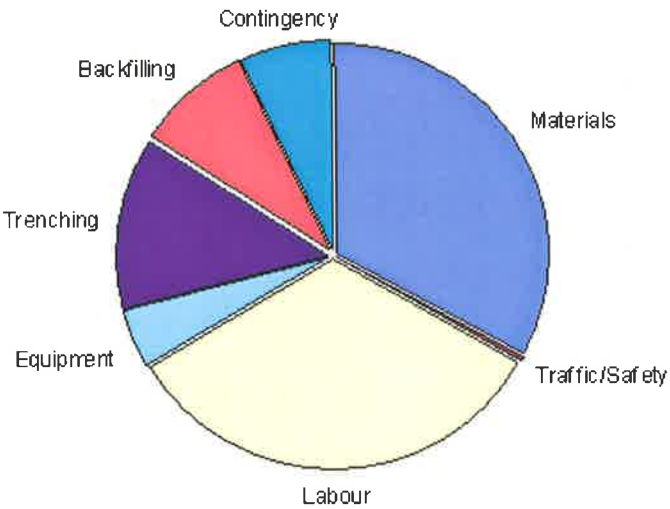

Figure D.2 Components of overall cost structure

Adapted from Gumerman *et al*, 1992

The soil was assumed to be sandy to light soil for excavation purposes, which is suitable for backfilling. Three different studies were used to obtain a cost structure relating to pipe costs. They had three different methods of cost determination.

D.5.1 The US Department of Commerce Report

The determination of pipeline cost from this report can be broken up into four separate categories:

- Base pipeline installation cost;
- Cost of trenching;
- Cost of backfilling; and
- Traffic cost.

D.5.1.1Pipeline Cost

The base pipeline installation cost is for an installed pipe in a previously excavated trench. It breaks up into a four-prong cost structure of its own:

- Labour cost is the value placed on labour associated with pipe installation and operation of equipment used during the construction process such as trucks, backhoes and bulldozers. The labour cost assumed in this report was $15/hour in 1990$US. This corresponds to about $26/hour in 1999$AUS.
- Equipment cost is the cost of hire or purchase of operating equipment. This cost excludes the fee associated with the operator as this is taken into account in the labour cost component.
- Materials cost covers the pipe material as well as many of the fittings. It includes valves, hydrants, sand bedding, backfill material and pavement replacement.
- Contingency cost is an added cost to allow for some of the unknown events that can occur with any construction project.

When these costs are all included for an analysis of a DICL pipeline class 50, the relationship found in Fig D.3 is observed.

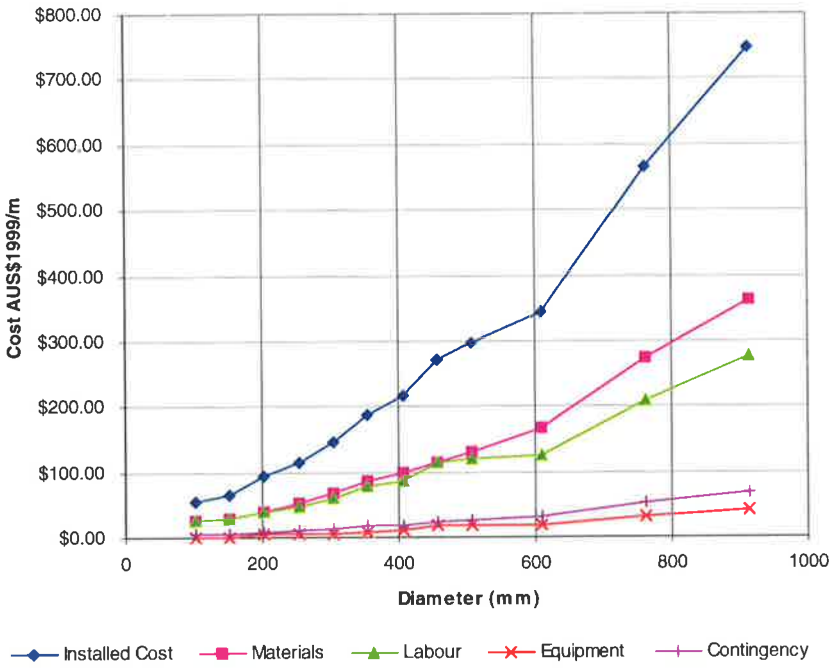

Figure D.3 Costs associated with pipe instalment in existing trench

From this it is clearly shown how the costs rise with increasing diameter. Materials and labour form the bulk of the costs as larger pipes take longer to install as seen in Figure D.4.

Larger diameters require larger trenches and hence take a longer time to lay a unit length than a smaller diameter line.

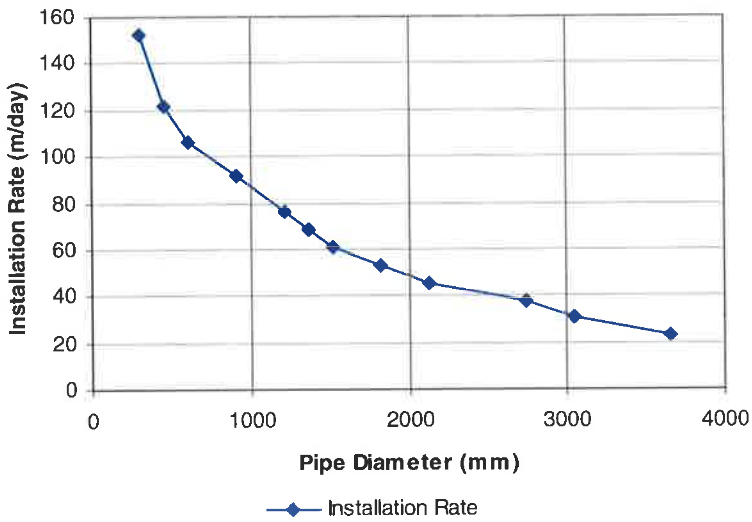

Figure D.4 Installation Rate (m/day) for different pipe diameters

Equipment cost and contingency costs also increase with diameter but have less impact on the total cost. The contingency cost is about 10% of the total cost.

D.5.1.2Trenching Cost

Trenching is the most common way to lay a pipe that is not too deep underground. Cost varies with soil type and type of trench. A 1:1 side slope is more expensive than a vertical wall trench as the amount of excavated material is increased. The soil type being excavated has a bearing on cost, with clay being much more effort to work with than a sandy soil. Another factor, which changes cost, is the depth of cover sought over the pipe as shown in Figure D.5.

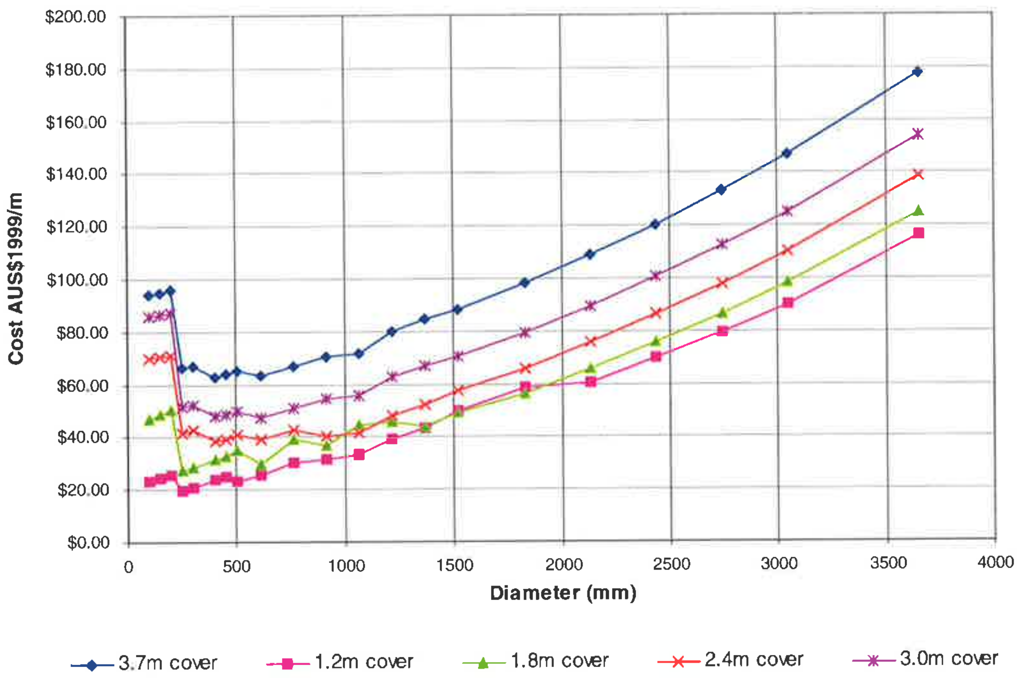

Figure D.5 Trenching Costs for Sandy Soil

The greater the depth of the trench, the higher the costs incurred. As Yorke Peninsula is a rural network travelling mainly through farmland, a 1.2m (4ft) depth of cover was allowed for.

The trenching graph is a little deceptive as costs are higher for the small diameters than some of their slightly larger counterparts. This is because of the machinery used. Backhoes are required for narrow trenches (eg 150mm) and they were expected to require an operator 100% of the time, a labourer for grade checking 100% of the time, a foreman 15% of the time and an oiler for 10% of the time.

This compares to the larger trenches, which can be excavated using a bulldozer, which requires the skills of an operator 100% of the time. Aside from that, the relationship is reasonably consistent with diameter and depth of cover.

D.5.1.3Backfilling

Once the pipe has been installed, earth needs to be put back in the trench to bury the pipeline. The costs were developed based on a 90% compaction of the backfill material.

Again, costs varied with native soil type as sandy soils require less effort in comparison to clayey soils. Imported backfill is more expensive again due to the purchase price. Figure D.6 is an example showing costs associated with backfilling with sandy soils.

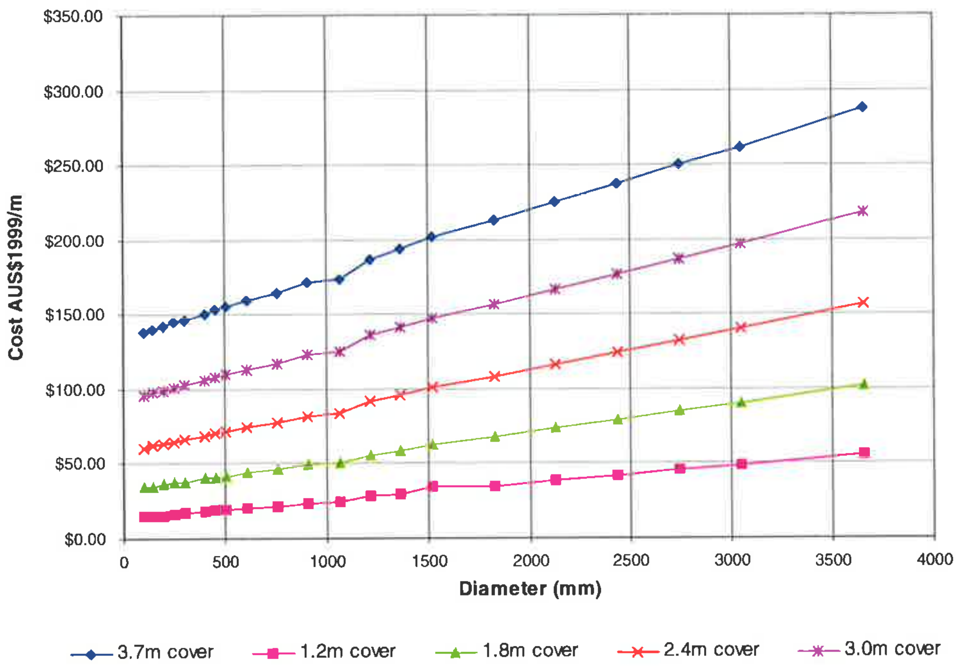

Figure D.6 Backfilling costs for Sandy Soil

The relationship found from backfilling costs is pretty evenly spread dependant on depth of cover.

D.5.1.4Traffic Costs

With any construction project that may be located on, over, under or alongside a road, traffic control is an important consideration. In city networks, traffic control can be an extremely costly process, and even in less populated regions, some consideration of the issue is required.

Costs in low traffic areas may be as simple as the cost of cones, flashers and direction signs as well as time taken to set up or shift traffic warnings.

If the location is more heavily traffic laden, not only are cones and flashers required, but flagmen with stop/slow signs may need to be employed to prevent any issues occurring.

Figure D.7 gives a cost in \$/m of pipe laid for two different traffic conditions.

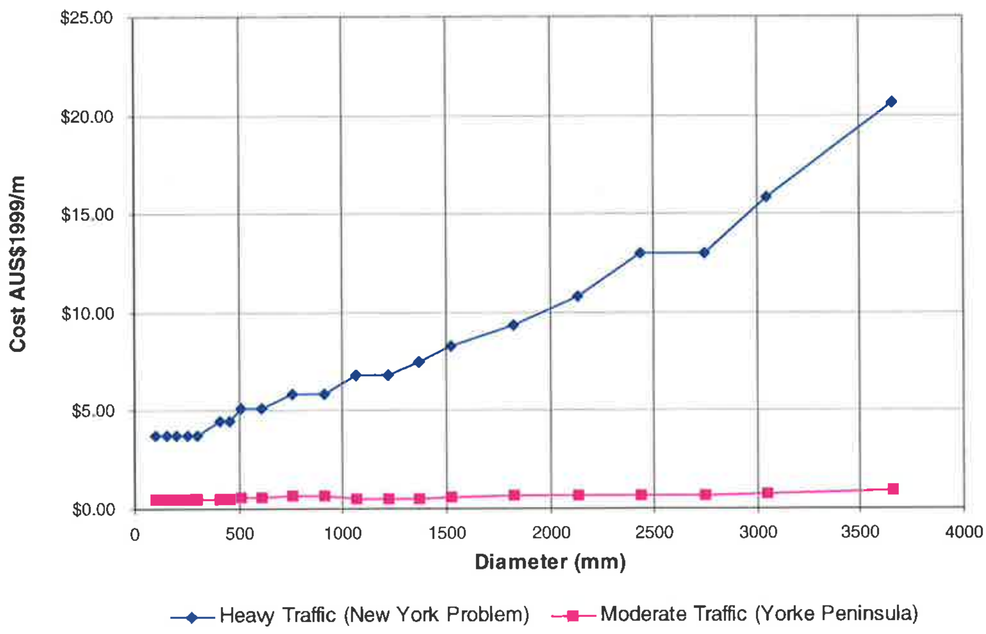

Figure D.7 Cost \$/m for different traffic conditions

The moderate traffic condition, which would be present at the Yorke Peninsula doesn't really encounter much variation with diameter whereas the heavy traffic scenario, which would be for a system like the New York network, has a large increase.

D.5.1.5Total Cost Structure

When all four of these cost components are added together, a representation of the total system cost relating to pipes is obtained.

These costs have been applied to conditions assumed for the Yorke Peninsula problem, the cost predictions are shown in Figure D.8. The following series of assumptions were made to get the total costs for different pipe diameters:

- Moderate-Low traffic conditions;
- Sandy soil or light soil;
- 1.2m (4ft) of cover for trenches;
- Native soil used as backfill material; and
- DICL pipes used across the system.

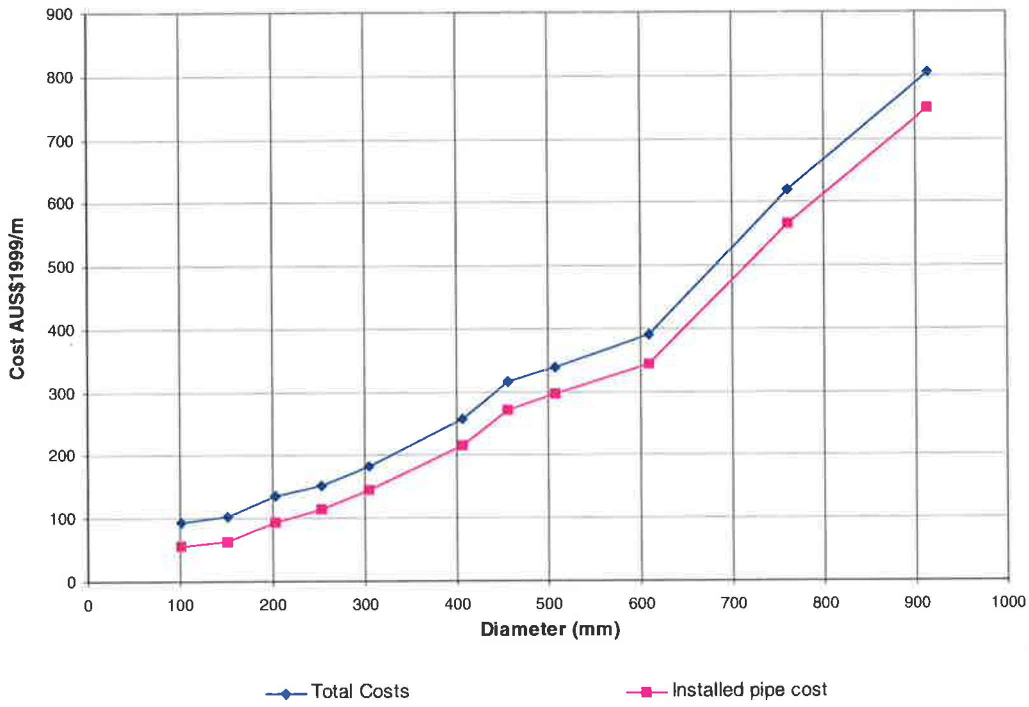

Figure D.8 Accumulated associated pipe costs for Yorke Peninsula

From this the relative cost structure of the US Commerce Department report relating to the Yorke Peninsula study is obtained.

D.5.2 SA Water Pipe Information

SA Water had two main network-associated costs. The first is the supply cost, which considers the purchase of the pipe material. The second cost is the laying cost which allows for trenching costs, labour costs and some equipment cost.

A third cost bracket consisted of a number of extra costs resulting from:

- fittings and thrust blocks;
- excavation work;
- pavement reinstatement;
- service relocation;
- preliminaries;
- quality assurance;
- testing;
- administration; and
- engineering and survey.

The sum of these parts comes to approximately 30% on top of the system installation costs. For a DICL pipe system, the price per metre of pipeline based on SA Water data is shown in Figure D.9.

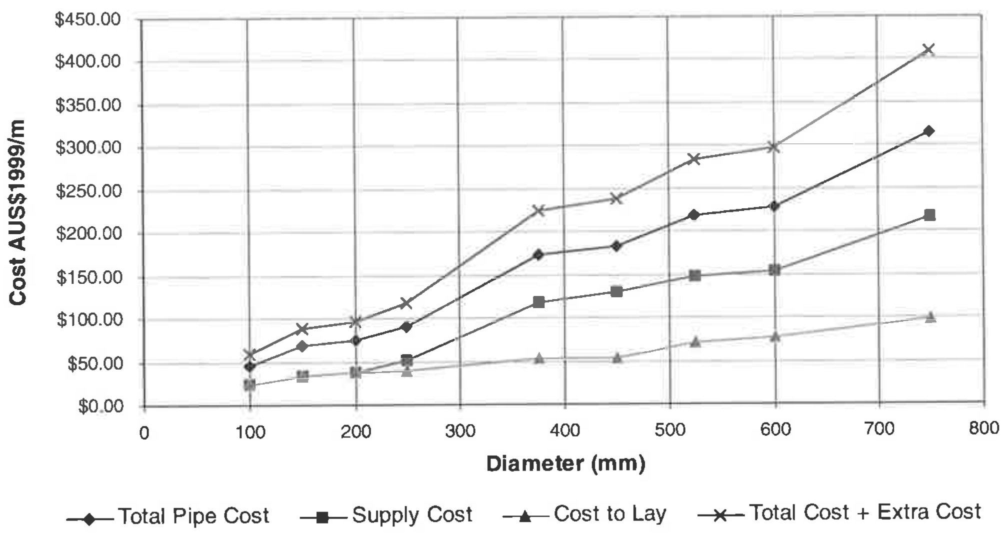

Figure D.9 SA Water cost regime for DICL pipelines

D.5.3 Rawlinsons

Rawlinsons based its costs on a DICL (Class K9) to A.S.2280-1991 with rubber ring joints laid in a trench. The first cost is the supply cost, which considers the cost of acquiring and laying a pipe in a trench.

The second cost considered is the cost to dig the trench and then backfill the earth to bury the pipeline. The trenching cost varies with the type of soil, as sand is easier to trench than light soil, which takes less effort than clay. The backfill cost varies with type of soil required to refill the trench. Clean fill is quite expensive whereas filling with the excavated soil is often included in the cost of trenching.

Both the supply cost and laying costs are shown in Figure D.10. The sum of these costs is also depicted.

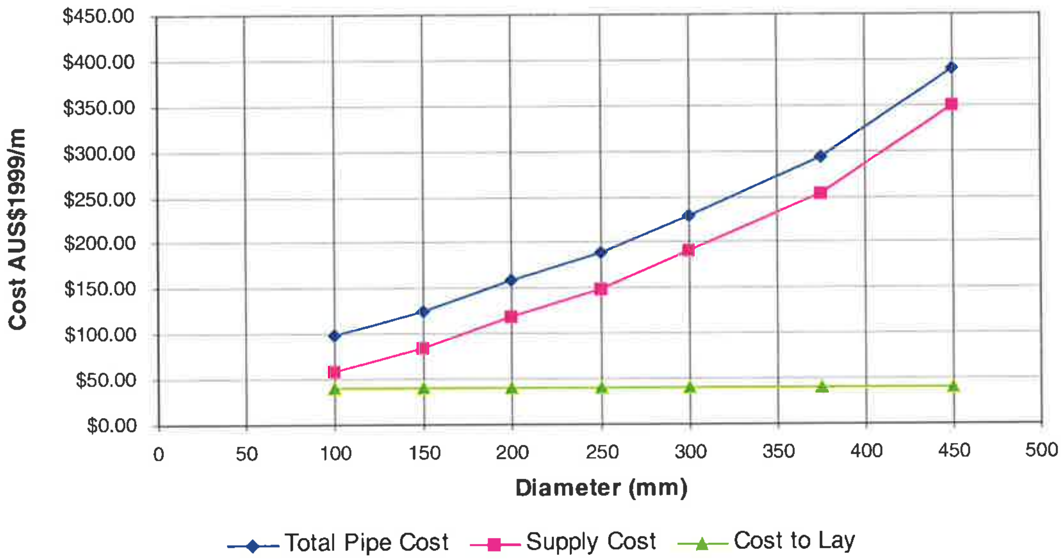

Figure D.10 Rawlinsons cost regime for DICL pipeline

From the graph, it is seen that the cost of trenching is constant regardless of diameter. Backfilling is not an issue cost wise as in a rural network the excavated soil is usually adequate backfill. The material cost is a steadily increasing level with increased cost for larger diameters.

D.5.4 Comparison of the Different Cost Regimes

Figure D.11 below examines the three different studies against one another.

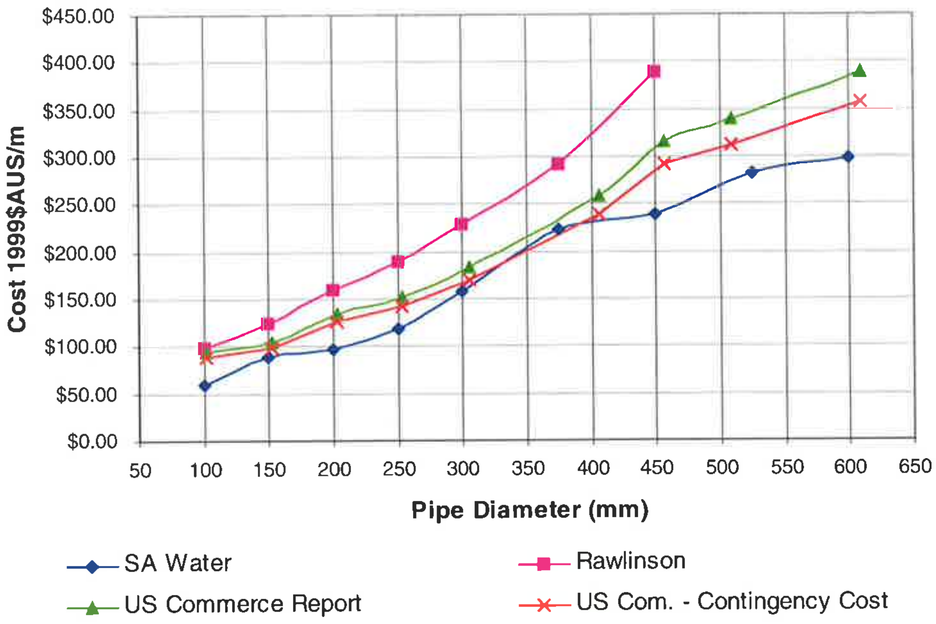

Figure D.11 Comparison of different cost regimes

The Rawlinsons study is clearly the upper bound cost of the three. This is potentially due to its conservative nature, allowing for different scales of purchase of the product. Rawlinsons allows for small-scale works, which would attract a higher cost per unit than a large-scale work. Rawlinsons also considers the contractor's margin to an extent in its prices although it specifies that in some cases, this may eventuate to an increased level of up to 10%. Rawlinsons supply cost was easily the largest of the three but it had a reasonably competitive laying cost.

The SA Water costs were at the lower bound of the group. The reasons for this may be due to the opposite of the Rawlinsons argument with SA Water having experience with larger scale projects hence reduced unit price. Being a government department may also have potential cost reductions in some areas.

The report put out for the US Department of Commerce fits quite comfortably between the two Australian studies. It is slightly closer to the SA Water data but maintained a consistent gap from it. This gap could be partially due to the contingency cost allowed for by the US report, which is not factored into the SA Water data.

When the contingency cost is removed from the US Commerce Department figures, the correspondence with the SA Water data is even better. This can be seen above in Figure D.11.

When examining the three studies, the inclination is to use the US Department of Commerce costs as they fit with Australian comparisons well and could be adapted globally. It also presented itself as the most in depth examination with relation to the background behind the cost estimates.

D.6 Tank Cost Analysis

Networks often have controls throughout the system, which assist in regulating flow to the consumer base. One method of control is storage reservoirs. The Yorke Peninsula network has a large number of tanks already operational throughout and costing any additional tanks becomes a valuable component.

For the Yorke Peninsula network, new reservoir capacities for extended population growth will be considered viable from a minimum of 0.4ML. The volumes will extend to about 5ML which should satisfy most country towns as a balancing storage.

Rawlinsons was unable to provide tanks of sufficient capacity to be an adequate comparison for this infrastructure, however, both the US Commerce Department and SA Water had data, which related well.

D.6.1 US Commerce Department Report

Construction costs for the tanks in this report are for a standard which caters for the requirements of ANSI/AWWA D110-86 "AWWA Standard for Wire Wound Circular Prestressed-Concrete Water Tanks." The cost analysis for this is seen in Figure D.12.

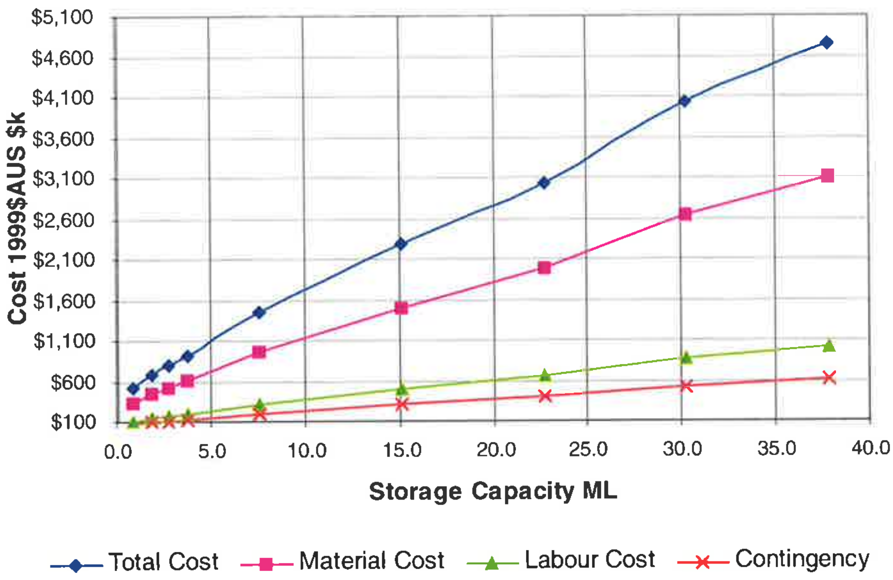

Figure D.12 Construction costs for concrete tanks

The two main cost groups are for materials which includes the steel/concrete for construction, and the labour cost, which is the cost of construction.

All of the costs seem to vary at an approximately linear rate with capacity, and it is the materials cost, which makes up the major share of the cost.

D.6.2 SA Water Cost

SA Water based their data on historical estimates and actual costs obtained from the departments financial records. They broke up the cost basis into four categories:

- Siteworks;
- Pipework;
- Concrete; and
- Roof.

The last three components make up the construction component and labour costs therein. The siteworks cost is the establishment of a site for the tank to be constructed on. These costs are represented in Figure D.13.

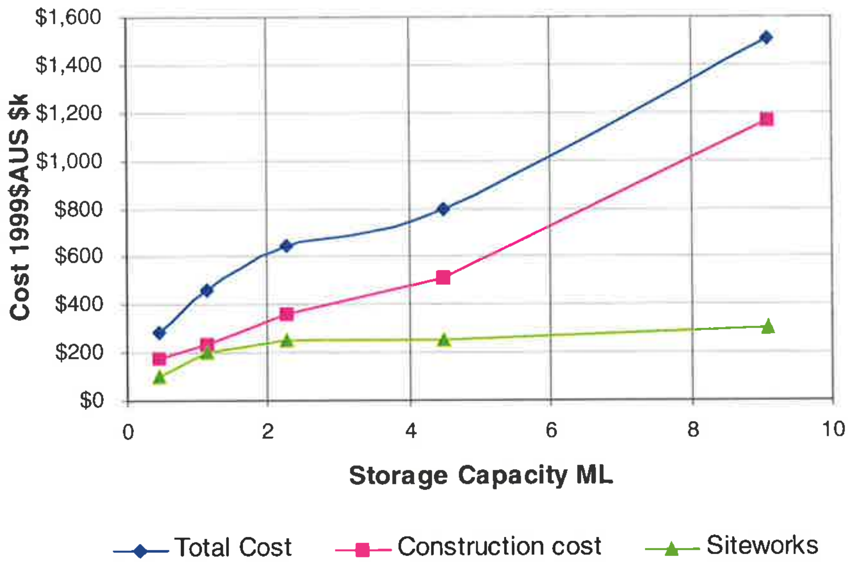

Figure D.13 SA Water cost regime for concrete tanks

Siteworks costs do not experience severe fluctuation for an increase in tank capacity, as many of the costs such as roads and services are required regardless of tank volume. The other costs vary at a reasonably linear rate with capacity.

Again it is the construction cost which forms the major portion of the total cost.

D.6.3 Comparison of Data Sets

When comparing the two cost structures over a similar range of tank capacities, it is clearly seen that they correspond reasonably well. The SA Water costs are placed at a constantly higher level, which may be due in part to the contingency, cost issue discussed in the pipe analysis section. Figure D.14 graphs the two data sets.

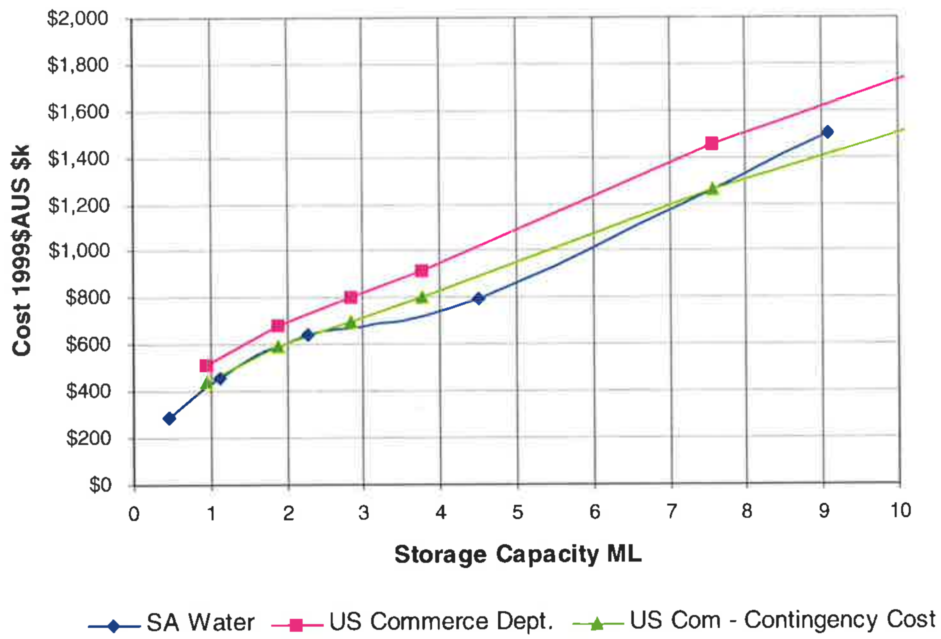

Figure D.14 Comparison of the cost regimes for concrete tank construction

When the contingency cost is removed from the US Commerce Department Report data set, the match with the SA Water data is very close as shown above in figure C.14.

The acceptability of either cost structure is not a problem, but to remain consistent with the pipe analysis choice of costing, the US Department of Commerce report is the preferred cost structure for this study.

D.7 Pump Cost Analysis

Yorke Peninsula has four main pump stations regulating supply throughout the distribution system. As these pumps are fairly old, a question to be addressed in the optimisation is whether it is preferable to expand the existing stations or build new ones.

Expansion of the existing stations will be lower in cost in the short term, but replacement of existing pumps down the track could be an expensive practice. However, more current asset is required to establish new pumping stations.

The US Department of commerce report provides a detailed examination of pump costing, examining establishment, expansion and maintenance.

D.7.1 Establishment of New Pumping Facilities

The establishment of a pumping station considers the total construction of a new pumping station with all associated fittings and sitework. As the Yorke Peninsula model involves the replacement of existing works, some of the component costs such as sitework and material cost may be lower than recorded.

Table D.4 Pumping Station Options

Pumping Cap. ML/day	TDH (m)	Capacity of each pump ML/day	No on Duty	No on Standby	Power per pump in kW	Speed rpm Horizontal pump
1.9	30.42	11.0	2	1	5.6	1750
	60.84				11.2	3500
	91.26				18.6	3500
3.8	30.42	22.1	2	1	11.2	1750
	60.84				22.4	3500
	91.26				29.8	3500
18.9	30.42	110.4	2	1	44.7	1750
	60.84				93.2	1750
	91.26				149.1	1750
37.9	30.42	220.9	2	1	93.2	1750
	60.84				186.4	1750
	91.26				260.9	1750
94.6	30.42	549.0	2	1	223.6	1180
	60.84				447.2	1180
	91.26				596.3	1180
378.5	30.42	877.1	5	1	335.4	1180
	60.84				670.9	1180
	91.26				931.8	1180
757	30.42	1457.6	6	1	521.8	590
	60.84				1118.1	590
	91.26				1565.3	710

The cost data looks at firm pumping capacities in the range of 1.9ML/day to 757ML/day. The firm pumping capacity is the pumping capacity with the largest pump out of service, so it is really the dependable pumping ability of the station. Some typical pump station options are given in Table D.4. This gives three head options for seven different pump capacities. The following sub-sections provide cost assumptions for a completely new pump infrastructure.

D.7.1.1 Pump Mechanics

The pump cost data seen in Table D.5 is for a horizontal centrifugal pump. This was selected, because it is a widely used pump in water systems globally. The connections to the pump are considered to be ductile iron and all check and gate valves are included in the connection costs.

Table D.5 Cost data for a horizontal centrifugal pump (AUS$1999)

30.42mTDH	Firm Pumping Capacity ML/day						
	1.9	3.8	18.9	37.9	94.6	378.5	757.0
Pump	$11,621	$21,335	$40,068	$60,015	$201,553	$683,579	$1,119,814
Connections	$13,529	$22,202	$49,261	$116,561	$212,307	$1,454,405	$2,308,489
Labour	$31,395	$40,761	$49,608	$70,942	$141,191	$384,546	$610,382
Equipment	$3,989	$4,857	$10,407	$12,142	$18,213	$41,108	$52,730
Contingency	$9,020	$13,356	$22,375	$39,027	$86,033	$384,546	$613,677
Total	$69,555	$102,511	$171,719	$298,686	$659,296	$2,948,184	$4,705,091
60.84mTDH							
	1.9	3.8	18.9	37.9	94.6	378.5	757.0
Pump	$12,315	$22,029	$54,811	$75,799	$232,080	$720,351	$1,780,324
Connections	$13,529	$22,202	$49,261	$116,561	$212,307	$1,454,405	$2,308,489
Labour	$31,568	$41,108	$51,516	$73,197	$146,048	$397,034	$635,532
Equipment	$3,989	$4,857	$11,274	$13,356	$20,467	$46,832	$61,056
Contingency	$9,193	$13,529	$24,457	$41,802	$91,583	$392,698	$717,749
Total	$70,595	$103,725	$191,319	$320,715	$702,485	$3,011,321	$5,503,149
91.26mTDH							
	1.9	3.8	18.9	37.9	94.6	378.5	757.0
Pump	$29,660	$49,955	$105,806	$195,308	$397,034	$1,538,356	$2,165,216
Connections	$15,611	$22,202	$49,261	$116,561	$212,307	$1,454,405	$2,308,489
Labour	$31,915	$41,629	$52,730	$75,799	$148,996	$407,788	$648,888
Equipment	$3,989	$5,377	$11,795	$14,223	$21,508	$50,822	$65,218
Contingency	$11,968	$17,866	$32,956	$60,362	$116,907	$517,758	$778,284
Total	$93,144	$137,028	$252,548	$462,253	$896,753	$3,969,129	$5,966,096

This data set was based on an inflation rate of 2.5% and an exchange rate of $0.72 as discussed at the start of the chapter. Figure D.15 shows the relationship between cost and firm pumping capacity.

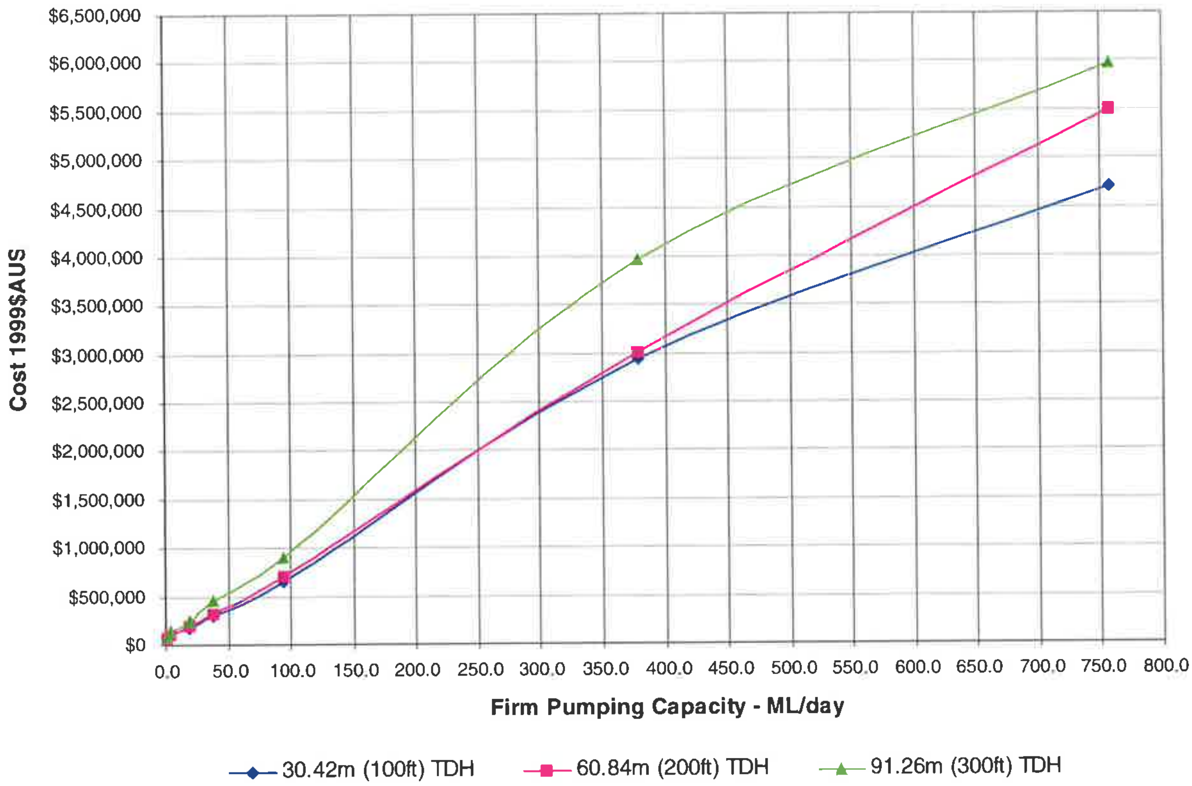

Figure D.15 Cost data for provision of a centrifugal pump

Logically, costs increase with increased heads and capacity. Of note is that for small capacities there are little cost between 60.84m and 30.42m head. For larger capacities increase, the total head has a more significant effect on cost.

D.7.1.2Sitework Costs

Sitework is considered in two levels: the moderate level is for fencing, paving, clearing the site, providing service roads and gates etc. The extreme sitework conditions are for heavily wooded areas with soil replacement required as well as the fencing and paving requirements asked for under moderate conditions. The sitework cost relationships as a function of pumping capacity are shown in Figure D.16.

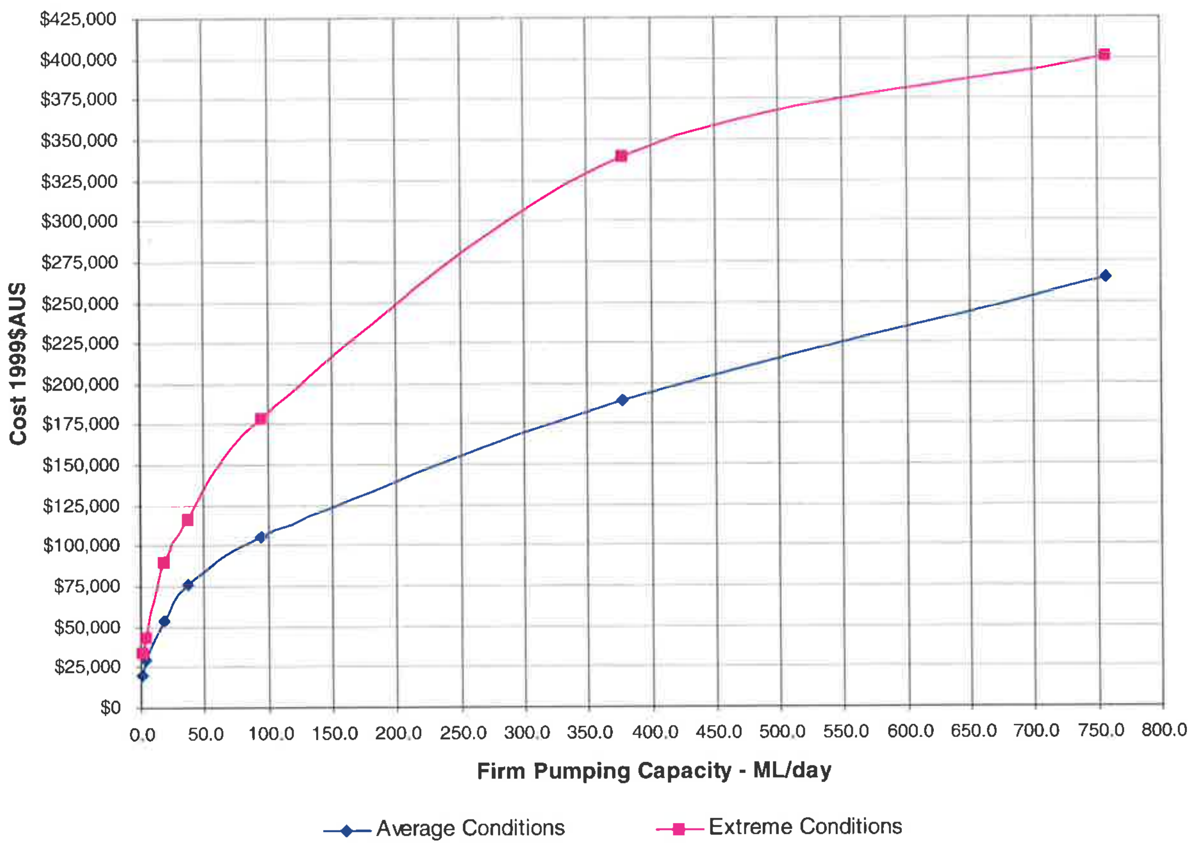

Figure D.16 Sitework costs for pumping station establishment

As expected, the extreme conditions are significantly more expensive than the average conditions. The cost difference for larger pump capacities is not considerably different to small capacities as much of the site costs of fencing and clearing would not vary with station size.

D.7.1.3 Building Costs

As with the sitework costs, two options occur with the building costs. A simple building design consists of a concrete slab, which can really only be considered for a mild dry climate.

The complex structure is a complete pumphouse, which is a building in its own right with electrical fittings and overhead cranes. The comparison is seen in Figure D.17. With the simple slab design, little change occurs with increased capacity. The complex pumphouse does have a large increase.

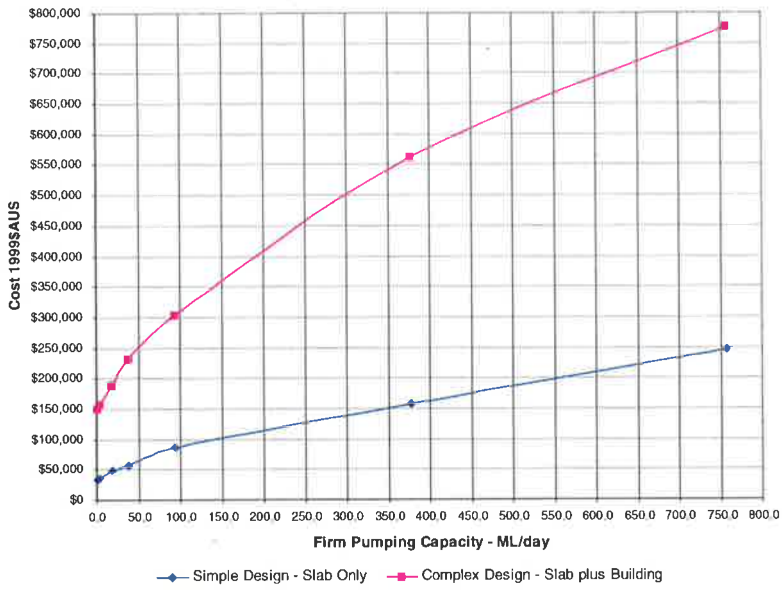

Figure D.17 Building costs for pumping station establishment

D.7.1.4Electricity Costs

This supplies costing for electrical equipment and instrumentation in the pumphouse. Figure D.18 shows how these costs vary with the firm pumping capacity of the pumping station.

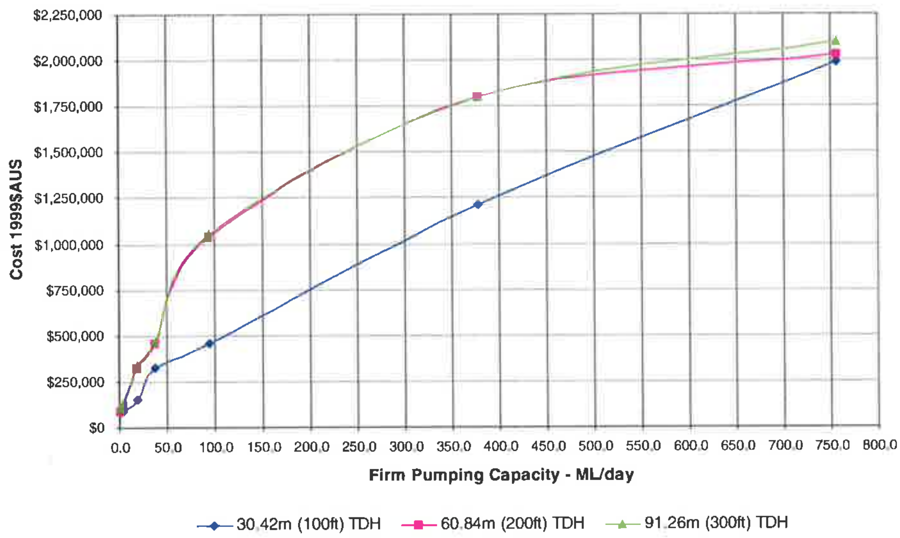

Figure D.18 Electrical & instrumentation costs for pumping station establishment

For small capacities it is evident that the instrumentation costs are considerably less. This would be partially due to fewer pumps in operation that require electrical connection.

D.7.1.5 Total Costs for Construction of a Pumping Station

The sum of the pump costs, site costs, building costs and electrical costs provide an overall cost structure for the construction aspects of pumping station establishment. These costs are shown in Table D.6. Operation and maintenance costs are considered later in the Appendix.

Table D.6 Total construction costs for pumping station establishment

30.42m TDH

Firm Pump Cap. ML/day	Power kW per pump	Power kW total	Number of pumps	Pump Cost	Sitework Cost	Building Cost	Elec. Cost	Total Cost
1.9	5.6	11.2	2	$69,555	$19,774	$149,690	$82,737	$321,756
3.8	11.2	22.4	2	$102,511	$29,140	$156,975	$89,502	$378,128
18.9	44.7	89.4	2	$171,719	$53,944	$189,584	$152,812	$568,059
37.9	93.2	186.4	2	$298,686	$76,146	$232,254	$327,653	$934,739
94.6	223.6	447.2	2	$659,296	$105,806	$302,676	$455,488	$1,523,266
378.5	335.4	1677	5	$2,948,184	$189,584	$561,295	$1,210,183	$4,909,246
757.0	521.8	3130.8	6	$4,705,091	$264,343	$776,550	$1,984,304	$7,730,288

60.84m TDH

Firm Pump Cap. ML/day Cap. ML/day	Power kW per pump kW	Power kW total kW	Number Pumps	Pump Cost	Sitework Cost	Building Cost	Elec. Cost	Total Cost
1.9	11.2	22.4	2	$70,595	$20,862	$157,929	$89,502	$338,888
3.8	22.4	44.8	2	$103,725	$30,744	$165,615	$118,122	$418,206
18.9	93.2	186.4	2	$191,319	$56,913	$200,019	$328,000	$776,251
37.9	186.4	372.8	2	$320,715	$80,337	$245,037	$455,488	$1,101,577
94.6	447.2	894.4	2	$702,485	$111,630	$319,335	$1,040,199	$2,173,649
378.5	670.9	3354.5	5	$3,011,321	$200,019	$592,188	$1,799,577	$5,603,105
757.0	1118.1	6708.6	6	$5,503,149	$278,892	$819,291	$2,031,310	$8,632,643

91.26m TDH

Firm Pump Cap. ML/day Cap. ML/day	Power kW per pump kW	Power kW total kW	Number Pumps	Pump Cost	Sitework Cost	Building Cost	Elec. Cost	Total Cost
1.9	18.6	37.2	2	$93,144	$20,862	$157,929	$112,918	$384,853
3.8	29.8	59.6	2	$137,028	$30,744	$165,615	$129,743	$463,130
18.9	149.1	298.2	2	$252,548	$56,913	$200,019	$339,795	$849,275
37.9	260.9	521.8	2	$462,253	$80,337	$245,037	$466,936	$1,254,563
94.6	596.3	1192.6	2	$896,753	$111,630	$319,335	$1,050,606	$2,378,324
378.5	931.8	4659	5	$3,969,129	$200,019	$592,188	$1,799,577	$6,560,913
757.0	1565.3	9391.8	6	$5,966,096	$278,892	$819,291	$2,099,651	$9,163,930

These costs will be used to form part of the distribution system cost structure.

D.7.1.6Expansion of Existing Pumping Facilities

The other option to station replacement is to expand the existing pumping stations. A similar distribution of costs is shown to those of new construction however the individual cost components are less. These costs basically represent the addition of one extra pump to an existing station. The x-axis of the following graphs represents the expanded capacity rather than the total capacity.

D.7.2 Pump Expansion

This examines the costs of adding one pump of similar size to the existing pumps. It also looks at the labour and equipment costs required making this expansion. Figure D.19 displays this relationship.

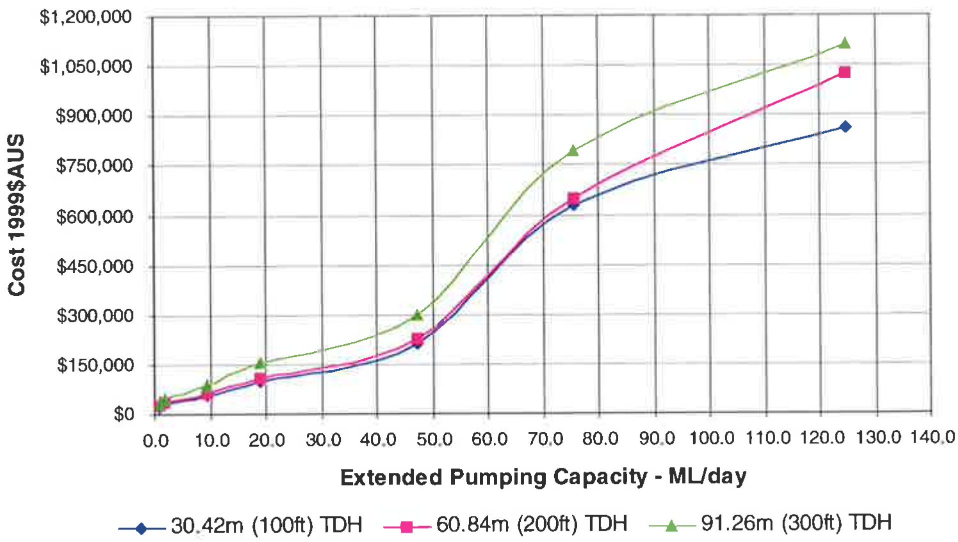

Figure D.19 Expansion costs of an additional pump

D.7.2.1Sitework Costs - Expansion

Sitework costs shown in Figure D.20 would be considerably smaller as much of the site preparation has been configured in the original station establishment.

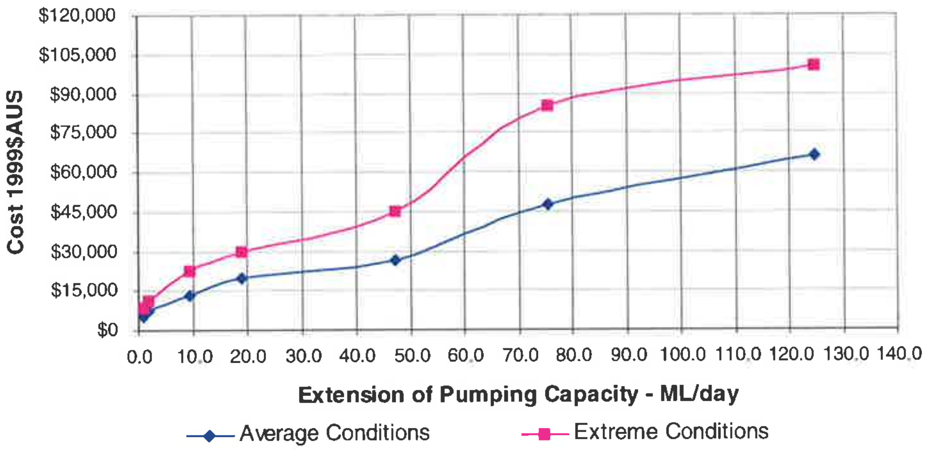

Figure D.20 Sitework costs for pump station expansion

D.7.2.2Building Costs - Expansion

The expanded building costs are a similar concept to an extension on a house. The extra space (slab or enclosed) must be paid for. The costs are shown in Figure D.21.

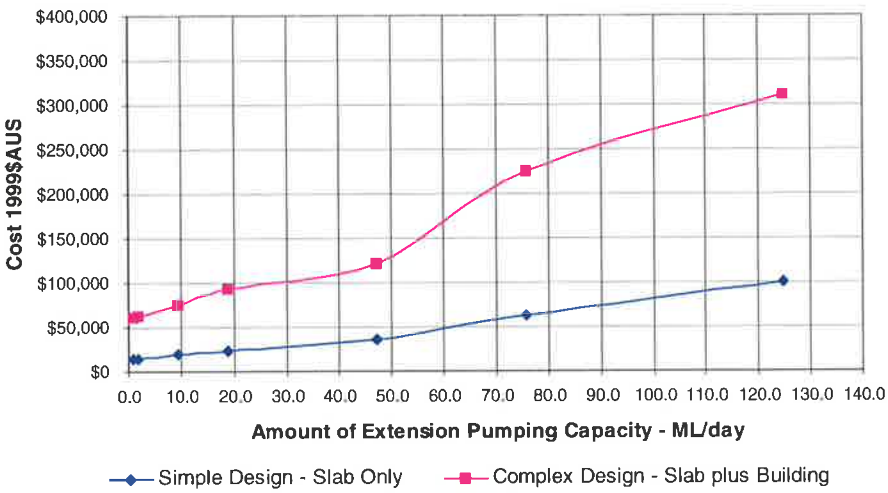

Figure D.21 Building costs for pump station expansion

D.7.2.3Electricity Costs - Expansion

Electrical costs (Figure D.22) for an expansion considers the cost of instrumentation and electrical equipment to cater for the data output of the new pumping ability.

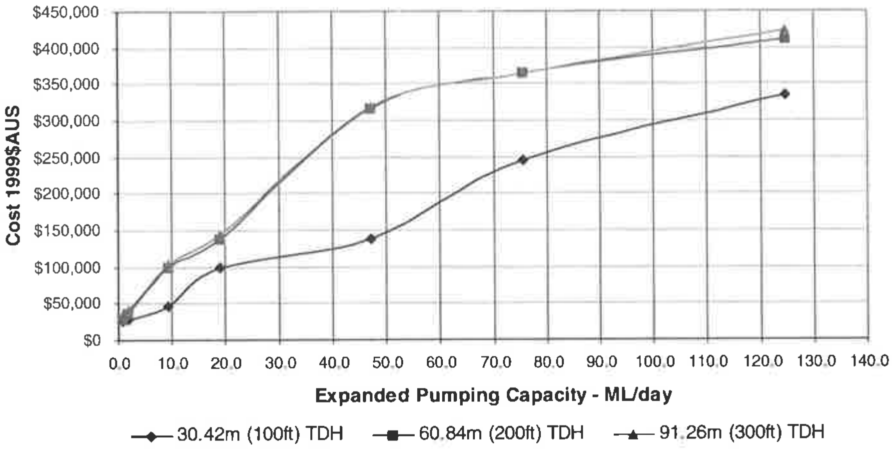

Figure D.22 Electrical costs of pump station expansion

D.7.3 Ongoing Maintenance and Operation Costs

The cost of a pumphouse does not end after completion of construction. Annual costs come into play. These include:

- Electrical energy cost;
- Labour cost; and
- Material cost.

The electrical energy cost is the fee for power to keep not only the pumps operational, but to power the lights, fans and equipment on an ongoing basis. The larger the pumping station, the greater the energy requirements to keep it running efficiently. The cost of electricity was assumed as \$0.08/kW in 1990\$US which is \$0.14/kW in 1999\$AUS.

The efficiencies of the pumps and the motors play an integral part in determining energy consumption. Pump and motor efficiencies are assumed to vary with the motor power (Gumerman *et al.*, 1992). Table D.7 lists the assumptions made for component efficiencies.

Table D.7 Assumed pump and motor efficiencies

Motor Power kW	Motor Efficiency %	Pump Efficiency %	Combined Efficiency %
< 3.75	79	75	59.3
3.75 to 11.25	88	80	70.4
11.25 to 75	92	80	73.6
75 to 112.5	94	85	79.9
>112.5	95.5	85	81.2

The total annual maintenance and operation costs are given in Table D.8.

Table D.8 Annual operation and maintenance costs of pumping stations

30.42m TDH							1992$US	1999$AUS
Firm Pumping Capacity (mgd)	Firm Pumping Cap. ML/day	Building	Energy kWh/yr Process	Total	Maintenance Materials $/yr	Labour Hr/Yr	Total Annual Costs $/yr	Total Annual Costs $/yr
0.5	1.9	11800	96600	108400	$1,000	365	$15,147	$26,273
1	3.8	12500	162800	175300	$1,000	365	$20,499	$35,556
5	18.9	18600	778800	797400	$1,900	455	$72,517	$125,783
10	37.9	22100	1557600	1579700	$3,700	685	$140,351	$243,443
25	94.6	36700	3530000	3566700	$6,500	910	$305,486	$529,875
100	378.5	75900	14118500	14194400	$29,500	1820	$1,192,352	$2,068,173
200	757.0	120900	28237000	28357900	$47,400	2730	$2,356,982	$4,088,260
60.84m TDH								
Firm Pumping Capacity (mgd)	Firm Pumping Cap. ML/day	Building	Energy kWh/yr Process	Total	Maintenance Materials $/yr	Labour Hr/Yr	Total Annual Costs $/yr	Total Annual Costs $/yr
0.5	1.9	11800	162800	174600	$1,000	365	$20,443	$35,459
1	3.8	12500	311500	324000	$1,200	365	$32,595	$56,537
5	18.9	18600	1557600	1576200	$3,000	455	$135,921	$235,759
10	37.9	22100	2869600	2891700	$4,500	685	$246,111	$426,887
25	94.6	36700	7060100	7096800	$9,800	910	$591,194	$1,025,445
100	378.5	75900	28237100	28313000	$32,300	1820	$2,324,640	$4,032,162
200	757.0	120900	56474100	56595000	$55,200	2730	$4,623,750	$8,020,042
91.26m TDH								
Firm Pumping Capacity (mgd)	Firm Pumping Cap. ML/day	Building	Energy kWh/yr Process	Total	Maintenance Materials $/yr	Labour Hr/Yr	Total Annual Costs $/yr	Total Annual Costs $/yr
0.5	1.9	11800	244300	256100	$1,200	365	$27,163	$47,115
1	3.8	12500	467300	479800	$1,600	365	$45,459	$78,850
5	18.9	18600	2152600	2171200	$3,600	455	$184,121	$319,364
10	37.9	22100	4235900	4258000	$5,900	685	$356,815	$618,907
25	94.6	36700	10589700	10626400	$11,600	910	$875,362	$1,518,343
100	378.5	75900	42358800	42434700	$41,700	1820	$3,463,776	$6,008,030
200	757.0	120900	84718400	84839300	$60,000	2730	$6,888,094	$11,947,619

Labour costs include people manning the station or checking the progress of the pumps throughout the year. This could be reading of data from the station, monitoring potential problem areas or cleaning in and around the site. A cost of $26/hour (1999$AUS) has been used for this. Materials cost includes replacement parts for the pumps, motors and electrical equipment. This cost was calculated as 2% of the construction components of pumps/motors, connections and electrical/instrumentation costs.

Table D.8 has been put into graphical form (Figure D.23) to show the relationships based on the different heads. As the electrical costs tend to shadow the other annual costs in the larger capacities, only the total annual costs were presented.

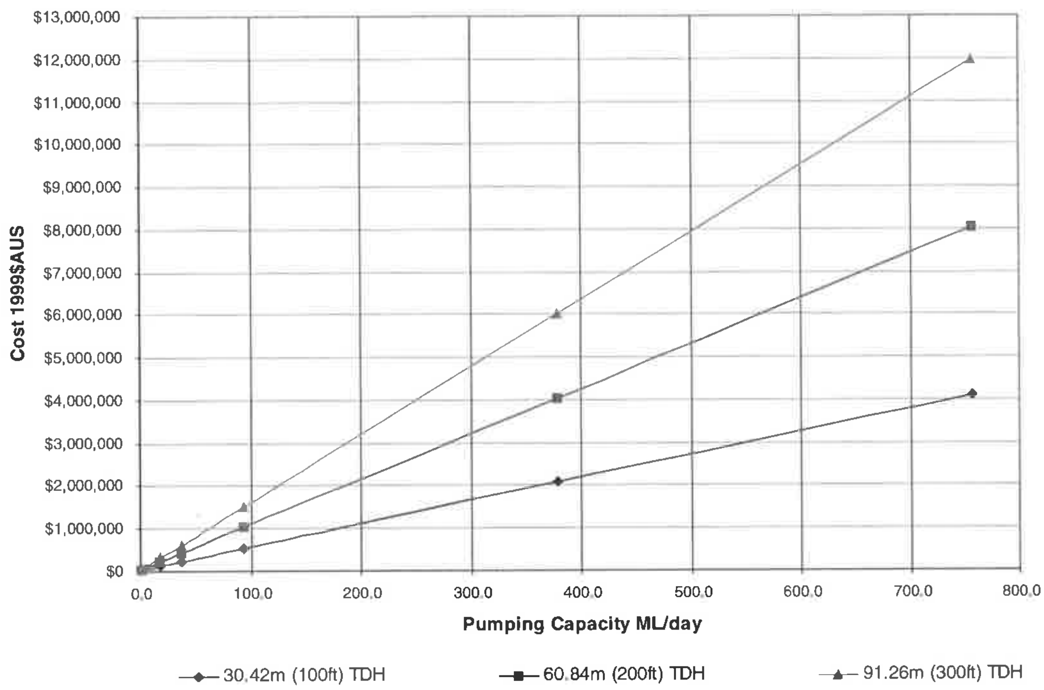

Figure D.23 Annual operation and maintenance costs of pumping stations

Two things of note come from this figure. The first is that low capacity pumping stations have a relatively low cost to maintain on an annual basis. The second and potentially more pivotal information is that the annual costs are strongly dependent on the head of the pumping station. This is important, as the cost differences between the construction of the different station types were relatively low (10%).

The relationships are roughly linear and from the graph, it can be seen, if the head requirement is doubled, so are the annual costs. This is reasonable given that the energy requirement of the pumps is proportional to the head provided. This will have a large bearing on pump selection for the network. By taking these costs over a life of 30 years for the system, with a discount rate of 4% per annum, the cost structures of the 21 different pump options provided in the report by Gumerman *et al.* (1992) are listed in Table D.9.

Table D.9 Present value of total costs for pumping stations

Pump Capacity ML/day	30.42m TDH	60.84m TDH	91.26m TDH
1.9	$776,068	$952,047	$1,199,569
3.8	$992,966	$1,395,846	$1,826,608
18.9	$2,743,104	$4,853,009	$6,371,723
37.9	$5,144,369	$8,483,328	$11,956,723
94.6	$10,685,886	$19,905,676	$28,633,567
378.5	$40,672,155	$75,327,388	$110,451,968
757.0	$78,424,625	$147,315,474	$215,762,552

These costs will be used for pump selection in the system.

D.7.4 Package Pumping Stations

Package pumping stations can often be an economical alternative. Although they are not under consideration for the Yorke Peninsula study, they have been included as an interesting option for future program optimisation. The costs of package pumping stations are shown in Figure D.24.

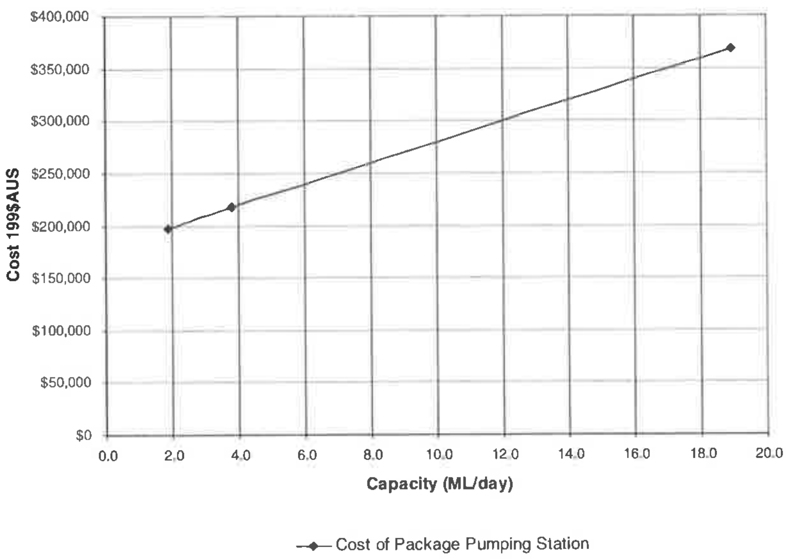

Figure D.24 Cost of package pumping station

D.7.5 Comparison of US Department of Commerce Report with SA Water Data

Through meetings with personnel from SA Water (Wozniak pers. com., 1998), an approximate relationship between pump power and cost was obtained. This estimate is SA Water's basis for pumping station costing, including building works, pump cost and all accessories.

It has an upper and lower limit to assist in catering for the potential cost situations that could arise. Figure D.25 details the different cost regimes.

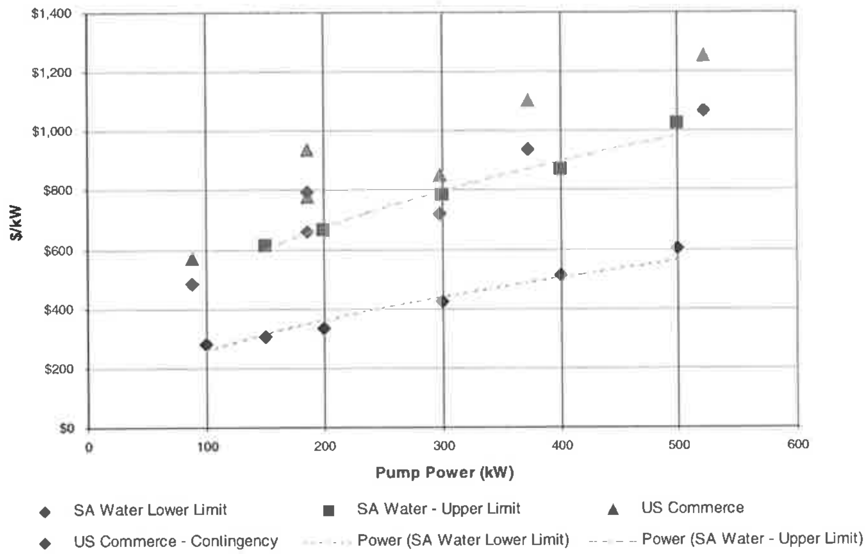

Figure D.25 Cost regime comparison for pumping stations

When data from the US Department of Commerce was included on the graph, it was noticed that it was consistently above the SA Water data in cost. The data was then replotted with the exclusion of the contingency costs, and the data points were found to follow a line very similar to the SA Water Upper Limit.

As the US Dept. of Commerce report data was based around the complex building costs, it would be expected to be on the high side. Another factor that could result in a gap in the costing is that SA Water may be able to cut costs due to the number of stations that they have constructed and may construct in the future.

Although the information obtained in the meeting with SA Water does not include the higher kW pumping stations, the correlation between the two studies is such that the US Department of

Commerce numbers could be used as a reasonable guide to the costs associated with the establishment of a pumping station.

D.8 Table of Costs for use in Study

The cost structure for the 10 different duplicate pipe options for the Yorke Peninsula network are shown below in Table D.10.

Table D.10 Pipeline costs for Yorke Peninsula study

Diameter (mm)	Diameter (Assumed mm)	Construction Cost ($/m)	All Pipe Costs ($/m)
0.0	0	$0	$0
101.6	100	$56	$94
152.4	150	$64	$104
203.2	200	$93	$134
254	250	$115	$152
304.8	300	$145	$183
406.4	400	$216	$258
457.2	450	$272	$316
508	500	$296	$339
609.6	600	$343	$389

The cost of the 8 different storage sizes for new tanks in the Yorke Peninsula Supply system are shown in Table D.11. The diameters of these storages are based on a uniform tank height of 5m.

The storage size stopped at 22.7ML as the relative diameter of the storage is 76m, any larger, it is more a reservoir than a storage tank and the volume of water in transit in Yorke Peninsula does not justify a storage of this size.

Table D.11 Tank costs for Yorke Peninsula study

Storage Capacity (ML)	Cost ($)	Diameter (m)
0.0	$0.00	0
0.9	$513.000	15.1
1.9	$681,000	22.0
2.8	$800,000	22.8
3.8	$920,000	31.2
7.6	$1,454,000	44.0
15.1	$2,296,000	62.0
22.7	$3,035,000	76.0

Twenty-five options for pumps currently exist, this will be shortened to 14 options for selection at the four locations at the network, and some new locations as booster pumps. The options prior to manipulation are seen in Table D.12.

Table D.12 Base pump costs for Yorke Peninsula study

Pump Capacity	30.42m TDH	60.84m TDH	91.26m TDH
0.0	$0.0	$0.0	$0.0
1.9	$776,068	$952,047	$1,199,569
3.8	$992,966	$1,395,846	$1,826,608
18.9	$2,743,104	$4,853,009	$6,371,723
37.9	$5,144,369	$8,483,328	$11,956,723
94.6	$10,685,886	$19,905,676	$28,633,567
378.5	$40,672,155	$75,327,388	$110,451,968
757.0	$78,424,625	$147,315,474	$215,762,552

These pump costs require some altering before they can be adapted to the optimisation package as the pumps will not be running at full capacity for 100% of the time.

D.9 Adaptation to EPANET

The pipe costs and the tank costs are fairly easy to include in EPANET. The pump costs do present a few areas of concern.

The issues associated with the pumps that can cause difficulty when modelling are partially due to the number of pumps in a given pumping station changing with the required capacity.

Another concern is associated with the annual operation costs of the pumps. They are based on a pumping station running continuously at capacity. The pumps used in Yorke Peninsula feed up to tanks and run at capacity for a fraction of the time. It is the determination of this fraction that requires addressing as a 5% error margin can result in a cost difference of up to $10,000,000 in some cases.

D.9.1 Methodology of Pump Data Inclusion

By knowing the average daily demand for the network and dividing this by the pump capacity, the percentage of operation can be determined using Equation D.1. The percentage of operation is already used to a degree in EPANET as controls on tanks switch the pumps on and off based on demand affecting the tank level. By determining the operation percentage through demand knowledge, this can be adequately measured in the cost regime.

$$\%\text{utilisation of pumping} = \frac{\text{Average Daily Demand}}{\text{Pump Station Daily Demand}} \tag{D.1}$$

This gives the pumping percentage for a given demand. Two parameters must be added to this equation to allow a more complete knowledge of the percentage. The first is a factor, which considers population fluctuation, which will alter the average daily demand. The second issue is more specific to the Yorke Peninsula study.

There are four main pumping stations on the peninsula: Paskeville, Kainton Corner (K.C.), Maitland and Mount Rat. Both Paskeville and K.C. are subject to the total system demand as they are located between Upper Paskeville Reservoir and the network. Maitland has approximately half of the systems water supply passing through its pumphouse and Mount Rat has more than a third. The current and projected demands based on a 2% population growth rate (EWS, 1987) for the different pumping stations are shown in Table D.13.

Table D.13 Demand projection for system pumping stations

Year from 1999	Population ratio	Mount Rat P.S. ML	MaitlandP.S. ML	K.C. P.S. ML	Paskeville P.S. ML
1999	1	37.64	47.42	87.96	87.96
2009	1.22	45.88	57.80	107.22	107.22
2019	1.49	55.93	70.46	130.70	130.70
2029	1.81	68.18	85.89	159.33	159.33

Determining the pumping percentages for each of the four stations incorporates the ratios of 0.43, 0.54, 1.0, and 1.0 for Mt Rat, Maitland, Kainton Corner and Upper Paskeville pumping stations respectively. This is calculated using Equation D.2.

$$pumping\ \% = \frac{(Average\ Daily\ Demand) \times (1 + rate)^{(no.\ years)}}{Pump\ Station\ Daily\ Demand} \times Pump\ Station\ Ratio \tag{D.2}$$

For a pump station ratio of 1.0 which holds for both Kainton Corner Pumping Station (K.C.P.S.) and Paskeville Pumping Station (P.P.S.) using a population growth rate of 2% (EWS, 1987), the following relationships for the different pumps are listed in Table D.14.

Table D.14 Operation percentages for different capacities at P.P.S. and K.C.P.S.

Pump Capacity	1999	2014	2029
l/s	%	%	%
22.0	399.8	538.1	724.2
44.0	199.9	269.1	362.1
219.0	40.2	54.1	72.8
439.0	20.0	27.0	36.3
1095.0	8.0	10.8	14.6
4381.0	2.0	2.7	3.6
8762.0	1.0	1.4	1.8

The other limiting factor on pump selection is that it cannot have a pumping percentage that is greater than 100% for the peak hydraulic case, which is the 2029 case. So, for the Upper Paskeville and Kainton Corner Pumping stations, all bar the 22L/s and 44L/s pumps are applicable. The 219 L/s pump would struggle during the peak period to keep up with demand.

Maitland Pumping Station (M.P.S.) has a ratio of 0.54 (54% of the system water passes through that point). This is presented in Table D.15.

Table D.15 Operation percentages for different capacities at M.P.S.

Pump Capacity	1999	2014	2029
l/s	%	%	%
22.0	215.9	290.6	391.1
44.0	108.0	145.3	195.5
219.0	21.7	29.2	39.3
439.0	10.8	14.6	19.6
1095.0	4.3	5.8	7.9
4381.0	1.1	1.5	2.0
8762.0	0.5	0.7	1.0

From this table it is seen that the Maitland Pumping Station may have another pump option in the 219L/s pumphouse as even during peak times, it could be under capacity.

Mount Rat Pumping Station (M.R.P.S.) has an even lower percentage (43%) of water passing through its pumping station. This is shown in Table D.16.

Table D.16 Operation percentages for different capacities at M.R.P.S.

Pump Capacity	1999	2014	2029
l/s	%	%	%
22.0	172	231	311
44.0	86	116	156
219.0	17	23	31
439.0	9	12	16
1095.0	3	5	6
4381.0	1	1.2	2
8762.0	0.2	0.6	1

As with the Maitland Pumping station, Mount Rat has three pump capacity options that do not overextend the pumping ability of the outfit.

So, from these tables, there are only two pump capacity options that cannot adequately cater for the needs of the network. Each of the remaining pump capacities has three head ranges that it can work in. The other lower capacities have been included as booster pump stations, which may come into consideration for future analysis. To gain the pumping percentage that must be factored to provide the weighting for operation costs in the network, the average over the 30 year period is taken as this represents the median level of required capacity. For the life of this study, Table D.17 shows the data projected for the year 2014 (mid way through 30 year cycle).

Table D.17 Summary of median operation percentages for each pumping station

Capacity	Paskeville	Kainton	Maitland	Mount Rat
L/s	%	%	%	%
22.0	538.1	538.1	290.6	231
44.0	269.1	269.1	145.3	116
219.0	54.1	54.1	29.2	23
439.0	27.0	27.0	14.6	12
1095.0	10.8	10.8	5.8	5
4381.0	2.7	2.7	1.5	1.2
8762.0	1.4	1.4	0.7	0.6

This means that a 219L/s capacity pumping station at Mount Rat would be expected to be operational for 23% of the year. A similar pumping station at Paskeville would require 54.1% average operation time as a greater volume of water must pass through it. If a 44 L/s pumping station was located in the same spot, it would not be feasible as even half way through the expected tenure of the system alterations, it is expected to run at 269.1% capacity over the year.

These are the operation percentages over the entire year. If the analysis was undertaken only for the peak pumping period, the operation percentage could be several times greater. This aspect is taken into consideration by the optimisation program, which penalises a shortfall in supply or pressure head. By penalising these two areas, the algorithm selects pumping stations that meet the conditions set by the network demands.

Represented in the Table D.18 are the costs that have been used in the GA optimisation analysis of the Yorke Peninsula network.

Table D.18 Base data for pumps for instalment in optimisation program

Firm Capacity	Firm Capacity	TDH	EPANET Design Capacity (Assumed)	EPANET Design Head (Assumed)	30 Years of maintenance costs	Construction costs	Total cost assuming 100% annual pumping
ML/day	L/s	m	L/s	m	$K	$K	$K
0	0	0	0	0	$0	$0	$0
1.9	22	30.42	11	30	$454	$322	$776
1.9	22	60.84	11	60	$613	$339	$952
1.9	22	91.26	11	100	$815	$385	$1,200
3.8	44	30.42	22	30	$615	$378	$993
3.8	44	60.84	22	60	$978	$418	$1,396
3.8	44	91.26	22	100	$1,363	$463	$1,827
18.9	219	30.42	109	30	$2,175	$568	$2,743
18.9	219	60.84	109	60	$4,077	$776	$4,853
18.9	219	91.26	109	100	$5,522	$849	$6,372
37.9	439	30.42	219	30	$4,210	$935	$5,144
37.9	439	60.84	219	60	$7,382	$1,102	$8,483
37.9	439	91.26	219	100	$10,702	$1,255	$11,957
94.6	1095	30.42	547	30	$9,163	$1,523	$10,686
94.6	1095	60.84	547	60	$17,732	$2,174	$19,906
94.6	1095	91.26	547	100	$26,255	$2,378	$28,634
378.5	4381	30.42	2190	30	$35,763	$4,909	$40,672
378.5	4381	60.84	2190	60	$69,724	$5,603	$75,327
378.5	4381	91.26	2190	100	$103,891	$6,561	$110,452
757	8762	30.42	4379	30	$70,694	$7,730	$78,425
757	8762	60.84	4379	60	$138,683	$8,633	$147,315
757	8762	91.26	4379	100	$206,599	$9,164	$215,763

By factoring the pump-time percentages into the operation costs, a more accurate determination of ongoing costs results. This in addition to the construction costs gives the total costs for the pumping station using Equation D.3.

$$\textit{pump station } \cos t = \textit{pumping}\% \times \textit{operation } \cos ts + \textit{construction } \cos ts \qquad \text{(D.3)}$$

The accumulated costs of all the pumping stations in the network makes up the total pump cost which along with the pipe and tank costs is the basis for the construction costs of the optimisation program.

The cost data used for the optimisation runs used the following assumptions in the selection process:

- A 2.5% annual inflation rate for the 1990-1999 period;
- An exchange rate of 0.72 between $US and $AUS;
- A 30 year life was assumed for the problem;
- A 4% discount rate was assumed for the next 30 years to 2029;
- Pipes are Ductile Iron and Concrete Lined (DICL);
- Trenching, backfilling and traffic costs are included in pipe costs;
- Tanks are above ground concrete;
- Each pumping station has been considered in EPANET to be a single pump with the head and capacities representative of the total station;
- Maintenance costs for the pumping station annually is 50% of the cost of running the station at full capacity for the full year;
- New pumping stations are to be constructed replacing the current stations
- Pumping station taken as an unspecified number of pumps with the same head and an accumulated capacity; and
- Design Capacity is around half the actual running capacity for the pumping station, which enables the pumps to operate well within their limits.

Of these assumptions, perhaps the most important is the reduction in the annual costs accumulated over the life of the pumping station. This represents the greatest cost difference.

In the case of the largest capacity pumps, the cost difference can be as high as $170m, which could well shadow the entire cost summary for the system.

D.10 Conclusions of Infrastructure Cost Analysis

The costing of water supply components is much more complicated than applying a purchase cost and applying a contingency factor. Specific conditions dictate different cost scenarios and the make-up of the total cost regime can be widely varied.

The pipe costs from Rawlinsons, SA Water and the US Department of Commerce report matched up reasonably well, with the US report taking the middle ground and hence being the preferred design option. Of the data sources, the US report was the most developed in its background as well.

The tank costs of SA Water and the US Department of Commerce matched together reasonably well. Due to this, the US report was selected as the tank data to keep it in line with the pipe costing.

Pumps presented a much more thoughtful approach to the development of its cost regime. Construction costs and operation costs were taken with a factor applied to the operation costs to gain a more realistic dollar value based on the percentage of time the pumps are operational. This methodology seems to be viable, and the costs produced from this should assist in the provision of a complete network design.

These costs make up the hydraulic section of the system costs. The other areas that require costs to be applied are to allow for the waterborne disease issues, disinfection by-product problems, taste and odour concerns, and the costs associated with supply and storage of disinfection.

Appendix E Description of Constants

Table E.1 Description of constants used in programming

Constant	Description of constant
POPSIZE	Population size of the genetic algorithm
TOURSIZE	Size of the tournament selection groups for the genetic algorithm
probcross	Probability of crossover for the genetic algorithm
maxgen	Number of generations for the genetic algorithm
select	Switch that determines selection type (roulette, tournament etc)
cross	Switch determining crossover type (uniform crossover, single-point etc)
seed	Picks the seed for the random number generator
probmut	Probability of mutation for gravity fed studies
pipemut	Probability of pipe mutation for controlled systems
pumpmut	Probability of pump mutation for controlled systems
tankmut	Probability of tank mutation for controlled systems
dosemut	Probability of dose mutation for controlled systems
crpipemut	Probability of pipe creep mutation for controlled systems
crpumpmut	Probability of pump creep mutation for controlled systems
crtankmut	Probability of tank creep mutation for controlled systems
crdosemut	Probability of dose creep mutation for controlled systems
creepswitch	Switch that determines if creep mutation is implemented
QUICKRUN	Switch that reduces the number of component options, thus quickening run
qualitymethod	Switch that selects the different quality methods (social or penalty cost)
dosemethod	Switch that selects the different dosing methods (random or incremented)
minresidual	Minimum allowable disinfectant residual for the quality penalty method
maxresidual	Maximum allowable disinfectant residual for the quality penalty method
lowrespenalty	Penalty cost applicable to solutions with too low residual levels
highrespenalty	Penalty cost applicable to solutions with too high residual levels
NUMPIPES	Number of pipes for implementation in the system
NUMTYPE	Number of different pipe options (ie. different diameters) for the system
pressonly	Switch that allows a solution to be solved for pressure only
capacityonly	Switch that allows a solution to be solved for capacity only

NUMBERPUMPS	Number of pumps for implementation in the system
PUMPOPTIONS	Number of different pump options (heads and capacities) For the system
paskps	Fraction of supply passing through Upper Paskeville Pumping Station
kcps	Fraction of supply passing through Kainton Corner Pumping Station
maitps	Fraction of supply passing through Maitland Pumping Station
mtrtps	Fraction of supply passing through Mount Rat Pumping Station
mintankpsb	Fraction of supply passing through Minlaton Tank Booster Pumping Station
minpsb	Fraction of supply passing through Minlaton Booster Pumping Station
yktnpsb	Fraction of supply passing through Yorketown Booster Pumping Station
ardpsb	Fraction of supply passing through Ardrossan Booster Pumping Station
ptvicpsb	Fraction of supply passing through Port Victoria Booster Pumping Station
currentdemand	Current demand of system
meddemfact	Median annual demand for the life of the system
NUMBERTANKS	Total number of tanks in the system
TANKOP	Switch determining if seasonal tank operation is to be implemented
NUMNEWTANKS	Number of tanks which are to be altered in the system
NEWTANKS	Number of tanks which are constructed from scratch
ADDTANKS	Tanks which have been capacity extended
TANKOPTIONS	Number of tank options (capacity) in the system
CHLORPEN	Penalty applicable to irregular dosing regime
TANKERRORCOST	Penalty applicable to tank level dropping below minimum allowable level
PUMPERRORCOST	Penalty applicable to unsatisfactory pump performance
PRESPENALTY	Penalty applicable for pressure deficits observed throughout the system
NUMNODES	Total number of nodes in the system
Kbsum	Bulk summer disinfectant decay rate
Kbspr	Bulk spring disinfectant decay rate
Kbaut	Bulk autumn disinfectant decay rate
Kbwin	Bulk winter disinfectant decay rate
Kwsum	Wall summer disinfectant decay rate
Kwspr	Wall spring disinfectant decay rate
Kwaut	Wall autumn disinfectant decay rate
Kwwin	Wall winter disinfectant decay rate
Kbatsum	Bulk summer decay rate for Arthurton tank gives 2^{nd} order model for system
Kbatspr	Bulk spring decay rate for Arthurton tank gives 2^{nd} order model for system
Kbataut	Bulk autumn decay rate for Arthurton tank gives 2^{nd} order model for system
Kbatwin	Bulk winter decay rate for Arthurton tank gives 2^{nd} order model for system
raintankcost	Cost of a rainwater tank for a 30-year life
tastelevel	Level at which taste and odour becomes a noticeable problem
Maxdose	Maximum possible dose for the system

numperhouse	Number of consumers per residence
DOSETYPE	Number of different dosing options (level of dosing)
EXTRACHROM	Number of seasonal and spatial dosing points considered in system
Numstations	Number of dosing stations in the system
Numseasons	Number of dosing seasons in system (4 seasons used, could be more or less)
opcostpask	Operational costs applicable to Upper Paskeville dosing station
opcostmait	Operational costs applicable to Maitland dosing station
opcostmtrat	Operational costs applicable to Mount Rat dosing station
SUMHOURS	Number of simulated hours to provide residual convergence for summer
SPRINGHOURS	Number of simulated hours to provide residual convergence for spring
AUTHOURS	Number of simulated hours to provide residual convergence for autumn
WINTHOURS	Number of simulated hours to provide residual convergence for winter
TOTHOURSHYD	Number of simulated hours to test the hydraulic aspects of the system
HstepFACTOR	Hydraulic time-step factor to produce faster simulations
ERRORS	Switch which processes any system errors and removes solution from run
CONPERDAY	Consumption per day for consumers
AVFACTOR	Factor which determines population from design supply of system
PRISTINERISK	Risk of waterborne disease illness from adequately treated water
POLLUTEDRISK	Risk of waterborne disease illness from untreated water
WI	Average water temperature in winter
AU	Average water temperature in autumn
SP	Average water temperature in spring
SU	Average water temperature in summer
THMlevel	THM level in μg/L for disinfectant level mg/L consumed
THMtemp	THM rise in μg/L adjusted for temperature rise or fall
addedrisk	Potential added risk of cancer contraction from exposure to THMs
price	Price of disinfectant
SICKCOST	Cost of illness due to waterborne disease contraction
DEATHCOST	Cost of death due to water borne disease contraction
morbtomort	Risk of mortality from waterborne disease morbidity
CANCCOST	Cost of cancer contraction
MINPOP	Minimum population before analysis of quality costs occur
ANNUAL	Rate of increase to determine present costs
PROJLIFE	Life of the study
POPGROWTH	Rate of population growth throughout life of the study
lowqual	Minimum residual level before waterborne disease could become a factor
bestpressure	Desired pressure levels at supply nodes in system

Table E.2 Typical values and accuracy of constants used in programming

Constant	Applied to NYT/YP	Typical Values (NYT:YP)	Comments on accuracy
POPSIZE	NYT/YP	100-1000	Configured to provide convergence
TOURSIZE	NYT/YP	2-7	Configured to provide convergence
probcross	NYT/YP	0.3-1.0	Configured to provide convergence
maxgen	NYT/YP	50-200	Configured to provide convergence
select	NYT/YP	switch	
cross	NYT/YP	switch	
seed	NYT/YP	switch	
probmut	NYT/YP	0.001-0.05	Configured to provide convergence
pipemut	YP	0.001-0.05	Configured to provide convergence
pumpmut	YP	0.001-0.05	Configured to provide convergence
tankmut	YP	0.001-0.05	Configured to provide convergence
dosemut	YP	0.001-0.05	Configured to provide convergence
crpipemut	YP	0.001-0.05	Configured to provide convergence
crpumpmut	YP	0.001-0.05	Configured to provide convergence
crtankmut	YP	0.001-0.05	Configured to provide convergence
crdosemut	YP	0.001-0.05	Configured to provide convergence
creepswitch	YP	switch	
QUICKRUN	YP	switch	
qualitymethod	YP	switch	
dosemethod	YP	switch	
minresidual	NYT/YP	(0.3-0.5) : (0.5-1.5)	Fine as based on system requirements
maxresidual	NYT/YP	(1.5-2.0) : (3.0-7.0)	Fine as based on system requirements
lowrespenalty	NYT/YP	$10m 1969US - na	Arbitrary to prevent low residuals
highrespenalty	NYT/YP	$10m 1969US - na	Arbitrary to prevent high residuals
NUMPIPES	NYT/YP	21 : 38	Fine as based on system setup
NUMTYPE	NYT/YP	16 : 9	Fine as based on system setup
pressonly	YP	switch	
capacityonly	YP	switch	
NUMBERPUMPS	YP	9	Fine as based on system setup
PUMPOPTIONS	YP	14 or 17	Fine as based on system setup
paskps	YP	100%	Accurate – based on current demands
kcps	YP	100%	Accurate – based on current demands
maitps	YP	54%	Accurate – based on current demands
mtrtps	YP	43%	Accurate – based on current demands

mintankpsb	YP	40%	Accurate – based on current demands
minpsb	YP	10%	Accurate – based on current demands
yktnpsb	YP	20%	Accurate – based on current demands
ardpsb	YP	10%	Accurate – based on current demands
ptvicpsb	YP	5%	Accurate – based on current demands
currentdemand	YP	87.96ML/day	Accurate – based on current demands
meddemfact	YP	1.35	Mod. accurate – growth rate estimate
NUMBERTANKS	YP	30	Fine as based on system setup
TANKOP	YP	switch	
NUMNEWTANKS	YP	7	Fine as based on system setup
NEWTANKS	YP	4	Fine as based on system setup
ADDTANKS	YP	3	Fine as based on system setup
TANKOPTIONS	YP	5 or 8	Fine as based on system setup
CHLORPEN	YP	$1,000,000 +/-	Arbitrary to prevent irregular residuals
TANKERRORCOST	YP	$1,000,000 +/-	Arbitrary to prevent tanks emptying
PUMPERRORCOST	YP	$1,000,000 +/-	Arbitrary to prevent pump failure
PRESPENALTY	NYT/YP	$500,000-10,000,000	Arbitrary to prevent pressure deficit
NUMNODES	NYT/YP	20 : 327	Fine as based on system setup
Kbsum	NYT/YP	-1.85 : -0.035 /day	Mod acc – calc. from recorded data
Kbspr	NYT/YP	-1.18 : -0.025 /day	Mod acc – calc. from recorded data
Kbaut	NYT/YP	-1.10 : -0.025 /day	Mod acc – calc. from recorded data
Kbwin	NYT/YP	-0.85 : -0.001 /day	Mod acc – calc. from recorded data
Kwsum	NYT/YP	0 /day	Assumed to be part of bulk decay
Kwspr	NYT/YP	0 /day	Assumed to be part of bulk decay
Kwaut	NYT/YP	0 /day	Assumed to be part of bulk decay
Kwwin	NYT/YP	0 /day	Assumed to be part of bulk decay
Kbatsum	YP	-0.20 /day	Mod acc – calc. from recorded data
Kbatspr	YP	-0.11 /day	Mod acc – calc. from recorded data
Kbataut	YP	-0.08 /day	Mod acc – calc. from recorded data
Kbatwin	YP	-0.06 /day	Mod acc – calc. from recorded data
raintankcost	YP	$3,000	Accurate – from recent study
treatcost	NY	$38m-75m 1969$US	Mod acc – adapted from recent study
tastelevel	NYT/YP	(1.5-2.0) : (3.0-7.0)	Based on system requirements
numperhouse	YP	2.0	Accurate – from recent study on Y.P.
DOSETYPE	NYT/YP	16 : 9 or 16	Fine as based on system setup
EXTRACHROM	NYT/YP	4 : 12	Fine as based on system setup
Numstations	YP	3	Fine as based on system setup
Numseasons	YP	4	Fine as based on system setup
opcostpask	YP	$30,000/year	Mod acc - adapted from recent study

opcostmait	YP	\$30,000/year	Mod acc - adapted from recent study
opcostmtrat	YP	\$30,000/year	Mod acc - adapted from recent study
SUMHOURS	NYT/YP	50 : 250 sim. hours	Fine as based on quality convergence
SPRINGHOURS	NYT/YP	50 : 400 sim. hours	Fine as based on quality convergence
AUTHOURS	NYT/YP	50 : 400 sim. hours	Fine as based on quality convergence
WINTHOURS	NYT/YP	100 : 850 sim. hours	Fine as based on quality convergence
TOTHOURSHYD	YP	50	Fine as based on system setup
HstepFACTOR	NYT/YP	1	Fine as does not effect accuracy
ERRORS	YP	switch	
CONPERDAY	NYT/YP	150-800 L/day	Mod acc – adapted from recent study
AVFACTOR	NYT/YP	0.5	Moderate accuracy
PRISTINERISK	NYT/YP	0.0001/per annual risk	1/10000 is acceptable risk for WQ
POLLUTEDRISK	NYT/YP	0.1/per annual risk	Unknown quantity for total WQ idea
WI	NYT/YP	3°C : 13°C	Accurate – Recorded data
AU	NYT/YP	8°C : 20°C	Accurate – Recorded data
SP	NYT/YP	8.5°C : 21°C	Accurate – Recorded data
SU	NYT/YP	15°C : 27°C	Accurate – Recorded data
THMlevel	NYT/YP	25μg/mg : 5μg/mg	Low-Mod Acc. based on recent studies
THMtemp	NYT/YP	0.5μg/mg	Low-Mod Acc. based on recent studies
addedrisk	NYT/YP	1×10^{-7} to 1×10^{-5}/μg	Low Accuracy based on recent studies
price	NYT/YP	\$1.2/L	Accurate – Current Cost
SICKCOST	NYT/YP	\$300-7500	Mod acc – adapted from recent studies
DEATHCOST	NYT/YP	\$1m-\$5m	Mod acc – adapted from recent studies
morbtomort	NYT/YP	0.0001	Low-Mod acc – from recent study
CANCCOST	NYT/YP	\$300,000-\$3m	Mod acc – adapted from recent studies
MINPOP	YP	50	
ANNUAL	YP	0.04/year	Mod accuracy for past decade
PROJLIFE	YP	30 years	30 year life is a reasonable study span
POPGROWTH	YP	0.02/year	Mod accuracy – based on last 30 years
lowqual	NYT/YP	(0.3-0.5) : (0.5-1.5)mg/L	Fine as based on system requirements
bestpressure	NYT/YP	20.0m	Fine as based on system requirements

Appendix F New York Tunnels

F.1 Different Solutions to the New York Tunnels Problem

The different solutions found in this Appendix are:

1. The current optimal feasible hydraulic solution for the system (Murphy *et al.*, 1993);
2. The Morgan and Goulter (1985) hydraulic solution;
3. The GA hydraulic solution using approximate Hazen-Williams values;
4. The GA quality and hydraulic analysis preferred solution;
5. The GA solution quality and hydraulic solution for a high pressure penalty loading; and
6. The GA quality and hydraulic solution with an increased quality penalty.

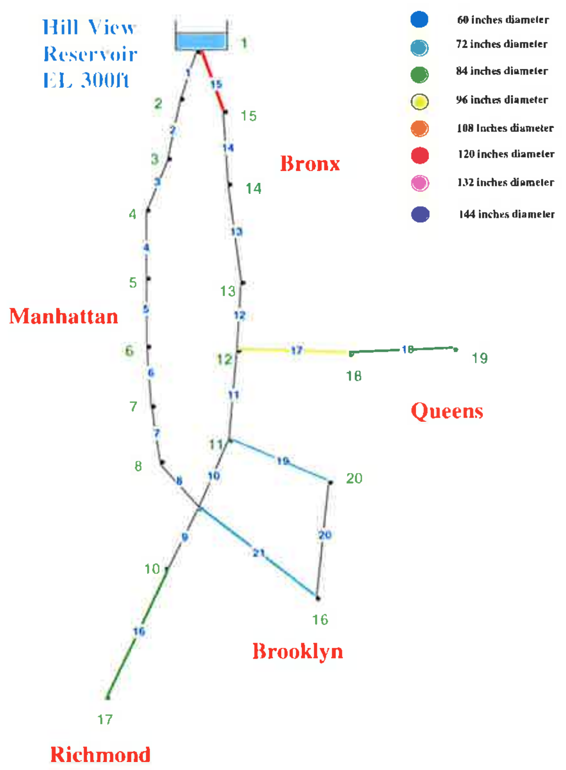

Figure F.1 NYT - Murphy *et al.*, 1993 solution $38.8m – feasible

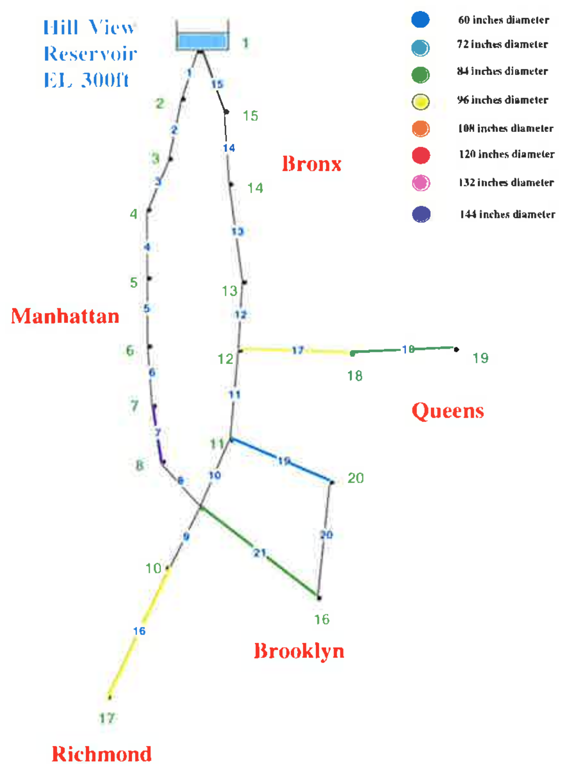

Figure F.2 NYT - Morgan and Goulter (1985) solution $39.20m – infeasible

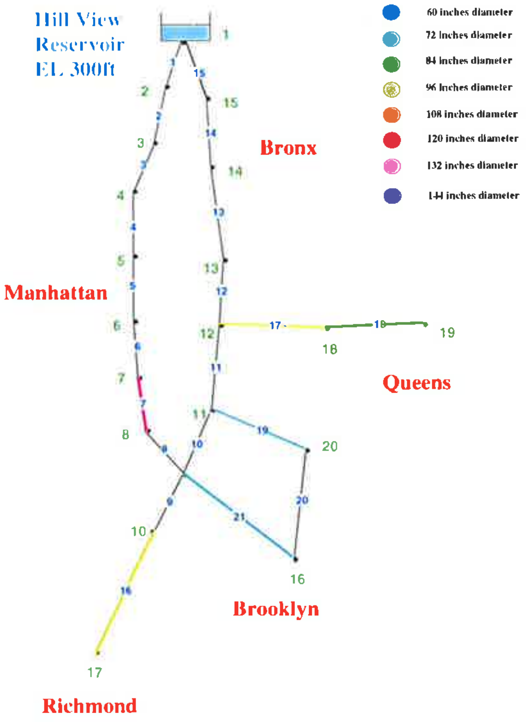

Figure F.3 **NYT - Solution using Approx Hazen Williams Values $38.13m – infeasible**

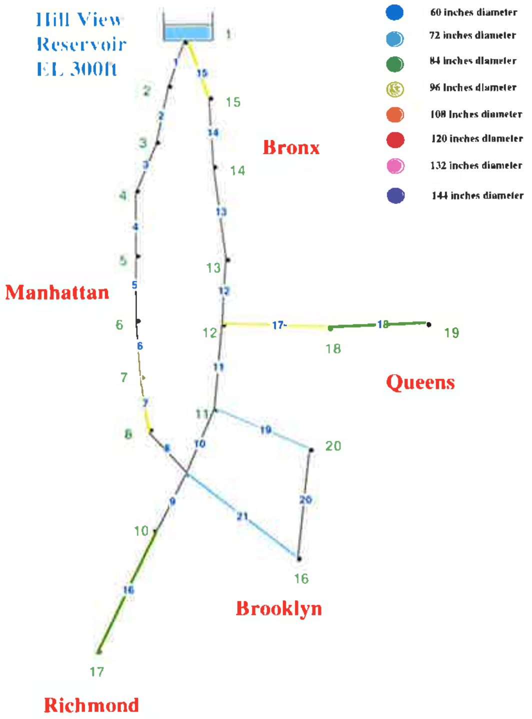

Figure F.5 **NYT - Quality with Increased Pressure Loading $40.26m - feasible**

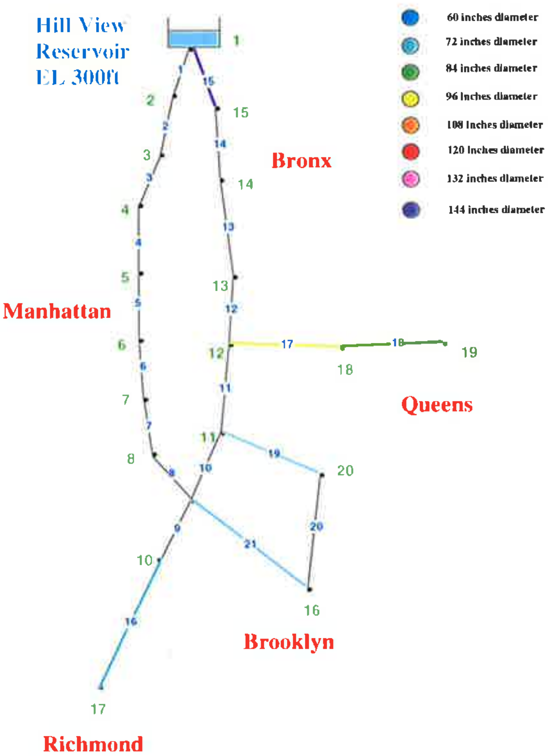

Figure F.4 **NYT - Best Quality and Hydraulic Solution $39.21m - infeasible**

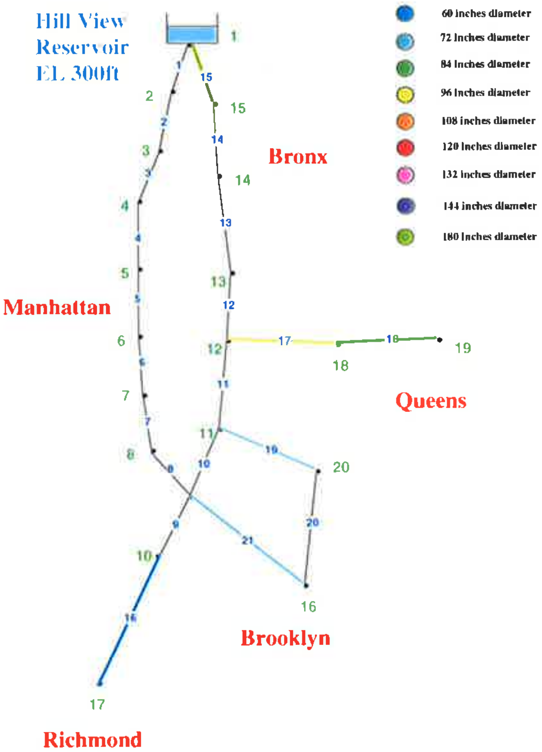

Figure F.6 **NYT - Quality with Increased Quality Penalty $40.61m – infeasible**

F.2 EPANET summer simulation results

The EPANET solution set for the flows through the different duplicate pipes from the results of the preferred hydraulic (\$38.80m) and the preferred combined (\$39.21m) simulations have been listed in Table F.1.

Table F.1 Use of duplicate pipes for the different solutions

Hydraulic optimal solution of \$38.80m						Preferred quality and hydraulic solution of \$39.21m					
Pipe	Diam inches	Flow gpm	Vel ft/s	HL ft/kft	Cl mg/L	Pipe	Diam inches	Flow gpm	Vel ft/s	HL/m ft/kft	Cl mg/L
D1	0	0	0	0.42	0	D1	0	0	0	0.41	0
D2	0	0	0	0.34	0	D2	0	0	0	0.33	0
D3	0	0	0	0.27	0	D3	0	0	0	0.26	0
D4	0	0	0	0.2	0	D4	0	0	0	0.2	0
D5	0	0	0	0.15	0	D5	0	0	0	0.15	0
D6	0	0	0	0.1	0	D6	0	0	0	0.1	0
D7	0	0	0	0.3	0	D7	0	0	0	0.29	0
D8	0	0	0	0.16	0	D8	0	0	0	0.15	0
D9	0	0	0	0	0	D9	0	0	0	0	0
D10	0	0	0	0.01	0	D10	0	0	0	0.01	0
D11	0	0	0	0.09	0	D11	0	0	0	0.09	0
D12	0	0	0	0.24	0	D12	0	0	0	0.24	0
D13	0	0	0	0.3	0	D13	0	0	0	0.3	0
D14	0	0	0	0.35	0	D14	0	0	0	0.36	0
D15	120	98905	2.81	0.27	1.31	D15	144	143200	2.82	0.22	1.32
D16	84	14715	0.85	0.05	0.21	D16	72	12261	0.97	0.07	0.26
D17	96	67996	3.01	0.41	0.8	D17	96	67996	3.01	0.41	0.85
D18	84	35358	2.05	0.23	0.61	D18	84	35358	2.05	0.23	0.67
D19	72	46905	3.7	0.83	0.7	D19	72	46911	3.7	0.83	0.79
D20	0	0	0	0.02	0	D20	0	0	0	0.02	0
D21	72	34536	2.72	0.47	0.53	D21	72	34531	2.72	0.47	0.59
1	180	361713	4.56	0.42	1.36	1	180	359019	4.53	0.41	1.36
2	180	322306	4.06	0.34	1.26	2	180	319612	4.03	0.33	1.26
3	180	282899	3.57	0.27	1.17	3	180	280205	3.53	0.26	1.17
4	180	245283	3.09	0.2	1.12	4	180	242590	3.06	0.2	1.12
5	180	207667	2.62	0.15	1.05	5	180	204974	2.58	0.15	1.05
6	180	170051	2.14	0.1	0.92	6	180	167358	2.11	0.1	0.92

7	132	132435	3.1	0.3	0.76	7	132	129742	3.04	0.29	0.81
8	132	94819	2.22	0.16	0.69	8	132	92126	2.16	0.15	0.74
9	180	24949	0.31	0	0.43	9	180	24949	0.31	0	0.48
10	204	71704	0.7	0.01	0.64	10	204	74388	0.73	0.01	0.7
11	204	220138	2.16	0.09	0.83	11	204	222831	2.19	0.09	0.89
12	204	369961	3.63	0.24	0.93	12	204	372655	3.66	0.24	0.98
13	204	419903	4.12	0.3	1.09	13	204	422596	4.15	0.3	1.08
14	204	459313	4.51	0.35	1.22	14	204	462006	4.53	0.36	1.21
15	204	399817	3.92	0.27	1.34	15	204	358216	3.52	0.22	1.33
16	72	9807	0.77	0.05	0.2	16	72	12261	0.97	0.07	0.26
17	72	31885	2.51	0.41	0.78	17	72	31885	2.51	0.41	0.83
18	60	14582	1.65	0.23	0.59	18	60	14582	1.65	0.23	0.65
19	60	29026	3.29	0.83	0.69	19	60	29029	3.29	0.83	0.79
20	60	3429	0.39	0.02	0	20	60	3439	0.39	0.02	0
21	72	34536	2.72	0.47	0.53	21	72	34531	2.72	0.47	0.59

From this table, it can be seen that pipe 16 and duplicate pipe D16 have significantly faster velocities for the $39.21m solution than the $38.80m solution. Although pipe 15 for the $38.80m result has a faster velocity than the $39.12m solution, the duplicate for this pipe (D15) is relatively standard for both results. Pipes 10 to 14 for the $39.21m solution have slightly increased velocities to the $38.80m solution.

Appendix G Yorke Peninsula System

G.1 Residence Times for the Yorke Peninsula System

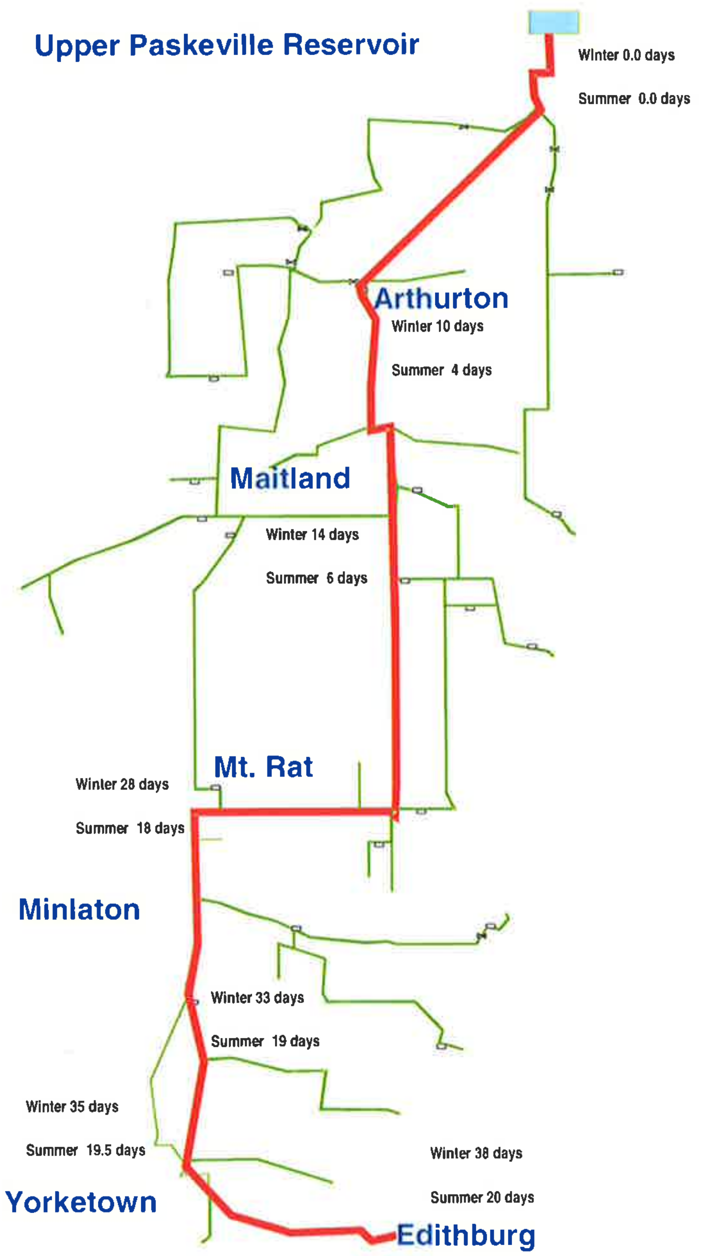

Figure G.1 Approximate residence times for Yorke Peninsula Trunk Main

G.1.1 Paskeville Pumping Station to Arthurton Tanks

Even though Arthurton Tanks act as the gravity fed source throughout the majority of the year, the actual water supply comes from Paskeville and is pumped when required up to the tanks. Typical 1998 supply ranged from 3.8ML/day during winter through to over 14ML/day during summer. This range is the average for the seasons, peak flows during this period would be significantly greater during the summer months whereas winter would see lower flows during some periods. The approximate residence times from Paskeville to Arthurton Tanks are shown in Table G.1.

Table G.1 Residence times from Paskeville to Arthurton Tanks

Length	Diameter	Rel Volume	Autumn	Winter	Spring	Summer
km	mm	ML	Res. Time	Res. Time	Res. Time	Res. Time
Incoming Supply (ML/day)			6.4	3.8	7.8	14.0
Average for section (ML/day)			5.675	3.325	6.7	12.6
Paskeville Pumping Station – Arthurton						
0.5	714	0.2	0.03	0.06	0.03	0.01
7.1	511	1.5	0.26	0.44	0.22	0.12
8.9	714	3.5	0.62	1.06	0.53	0.28
9.8	511	2.0	0.35	0.60	0.30	0.16
Total						
26.3		7.2	1.26	2.16	1.07	0.57
Additional Residence Time due to Tanks						
Arthurton Tanks (ML)			38	27.5	37	46.5
Residence Time (days)			6.7	8.3	5.5	3.7
Added Residence Time			7.96	10.43	6.59	4.26
Total Residence time			7.96	10.43	6.59	4.26

The Arthurton tanks effect the main residence levels. During winter, it has been assumed that the two small tanks have been shut off and the main tank left as a balance. Autumn and spring are taken to have one of the small tanks operational in addition to the main tank and summer assumes the full storage capacity. Up to this stage the residence time ranges from approximately four (4) days in summer through to over ten (10) days in winter.

G.1.2 Arthurton Tanks to the Maitland Tanks

All the water coming into this part of the network is gravity fed from Arthurton Tanks. It then runs through approximately 20km of pipeline to feed into the Maitland Tanks. The Maitland tanks act as a balancing storage for the Maitland pumping station and the lower system. Table G.2 lists the detention time between Arthurton Tanks and Maitland Tanks.

Table G.2 Residence time from Arthurton Tanks to Maitland Tanks

Length	Diameter	Rel Volume	Autumn	Winter	Spring	Summer
km	mm	ML	Res. Time	Res. Time	Res. Time	Res. Time
Incoming Supply (ML/day)			5.0	2.9	5.6	11.2
Average for section (ML/day)			4.225	2.375	4.5	9.8
Arthurton – Maitland						
0.5	511	0.1	0.02	0.04	0.02	0.01
0.7	511	0.1	0.03	0.06	0.03	0.01
8.3	511	1.7	0.40	0.71	0.38	0.17
1.5	461	0.3	0.06	0.11	0.06	0.03
4.5	616	1.4	0.32	0.57	0.30	0.14
4.5	461	0.7	0.18	0.31	0.17	0.08
Total						
20.0		4.3	1.02	1.81	0.95	0.44
Additional Residence Time due to Tanks						
Maitland Tanks (ML)			9	4.5	9	13.5
Residence Time (days)			2.1	1.9	2.0	1.4
Added Residence Time			3.15	3.70	2.95	1.82
Total Residence time			11.11	14.13	9.55	6.08

Maitland has a 9ML tank and a 4.5ML tank. The operation of these tanks has been assumed to be the smaller tank in winter, the larger tank in spring and autumn and both in summer. The accumulation of the residence times produces a period of approximately six (6) days in summer to over 14 days in winter.

G.1.3 Maitland Tanks to Mount Rat Tanks

The water is dosed and gravity fed down to the Maitland Pumping Station where it supplies Curramulka through Curramulka tank which also acts as a storage for the Mount Rat Pumping Station which feeds water up to Mount Rat Tanks, the balance for the lower third of the system. Table G.3 lists the residence times for this section of the system.

Table G.3 Residence times for Maitland Tanks to Mount Rat Tanks

Length	Diameter	Rel Volume	Autumn	Winter	Spring	Summer
Km	Mm	ML	Res. Time	Res. Time	Res. Time	Res. Time
Incoming Supply (ML/day)			3.5	1.9	3.4	8.4
Average for section (ML/day)			2.966	1.672	3.182	6.86
Maitland - Mount Rat						
1.2	411	0.2	0.06	0.10	0.05	0.02
3.3	411	0.4	0.15	0.26	0.14	0.06
4.0	411	0.5	0.18	0.32	0.17	0.08
0.1	411	0.0	0.00	0.00	0.00	0.00
2.9	361	0.3	0.10	0.18	0.09	0.04
2.7	361	0.3	0.09	0.17	0.09	0.04
3.6	361	0.4	0.12	0.22	0.12	0.05
12.8	361	1.3	0.44	0.79	0.41	0.19
0.9	311	0.1	0.02	0.04	0.02	0.01
0.1	311	0.0	0.00	0.00	0.00	0.00
1.0	361	0.1	0.03	0.06	0.03	0.01
1.2	361	0.1	0.04	0.07	0.04	0.02
3.0	361	0.3	0.10	0.19	0.10	0.05
2.8	361	0.3	0.10	0.17	0.09	0.04
Total						
39.7		4.3	1.45	2.57	1.35	0.63
Additional Residence time due to tanks						
Curramulka Tank (ML)			9	9	9	9

Residence Time (days)	3.0	5.4	2.8	1.3
Mount Rat Tanks (ML)	9	4.5	9	13.5
Residence Time (days)	3.0	2.7	2.8	2.0
Added Residence Time	13.49	14.27	13.18	11.60
Total Residence Time	24.59	28.40	22.73	17.67

Curramulka tank is around 9ML and is assumed to retain this volume year round. Mount Rat tanks take on a similar configuration to that seen of the Maitland Tanks earlier. Up to this point in the system, the residence times have a range of nearly 18 days through to over 28 days dependant on season.

G.1.4 Mount Rat Tanks to Minlacowie Tank

This section of the system is gravity fed. Minlacowie tank acts as a control for the lower part of this region down to Yorketown and Edithburg. The residence time for this section is shown in Table G.4.

Table G.4 Residence time from Mount Rat Tanks to Minlacowie Tank

Length	Diameter	Rel Volume	Autumn	Winter	Spring	Summer
Km	mm	ML	Res. Time	Res. Time	Res. Time	Res. Time
Incoming Supply (ML/day)			2.4	1.4	3.0	5.3
Mount Rat – Minlacowie Tank						
4.8	461	0.8	0.3	0.5	0.3	0.1
2.9	411	0.4	0.2	0.3	0.1	0.1
3.4	411	0.4	0.2	0.3	0.2	0.1
3.7	411	0.5	0.2	0.3	0.2	0.1
2.7	411	0.4	0.1	0.2	0.1	0.1
5.7	361	0.6	0.2	0.4	0.2	0.1
1.3	361	0.1	0.1	0.1	0.0	0.0
Total						
24.5		3.2	1.3	2.2	1.1	0.6
Additional Residence time due to Tanks						

Minlacowie Tank (ML)	3	3	3	3
Residence Time (days)	1.2	2.1	1.0	0.6
Added Residence Time	2.55	4.29	2.09	1.16
Total Residence Time	27.14	32.69	24.82	18.84

The accumulated residence time ranges between about 19 days in summer and 33 days in winter.

G.1.5 Minlacowie Tank to Yorketown and Edithburg

The demand through this section is now much focussed on Yorketown and Edithburg. Both of these towns have storage facilities that supply their region and the area is gravity fed. The residence times for the bottom end of the system are shown in Tables G.5 and G.6.

Table G.5 Residence times from Minlacowie Tank to Yorketown

Length	Diameter	Rel Volume	Autumn	Winter	Spring	Summer
km	mm	ML	RT	RT	RT	RT
Incoming Supply (ML/day)			1.6	1.0	2.0	3.8
Minlacowie Tank – Yorketown						
13.1	231	0.5	0.3	0.5	0.3	0.1
4.6	231	0.2	0.1	0.2	0.1	0.1
16.8	186	0.5	0.3	0.5	0.2	0.1
Total						
34.5		1.2	0.7	1.2	0.6	0.3
Additional Residence time due to Tanks						
Yorketown Tank (ML)			1	1	1	1
Residence Time (days)			0.6	1.0	0.5	0.3
Total Residence time			28.5	34.9	25.9	19.4

Table G.6 Residence times from Yorketown to Edithburg

Length	Diameter	Relative Volume	Autumn	Winter	Spring	Summer

km	mm	ML	Res. Time	Res. Time	Res. Time	Res. Time
Incoming Supply (ML/day)			0.8	0.5	1.0	1.9
Yorktown – Edithburg						
5.7	231	0.2	0.3	0.5	0.2	0.1
7.9	186	0.2	0.3	0.4	0.2	0.1
Total						
13.6		0.5	0.6	0.9	0.5	0.2
Additional Residence time due to tanks						
Edithburg Tank (ML)			1	1	1	1
Residence Time (days)			1.3	2.0	1.0	0.5
Total Residence time			30.33	37.79	27.37	20.18

The approximated total residence time throughout the length of the network from Paskeville down to Edithburg has a range of about 20 days in summer through to around 38 days during the winter months.

G.2 Assumed Tank Operation for EPANET Simulation

The operation of the main storage facilities in the network is critical to the residence time for the supply. This directly affects the amount of disinfection consumed throughout the distribution system.

Due to the variation in water consumption at different times of the year, the residence times alter significantly. As seen from Figure G.2 examining the monthly consumption data for the Peninsula, the winter months have about a third (33%) of the demand as the summer months. If the system were not altered to cater for this, the residence times in winter would be three (3) times those experienced in summer. As the predicted summer residence time for 1998 demand in the network from Paskeville to Edithburg is approximately 17 days this would equate to nearly a two month residence time during the winter period.

To counterbalance this potential increase in residence time, the tanks are operated to significantly reduce the residence times. Although the operation of the tanks in the network is highly dependent on seasonal demand and quality issues, the pattern of tank rotation is variant due to maintenance, quality irregularities and holiday peaks. To present the residence time issue in a form suitable for simulation and optimisation using an EPANET program, data obtained from the system on the main tanks operated and the relative sizes of the storage facilities (Hogben, 1999) was adapted as seen in Table G.7.

Table G.7 Assumed Seasonal Tank Operation for Yorke Peninsula

Location	Total	Summer	Autumn	Winter	Spring
Arthurton	1×27.5ML 2×9.5ML	1×L + 2×S	1×L + 1×S	1×L	1×L + 1×S
Maitland	1×9ML 1×4.5ML	1×L + 1×S	1×L	1×S	1×L
Mt Rat	1×9ML 1×4.5ML	1×L + 1×S	1×L	1×S	1×L

Number of Tanks in Operation (L = larger tank, S = smaller tank)

Pictures of these tanks can be seen in Figure G.2, Figure G.3 and Figure G.4 for Maitland, Mount Rat and Arthurton and respectively.

Figure G.2 Maitland Tanks

Figure G.3 Mount Rat Tanks

Figure G.4 Arthurton Tanks

This operational procedure results in the relative volumes of the tank locations (relative volume being the total tank volume at that storage node) as listed in Table G.8. As can be read off of Table G.8, the winter storage volume is about half of the observed capacity for the summer months.

Table G.8 Relative Volume in Operation (ML)

Location	Actual	Summer	Autumn	Winter	Spring
Arthurton	46.5	46.5	37.0	27.5	37.0
Maitland	13.5	13.5	9.0	4.5	9.0
Mt Rat	13.5	13.5	9.0	4.5	9.0

Two methods that can be used to adjust the volume when altering the EPANET code are:

- Adjusting the height of the storage location; and
- Adjusting the relative diameter of the storage.

Both of these methods are shown in Figure G.5. Adjusting the tank height would assist in volume reduction, however, this would have two major effects on the overall system:

- It would alter the starting head (HGL) for the water, hence change the pressure levels throughout the system;
- The tanks have valves controlling their operation. These valves, as well as the pumps feeding the tanks are turned on and off based on the water elevation inside the tank. By altering the height of the tank, these levels could be interfered with or even made redundant, thus upsetting the operational process for the tanks.

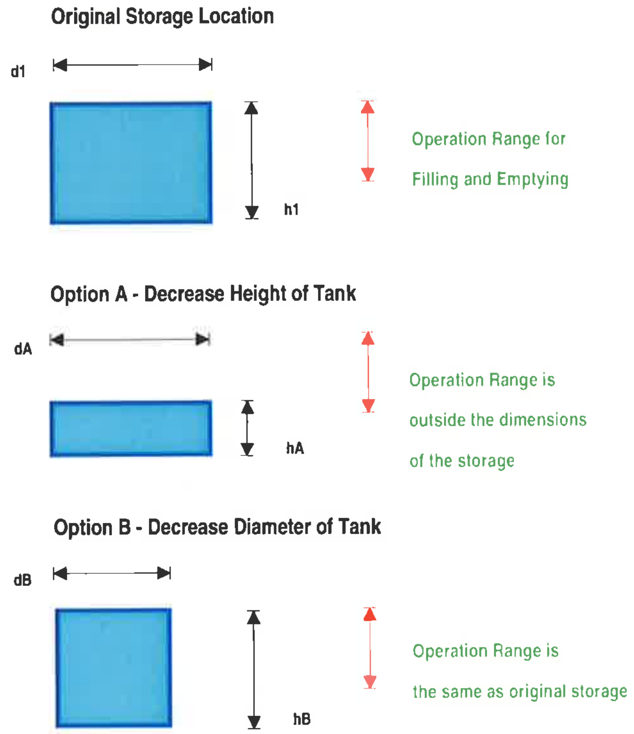

Figure G.5 Options for Alteration of Storage Volume

Both of these methods make Option A inferior as Option B does not have these problems. As the height is not affected, the pressures or control heights can remain the same hence, the overall system structure is not significantly altered.

As Option B is the selected method, it becomes important to determine the relative diameters of these storage locations. Even though several tanks may exist at a node, they have been assumed to be one tank to assist in the programming methodology for this study. These views are illustrated in Figure G.5 examining the two options.

The seasonal volumes of the storage locations have been calculated. Now, these volumes must be allocated a diameter allowing for that capacity. Table G.9 lists the multiplication factors required to create that parameter.

Table G.9 Multiplication Factor for Diameter Conversion

Location	Summer	Autumn	Winter	Spring
Arthurton	1	0.88	0.77	0.88
Maitland	1	0.82	0.58	0.82
Mt Rat	1	0.82	0.58	0.82

Once these factors are applied to the existing diameter, the relative seasonal diameters can be ascertained. These measurements can be found in Table G.10.

Table G.10 Assumed Tank Diameters (m)

Location	Actual	Summer	Autumn	Winter	Spring
Arthurton	84.5	84.5	74.5	65.5	74.5
Maitland	56.2	56.2	45.9	32.4	45.9
Mt Rat	51	51.0	41.6	29.4	41.6

With the storage diameters known, and the operational procedure assumed, the model can be adapted to allow a simulation that more closely represents the real life conditions experienced along the network.

G.3 Water Quality Simulation Results

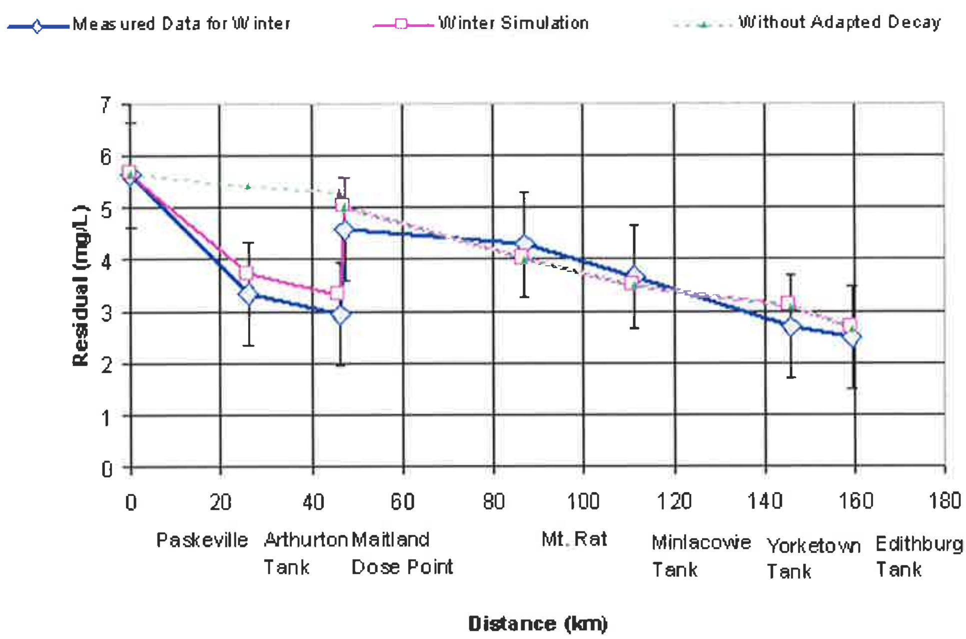

Figure G.6 Comparison of simulated and recorded residuals for winter

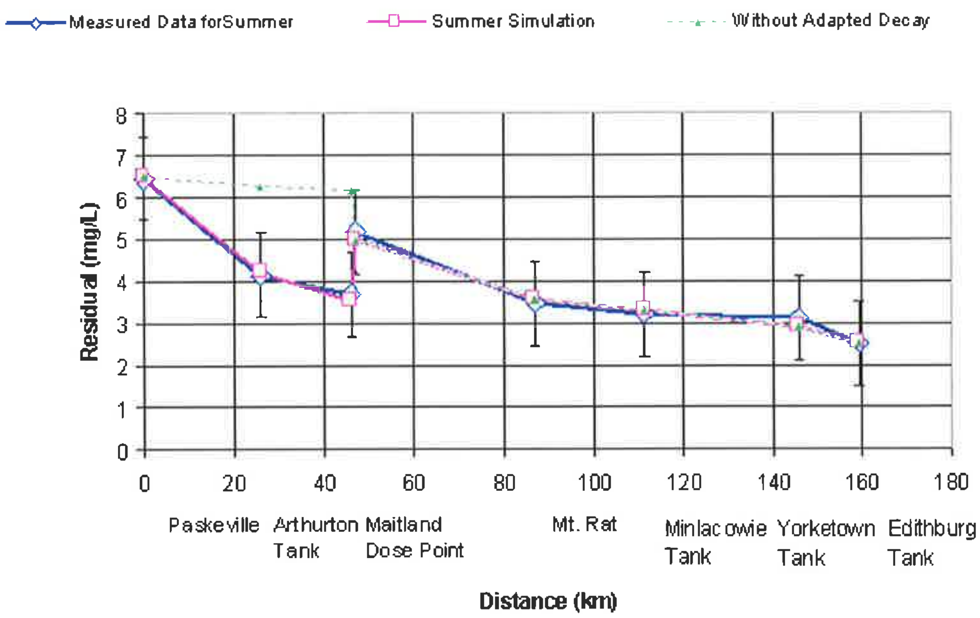

Figure G.7 Comparison of simulated and recorded residuals for summer

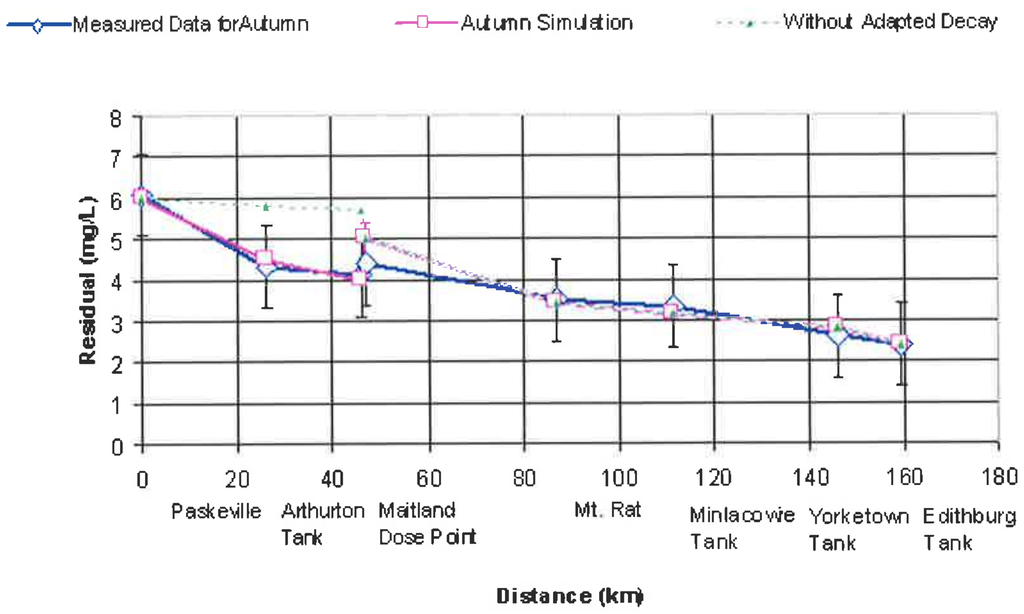

Figure G.8 Comparison of simulated and recorded residuals for autumn

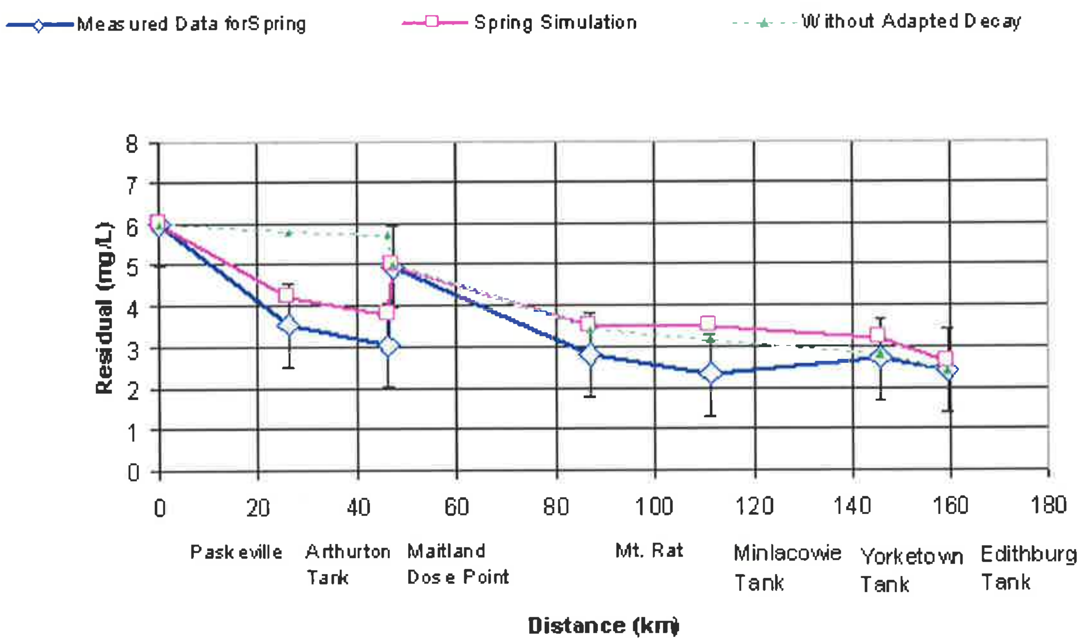

Figure G.9 Comparison of simulated and recorded residuals for spring

Appendix H Yorke Peninsula Simulation

H.1 Pressure Analysis of Yorke Peninsula System

Table H.1 Duplicate pipes selected from pressure analysis of Yorke Peninsula System

Pipe ID	Node In	Node Out	Length	Population = 2000 All pipes	Population = 250 All pipes	Population = 250 Selected Pipes
			metres	diameter (mm)	diameter (mm)	diameter (mm)
285	1402	14014	599	0	150	0
19012	501	807	110	0	300	0
19013	1010	1011	3297	0	0	0
19014	1011	1012	1514	0	0	0
19015	1012	1013	5458	0	0	0
19016	1014	653	927	0	0	0
19017	654	1015	1457	0	0	0
19018	1013	14014	791	0	200	0
19027	1017	605	231	0	150	0
19029	1101	607	156	0	250	0
19031	1103	609	159	0	150	0
19032	809	611	628	0	150	0
19034	610	1105	6850	0	0	100
19035	1105	1106	3839	100	100	0
19038	812	1116	501	100	100	0
19046	1403	817	527	0	100	0
19047	591	818	1043	0	100	0
19053	617	824	418	0	400	0
19055	615	826	715	0	150	0
19063	621	828	727	0	150	0
19066	1413	1423	5436	0	100	0
19068	1422	1421	4834	0	0	100
19069	1418	829	637	0	100	0
19071	830	1421	3073	250	0	0
19074	625	832	631	100	0	100
19082	1408	1114	2648	0	0	100
19084	628	837	510	150	0	0
19087	838	1303	4498	100	0	0
19105	844	1424	1216	0	100	0
19108	637	1700	184	0	250	0
19116	1706	847	1347	0	200	0
19124	1803	640	186	0	200	0
19126	848	853	90	0	100	0
19136	1900	1901	16837	0	200	0
19137	647	857	440	0	100	0
19139	2000	858	507	0	250	0
19140	2000	859	379	0	250	0
19141	1904	650	64	150	150	0

19155	856	1900	2482	150	250	150
19156	1406	704	44	0	450	0
19157	802	805	722	0	200	0
19158	1009	1010	4699	100	150	100
19161	816	867	6520	100	100	0
19162	674	868	299	0	150	0
19165	1409	652	163	0	250	100
19166	869	812	346	0	150	0
19168	871	5180	56	0	300	100
19169	5190	869	80	100	300	0
19170	658	1009	160	0	150	0
19175	804	602	457	0	150	0
19186	865	1904	541	0	200	0
19188	711	1902	13064	250	0	250
19189	852	1803	316	0	200	0
19191	831	624	699	100	0	0
19193	829	622	541	0	150	0
19194	843	634	89	0	300	0
19196	827	620	747	0	300	0
19197	823	616	512	0	300	0
19198	820	1412	1279	0	150	0
19200	833	1409	443	0	300	0
19201	834	1500	2037	0	150	0
19203	837	1300	2694	250	400	400
19205	842	1115	1653	150	150	150
19206	817	614	496	100	150	100
Total Cost				\$5.4m 1999\$AUS	\$8.03m 1999\$AUS	\$5.02m 1999\$AUS

H.2 Capacity Analysis of Yorke Peninsula System

Table H.2 Selection process of new pipes for Yorke Peninsula System

Pipe Number	Pipe ID	EWS (1987)	Cap - Full	Cap - 79	Cap - 38	Selected for Design
1	285		Y		Y	Y
2	2222					
3	19001		Y			Y
4	19002		Y			
5	19003					
6	19004					
7	19005		Y			
8	19006					
9	19007					
10	19008	Y				
11	19009	Y				
12	19010					
13	19011		Y	Y		Y
14	19012		Y			
15	19013					
16	19014		Y	Y	Y	Y
17	19015					
18	19016					
19	19017		Y			
20	19018	Y	Y	Y	Y	Y
21	19019					
22	19020					
23	19021		Y	Y		Y
24	19022					
25	19023					
26	19024					
27	19025					
28	19026					
29	19027		Y		Y	
30	19028					
31	19029		Y	Y		Y
32	19030					
33	19031		Y		Y	Y
34	19032		Y			
35	19033					
36	19034		Y	Y	Y	Y
37	19035					
38	19036					
39	19037		Y			
40	19038					
41	19039		Y			
42	19040					
43	19041			Y		Y
44	19042					
45	19043					
46	19045					
47	19046		Y			
48	19047					
49	19048					
50	19049					
51	19050		Y			
52	19051					

53	19052					
54	19053		Y			
55	19054					
56	19055					
57	19056					
58	19057					
59	19058					
60	19059					
61	19060					
62	19061					
63	19062					
64	19063		Y			
65	19064					
66	19065					
67	19066					
68	19067					
69	19068		Y	Y	Y	Y
70	19069					
71	19070					
72	19071		Y			
73	19072					
74	19073					
75	19074		Y	Y	Y	Y
76	19075					
77	19076					
78	19077					Y
79	19078					
80	19079		Y			
81	19080					
82	19081					
83	19082	Y				Y
84	19083					Y
85	19084	Y				
86	19085					
87	19086					
88	19087	Y				
89	19088					
90	19090					
91	19091					
92	19092					
93	19093			Y		Y
94	19094		Y			
95	19095					
96	19096		Y	Y	Y	Y
97	19097		Y	Y		Y
98	19098					
99	19099		Y			
100	19100					
101	19101					
102	19102					
103	19103			Y		Y
104	19104					
105	19105					
106	19106					
107	19107					
108	19108		Y		Y	Y
109	19109					
110	19110		Y			

111	19111					
112	19112					
113	19113					
114	19114					
115	19115					
116	19116					
117	19117		Y			
118	19118					
119	19119					
120	19120					
121	19121					
122	19122	Y				
123	19123	Y				
124	19124	Y	Y			Y
125	19125	Y	Y	Y		Y
126	19126	Y	Y			
127	19127	Y	Y	Y		Y
128	19128	Y	Y	Y		Y
129	19129	Y				
130	19131		Y	Y		Y
131	19132		Y	Y	Y	Y
132	19133					
133	19134					
134	19135					
135	19136					
136	19137					
137	19138					
138	19139		Y			
139	19140					
140	19141		Y	Y		Y
141	19142					
142	19143					
143	19144					
144	19145					
145	19146		Y			
146	19147					
147	19148					
148	19149					
149	19150		Y		Y	Y
150	19151					
151	19152					
152	19153		Y			
153	19154					
154	19155		Y	Y	Y	Y
155	19156					
156	19157		Y			
157	19158		Y	Y	Y	Y
158	19159					
159	19160					
160	19161		Y	Y		Y
161	19162		Y			
162	19163					
163	19164					
164	19165		Y	Y	Y	Y
165	19166					
166	19167					
167	19168		Y		Y	Y
168	19169		Y			

169	19170					
170	19171					
171	19172					
172	19173		Y			
173	19174					
174	19175					
175	19176					
176	19177					
177	19178					
178	19179					
179	19180		Y			
180	19181					
181	19182					
182	19183					
183	19184					
184	19185					
185	19186		Y			
186	19187		Y			
187	19188		Y	Y	Y	Y
188	19189		Y			
189	19190		Y			
190	19191		Y	Y		Y
191	19192					
192	19193		Y			
193	19194					
194	19195					
195	19196					
196	19197					
197	19198					
198	19199			Y	Y	Y
199	19200		Y			
200	19201					
201	19202					
202	19203		Y	Y	Y	Y
203	19204		Y	Y		Y
204	19205		Y	Y	Y	Y
205	19206		Y	Y	Y	Y
206	19286	Y				

H.3 EPANET Input File for the Yorke Peninsula

The EPANET input file considers a typical quality run. Dummy pipes have been set up so that EPANET can cater for two different demand patterns (industrial and residential) at some of the more significant demand nodes. The main storage at Upper Paskeville (Node 500) is assumed as a reservoir, as the level is kept relatively constant through pumping from the River Murray.

```
[TITLE]
 EPANET Example Network 1

[JUNCTIONS]
;------------------------------
;      Elev    Demand    Pattern
; ID    m      lps
;------------------------------
;nodes in 590s are dummy nodes before
dosing points
  590  150
  591  220
  592  120
  637  130
  600  145
  601  145
  602  105
  603  125
  604  125
  605  200
  606  200
  607  120
  608  120
  609  100
  610  100
  611  110
  612  220
  614  145
  615  145
  616  105
  617  105
  618  150
  619  150
  620  75
  621  75
  622  105
  623  105
  624  145
  625  145
  626  110
  627  115
  628  115
  629  115
  630  115
  631  115
  632  115
  633  115
  634  155
  635  155
  636  130
  638  110
  639  110
  640  25
  642  30
  643  30
  644  20
  646  85
  647  85
  648  15
  649  30
  650  40
  651  40
  652  110
  653  50
  654  50
  657  120
  658  120
  659  110
  660  110

;Nodes to connect TCV for potential tanks
;All of these have pumps to get water to an
acceptable pressure
;so assumption that towns  elevation 21m
(4m high) below the tank
;elevation of 25m.  This only holds for the
seaside towns of
;Pt Victoria, Wool Bay and Coobowie

;Check valve connecting nodes
;decided to connect straight to demand
nodes here
;Pt Victoria
; 661  25
;Wool Bay
; 662  25
;Coobowie
; 663  25
;Petersville
  664  120

;Throttle Control Valve Connecting nodes
;Pt Victoria
 671  25
;Wool Bay
 672  25
;Coobowie
 673  25
; Petersville
 674  120

  701  150
  702  145
  703  145
  704  150
  705  150
  706  95
  707  95

; 708-709 at Minlaton tank as optional pump
  708  95
  709  95
```

```
; 710-711 at YKTOWN booster
  710  85
  711  85

; 712-713 booster to fill MIN Tank
  712  75
  713  75

; 714-715 booster to fill Sth Kilkerran
Tank and supply Pt Victioria
  714  120
  715  120

; 716-717 booster to fill Ardrossan Tank
  716  140
  717  140

  800  150
  801  145
  802  140
  803  115
  804  115
  805  130
  806  115
  807  115
  809  115
  810  120
  811  120
  812  220
  813  225
  814  200
  815  200
  816  195
  817  150
  818  215
  819  150
  820  150
  823  110
  824  105
  825  90
  826  140
  827  60
  828  50
  829  145
  830  120
  831  145
  832  140
  833  110
  834  100
  836  120
  837  105
  838  55
  839  125
  840  105
  841  125
  842  100
  843  150
  844  150
  845  120
  846  122
  847  95
  848  95
  849  95
  850  90
  851  20
  852  30
  853  95
  854  10
  856  80
  857  85
  858  35
  859  40
  860  42
  861  40
  862  10
  863  12
  864  100
  865  40
  867  120
  868  115
  869  230
  870  145
  871  225
  872  100
  875  30
  876  110

;approximations have been made on some of
the demands as they
;have separate patterns which EPANET has
difficulty with
;so two supply nodes with separate pattern
were used
;to allow for this

  656   140     0.27    1

; node 1000 has potential water flow in
from trunk main from murray
; assuming 500 is a reservoir not a tank

  1002  135     .26     1
  1003  140     .1      1
  1004  130     .21     1
  1005  115     .23     1
  1006  95      .62     1
  1007  100     .91     1
  1008  110     .06     1
  1009  115     .65     1

;splitting demands into industrial and
residential by establishing
;dummy nodes

  1010  110
  1080 110     1.14    6
  1090 110     .82     1

  1011  100     .2      1

; dummies
  1012  80
  1082 80      .63     6
  1092  80     1.79    1

  1013  55      .8      1
  1014  60      .54     1

; 1085 and 1095 are extensions of 1015
  1015  20
  1085  20     4.99    6
  1095  20     1.92    1

  1016  140     .42     1
  1017  215     .37     1
  1100  160     1.16    1
  1101  110     .3      1

  1102  150     1.0     1

  1200  80      .61     1
  1103  95      .13     1
  1104  115     .05     1
  1105  70      0.31    1
  1106  60      .68     1
  1107  80      .4      1
  1108  120     0.18    1
```

```
 1109  160     .54     1
 1110  170     .17     1
 1111  160     0.06    1

; dummies
 1112  160     0.47   1

 1113  120     .25     1
 1114  135     0.03    1
 1115  90     .62     1
 1300  90      2.13    1

; dummies
 1302  5
 1382  5      1.59    6
 1392  5      0.99    1

 1303  30      0.75    1

; dummies
 1116  220
 1186  220    0.28    6
 1196  220    0.6     1

; dummy node to allow 1402 to connect to
the ardrossan line

 14014 86
 1117  170     0.41    1
 1118  170     .16     1

;dummies
 1400  195
 1480  195     3.87   6
 1490  195    2.3     1


 1401  95     .57     1
 1402  90      .14     1
 1403  160     .14     1
 1404  120     0.03    1
 1405  115     .38     1
 1406  145     .14     1
 1407  140     .15     1
 1408  120     .13     1
 1409  110     .45     1
 1500  80      0.04    1
 1501  75      .65     1
 1503  75      0.75    1
 1700  120     0.06    1
 1701  75      .69     1
 1702  80      .71     1
 1703  70      .5      1
 1704  65      1.3     1

; dummies
 1705  60
 1785  60     2.72    6
 1795  60     1.65    1

 1706  90      .63     1
 1800  100     .07     1
 1801  75      .95     1
 1802  95     .04     1
 1803  15      .83     1

; dummies
 641   15
 2418  15     2.49    6
 2419  15     1.5     1

; dummies
 645   15
 2458  15     1.41    6
 2459  15      3.14    1

 1707  75      0.49    1
 1708  80      .86     1
 1709  100     0.42    1
 1900  70      1.11    1

; dummies
 1901  30
 1981 30      1.41    6
 1991 30      3.14    1

 1902  45      0.73    1

; dummies
 1903  5
 1983  5      0.49    6
 1993  5      1.0     1

; dummies
 1904  40
 1984  40     1.41    6
 1994  40     0.99    1

 2000  40      0.02    1
 2001  40      0.18    1

; dummies
 2002  5
 2082  5      0.95    6
 2092  5      0.8     1

; dummies
 2003  25
 2083  25     0.26    6
 2093  25     0.32    1

 2004  20      0.9     1

; dummies
 2005  10
 2085  10     1.74    6
 2095  10     1.73    1

 1410  140     .33     1
 1411  145     .18     1
 1412  130     0.05    1
 1413  115     0.12    1

; dummies
 1414  20
 1484  20     0.5     6
 1494  20     0.48    1

 1415  100     0.86    1
 1416  125     0.07    1
 1417  135     0.39    1
 1418  150     .68     1
 1419  145     0.07    1
 1420  100     .56     1
 1421  90      1.13    1
 1422  75      .54     1
 1423  100    .3       1

; dummies
 1600  70
 1680  70     0.5     6
 1690  70     1.04    1

 1424  130     .2      1
 1425  100     1.25    1
 1426  100     .53     1
;  3000  150
```

```
 4010   25
 4020   25
 4030   25
 4040   120

  5010  105
  5020  110
  5030  140
  5040  220
; node for maitland dosing
  5050  220
  5060  143
  5070  104
  5080  74
  5090  142
  5100  150
  5110  150
  5120  111
  5130  130
  5140  130
  5150  113
  5160  91
  5170  154
  5180  240
  5190  240
  5210  110
  5220  106
  5230  111
  5240  111
  5250  45
  5260  54
  5270  34
  5280  50
  5290  70

[TANKS]
;------------------------------------------
-
;     Elev    Init    Min      Max      Diam.
; ID   m     Level  Level     Level      m
;------------------------------------------
-
;500  151.92    7.06     1.08   7.06
180
; settng 500 (Upper Pask.) as a reservoir

;New tanks (401-409) added into the GA

;Pt Victoria
401 25.00       5.0     1.0     5.0     15

;Wool Bay
402 25.0        5.0     1.0     5.0     15


;Coobowie
403 25.0        5.0     1.0     5.0     15

;Petersville
404 120.0       5.0     1.0     5.0     15

;existing tanks
500  158.98
501  105.73     4.46    1.27    4.46
11.34
502  111.05     4.95    1.95    4.95
11.34
503  141.9      4.9     2.1     4.9
11.34

;Maitland
504  220        6.6     1       6.6        51
506  142.34     4.46     1.66    4.46
11.34
507  103.85     4.95     2.05    4.95
11.34
508  73.94      4.46     1.06    4.46
11.34
509  141.85     4.95     1.15    4.95
11.34
510  150.9      4.5      1.1     4.5
11.28
511  150.9      4.5      1.1     4.5
11.28
512  111        5.6      1.82       5.6
16.77

;Mount Rat
513  131.46     5.45     2.04    5.45
56.2

515  113.08     5.22     1.92    5.22
23.69
516  91.08      5.42     1.92    5.42
32.51
517  154.37     7.45     0.63    7.45
39.26

;Arthurton
518  238.18     9.65     3.82    9.65
84.5

521  110        6.6      2       6.6
16.77
522  105.5      5.4      2       5.4
16.77
523  111.3      5.2      1.7     5.2
23.72
524  111.55     4.95     1.95    4.95
11.34
525  44.62      6.28     1.38    6.28
13.864
526  53.92      7.28     1.08    7.28
13.864
527  33.72      7.28     1.28    7.28
13.864
528  50.36      7.24     1.64    7.24
13.91
529  70.71      5.42     1.29    5.42
32.51


[QUALITY]
;------------------------
;       TO      Initial
;Nodes  nodes   Concen. mg/L
;------------------------
;assuming reservoir has complete chlorine
decay
 500            0.0
;starting concentration
 501     5290   0.0
 590            0.0
[SOURCES]
;-------------------------
;       chlor    Source
; ID    mg/L     Pattern
;-------------------------
 800    4.0       11
 816     4.0       11
 846     4.0       11
; 590    0.0       11

[PIPES]
;------------------------------------------
-----------
```

```
;       Start      End       Length     Diam
Rough. Check
;ID     Node       Node        m         mm
Coeff. Valve
;-------------------------------------------
-----------

;pipes dictating flow into dosing node
1   590          800     10         714     120
CV
2   591          816     5          411     130
CV
3   592          846     5        461      130
CV

; new pipe options
285     1402    14014   600     150      120
19001        701       590      461      714
130
19011       807       806       349       92
120
19014       1011      1012      1515      242
120
19018       1013      14014     7292      190
120
19021       1004      603       1273      186
135
19029       1101      607       157       133
130
19031       1103      609       159       141
130
19034       610       1105      6850      144
120
19041       1117      1118      1528      461
130
19068       1422      1421      4834      92
120
19074       625       832       632       144
120
19077       1407      1408      5325      141
130
19082       1408      1114      2648      66
120
19083       1114      836       525       141
130
19093       1110      1111      3136      186
135
19096       1111      1112      1098      141
130
19097       630      839     513     141      130
19103      1418      843       911       311
125
19108      637       1700       184          461
130
19124      1803      640       187       141
130
19125      1802      852       17860     141
130
19127      853       642       14229     141
130
19128      642       854       4051      141
130
19131      1708      646       516       231
135
19132      647       856       546       231
135
19141      1904      650       65        133
130
19150      651       2000      55        231
135
19155      856       1900      2483      186
135
19158      1009      1010      4700      231
135
19161       816       867       6520      92
120
19165       1409      652       163       141
130
19168       871       5180       57        511
130
19188       711       1902      13065     231
135
19191       831       624       699       144
120
19199       819       618       631       66
120
19203       837       1300      2695      186
135
19204       841       632       388       92
120
19205       842       1115      1653      144
120
19206       817       614       497       66
120

;existing pipes
9001          701       590       461       714
130
9002          656       1016      7087      511
130
9003          801       1002      4360      186
135
9004          800       702       8788      714
130   CV
9005          656       801       52        231
135
9006          717       600       5102      311
125
9007          601       802       1514      231
135
9008           1010      803       1865      141
130
9009           803       804       2337      92
120
9010        805       657       1013      231
135
9011        807       806       349       92
120
9012        501       807       110       92
120
9013        1010      1011      3298      242
120
9014        1011      1012      1515      242
120
9015        1012      1013      5458      190
120
9016        1014      653       927       190
120
9017        654       1015      1458      242
120
9018        1013      14014     7292      190
120
9019        1002      1003      1959      186
135
9020        1003      1004      2312      186
135
9021        1004      603       1273      186
135
9022        1005      1006      3270      186
135
9023        1006      1007      2438      186
135
9024        1007      1008      3481      186
135
9025        604       1005      2980      186
135
9026        1016      1017      9801      511
135
```

```
9027      1017      605       231       311
125
9028      606       1101      8150      311
125
9029      1101      607       157       133
130
9030      608       1103      2050      130
130
9031      1103      609       159       141
130
9032      809       611       629       133
130
9033      810       1102      998       141
130
9034      610       1105      6850      144
120
9035      1105      1106      3840      120
115
9036      1103      1104      2613      141
130
;93601    811       1008      800       186
135
9037      1104      811       2446      186
135
9038      812       1116      502       511
130
9039      1017      813       651       511
130
9040      1116      1117      8276      511
130
9041      1117      1118      1528      461
130
9042      1118      814       4570      616
130
9043      1118      815       4460      461
130
; 613 changed to 591 for dosing puposes
;9044       591       816       5     411
130  CV
9045      1403      1406      4030      411
130
9046      1403      817       528       66
120
9047      591       818       1043      311
125
9048      1410      1411      2711      361
125
9049      1411      819       865       66
120
9050      619       820       617       144
120
9051      1412      1413      3603      144
120
9052      1413      823       1931      92
120
9053      617       824       418       144
120
9054      824       825       2060      144
120
9055      615       826       716       141
130
9056      826       1404      2065      141
130
9057      1404      1405      1755      141
130
9058      1412      1416      1385      133
130
9059      1416      1415      3319      133
130
9060      1405      1416      7285      92
120
9061      1415      825       3336      92
120
9062      825       827       5585      92
120
9063      621       828       728       92
120
9064      1411      1417      3611      361
125
9065      1417      1418      12832     361
125
9066      1413      1423      5437      92
120
9067      1423      1422      2866      92
120
9068      1422      1421      4834      92
120
9069      1418      829       638       144
120
9070      623       830       607       144
120
9071      830       1421      3074      144
120
9072      1418      1419      3158      144
120
9073      1419      831       1057      144
120
9074      625       832       632       144
120
9075      1419      1420      7278      144
120
9076      1406      714      2370      186
135
9077      1407      1408      5325      141
130
9078      1408      833       2295      141
130
9079      626       834       534       186
135
9080      1500      1501      4836      141
130
9081      1501      1503      12083     141
130
9082      1408      1114      2648      66
120
9083      1114      836       525       141
130
9084      628       837       511       186
135
9085      1300      838       5321      186
135
9086      838       671      5919      141
130
9087      838       1303      4499      141
130
9088      629       810       364       141
130
9090      1101      1108      2787      231
135
9091      1108      1109      7691      231
135
9092      1109      1110      2491      186
135
9093      1110      1111      3136      186
135
9094      1112      1113      5122      141
130
9095      1101      809       4240      141
130
9096      1111      1112      1098      141
130
9097      630       839       514       92
120
9098      1106      1107      8652      141
130
9099      631       840       554       141
130
```

9100	1113	1114	5545	141	130
9101	1113	841	1601	92	120
9102	633	842	551	144	120
9103	1418	843	911	311	125
9104	635	844	964	361	125
9105	844	1424	1217	361	125
9106	1424	1425	4827	141	130
9107	707	636	1093	361	125
9108	637	1700	184	461	130
9109	1700	845	635	296	130
9110	1700	592	520	461	130
9111	1701	1702	2909	411	130
9112	1702	1703	3387	411	130
9113	1703	1704	3667	411	130
9114	1704	1705	2683	411	130
9115	1705	1706	5720	361	125
9116	1706	847	1347	361	125
9117	1800	848	1449	186	135
9118	1800	849	2339	231	135
9119	849	850	1567	186	135
9120	660	708	314	186	135
9121	850	1801	2917	141	130
9122	849	1802	1184	186	135
9123	641	851	432	141	130
9124	1803	640	187	141	130
9125	1802	852	17860	141	130
9126	848	853	91	186	135
9127	853	642	14229	141	130
9128	642	854	4051	141	130
9129	854	644	341	141	130
9131	1708	646	516	231	135
9132	647	856	546	231	135
9133	1705	1707	1338	311	125
9134	1707	1709	3316	311	125
9135	1709	1708	4834	311	125
9136	1900	1901	16838	186	135
9137	647	857	440	231	135
9138	1904	650	65	231	135
9139	2000	858	508	133	130
9140	2000	859	379	141	130
9141	1904	650	65	133	130
9142	2001	673	6806	96	130
9143	1902	860	4635	231	135
9144	1902	861	8328	141	130
9145	861	672	7542	96	130
9146	649	862	403	186	135
9147	2003	863	7886	186	135
9148	2003	2004	5467	141	130
9149	651	2003	5670	231	135
9150	651	2000	55	231	135
9151	1424	1426	3034	361	125
9152	1426	706	2805	361	125
9153	639	864	626	141	130
9154	1901	865	1096	186	135
9155	856	1900	2483	186	135
9156	1406	704	45	411	130
9157	802	805	723	231	135
9158	1009	1010	4700	231	135
9159	1017	1100	5991	141	130
9160	816	1403	3337	411	130
9161	816	867	6520	92	120
9162	674	868	299	144	120
9163	1401	1402	4468	92	120
9164	868	1401	1726	144	120
9165	1409	652	163	141	130
9166	869	812	347	511	130
9167	870	1410	2924	361	125
9168	871	5180	57	511	130
9169	5190	869	80	511	130
9170	658	1009	160	231	135
9171	818	1400	5469	311	125
9172	815	612	782	461	130
9173	814	612	740	616	130
9174	806	872	946	92	120
9175	804	602	457	92	120

```
9176      840      1200      1791      141
130
9177      839      1102      7180      92
120
9178      813      871       374       511
130
9179      864      1503       1427      141
130
9180      845      638       548       296
130
9181      846      712      4756      461
130
9182      862      2005      167       186
135
9183      863      648       463       186
135
9184      858      2001      3709      141
130
9185      859      875       1228      133
130
9186      865      1904      542       186
135
9187      860      1904      419       231
135
9188      711      1902      13065     231
135
9189      852      1803      317       141
130
9190      847      659       445       361
125
9191      831      624       699       144
120
9192      832      1600      5222      144
120
9193      829      622       542       144
120
9194      843      634       90        311
125
9195      828      1414      1140      92
120
9196      827      620       748       92
120
9197      823      616       513       92
120
9198      820      1412       1279      144
120
9199      819      618       631       66
120
9200      833      1409      444       141
130
9201      834      1500      2038      186
135
9202      836      627       424       141
130
9203      837      1300      2695      186
135
9204      841      632       388       92
120
9205      842      1115      1653      144
120
9206      817      614       497       66
120

;Dummy pipes to dummy nodes to allow two
;demand patterns to occur i.e. residential
and
;industrial
210     1010     1080     5        231      120
211     1010     1090     5        231      120
212     1015     1085     5        231      120
213     1015     1095     5        231      120

214     1116     1186     5        231      120
215     1116     1196     5        231      120
216     2003     2083     5        231      120
217     2003     2093     5        231      120

220     1012     1082     5        231      120
221     1012     1092     5        231      120
222     1400     1480     5        231      120
223     1400     1490     5        231      120
224     1302     1382     5        231      120
225     1302     1392     5        231      120
226     1705     1785     5        231      120
227     1705     1795     5        231      120
228     641      2418     5        231      120
229     641      2419     5        231      120
230     645      2458     5        231      120
231     645      2459     5        231      120
232     1901     1981     5        231      120
233     1901     1991     5        231      120
234     1903     1983     5        231      120
235     1903     1993     5        231      120
236     1904     1984     5        231      120
237     1904     1994     5        231      120
238     2002     2082     5        231      120
239     2002     2092     5        231      120
240     2005     2085     5        231      120
241     2005     2095     5        231      120
242     1414     1484     5        231      120
243     1414     1494     5        231      120
244     1600     1680     5        231      120
245     1600     1690     5        231      120


251      501      5010      5       200      120
252      502      5020      5       200      120
253      503      5030      5       200      120

;Maitland
254      504      5050      5       500      120
2541     504     5040     5        500      120

256      506      5060      5       1000
120
257      507      5070      5       1000
120
258      508      5080      5       1000
120
259      509      5090      5       1000
120
260      510      5100      5       1000
120
261      511      5100      5       1000
120
2601     510      5110      5       1000
120
2611     511      5110      5       1000
120
262      512      5120      5       1000
120

;Mt Rat
263      513      5140      5       1000
120
2630      513     5130      5       1000
120

265      5150      515      5       1000
120   CV
266      516      5160      5       1000
120
267      517      5170      5       1000
120
268      518      5180      5       1000
120
```

```
;269      407      5180     5        300
120
;270      520      5180     5        300
120
2680     518      5190     5        300      120
;2690     407      5190     5        300
120
;2700     520      5190     5        300
120
271      521      5210     5        200      120
272      522      5220     5        200      120
273      523      5230     5        200      120
274      524      5240     5        200      120
275      525      5250     5        200      120
276      526      5260     5        200      120
277      527      5270     5        200      120
278      528      5280     5        200      120
279      529      5290     5        200      120
; Min pump option
280      709      1800     5        200      120
281      857      710      5        200      120
282      713      1701     5        400      120
;ard
283      656      716      5        300      120
;ptvic
284      715      1407     5        300      120
;from petersville to Ardrossan
286      14014    1014     3200     190      120


;dummy pipes to allow for TCV at new tanks
for GA
;Minlaton
;370     5150     407      5        300      120
         CV
;Pt Victoria
371      401      4010     5        300      120
;Wool Bay
372      402      4020     5        300      120
;Coobowie
373      403      4030     5        300      120
;Petersville
374      404      4040     5        300      120
;
;375     405      4050     5        300      120
;376     406      4060     5        300      120
;377     407      4070     5        300      120
;378     408      4080     5        300      120
;379     409      4090     5        300      120

;pipes to connect new tank TCV node with
final destination
;Minlaton
;380
;Pt Victoria
381      671      1302     5        150      120
;Wool Bay
382      672      1903     5        150      120
;Coobowie
383      673      2002     5        150      120

;Connects to tank
;Petersville
384      867      664      5        150      120



;New check valves added in to cater for new
tanks
;Minlaton
;385     407      660      20       300      120
         CV
;Pt Victoria
386      401      1302     20       100      120
         CV
;Wool Bay
387      402      1903     20       100      120
         CV
;Coobowie
388      403      2002     20       100      120
         CV
;Petersville
389      404      674      1        300      120
         CV


402      500      701      1        714      120
CV
412      5040     591      1        411      120
CV
414      503      615      1        141      120
CV
416      507      617      1        144      120
CV
418      506      619      1        144      120
CV
420      508      621      1        144      120
CV
422      509      623      1        144      120
CV
424      5100     625      1        144      120
CV
425      512      626      1        186      120
CV
427      523      628      1        186      120
CV
428      502      629      1        141      120
CV
430      521      631      1        141      120
CV
432      522      633      1        144      120
CV
434      517      635      1        361      120
CV
435      706      707      1        361      120
CV
437      5130     637      1        461      120
CV
440      524      639      1        141      120
CV
442      528      641      1        141      120
CV
445      525      645      1        141      120
CV

447      515      660      1        186      120
CV

449      516      647      1        231      120
CV
451      527      649      5        186      120
CV
453      526      651      1        231      120
CV
457      529      654      1        141      120
CV
458      705      870      1        361      120
CV
459      704      705      1        361      120
CV

461      703      656      1        714      120
CV

;check valve that bypasses potential pump
option at Minlaton
```

```
463     708     709     1       200     120
        CV

465     876     660     1       231     120
        CV

;check valve that bypasses booster station
at Utopia on to Yorketown

466     710     711     1       231     120
        CV

;check valve bypassing the fill up 515 (Min
Tank) pump

467     712     713     1       400     120
        CV

;check valve for Pt Victoria Booster

468     714     715     1       300     120
        CV

;check valve bypassing the Ardrossan
Booster

469     716     717     1       300     120
        CV

; valve which places Weetulta Tank off line

470     611     629     1       300     120
        CV

;4000    611     629     10      141     120


[PUMPS]
;------------------------------------------
--------------------------
;       Start   End     h1      q1
;ID     Node    Node    m       l/s
;------------------------------------------
--------------------------


;Upper Paskeville Pumping Station
 300    500     701     65      50

;Kaiton Corner Pumping Station
 302    702     703     65      50

;Maitland Pumping Station
 304    704     705     65      50

;Mt Rat Pumping Station
 306    706     707     65      50

;Optional Pumping Station at Minlaton Tank
;- May help during peak periods -
; supply Pt Vincent and Stansbury

 308    708     709     65      50

;Optional Pumping Station (Booster Variety)
;to supply Yorkewtown etc)

 310    710     711     65      50

;Optional Booster Station to fill tank 515
at Minlaton

 312    712     713     65      50

;Optional Booster Station to Port Victoria
;Supplying South Kilkerran Tank

 314    714     715     65      50

;Optional Booster Station to supply
Ardrossan

 316    716     717     65      50

[VALVES]
;------------------------------------------
------
;ID     start   end     diam    type
setting
;------------------------------------------
------

;valves added to accompany NEWTANKS

;Pt Victoria
391     671     4010    150     TCV     5
;Wool Bay
392     672     4020    150     TCV     5
;Coobowie
393     673     4030    150     TCV     5
;Petersville
394     664     4040    150     TCV     5

403     600     601     311     PRV     60
404     602     5010    92      TCV     5
405     603     604     186     PRV     40
406     605     606     311     PRV     30
407     607     608     133     PRV     70
408     609     610     141     PRV     45

409     611     5020    133     TCV     5

410     612     5050    461     TCV     5
411     612     5050    461     TCV     5

413     614     5030    66      TCV     5
415     616     5070    144     TCV     5
417     618     5060    144     TCV     5
419     620     5080    144     TCV     5
421     622     5090    92      TCV     5
423     624     5110    144     TCV     5
426     627     5230    141     TCV     5
429     630     5210    92      TCV     5
431     632     5220    92      TCV     5
433     634     5170    311     TCV     5
436     636     5140    361     TCV     5
439     638     5240    141     TCV     5
441     640     5280    141     TCV     5
443     642     643     141     PRV     50
444     644     5250    141     TCV     5
446     659     5150    231     TCV     5
448     646     5160    231     TCV     5
450     648     5270    231     TCV     5
452     650     5260    231     TCV     5
454     652     5120    141     TCV     5
456     653     5290    141     TCV     5
462     657     658     231     PRV     30
464     659     876     231     TCV     5

460     702     703     714     TCV     5


[CONTROLS]
;------------------------------------------
LINK 300 OPEN IF NODE 518 BELOW 6.32
LINK 300 CLOSED IF NODE 518 ABOVE 9.62
```

```
;LINK 301 OPEN IF NODE 518 BELOW 6.32
;LINK 301 CLOSED IF NODE 518 ABOVE 9.62

LINK 302 OPEN IF NODE 518 BELOW 6.32
LINK 302 CLOSED IF NODE 518 ABOVE 9.62

;LINK 303 OPEN IF NODE 518 BELOW 6.32
;LINK 303 CLOSED IF NODE 518 ABOVE 9.62

LINK 304 OPEN IF NODE 517 BELOW 6.63
LINK 304 CLOSED IF NODE 517 ABOVE 7.43


;LINK 305 OPEN IF NODE 517 BELOW 6.63
;LINK 305 CLOSED IF NODE 517 ABOVE 7.43

LINK 306 OPEN IF NODE 513 BELOW 5.04
LINK 306 CLOSED IF NODE 513 ABOVE 5.44

LINK 308 OPEN IF NODE 528 BELOW      5.04
LINK 308 CLOSED IF NODE 528 ABOVE 7.24

LINK 310 OPEN IF NODE 526 BELOW 6.08
LINK 310 CLOSED IF NODE 526 ABOVE 7.28

LINK 312 OPEN IF NODE 515 BELOW 3.92
LINK 312 CLOSED IF NODE 515 ABOVE 5.22

LINK 316 OPEN IF NODE 529 BELOW 2.32
LINK 316 CLOSED IF NODE 529 ABOVE 5.42

LINK 314 OPEN IF NODE 523 BELOW 4.7
LINK 314 CLOSED IF NODE 523 BELOW 5.2

;controls for new tanks

LINK  391  OPEN IF NODE 401 BELOW 2.5
LINK  391  CLOSED IF NODE 401 ABOVE 5.0

LINK  392  OPEN IF NODE 402 BELOW 2.5
LINK  392  CLOSED IF NODE 402 ABOVE 5.0

LINK  393  OPEN IF NODE 403 BELOW 2.5
LINK  393  CLOSED IF NODE 403 ABOVE 5.0

LINK  394  OPEN IF NODE 404 BELOW 2.5
LINK  394  CLOSED IF NODE 404 ABOVE 5.0

LINK  404  OPEN IF NODE 501 BELOW 1.36
LINK  404  CLOSED IF NODE 501 ABOVE 4.46

LINK  410  OPEN IF NODE 504 BELOW 4.3
LINK  410  CLOSED IF NODE 504 ABOVE 6.6

LINK  411  OPEN IF NODE 504 BELOW 4.3
LINK  411  CLOSED IF NODE 504 ABOVE 6.6

LINK  413  OPEN IF NODE 503 BELOW 3.1
LINK  413  CLOSED IF NODE 503 ABOVE 4.9

LINK  415  OPEN IF NODE 507 BELOW 3.25
LINK  415  CLOSED IF NODE 507 ABOVE 4.95

LINK  417  OPEN IF NODE 506 BELOW 4.15
LINK  417  CLOSED IF NODE 506 ABOVE 4.95

LINK  419  OPEN IF NODE 508 BELOW 2.06
LINK  419  CLOSED IF NODE 508 ABOVE 4.46

LINK  421  OPEN IF NODE 509 BELOW 2.15
LINK  421  CLOSED IF NODE 509 ABOVE 4.95

LINK  423  OPEN IF NODE 511 BELOW 2.1
LINK  423  CLOSED IF NODE 511 ABOVE 4.5

LINK  426  OPEN IF NODE 523 BELOW 2.7
LINK  426  CLOSED IF NODE 523 ABOVE 5.2

LINK  429  OPEN IF NODE 521 BELOW 5.0
LINK  429  CLOSED IF NODE 521 ABOVE 6.6

LINK  431  OPEN IF NODE 522 BELOW 3.0
LINK  431  CLOSED IF NODE 522 ABOVE 5.4

LINK  433  OPEN IF NODE 517 BELOW 6.63
LINK  433  CLOSED IF NODE 517 ABOVE 7.43

LINK  436  OPEN IF NODE 513 BELOW 5.04
LINK  436  CLOSED IF NODE 513 ABOVE 5.44

LINK  439  OPEN IF NODE 524 BELOW 3.95
LINK  439  CLOSED IF NODE 524 ABOVE 4.95

LINK  441  OPEN IF NODE 528 BELOW 6.64
LINK  441  CLOSED IF NODE 528 ABOVE 7.24

LINK  444  OPEN IF NODE 525 BELOW 3.88
LINK  444  CLOSED IF NODE 525 ABOVE 6.28

LINK  446  OPEN IF NODE 515 BELOW 3.92
LINK  446  CLOSED IF NODE 515 ABOVE 5.22

LINK  448  OPEN IF NODE 516 BELOW 2.92
LINK  448  CLOSED IF NODE 516 ABOVE 5.42

LINK  450  OPEN IF NODE 527 BELOW 6.28
LINK  450  CLOSED IF NODE 527 ABOVE 7.28

LINK  452  OPEN IF NODE 526 BELOW 6.08
LINK  452  CLOSED IF NODE 526 ABOVE 7.28

LINK  454  OPEN IF NODE 512 BELOW 2.82
LINK  454  CLOSED IF NODE 512 ABOVE 5.42

LINK  456  OPEN IF NODE 529 BELOW 2.32
LINK  456  CLOSED IF NODE 529 ABOVE 5.42

LINK  460  OPEN IF NODE 518 ABOVE 9.62
LINK  460  CLOSED IF NODE 518 BELOW 6.32

LINK  464  OPEN   IF NODE 515 BELOW 2.92
LINK  464  CLOSED IF NODE 515 ABOVE 5.22

;LINK  467  OPEN   IF NODE 515 ABOVE 5.22
;LINK  467  CLOSED   IF NODE 515 BELOW 2.92


[PATTERNS]
;---------------------------------------
;1 2.08 2.08
;1 0.4 0.24 0.17 0.18 0.22 0.57 0.85 1.07
1.25 1.3 1.15 1
;1 0.95 1.18 1.21 1.27 1.45 1.7 1.97 2.1
1.55 0.97 0.72 0.54

;----------------------------------------
;Industrial Areas ie. farms and industries
;----------------------------------------


;Peak Slow period (*0.5), during wet period
with less industrial
;need for water to irrigate etc., Peak
Winter period
1 0.32 0.49    0.625   0.685
1 0.73 0.75     0.75   0.75     0.75    0.75
       0.75    0.74
```

```
1 0.72  0.675   0.61    0.52    0.43    0.32
0.215   0.115   0.09    0.055   0.055   0.11


;Slow Period (trough) (*0.7), Autumn
2 0.448 0.686   0.875   0.959
2 1.015 1.05    1.05    1.05    1.05    1.05
        1.05    1.036
2 1.008 0.945   0.854   0.728   0.602   0.448
        0.301   0.161   0.126 0.077     0.077
        0.154

;Average Industrial, Spring
3 0.64  0.98    1.25    1.37
3 1.45  1.5     1.5     1.5     1.5     1.5
        1.5     1.48
3 1.44  1.35    1.22    1.04    0.86    0.64
0.43    0.23    0.18  0.11      0.11    0.22


;Peak Week (*2.5), Summer
4 1.60  2.45    3.13    3.43
4 3.63  3.75    3.75    3.75    3.75    3.75
        3.75    3.70
4 3.60  3.38    3.05    2.60    2.15    1.60
1.08    0.58    0.45    0.28    0.28    0.55


;Peak Day (*2.7)
5 1.728 2.646   3.375   3.699
5 3.915 4.05    4.05    4.05    4.05    4.05
        4.05    3.996
5 3.888 3.645   3.294   2.808   2.322   1.728
        1.161   0.621   0.486 0.297     0.297
        0.594

;last one put in to provide peak hydraulic
penalty - only for 24hr run

;------------------------------------------
------
;Residential Water Supply - Hot weather
high peak
;------------------------------------------
------

;Average Day
;10 0.4 0.24    0.17    0.18    0.22    0.57
        0.85    1.07
;10 1.25        1.3     1.15    1       0.95
        1.18    1.21    1.27
;10 1.45        1.7     1.97    2.1     1.55
        0.97    0.72    0.54

;Peak Day (*2.7)
;10 1.08        0.65    0.46    0.49    0.59
        1.54    2.30    2.89
;10 3.38        3.51    3.11    2.70    2.57
        3.19    3.27    3.43
;10 3.92        4.59    5.32    5.67    4.19
        2.62    1.94    1.46

;------------------------------------------
-----
;Residential Water Supply - Hot weather low
peak
;------------------------------------------
-----

;Average Day Spring
8 0.45  0.53    0.74    1.06
8 1.26  1.3     1.25    1.2     1.17    1.16
        1.18    1.21
8 1.3   1.48    1.61    1.66    1.46    1.04
0.7     0.52    0.45    0.43    0.41    0.42


;Low Day (trough) (*0.7)
;7 0.315 0.301 0.287    0.294   0.315   0.371
        0.518   0.742
;7 0.882 0.91    0.875 0.84     0.819   0.812
        0.826   0.847
;7 0.91 1.036    1.127   1.162   1.022   0.728
        0.49    0.364

;Peak Day (*2.7)
;9 1.215 1.161 1.107    1.134   1.215   1.431
        1.998   2.862
;9 3.402 3.51    3.375 3.24     3.159   3.132
        3.186   3.267
;9 3.51 3.996    4.347   4.482   3.942   2.808
        1.89    1.404

;------------------------------------------
--------
;Residential Water Supply - Hot weather
medium peak
;------------------------------------------
--------

;Average Day
;8 0.33 0.28    0.3     0.31    0.34    0.45
        0.78    1.24
;8 1.44 1.4     1.39    1.34    1.28    1.21
        1.14    1.17
;8 1.28 1.49    1.74    1.87    1.26    0.82
0.62    0.47

;Low Day (trough) (*0.7)
;7 0.231 0.196 0.21     0.217   0.238   0.315
        0.546   0.868
;7 1.008 0.98    0.973 0.938    0.896   0.847
        0.798   0.819
;7 0.896 1.043 1.218    1.309   0.882   0.574
        0.434   0.329

;put in 6 (should be 9) for testing puposes
;Peak Week (*2.5)
;6 0.83 0.7     0.75    0.775   0.85    1.125
        1.95    3.1
;6 3.6  3.5     3.48    3.35    3.2     3.025
        2.85    2.925
;6 3.2  3.725   4.35    4.675   3.15    2.05
1.55    1.175

;month flow 2.1 * average
9 0.70 0.95 1.68 2.60
9 3.00 2.98 2.95 2.83 2.70 2.52 2.40 2.45
9 2.70 3.12 3.65 3.95 2.66 1.71 1.39 1.00
0.67 0.60 0.63 0.65



;Peak Day (*2.7)
;9 0.891 0.756 0.81     0.837   0.918   1.215
        2.106   3.348
;9 3.888 3.78    3.753 3.618    3.456   3.267
        3.078   3.159
;9 3.456 4.023 4.698    5.049   3.402   2.214
        1.674   1.269

;last one put in to provide hydraulic peak
only good for 24hr run

;--------------------------------------
;Residential Water Supply - Winters Day
;--------------------------------------
```

```
;Curve without winter factor - probably
more Autumn/Spring Day
7 0.07  0.13    1.48    1.63
7 1.61  1.56    1.47    1.36    1.24    1.12
        1.06    1.09
7 1.16  1.26    1.3     1.25    1.15    1.02
0.88    0.73    0.6     0.4     0.24    0.11


;Probably More typical winter Day (*0.7)
;6 0.42 0.28    0.168   0.077   0.049   0.091
        1.036   1.14
;6 1.13   1.092         1.029   0.952   0.868
        0.784   0.742   0.76
;6 0.81 0.88    0.91    0.875   0.805   0.714
        0.616   0.511

;Low Period (*0.5)
6 0.04  0.07    0.74    0.82
6 0.81  0.78    0.74    0.68    0.62    0.56
        0.53    0.55
6 0.58  0.63    0.65    0.63    0.58    0.51
0.44    0.37    0.30    0.20    0.12    0.06


;Very low Period (*0.3) wet and cool with
little water demand
;for irrigation and domestic use down
;6 0.18 0.12    0.07    0.03    0.02    0.04
        0.44    0.49
;6 0.48 0.47    0.44    0.41    0.37    0.34
        0.32    0.33
;6 0.35 0.38    0.39    0.38    0.35    0.31
0.26    0.22


; pattern for dosing
11 1

; pattern for determination of supply
hydraulics at end of simulation

12      3.53    4.39    4.50    4.72
12      5.39    6.32    7.33    7.81    5.77
        3.61    2.68    2.01
12      1.49    0.89    0.63    0.67    0.82
        2.12    3.16    3.98    4.65    4.84
4.28    3.72

;pattern for determination of pressure
ability at end of simulation

13      7.5


[REACTIONS]
;-------------------------------------------
---
; altering decay of chloroamine from
0.048/day to 0.1/day
; no real justification, just playing
around

 GLOBAL BULK  -0.1          ; Bulk decay
coeff.
 GLOBAL WALL  -0.0         ; Wall decay
coeff.
 TANK   500     0.0

[REPORT]
;-------------------------------------------
----
LINKS NONE
NODES NONE

[TIMES]
;-------------------------------------------
-----
 DURATION  2000       ; 12 hour simulation
period
 REPORT TIMESTEP 500    ;
 REPORT START  0;
 HYDRAULIC TIMESTEP  1 ;
 QUALITY TIMESTEP    1;
 PATTERN TIMESTEP     1;
 MINIMUM TRAVELTIME    1;

[OPTIONS]
;-------------------------------------------
----
 UNITS     SI
 ACCURACY 0.002
 QUALITY   Chloramine  ; Chlorine analysis
 MAP       yorke.map  ; Map coordinates file

[END]
```

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Corrigenda

When describing the solutions in this thesis, replace "best" with good, "optimal" with efficient and "complete" with thorough.

Equation 2.3 bottom of Page 6, (2.4) should be (2.3)

Page 58, Paragraph 2, Line 5, Word 5, 'wass' should read 'was'

Page 70, Paragraph 3, Line 5, Word 5 , 'This Study' should read 'Tryby and Uber, 1999'

Page 70, Paragraph 4, Line 1, Word 1 , 'This Study' should read 'Tryby and Uber, 1999'

Page 70, Paragraph 4, Line 2, Word 4 , 'This Study' should read 'Boccelli *et al.*, 1998'

Page 74, Paragraph 4, Line 1, Word 5, 'produced' should read 'first published'

Page 76, Paragraph 3, Line 1, Word 10, 'passed' should read 'exceeded'

Page 80, Paragraph 3, Line 2, Word 15, 'four' should read 'for'

Page 192, Paragraph 2, Line 2, Word 5, '(EWS, 1986(b)),' should read (EWS, 1986(b))'

Page 234, Figure 9.19, 'Averrage' should read 'Average'

Page 354, Between References 59 and 60, A new reference should be inserted:

Gleeson C., and Gray N. (1997) *The Coliform Index and Waterborne Disease: Problems of Microbial Drinking Water Assessment.* E. & F.N. Spon, London

The issue of the optimal timing of replacement or rehabilitation of facilities is outside the scope of this thesis and is covered in detail by Engelhardt (1999).

The 'willingness to pay' curve (Chapter 5, Page 113, Figure 5.11) used in this thesis is assumed as linear although it is stated in page 113 and in Chapter 10, Page 253, Paragraph 2 that greater analysis into the shape of this curve would increase the model's accuracy. As the 'willingness to pay' curve depends on network size, location and socio-economic factors, the linear approach was used as a more general approach.

The method of costing illness and death due to inadequate water quality is a challenging issue. The philosophy of placing a value on human life could be objected to 'in principle' by some researchers on moral grounds. This 'social cost' analysis described in detail in Chapter 5, was used as an alternative mechanism to the more rigid 'penalty functions' used for the hydraulic constraints. The penalty function method was applied to water quality as well as the social cost analysis, however, the social cost is more effective as discussed in Chapter 10.

The quality model used could disadvantage smaller communities, as they have a low total cost due to their low population. Although funding tends to concentrate on water quality supplied to larger populations, this issue could be addressed in future work by specifying a minimum quality standard for every consumer in the system.

The seasonal level of chlorine residual determining the quality costs was taken by using the lowest level in the daily cycle after the model had converged to a cyclic pattern. Representative graphs can be found in Chapter 7 and 9. This process was used for each of the four seasons to give a total annual expected cost for the water quality. The low-level residual is the worst case scenario, as it means that a greater level of chlorine has been consumed (hence higher formation of THM's) and a higher risk of waterborne disease is present.

One of the main results stemming from this thesis is that residence time is a major factor in the optimisation for water quality and hydraulics. The application of a penalty function to residence time was tested in Chapter 7 and showed results similar to the other optimisation techniques (social cost, quality penalty). This analysis was mainly for comparison and discussion purposes and further work into this field is a major aspect that could be taken up in other research following on from this work.