University of Southern Queensland

Faculty of Engineering and Surveying

Optimisation of Gold Coast City's Chlorine Dosing System for the Southern Region Distribution Network

A dissertation submitted by

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GoldCoastWater

EXECUTIVE SUMMARY

Gold Coast Water (GCW) have identified that the existing chlorine disinfection system is not effectively servicing the extremities of the city's water distribution network. This study was required by GCW in a bid to resolve the issue.

The purpose of this study was to provide a recommendation on a near optimal disinfection solution for Gold Coast City's Southern Regional potable water network based on the hydraulic behaviour of the system. Disinfection options were designed according to GCW's internal product specifications outlined in the Desired Standards of Service (DSS) water quality criteria.

Literature of the subject matter were extensively reviewed to determine how the two most widely used disinfectants, chlorine and chloramine, function in a distribution network as well as to determine the best approach to water quality network analysis and system design. Technical studies for four common phases of chlorine network modelling were discussed in detail, these include the distribution of chlorine through a network, factors affecting chlorine decay, optimal selection of chlorine booster stations, and chlorine scheduling.

"H_2ONET Analyser", developed by MWH Soft, Inc. (1996-2003), was the chosen hydraulic modelling software for this study. GCW's Hydraulic Network Model was last updated by Cardno MBK in October 2006. However, the model required further revision to incorporate more recent changes including the Pressure and Leakage Management Project (PLMP).

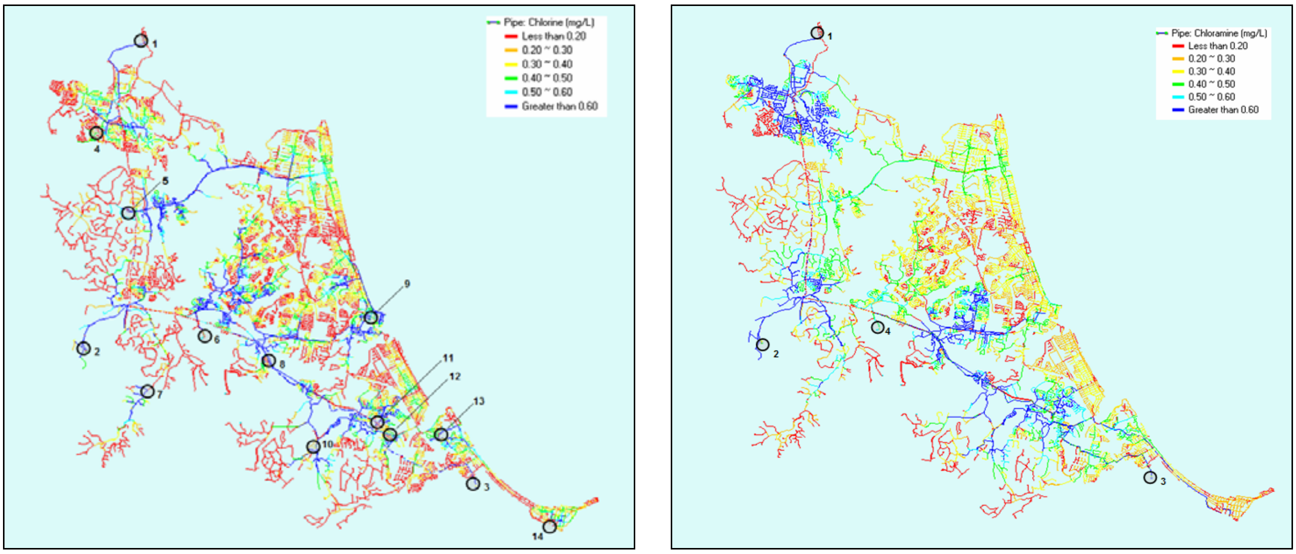
In attempt to calibrate the model, field data was extracted from pressure loggers and online PLMP monitors in PRV chambers. The outcome of the calibration indicated that model pipe friction coefficients did not require changing. The model was surprisingly accurate in simulating real time pressure and exhibited an average model error well below $\pm$10% over all field locations. This was verified with an alternative data set that also demonstrated an error well below $\pm$10%. A sensitivity analysis undertaken on the Global Demand Factor (GDF) revealed that it was relatively insensitive to small changes but did produce a better field curve when a GDF of 1 was adopted. An Average Day demand was adopted for the model design runs.

The Tugun Desalinisation Plant, pipeline and off takes were incorporated into the model for all design runs. Desalinisation will be activated late 2008 and will supply south of and including

Burleigh and Reedy Creek Water Supply Districts (WSD). The model therefore needed to include Desalinisation to represent the future operating regime of the network.

Initial network analysis was undertaken using the future operating conditions to compare the distribution of chlorine and chloramine through the distribution system. It was found that chloramine survived much longer where a residual ≥ 0.2 mg/L could be achieved in 73.1% of the study area during a 2007 AD demand scenario. This equated to almost a 500% increase over chlorine distribution.

Design runs for chlorine and chloramine were undertaken for the study after analysing the system flows and determining the best locations for potential disinfectant booster stations. Boosters were located at storage reservoirs, where possible, to utilise their mixing capabilities. The design runs included continued disinfectant dosing at Molendinar and Mudgeeraba water treatment facilities and the Tugun Desalinisation Plant. It was found that chlorine required an additional 11 re-chlorination stations while chloramine required only a single booster station. Figure 1(a) and (b) illustrate the movement of chlorine and chloramine respectively through the study area and the dosing locations required for more than 80% of the network (≥ 200 mm) to meet GCW's DSS.



(a) (b)

Figure 1: Chlorine and Chloramine distribution and required disinfectant source nodes

A summary of pipe residuals that satisfy the DSS minimum chlorine criteria for Options 1 and 2 are provided in Table 1.

Table 1: Summary of network residual for Options 1 and 2

Option	Disinfectant	Boosters Required	Main Size	Pipes Satisfying Disinfectant Residual Design Criteria (%)		
				≥ 0.2 mg/L	≥ 0.15 mg/L	≥ 0.1 mg/L
1	Chlorine	11	≥ 200 mm	81.0	83.8	86.6
			all	62.5	70.4	78.2
2	Chloramine	1	≥ 200 mm	83.3	92.7	98.1
			all	79.6	86.6	95.6

The total estimated cost for each option for the first and second year of operation is provided below in Table 2.

Table 2: Summary of estimated costs for Options 1 and 2

Option	Initial Capital Cost	Annual Chemical Cost	Ongoing Annual Maintenance Cost ($5,000/site)	Total Cost for 1st Year	Total Cost for 2nd Year
1	$1,073,050.00	$369,966.48	$55,000.00	$1,498,016.48	$424,966.48
2	$102,850.00	$421,857.06	$5,000.00	$529,707.06	$426,857.06

The results of the project present sufficient evidence that conventional chlorine dosing does not provide effective disinfection for Gold Coast City's Southern Regional water distribution network. Two solutions have been provided in this report that mitigate the existing disinfection deficiency to an appropriate level. The number of booster sources required for Option 1 is not practical from an operational and maintenance perspective even though ongoing costs were slightly lower. The initial capital costs for Option 2 are also much more practical. Therefore it is recommended that Option 2 be further investigated for future disinfection of the Southern Region potable water network. Given the future South East Queensland Potable water operating strategy (transfer of water to and from Brisbane/Logan) and that Brisbane Water disinfect with chloramine, Option 2 appears to be a valid proposal.

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I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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Signature

Date

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NOMENCLATURE

AD	Average Day
ADWG	Australian Drinking Water Guidelines
CDC	Chlorine Distribution Curve
DBP	Disinfection By-products
DPD	Diethyl-p-phenylene Diamine
DSS	Desired Standards of Service
DWDS	Drinking Water Distribution System
ET	Equivalent Tenement
FC	Financial Catchments
FOS	Factor of Safety
GA	Genetic Algorithm
GAC	Granular Activated Carbon
GCCC	Gold Coast City Council
GCDA	Gold Coast Desalinisation Alliance
GCW	Gold Coast Water
GDF	Global Demand Factor
HAA	Halo-acetic Acid
HACCP	Hazard Analysis Critical Control Point
HLZ	High Level Zone
HW	Hazen-Williams
IDM	Infrastructure Demand Model
ILZ	Intermediate Level Zone
LLZ	Low Level Zone
MD	Maximum Day
MDMM	Mean Day Maximum Month
MFR	Multi-Family Residential
MH	Maximum Hour

NOM	Natural Organic Matter
PF	Peaking Factor
PIP	Priority Infrastructure Plan
PLMP	Pressure Leakage Management Project
PRV	Pressure Reducing Valve
PSD	Planning Scheme Density
RDS	Regional Drought Strategy
SEQ	Southeast Queensland
SFR	Single-Family Residential
SRWPA	Southern Region Water Pipeline Alliance
THM	Trihalomethane
TOC	Total Organic Carbon
WHO	World Health Organisation
WPP	Water Purification Plant
WSD	Water Supply District
WSZ	Water Supply Zone
WTW	Water Treatment Works

CHAPTER 1. INTRODUCTION

1.1 PROJECT PURPOSE

The aim of this project is to deliver a near optimal solution for Gold Coast Water's (GCW) disinfection system based on the hydraulic behaviour of the network. The study will specifically seek to mitigate disinfection deficiencies within all areas of the southern region potable water network. A calibrated hydraulic network model of the study area, together with field testing, will be deployed to identify areas with poor chlorine residual. The model will be used to simulate alternative disinfection treatment options and/or pipeline reconfigurations to resolve existing network deficiencies. All required works included in the project recommendations will be designed according to GCW's internal product specifications.

1.2 OBJECTIVES

The project will integrate the tasks outlined in the project specification displayed in Appendix A. These tasks present a base for the project and will facilitate to achieve an optimised disinfection system for the southern region distribution system as desired by GCW. The objectives of the project are to:

- Discuss the appropriate literature related to network disinfection and optimisation
- Calibrate and validate GCW's hydraulic network model
- Undertake hydraulic and water quality network analysis of the study area
- Present near optimal disinfection solutions for the study area and accompany each solution with an economic analysis

1.3 INTRODUCTION

GCW is a directorate of Gold Coast City Council (GCCC) and is responsible for the provision of potable and recycled water products and wastewater services for customers of Gold Coast City. GCW's vision is to create a sustainable Waterfuture where water is valued for its contribution to the Gold Coast lifestyle (*Gold Coast Water_1, 2006*). Among others, GCW will deliver the following to achieve this vision (*Gold Coast Water_2, 2006*):

- Deliver high quality drinking water products and services
- Ensure the safety of the community by adhering to the Gold Coast City Council Occupational Health and Safety Policy and guidelines
- Satisfy the requirements of the following management systems Standards
  - AS/NZS ISO 9001 (Quality)
  - AS/NZS ISO 14001 (Environment)
  - AS/NZS 4801 (Occupational Health and Safety)
  - Codex Alimentarius Alinorm 97/13A (HACCP)

In providing safe drinking water, GCW undertakes chlorination as the method for disinfection of the drinking water distribution system (DWDS). Chlorine dosing takes place at the water purification plant (WPP) and is the last step in the treatment process. Like most water authorities, GCW attempts to maintain residual chlorine within all reaches of the network to minimise the potential for harmful waterborne pathogens and microbial contamination. However, recent field measurements undertaken for the Hazard Analysis Critical Control Point (HACCP) Plan have discovered that many peripheral parts of the network have deficient or no detectable levels of residual chlorine. This has created incentive for the investigation and analysis of GCW's disinfection system.

The Australian Drinking Water Guidelines (ADWG) and GCW's Desired Standards of Service (DSS) are the legislative frameworks that benchmark drinking water quality for the Gold Coast, including the requirements for chlorine disinfection. In Queensland, water authorities largely decide the quality of their drinking water, as there is no government body providing enforceable and accountable water quality standards. However, it is considered properly diligent and legally prudent for an organisation to comply with the 2004 World Health Organisation (WHO) based ADWGs. In practice, authorities generally attempt to provide a quality product that exceeds these guidelines. GCW utilises the guidelines as an external product specification while the DSS adopted in the HACCP Plan provide much more stringent internal specifications, which allow corrective action to be taken before external specifications are breached (*Gold Coast Water_2, 2006*).

1.4 THE WATER QUALITY ISSUE

It has been recognised by GCW that large areas within the distribution system are lacking residual chlorine (*Gold Coast Water_2, 2006*). This has been made apparent in recent review of the HACCP Plan. Theoretically, the amount of chlorine dosed should provide disinfection residual (>0.2 mg/L) throughout the Gold Coast network. However, due to the lasting drought in Southeast Queensland (SEQ) and the adoption of water restrictions through the Regional Drought Strategy (RDS) and plan, the city's water consumption has reduced and caused water to reside in the network longer. When the detention time of water in the network is prolonged, free chlorine has additional time to react with the bulk water and biofilm layers in pipes, consequently reducing chlorine concentration *(Gray, 1999)*. This is in addition to the natural reduction due to chlorine decay. It is assumed that the increase in water retention time could be one of the major contributing factors causing chlorine concentrations to fall below GCW's DSS.

Further, the implementation of the Pressure Leakage Management Project (PLMP) may also be a less obvious but a contributing factor to the recent decline in chlorine residual. The PLMP was adopted by GCW in mid 2004 to reduce the loss of water through leakage resulting from high network pressure. Lowering the pressure through the use of pressure reducing valves (PRV) subsequently reduces the flow by lowering leakage and water consumption. Like water restrictions, this also increases the retention time of water from treatment plant to tape. Approximately 70% of Gold Coast's distribution system will be influenced by the PLMP. The effects this ultimately has on chlorine concentrations is thought to be specific to the pressure-reduced zones and therefore relatively localised.

Indirectly, the RDS and the PLMP are the two factors thought to be most influential in reducing chlorine concentrations to levels below GCW's target limits. They affect the system in such a way that they enhance the more direct effects on chlorine reduction such as chemical reactions and physical breakdown.

1.5 HYDRAULIC NETWORK MODEL

A water network model comprises of a hydraulic model and an integrated Infrastructure Demand Model (IDM). The IDM predicts growth for planning purposes, this allows infrastructure to be sized to service future demand (consumption) without the requirement for continuous expensive upgrades. The hydraulic model simulates the movement of water through storage reservoirs and the reticulation system according to demand. Hence, planning horizons based on predicted demands can be incorporated into the model. This allows the staging of required infrastructure and an even distribution of capital costs over the scheduled time frame.

GCW utilises an IDM developed for Gold Coast City to effectively plan for water and wastewater infrastructure. GCW's hydraulic network model incorporates demands from the IDM creating planning horizons for 2006, 2011, 2016, 2021 and 2056. These planning horizons are further broken down into three scenarios by applying peaking factors (PF) to represent flows in different seasons of the year, these include Maximum Day (MD), Mean Day Maximum Month (MDMM) and Average Day (AD) demands.

Demands are usually represented in Equivalent Tenements (ET), where 1 ET is equivalent to approximately 3.2 persons. In the model, demand is placed on nodes. A node will contain an accumulated demand from the nearest properties. Each property is specified with a development type that has varying water use and demand; demand patterns are used to simulate these variations. The development types used by the IDM include, single-family residential (SFR), multi-family residential (MFR), commercial, industrial, irrigation, public use and tourist.

Given the current water restriction regime on the Gold Coast, it is probable that the existing model scenarios will not adequately represent the current demand patterns for the city. Therefore, the scenarios adopted for the study will be altered to account for the reduced consumption of the city. This will be achieved by evaluating the city's water consumption data for previous yeas to determine an appropriate consumption multiplier, which will be applied to the AD diurnal demand pattern. The reduced demand multiplier can also be gauged from the calibration process when comparing the average day flow patterns in the model to the actual field flow patterns. This will create a water-restriction scenario within the model that can then be used to simulate alternative treatment options, and allow a current and future near optimal solution to be achieved, with and without water restrictions, as the city expands.

1.6 DISSERTATION OVERVIEW

The study is divided into the following relevant chapters:

1. Study outline, objectives, issue and description (as discussed)
2. A description of Gold Coast Water's infrastructure including water practices and guidelines
3. Review of previous engineering studies
4. Model description including hydraulic and water quality frameworks
5. Description of study area
6. Model preparation; calibration/validation and model sensitivity
7. Network analysis and optimisation of the disinfection system
8. Conclusions and recommendations of suitable solutions

CHAPTER 2. DESCRIPTION OF GOLD COAST WATER INFRASTRUCTURE

2.1 INTRODUCTION

This chapter provides a brief overview of Gold Coast City's distribution system, the current water treatment and disinfection practices, and how these compare to national and international standards. Issues thought to be associated with inadequate network disinfection will be discussed as well as the potential public health concerns that may arise. This chapter will set the scene for later chapters, providing insight into the general operation of GCW's distribution system.

2.2 THE GOLD COAST

Gold Coast lies on the east coast of Australia in the southeast corner of Queensland. It contains more than 500,000 residents spanning 70 km of coastline from South Stradbroke Island to Rainbow Bay and covers 1,402 km^2. The relaxed coastal atmosphere and rich environment has contributed to the city becoming the fastest growing region in Australia with the population expected to reach almost 700,000 by 2021 (*Gold Coast Water_3, 2006*).

The Gold Coast is renowned for its subtropical climate and is recognised as being the most biologically diverse city in Australia. The vegetation ranges from coastal wetlands to mountain forests, which are home to more than 25 species of fish, 34 species of amphibians, 72 mammals, 323 birds and 71 reptiles (*Gold Coast Water_3, 2006*).

2.3 PHYSICAL DESCRIPTION OF SYSTEM

2.3.1 GOLD COAST WATER SUPPLY

Hinze Dam and Little Nerang Dam are the raw water supplies for the Gold Coast City. They are located just west of Molendinar and Mudgeeraba in the Gold Coast hinterland and have a combined storage capacity of 170 GL. The raw water is pumped to Molendinar and Mudgeeraba Water Purification Plants (WPP) where approximately 65,650 ML of water is treated each year with a maximum capacity of 290 ML/day. High quality treated water is then distributed via 3,082 kms of water mains servicing 78 reservoirs, 209,000 residential properties and 17,138 nonresidential properties accounting for an approximate 536,000 residents and an annual influx of 10.4 million tourists (*Gold Coast Water_3, 2006*).

Until mid 2006, the Gold Coast's northern suburbs of Beenleigh and Stapylton received approximately 20 ML/d from Brisbane/Logan water supply. This supply was suspended due to the water shortage in Brisbane dams, where at this time, the combined storage of Wivenhoe, Somerset and North Pine dams was approximately 26% full. GCW now supplies the whole northern region with potable water from Molendinar treatment plant.

The northern and the southern region distribution systems are general terms used to describe the existing network based on the current operations of the system. More commonly, the distribution system is identified by 6 Financial Catchments (FC), 18 Water Supply Districts (WSDs), and 83 Water Supply Zones (WSZ). WSDs describe the areas bounded by major reservoirs while WSZs are delineated by pressure differences governed by reservoir supply and pumps, these are further expressed as being either a high level Zone (HLZ), intermediate level zone (ILZ) or low level zone (LLZ) based on surface elevation. Figure 2.1 illustrates the layout and location of GCW's raw water dams, treatment plants, the distribution system and the corresponding FCs for the northern and southern supply regions.

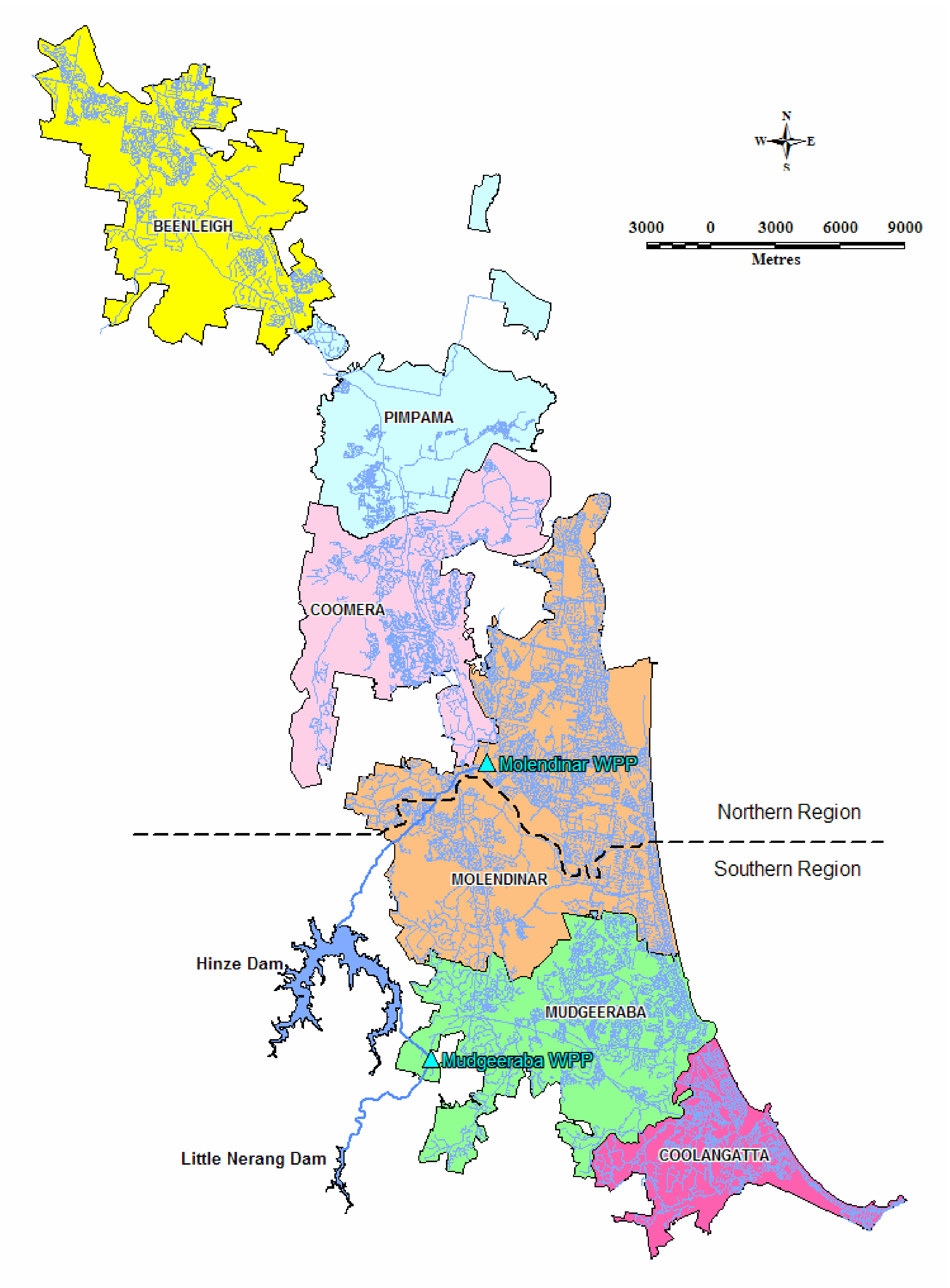


Figure 2.1: GCW's raw water storage dams, treatment plants, northern and southern regional supply network and FC boundaries

Figures 2.2 and 2.3 display the WSD and WSZ boundaries for the southern (study area) region. The latter also includes reservoirs and pump stations.

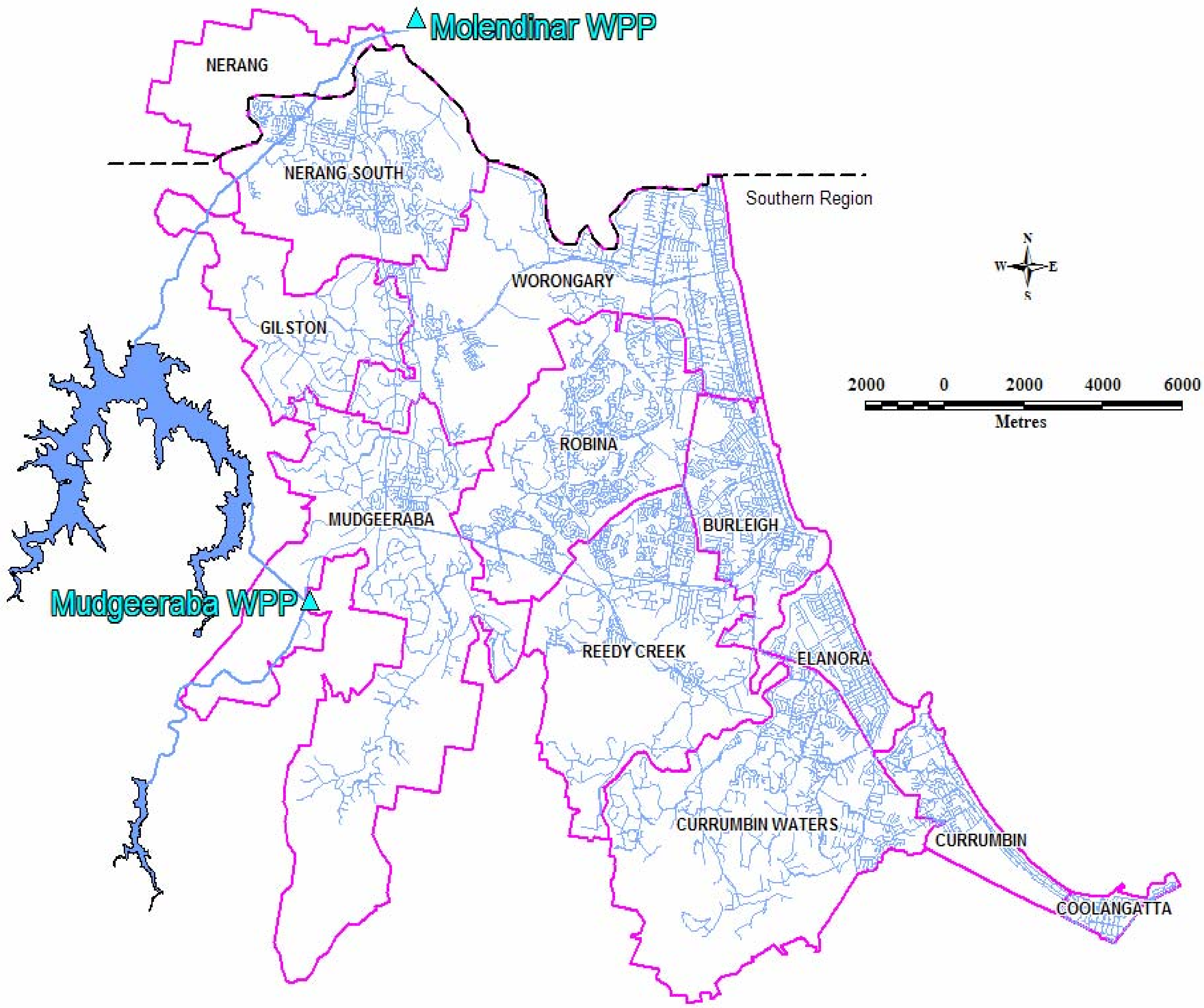


Figure 2.2: Southern region distribution system and WSDs

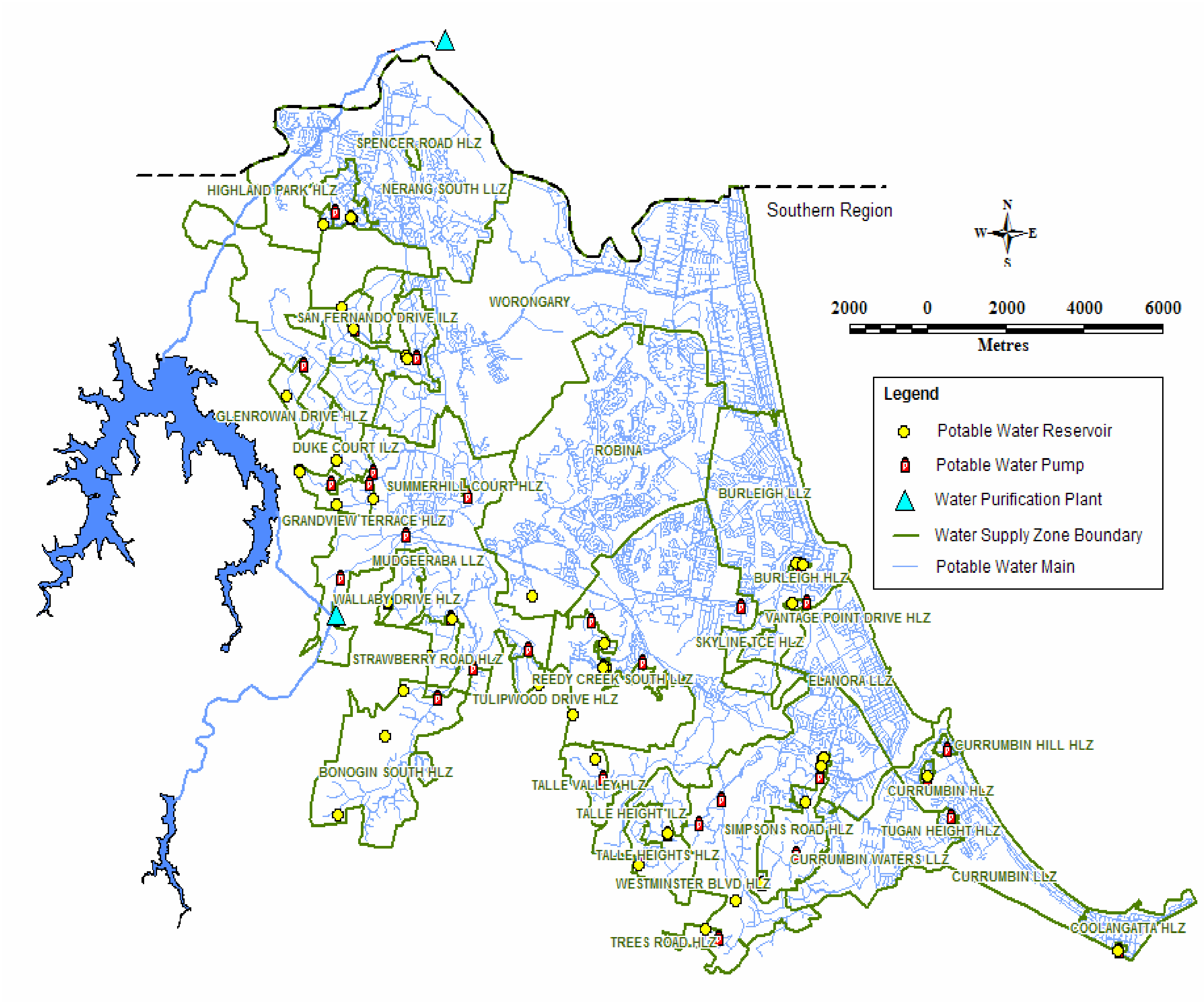


Figure 2.3: Southern region distribution system, WSZs, reservoirs and pumps

2.3.2 WATER TREATMENT AND DISINFECTION

The objective of water treatment is to provide a sufficient and continuous supply of drinking water that is aesthetically pleasing, and chemically and bacteriologically safe. In particular, treated water should obtain palatable, clear, colourless and odourless qualities as well as being reasonably soft, non-corrosive and low in organic matter content *(Gray, 1999).*

GCW operate two conventional treatment plants at Molendinar and Mudgeeraba (shown in Figures 2.2 and 2.3 above). Molendinar WPP is capable of producing 180 ML of the combined total of 290 ML/d. Molendinar WPP is supplied entirely by Hinze Dam while Mudgeeraba receives contributions from both Hinze Dam and Little Nerang Dam (*Gold Coast Water_2, 2006*). The conventional purification process involves 4 main steps including pre-treatment of chemical addition and rapid mixing, primary treatment including flocculation and sedimentation, secondary treatment using filtration and advanced treatment disinfection. This system has been proven to be effective for surface waters that are sometimes or always turbid *(AWWA, 1999)*. Figure 2.4 presents a flow diagram of GCW's treatment processes.

The disinfection phase of the treatment process is the main course of interest for this study. Disinfection is mandatory for all major treatment facilities and should essentially be fail-safe. It should be noted that disinfection does not fully sterilize the bulk water but reduces the risk of infection. There is always a degree of risk associated with large-scale reticulation systems. Risk is created from factors that cannot be controlled such as network contamination via intrusions through fittings, leaks and pipe bursts and from by-products produced from the reaction of water constituents with network infrastructure. For this reason it is important to have a continued monitoring system to detect and minimise these risks. GCW's HACCP Plan was developed solely to minimise risk by identifying critical control points within the system, these then trigger emergency response procedures that are outlined in the Incident Management Plan (*Gold Coast Water_2, 2006*).

Disinfection is a mechanism which controls the numbers of bacteria and viruses to acceptable levels. According to Gray (1999), there are three disinfection options commonly available including ozone, ultraviolet radiation and chlorine. Chlorine is the most common method used worldwide due to its lasting residual effect in water, ease of handling and cost effectiveness. Ozone and UV radiation do not have the same residual effect as chlorine, so they can only be adopted when the point of use is close by.

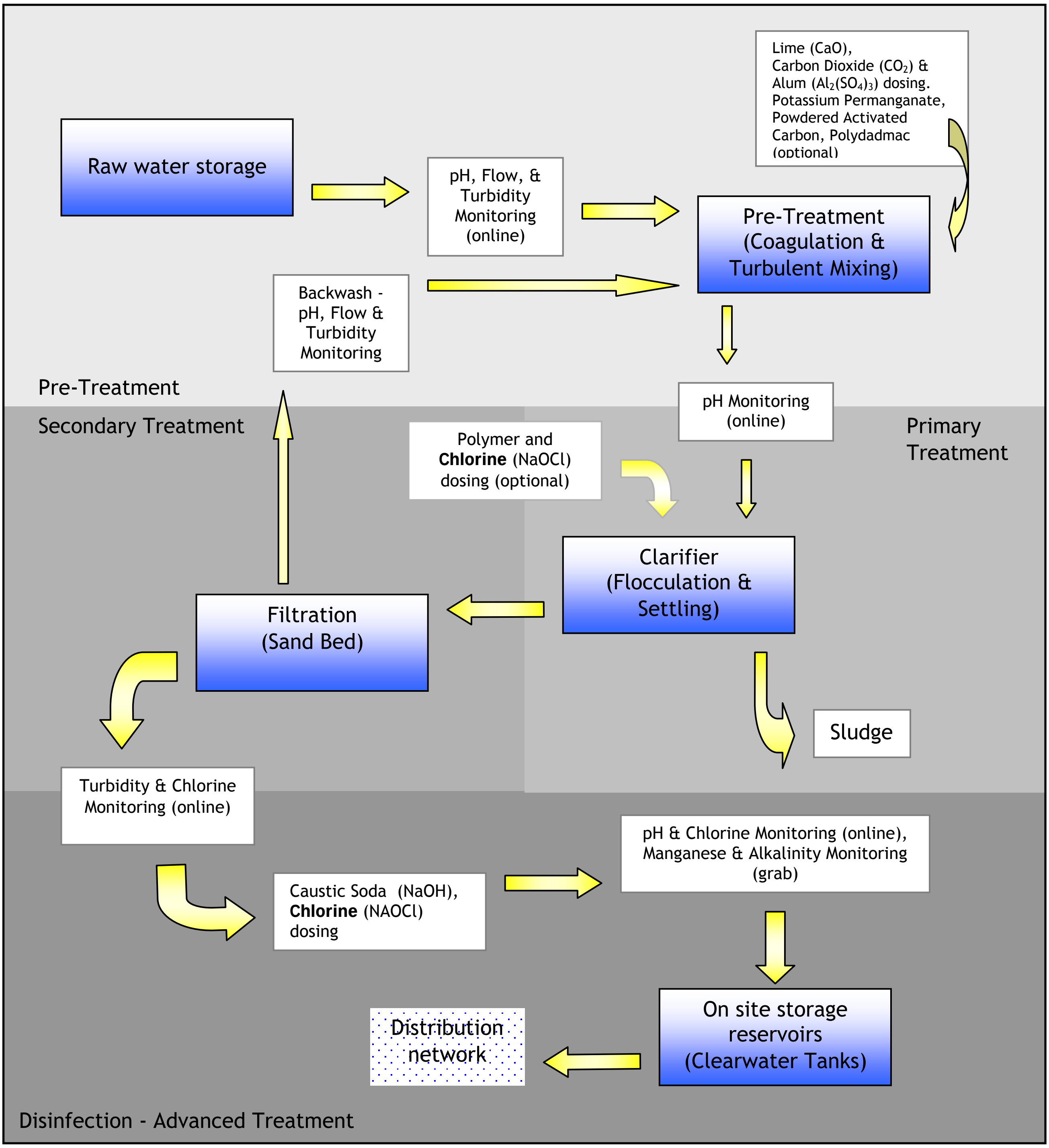


Figure 2.4: GCW's Water purification process

The chief purpose of chlorination is to deactivate micro-organisms. In addition, chlorine is used to oxidise metals or compounds that impart odours and bad taste, bleach colour causing components, improve coagulation by pre-oxidising organic matter, prevent micro-organism regrowth within the network and to provide a barrier against pathogens entering the reticulation system through cross-contamination. Chlorine ages and decays spontaneously within the distribution system by reacting with the bulk water, pipe walls and other network infrastructure to form various disinfection by-products (DBP), some of which can cause serious health problems *(AwwaRF, 2003)*.

GCW utilises chlorine at both purification plants to disinfect the potable water supply. Chlorine is added in the form of sodium hypochlorite (NaOCl) and is dosed according to the quality and availability of the sodium hypochlorite supply and the flow rate (Q) through the treatment facility. GCW dose with sodium hypochlorite at the pre-filter chlorination phase to acheive concentrations of 3.5 – 4.5 mg/L while for disinfection it is dosed at approximately 2.5 – 3.0 mg/L at the inlet of the clearwater tanks to maintain 1.0 – 1.5 mg/L at the outlet. The pH values at the outlet usually range between 6.9 and 7.5. In order to calculate the chlorine dose, the desired chlorine residual and the chlorine demand of the filtered water must be known. To demonstrate how the chlorine dose rate is attained using the above concentration of 1.5 mg/L, a sample calculation for hypochlorite delivery (based on 10% hypochlorite solution) in a flow of 2000 L/s is provided below. This technique is used to quantify the dose rates at the WPPs (*Gold Coast Water, 2004*).

1.5 mg/L x 7,200,000 L/hr x 1 g/1000 mg = **10800 g/hr**

10800 g/hr x 1 L/100 g = **108 L/hr**

GCW also operates two rechlorination (rechlor) facilities at Calypso Bay and Foxwell Rd in the northern region to boost chlorine concentrations in the extremities of the network. These facilities also dose with sodium hypochlorite. Until recently, a third rechlor station, dosing with gaseous chlorine, disinfected water coming into Beenleigh from Brisbane/Logan supply, which was generally slightly higher in pH. The advantage of using gaseous chlorine in this instance is that it lowers the pH of the water to around 7.5, where as sodium hypochlorite increases pH (*Gold Coast Water, 2004*). This rechlor facility became redundant when the delivery of water from Brisbane/Logan ceased.

Sodium hypochlorite is not a disinfectant in itself, but readily dissociates in the presence of hydrogen (H^+) or water to produce hypochlorous acid (HOCl), a weak acid. The reaction is presented by *Equation 2.1 (Gray, 1999)*.

Sodium Hypochlorite		Hydrogen		Hypochlorous Acid		Sodium	
$NaOCl$	+	H^+	$\longrightarrow$	$HOCl$	+	Na^+	*(Eqn. 2.1)*

Similarly, chlorine (Cl_2) reacts with water to form hydrochloric acid (HCl) and hypochlorous acid. The latter is often used as a chlorine source and readily breaks down producing associated forms of the hypochlorite ion (OCl^-), both of which are disinfectants. The reaction undergoes hydrolysis as shown in Equation 2.2 *(Gray, 1999)*.

Chlorine		Water		Hydrochloric Acid		Hypochlorous Acid	
Cl_2	+	H_20	$\longrightarrow$	HCl	+	$HOCl$	*(Eqn. 2.2)*
				$\updownarrow$		$\updownarrow$	
				$H^+ + Cl^-$		$H^+ + OCl^-$ (chlorite)	

The degree of dissociation is dependent on pH. As pH drops, less hypochlorous acid dissociates to chlorite. When pH is greater than 9.0, 100% of the chlorine is in the chlorite form while this falls to less than 50% at a pH below 7.5. At a pH less than 5.0, 100% of the chlorine is in the hypochlorous acid form. Hypochlorous acid is approximately 80 times more potent than chlorite making disinfection more effective under acid conditions *(Gray, 1999)*. At the disinfection phase of GCW's treatment process the water inhabits a pH of approximately 7.2, thus containing higher concentrations of the stronger disinfectant. Figure 2.5 illustrates this relationship between HOCl and pH at 20°C graphically (Jacques, 1985).

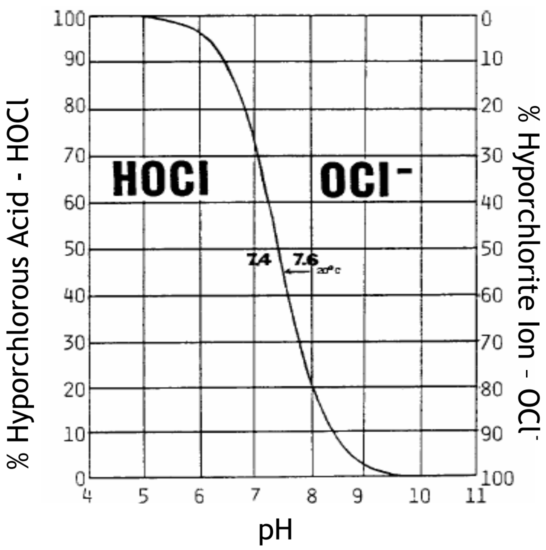


Figure 2.5: Ionisation curve HOCl as a function of pH

Hypochlorous acid and hypochlorite are free chlorine residuals and readily react with any organic matter and reducing agents present within the water. Generally, residuals last longer in cleaner water and decay quickly in turbid waters where they readily react and combine with natural organic matter (NOM). Combined chlorine residuals form when chlorine reacts with ammonia (NH_3) to form chloramines, which also retain disinfection potential. Monochloramines (NH_2Cl), dichloramines ($NHCl_2$) and trichloramines (NCl_3) are created depending on the available concentration of chlorine and ammonia and the pH of the water. Since drinking water is generally maintained around pH 7.2, and dichloramines and trichloramines form less readily at pH greater than 6, these bi-products are usually not common to distribution systems. Similarly, for substantial concentrations of dichloramines and trichloramines, a chlorine to nitrogen ratio (Cl:N) of 5:1, by weight, is required, also a condition not characteristic of drinking water. Chloramines, although much longer lasting, have far less disinfecting properties and require a contact time 100 times greater than free residuals for the same level of treatment. Combined residuals do however prevent chlorine from reacting with phenols that result in taste problems and trace organics, which can be toxic *(AwwaRF, 2003)*. The reactions forming combined residuals provided in Equations 2.3, 2.4 and 2.5.

Monochloramine:

$$\underset{\text{Hypochlorous Acid}}{HOCl} + \underset{\text{Ammonia}}{NH_3} \longrightarrow \underset{\text{Monochloramine}}{NH_2Cl} + \underset{\text{Water}}{H_20} \qquad \textit{(Eqn. 2.3)}$$

Dichloramine:

$$\underset{\text{Monochloramine}}{NH_2Cl} + \underset{\text{Hypochlorous Acid}}{HOCl} \longrightarrow \underset{\text{Dichloramine}}{NHCl_2} + \underset{\text{Water}}{H_20} \qquad \textit{(Eqn. 2.4)}$$

Trichloramine:

$$\underset{\text{Dichloramine}}{NH_2Cl} + \underset{\text{Hypochlorous Acid}}{2HOCl} \longrightarrow \underset{\text{Trichloramine}}{NCl_3} + \underset{\text{Water}}{2H_20} \qquad \textit{(Eqn. 2.5)}$$

It is relevant to note that Brisbane Water utilise chloramine for primary disinfection while Sydney Water use both chlorine and chloramine, the latter is applied to the larger systems where a longer residual is required while chlorine is used to disinfect the smaller systems (*Sydney Water Corporation, 2000*).

In the last 35 years, hundreds of compounds produced by drinking water disinfection have been identified. Some common disinfection by-products associated with chlorine include oxychlorides, oxybromines, trihalomethanes (THMs), haloalkanes, halopropanones, haloacetonitriles, halohydrines, haloacides, halodiacides, haloketones, haloaldehydes, halophenols, haloacetic acids (HAAs), halonitromethanes, halothiaophenes, chloroprocrin, chloral hydrate and chlorinated PAHs, just to mention a few. THMs in particular, are more understood. They were the first by-products found in chlorinated drinking water and have therefore been more widely studied. THMs are single carbon compounds having general formulae CHX_3 where X may be a single or combination of halogen atoms such as chlorine, iodine, bromine and fluorine *(Gray, 1999)*. It is well recognised that THMs are more often produced by chlorination than chloramination and are generally present in most chlorinated systems. HAAs have also recently been found to be ubiquitous in chlorinated waters, but at subordinate levels (*Symons et al., 1996a)*. Other widely studied compounds occurring in chlorinated drinking waters include the halopropanones, haloacetonitriles, chloroprocrin and chloral hydrate, which share similar analytical methods to THMs and hence have an extensive industry database *(AWWA, 1999)*. GCW test the distribution system every three months for THMs at every major reservoir site. According to the ADWGs, the minimum requirement is to at least test for THMs (undertaken quarterly by GCW), and for other compounds if THMs are above 0.25 mg/L (*ADWG, 2004*).

A number of factors contribute to the formation of THMs and chlorine decay including pH, temperature, organic matter content, inorganics, chlorine to nitrogen ratio (Cl:N), pipe material and condition, and disinfectant dose (*Symons et al., 1996a)*. Numerous authors and experimental work has supported this statement.

Fleishacker and Randtke (1983), Stevens, Moore and Miltner (1989), Summers et al (1996), Symons, Dreesman and Stevens (1987) all identified that the formation of THM increased with increasing pH. Fleishacker and Randtke (1983) also presented a small increase in chlorine decay with increasing pH of 5 – 10 over the first 24 hours, but none beyond this. They also demonstrated that by increasing the dose of chlorine the decay rates increased as did the formation of THM. Studies undertaken by Koechling et al (1997) and Valentine, Ozekin and Viesland (1997) showed that THMs and chlorine decay increased with an increase in temperature from roughly 10°C to 35°C over a 24 hour period, the former at a rate of 75-100%. Valentine, Ozekin and Viesland (1997) also stated that chloramine decay increased as bromide concentration increased while Symons, Dreesman and Stevens (1987) proved THMs increased in the presence of bromide. Positive correlation has been shown in laboratory studies between THM and chlorine demand and total organic carbon (TOC) using treated and raw water sources *(Summer et al, 1996).* A chlorine to nitrogen ratio greater than 5:1 suggests that THM formation increases significantly but remains relatively low at a ratio below 5:1 (*Symons et al., 1996a)*. Bryant, Fulton and Budd (1992) commented that THM increased as CI:N approached the break point value of 5:1.

Over time, inorganic and organic material that react with chlorine can build up on the inside of the pipe wall surface to form biofilm (colonies of bacteria protected by an exocellular coating). "Wall demand" is the term used when such reactions occur between the disinfectant and pipe wall. Wable et al (1991) and Brereton, Mavinic, and Crowe (1997) both established that pipes, when removed from the distribution system, contain absorbed organic material that proved to generate chloroform when the pipe reactors were filled with organic free chlorinated water. Similarly, they demonstrated that additional chlorine required to overcome the demand of the pipe wall, also contributes to higher levels of THMs. Clark (Clark) initially assumed a linear relationship between the loss of free chlorine and THMs formation but later demonstrated this experimentally.

The natural decay of chlorine is an issue that stands alone and generally cannot be controlled. An experiment undertaken by AwwaRF (2003) evaluated the potential for booster chlorination to minimise the volume of conventional (single dose) chlorine applied, while maintaining the required free chlorine residual. Prasad, Walters and Savic (2004) commented that booster chlorination can reduce the total disinfectant dose. The AwwaRF (2003) experiment expressed a conventional chlorination scenario with an equivalent mass booster chlorination scenario. Equation 2.6 demonstrates the rate of chlorine decay mathematically.

$$M_o = \sum_{i=0}^{n_b} B_i \qquad \textit{(Eqn. 2.6)}$$

where M_0 = is the conventional dose at time t_0

B_i = is the boost dose at t_i

n_b= is the number of boost doses after the first dose at dose B_0

The principle of booster chlorination is to reduce the average disinfectant decay by using a second point source and a total equivalent mass to increase chlorine residual at a point downstream *(AwwaRF, 2003).* Figure 2.6 demonstrates how chlorine concentrations vary over time using both conventional (M_o) and booster (B_o & B_1) chlorination.

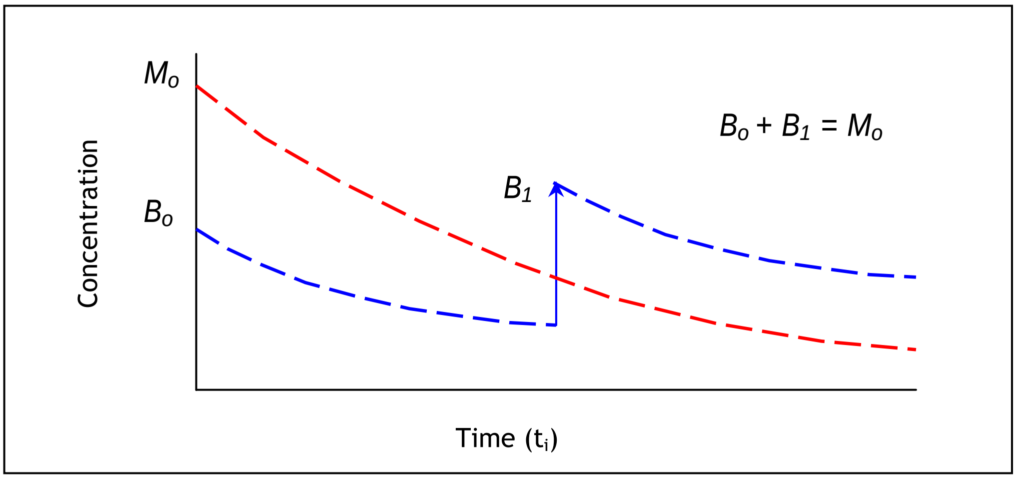


Figure 2.6: Theoretical advantage of booster chlorination *(AwwaRF, 2003)*

It is evident that a higher chlorine residual is achieved at a point downstream of B_1 after time t_i using the equivalent booster chlorination scenario. The results indicate that chemical volume can be reduced to achieve the same outcome as the conventional dosing system. Booster chlorination will therefore be considered in the design for the near optimum dosing options.

The idea of booster chlorination is well supported. Harmant, Nace, and Kiene (2004) indicated that dosing only at the treatment plant is often inadequate and does not provide sufficient chlorine residual throughout large networks. Further, they suggested that by boosting the system with chlorine at various locations, the production of by-products such as THMs and the number of consumer complaints are minimised. Ainley Group Consulting Engineers and Planners (2003) through experience have found that booster chlorination is best injected at the reservoir inlet, to achieve complete mixing prior to distribution.

The complex and multi-objective nature of disinfection has provided motivation to develop mathematical models that describe and predict chlorine decay and by-product formation. The water quality model and associated decay coefficients will be discussed in Chapters 4 and 6.

2.3.3 DESALINISATION AND SOUTHERN REGIONAL WATER PIPELINE (SRWP)

The Gold Coast Desalinisation Alliance (GCDA) is currently undertaking the construction of the Tugun Desalinisation Plant and pipeline. The pipeline extends from Tugun in the south to Molendinar WPP. At the same time the Southern Region Water Pipeline Alliance (SRWPA) are constructing a pipeline from Molendinar WPP to Brisbane in the north. The initiation of the projects were in response to the ongoing drought in south east Queensland and the diminishing levels of Wivenhoe, Summerset and North Pine dams. Tugun was allocated the most suitable site for the multimillion dollar project that is designed to produce 125 ML/d with the possibility of producing a further 47 ML/d. The operational philosophy in the initial years of operation is to send 125 ML/d north through the desalinisation pipeline and the SRWP to supply Brisbane. Desalinated water will mix in the Robina reservoirs and also at Molendinar treatment plant before reaching Brisbane *(GCDA, 2007)*.

The introduction of the Desalinisation Plant will dramatically alter the existing operating regime of Gold Coast's southern region distribution network. This new operational philosophy is illustrated graphically in Appendix B. The Desalinisation Plant and pipeline alignment is presented in Figure 2.7. The new regime is expected to be activated late 2008, where WSDs south of and including Reedy Creek and Burleigh will be fed purely by desalinated water. Robina and Worongary WSDs will be fed with a combination of desalinated and Hinze Dam water, Mudgeeraba and Gilston with Hinze Dam water from Mudgeeraba WPP while Nerang South will by fed with Hinze Dam water via Molendinar *(GCDA, 2007)*. This new operating strategy will have substantial implications for the modelling outcome of this study given that the Desalinisation Plant will also be dosing with chlorine at an anticipated rate of 1.5 mg/L.

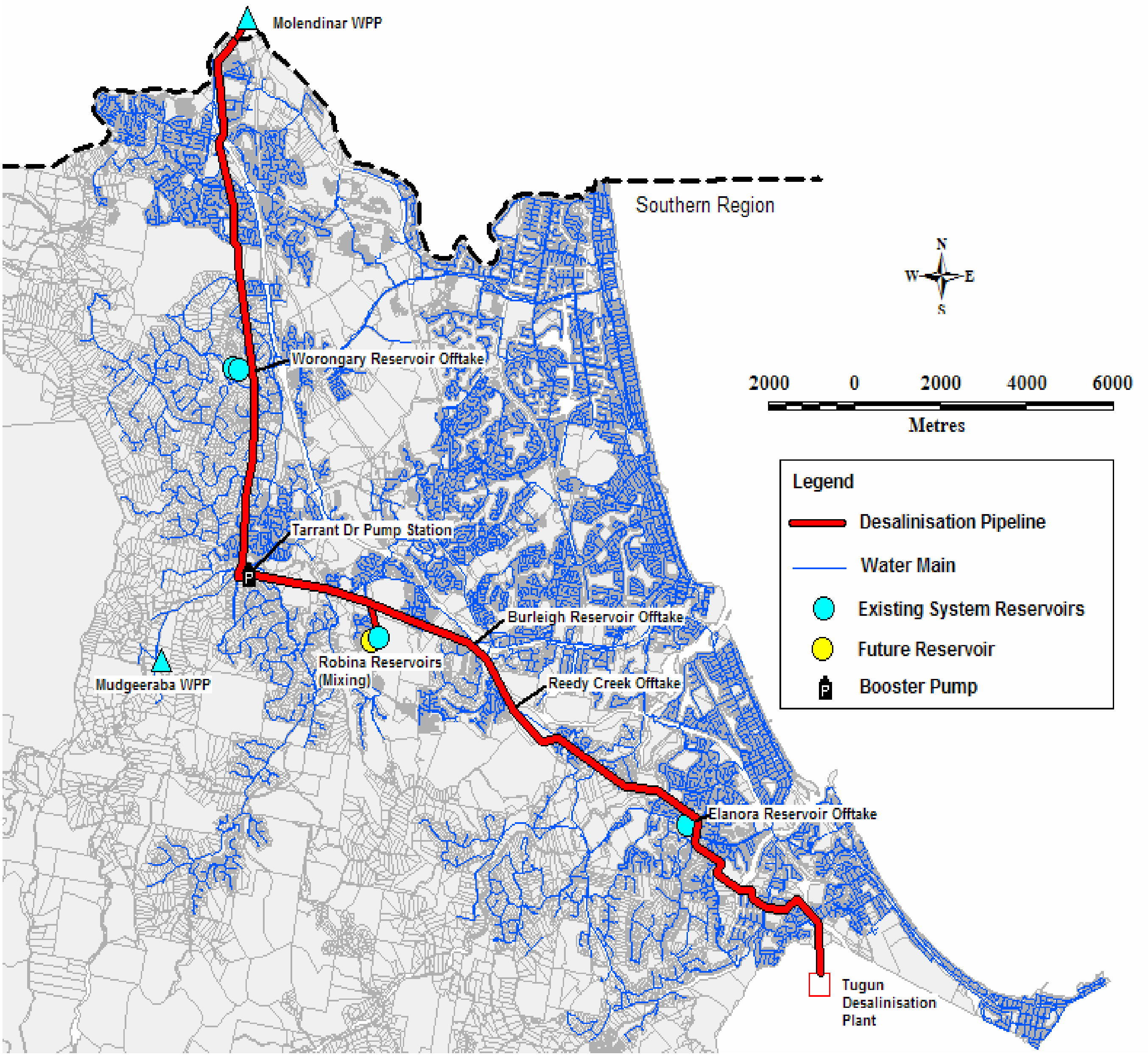


Figure 2.7: Desalinisation Plant location and pipeline alignment

2.4 MAINTENANCE PROTOCOLS

2.4.1 WATER QUALITY MONITORING

GCW monitor the city's DWDS through a combination of computer based hydraulic network modelling, field testing and physical assessment. Hydraulic network modelling has primarily been used for planning purposes, to size pipes and to assess the operating conditions of the network. MWH Soft, Inc. (1996-2003) developed the modelling suite "H_2ONET Analyser" which has been the chosen software package by GCW for network analysis. H_2ONET Analyser is a complete distribution modelling, analysis and design software integrated with an AutoCAD interface. Its has the ability to perform comprehensive hydraulic and water quality modelling, energy management, real time simulation and control, fire flow analysis and unidirectional flushing. H_2ONET's water quality module is not one that has been extensively used by GCW. However, this study has created opportunity to utilise the chemical, tracing and tracking elements within the module to estimate real time chlorine concentration and water age.

GCW routinely test the network for a wide range of physical, chemical, microbiological, radiological parameters including, organic compounds and pesticides. The generic water quality parameters are tested more frequently on a monthly basis, as they are often trigger indicators for less common water quality parameters and will likely determine whether further investigation is required. The less common parameters are tested quarterly, 6 monthly and annually. Turbidity is often removed from raw water supplies during the treatment process and pH neutralised so are only tested downstream in the distribution network. Parameters that may not be removed by the purification process are monitored at the raw water supplies so action can be taken before movement proceeds into the reticulation system. These are usually less common and only require testing with the occurrence of an incident, they may include alpha and beta radiation, pesticides and microbiological organisms such as cryptosporidium and Giardia. Table 2.1 presents a list of GCW's more frequently tested water quality parameters from the purification plant and the distribution system. The table provides the international WHO (2004) values, national ADWG (2004) standards, and GCW's DSS (2006) and the frequency at which the test is undertaken (*Gold Coast Water_2, 2006*).

Table 2.1: Water quality parameters tested by GCW and corresponding ADWG values

GUIDELINE PARAMETER	WHO VALUE (International Specification)	ADGW VALUE (External Specification)	DSS VALUE (Internal Specification)	GCW FIELD TEST FREQUENCY	SAMPLE SITE LOCATION
Physical & Chemical Parameters					
pH	Not Established	6.5 to 8.5	6.9 - 7.5	Monthly	Distribution network – all sites
Chlorine residual	5.0 mg/L	< 5.0 mg/L	≥ 0.2 mg/L & ≤ 1.5 mg/L	Monthly	Distribution network – all sites
True Colour	Not Established	< 15 HU	≤ 5 HU	Monthly	Distribution network – all sites
Turbidity	Not Established	< 5 NTU	≤ 0.1 NTU	Monthly	Distribution network – all sites
Total Dissolved Solids	Not Established	< 500 mg/L	No DSS Value	6 monthly	Distribution network – all sites
Conductivity	Not Established	No Guideline Value	No DSS Value	Monthly	Distribution network – selected sites
Hardness	Not Established	< 200 mg/L	No DSS Value	Quarterly	Distribution network – one site each major res. Zone
Aluminium	Not Established	< 0.2 mg/L	≤ 0.15 mg/L	Monthly	Distribution network – one site each major res zone
Iron	Not Established	0.3 mg/L	No DSS Value	Quarterly	Distribution network – one site each major res. zone
Manganese	≤ 0.4 mg/L	< 0.1 mg/L	≤ 0.01 mg/L	Monthly	Distribution network – all sites
Cyanide	< 0.07 mg/L	< 0.08 mg/L	No Value	Annually	Water Purification Plants
Taste & Odour	Not Established	Not Objectionable	No DSS Value	Reactive	Various Domestic taps
Alkalinity	Not Established	No Guideline Value	> 35 mg/L	Monthly	Distribution network – all sites
*Microbiological Quality**					
Total coliforms	-	< 1 CFU/100ml	No DSS Value	Bi-monthly	Distribution network – all sites
E. coli	-	< 1 CFU/100ml	No DSS Value	Monthly	Distribution network – all sites
Disinfection By Products					
Trihalomethanes	Ratio of respective guideline value < 1	< 0.25 mg/L	No DSS Value	Quarterly	Distribution network – one site each major res. zone

Table 2.1 cont'd

GUIDELINE PARAMETER	WHO VALUE (International Specification)	ADGW VALUE (External Specification)	DSS VALUE (Internal Specification)	GCW FIELD TEST FREQUENCY	SAMPLE SITE LOCATION
Organic Compounds					
Di phthalate	0.008 mg/L	< 0.01 mg/L	No DSS Value	Annually	Distribution network – one site each major res. zone

*Heterotrophic Plate Counts are routinely performed on reticulation samples to check compliance with the ADWG recommended levels. However, no external specification is included since no ADWG guideline value is published.

Note that the WHO (2004) does not provide values for many of the listed parameters; the reason why these values have not been established can be found in Table A4.2 (pg. 486) of the WHO Chemical Summary Tables (Chemicals for which guideline values have not been established). Similarly, the DSS only provide values for the commonly tested water quality parameters.

There are 114 water quality monitoring sites for the Gold Coast reticulation system, 59 of which are allocated to the southern region network. Figure 2.8 displays the location of these test sites within the southern region. The dot points are colour coded to represent the performance of the network in terms of chlorine residual; the data was collected from July 2006 to February 2007 and averaged over the 6 month period. The green points indicate that chlorine residuals comply with the DSS target values ($0.2 \leq Cl^- \leq 1.5$ mg/L) while those in red indicate that chlorine residual does not comply with the DSS values (< 0.2 mg/L).

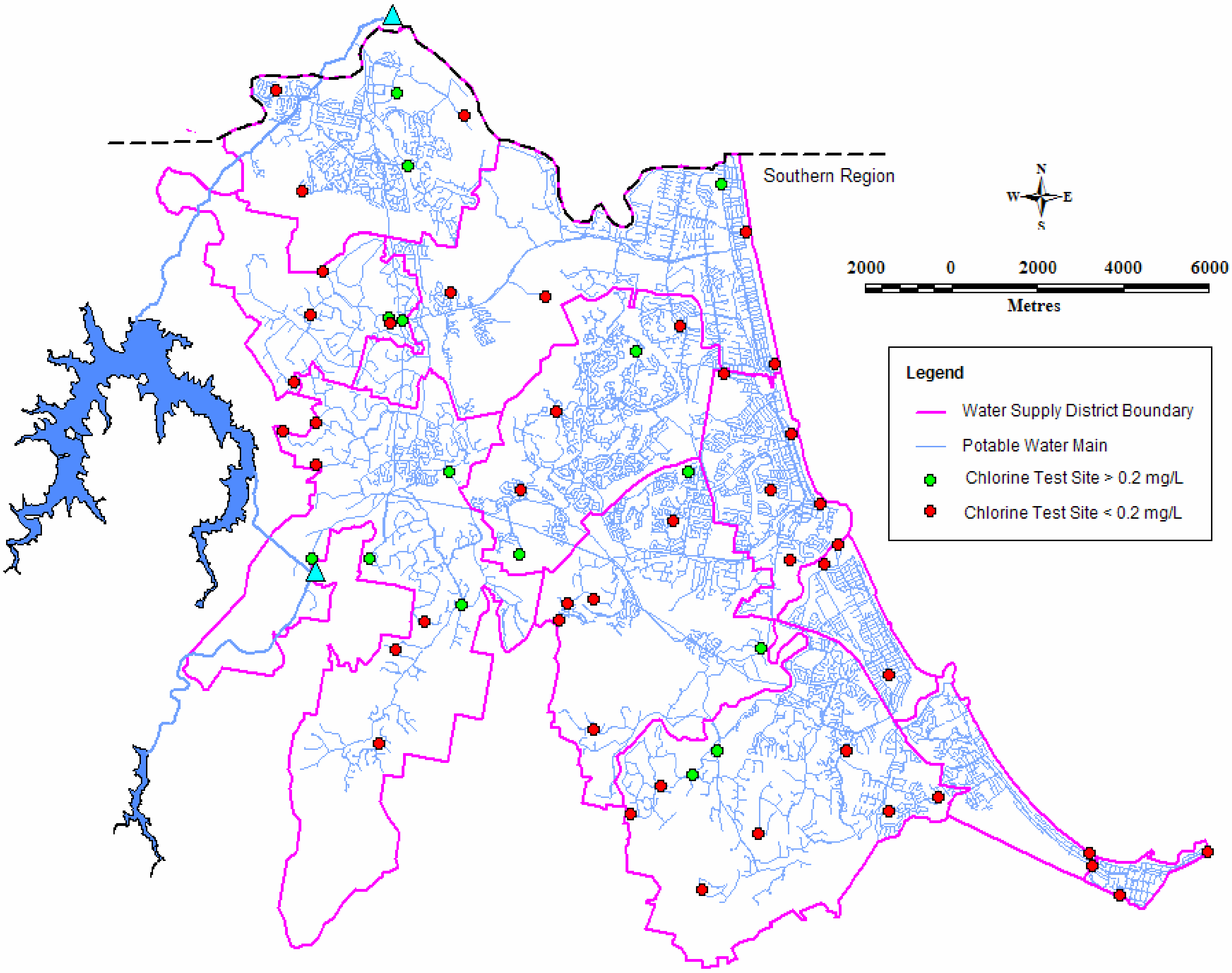


Figure 2.8: GCW's Southern region water quality site testing locations

It is evident by observing Figure 2.8 that the majority of the test sites do not have a chlorine residual within GCW's DSS criteria. More specifically, only 14 sites (23%) of the 59 tested, measured a residual in the required range. A broad trend can be seen between the location of the test site and the chlorine residual. The red points generally occur at the margins of the network while the green points are typically more centralized to the trunk main system or located close to the WPP.

The chlorine monitoring technique used by GCW is in line with that specified in the 21st Edition of the Standard Methods for the Examination of Water and Wastewater *(Eaton et al 2005)*. The manual procedure is a n,n-diethyl-p-phenylene diamine (DPD) colorimetric method that uses a calibrated photometric instrument to determine free chlorine in a test sample. The procedure is based on a sample volume of 10 mL. A DPD indicator reagent of 0.5 mL and a buffer reagent of

0.5 mL are added to a test tube or photometer cell containing the 10 mL test sample and mixed well. Alternatively, an indicator and buffer reagent tablet is added to the 10 mL sample and shaken until the tablet is fully dissolved. The colour of the solution is then compared to the calibrated samples in the instrument to determine the chlorine concentration.

The DPD colorimetric method is an informal approximation of chlorine concentration. A level of uncertainty is associated with the manual procedure where there are obvious margins for error, including the use of incorrect sample volumes as well as a rough visual determination of free chlorine by colour comparison. This may easily be done if a job is rushed or a sample is not taken correctly. The DPD reactor reagent also reacts with oxidised manganese (usually in concentrations > 2.6 mg/L) to produce colour, often causing this technique to produce misleading results at chlorine concentrations < 0.1mg/L (Eaton et al 2005). This may have contributed to the outliers in the Figure 2.8 which shows the presence of chlorine, where in actual fact, it may only be oxidised manganese. Consequently, this method may only provide practical approximations for residual chlorine above 0.1 mg/L, and therefore can not be used for model calibration.

Considering the number of test sites in the network below GCW’s specified chlorine criteria and the potential error associated with the existing chlorine test procedure, there is obvious need for further investigation and water quality modelling.

2.5 WATER MANAGEMENT INITIATIVES

2.5.1 WATER RESTRICTIONS

Low residual chlorine in GCW’s potable water network is believed to be directly associated to the Regional Drought Strategy and plan. Reduction in water consumption (demand) through the implementation of water restrictions means that water retention times in the network are extended and chlorine concentrations are reduced. The relationship between retention time and chlorine decay has been confirmed by Harmant, Nace, Kiene (2004). As mentioned previously, chlorine decay is dependent on a number of physical, chemical and biological properties of the bulk water and pipe work. In addition, chlorine decay is also relative to network operation characteristics such as demand, velocity, adequacy of treatment process, effectiveness and efficiency of treatment processes, storage facilities, and pipeline characteristics such as pipe material, age, condition (corrosion and tuberculation), pipe roughness, design, and maintenance of the distribution system *(Hart et al, 1992).* Typically, the longer water stays in the network, the longer free chlorine has to react with and be absorbed by its surroundings.

Distribution systems are usually designed to satisfy hydraulic reliability, this includes water supply and pressure for domestic and industrial demand as well as for fire flow demands. To meet these requirements large storage tanks are often incorporated into the system design resulting in long residence times, amplifying water quality deterioration, which is further exacerbated by water restrictions *(Clarke et al, 1995)*. GCW is no exception, rapid growth and development of the coastline is seeing wide advances in pipe work and system upgrades. It is not expected that this will slow in the near future, with planning scheme density (PSD) not occurring until 2056.

Gold Coast City Council together with other local councils have engaged in the RDS and plan to effectively mollify and mitigate drought conditions, which have long been apparent in Australia and southeast Queensland. The strategy (to date) imposes five levels of water restrictions, which are implemented when the combined storage of Brisbane's Wivenhoe, Somerset and North Pine Dams fall below a pre-determined trigger levels. Table 2.2 presents the water restriction details for five levels. Level four water restrictions became active in Gold Coast City on November 1st, 2006 while level five restrictions were activated on the April 10th, 2007 and applied to all local governments in SEQ (*Queensland Water Commission, 2006*).

Table 2.2: RDS and plan water restrictions

Water Restriction Level	Combined Supply (Wivenhoe, Somerset & North Pine Dams)	Regional Water Conservation Measures	Targeted % Reduction in Consumption
1	40 %	Voluntary implement water saving practices at home	5
2	35 %	Mandatory Ban on sprinklers	10
3	30 %	Mandatory ban on outdoor hosing and continued ban on sprinklers	15
4	25 %	Mandatory ban on outdoor hosing, efficiency measures for pool owners using town water and restrictions for business, industry & government agencies (concessions apply in some circumstances)	20
5	20%	Mandatory ban on outdoor hosing, efficiency measures for pool owners using town water and restrictions for business, industry & government agencies	25

2.5.2 PRESSURE AND LEAKAGE MANAGEMENT PROJECT

By reducing network pressure, the PLMP changes the nature and direction of the flow in the system. Ultimately, this reduces water consumption and water lost through leakage. Therefore, like water restrictions, the PLMP will increase the retention time of the bulk water, indirectly affecting chlorine distribution.

Approximately 70 District Metered Areas (DMA or pressure reduced zone) will be implemented throughout GCW's potable water network. This will cover about 70% of the GCW's supply area excluding the high density coastal strip. Almost half of the DMAs were installed by late 2006 while the remaining packages will be undertaken and completed by the end of 2007. The total water savings for the project are expected to reach between 13 and 15 ML/day; the savings as of January 2007 were 4.8 ML/d.

The average pressure within the GCW supply area is 60 m, while many areas receive water pressure above 80 m. The philosophy behind the DMA is that by closing off boundary valves of an identified area, and feeding the isolated network with a dedicated main through a Pressure Reducing Valve (PRV), the network pressures can be reduced. Settings are applied to the PRV to reduce the pressure in the DMA downstream, often the maximum and minimum pressure values are set at 45 m and 25 m respectively depending on the variation in the ground surface level. The PRV operates according to the downstream flow requirements and opens up to maximum pressure during peak demand periods such as maximum hour (MH). Under extreme flow conditions the PRV has the capability to open up to an absolute maximum pressure of 55 m to service domestic and fire fighting demands (*Gold Coast Water, 2005*). By pressure reduction the following issues can be avoided:

- Leakage from water mains, joints, fittings and service connections
- Water main and water service connection pipe bursts
- Water hammer, fatigue, and premature aging of pipe materials and fittings
- Premature aging and failure of customers hot water cylinders
- Leakage from customer pipes, toilets, showers and taps

While the amount of water lost through leakage and consumption is reduced, leaks can be detected, located and repaired by monitoring minimum night flows, thus preventing long-term water loss.

Before a DMA can be installed, network analysis is undertaken using GCW's hydraulic network model to ensure the system would continue to comply with GCW's DSS flow and pressure criteria under the proposed boundary conditions. The DSS state that residual mains pressure should not exceed 80 m nor should it fall below 22 m at the property boundary of service under standard operating conditions. At the same time flow should not exceed a velocity of 2.5 m/s. Of course, there are exceptions in the network where pressure criteria cannot conform to these targets, such as directly downstream of pumps and reservoirs. Similarly, 12 m of pressure must be maintained during fire flows; this includes a 30 l/s demand for all commercial/industrial developments and 15 l/s demand for all remaining development types (*Gold Coast Water_4, 2006*). Fire fighting requirements have significantly reduced the area of the city at which DMAs can be established without significant augmentation. DMAs have generally been selected to minimise the need for augmentation. If the DSS criteria are not satisfied, then augmentation of the system may be required. This depends on the extent of failure and whether the failure is an existing network deficiency (usually due to fire flow) or occurs as a result of the network isolation. It is then decided whether DMA is established permanently or temporarily. This is determined by weighing up economic cost with water savings.

Where there is no provision for pressure reduction or the cost of augmentation out weighs the benefits of water saved, a DMA will be considered as a temporary DMA. Annual leakage management will then be performed by the use of temporary flow probes. Temporary DMAs will be "locked down" by closing the boundary valves over the late night/early morning period to monitor minimum night flows through a single feed, and detect/repair leaks.

The southern region potable water network ultimately contains 30 permanent DMAs. The PRVs are monitored on a daily basis where upstream, downstream pressure and flow are recorded and sent back via telemetry/sms to a host computer. This SCADA system allows 24 hour monitoring of the DMA demand and continued leakage control. The field data provided by PRVs will later be used for the calibration of GCW's hydraulic network model. All permanent DMAs proposed and currently operating within the southern region will be incorporated into the model prior to the modelling and optimisation of the disinfection system. Figure 2.9 presents the boundaries for all permanent DMAs within the southern region distribution system. The green shaded areas represent the permanent DMAs installed as of 15th January 2007; the red areas indicate those that are yet to be installed.

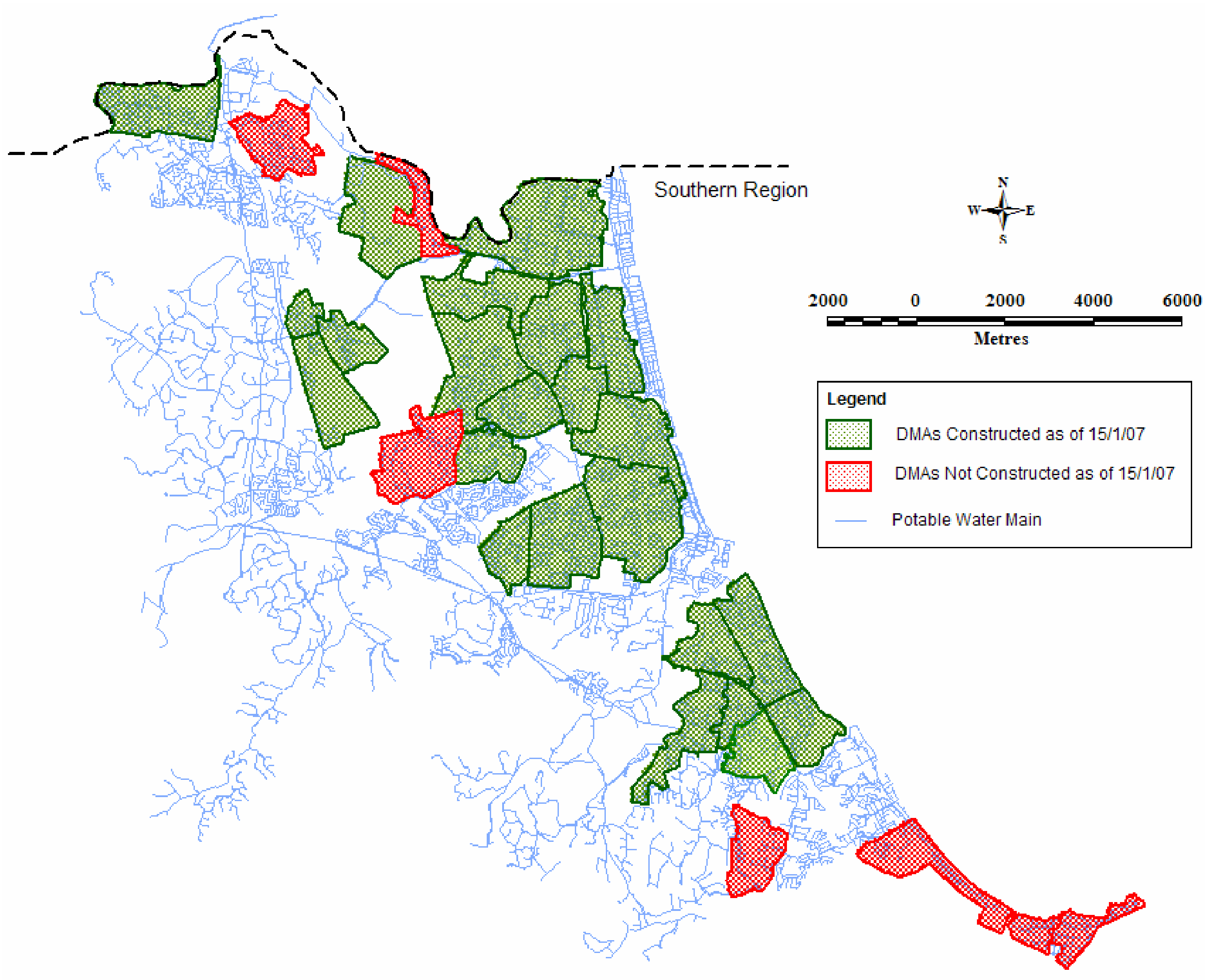


Figure 2.9: Permanent DMA boundaries for the southern region distribution system

GCW is the first water authority in Australia to undertake pressure and leakage management on such an extensive scale. As a result, a number of political issues have emerged, which have slowed the pace and limited the scope of the project. Such issues include those associated with high-rise buildings that do not have booster pumps and/or on-site fire fighting capacity (4 hours), flow and pressure testing for hydraulic consultants, and customer service agreements including fire sprinkler installations. Preliminary field investigations estimated that for the Gold Coast network, 60 DMAs could be commissioned with no adverse effects on the customer, while 8 DMAs have high-rise building issues and 7 DMAs have fire sprinkler installations and cannot be commissioned until appropriate modifications are completed.

The relationship between chlorine decay and the PLMP is quite clear, however the extent to which chlorine is affected is not apparent. By overlaying Figure 2.8, which shows the performance of chlorine in the southern system, on the DMA boundaries from Figure 2.9, a visual relationship between the PLMP and residual chlorine can be seen, this is illustrated in Figure 2.10. However, since many of the DMAs have been installed within the network during the past 6 months and the chlorine data used was an average over a 6-month period from July 2006 to February 2007, Figure 2.9 will not accurately represent the relationship. The circled points in Figure 2.9 show the test sites that lie within a DMA where free chlorine failed to satisfy GCW's DSS minimum chlorine criteria of 0.2 mg/L. Table 2.3 provides installation details for the DMAs with the respective water quality test site. Note that the first column of Table 2.3 corresponds to the sample site number in Figure 2.9. Similarly the last column refers to the number of chlorine samples that were affected by the DMA relative to its installation date from the 6 samples collected during the 6-month period.

Table 2.3: DMA information relative to chlorine concentration < 0.2 mg/L at the test site

No.	DMA Name	Date Installed	No. of Test Sites	No. of Samples Effected
1	Winderadeen Crt, Nerang	13/11/2006	1	3
2	Jura Pde, Merrimac	3/07/2006	1	6
3	Cheltenham Dr, Robina	Not installed	1	0
4	Rio Vista Blvd South, Worongary	26/06/2006	1	6
5	Gold Coast Hwy, Miami	28/09/2006	1	4
6	Mattocks Rd, Reedy Creek	18/09/2006	1	4
7	Deodar Dr, Burleigh	4/09/2006	1	5
8	Philippine Pde North, Elanora	17/01/2007	2	1
				1
9	Philippine Pde South, Elanora	15/01/2007	1	1
10	Golden Four Dr, Bilinga	Not installed	1	0
11	Stapylton West, Coolangatta	Not installed	2	0
				0
12	Stapylton East, Coolangatta	Not installed	1	0

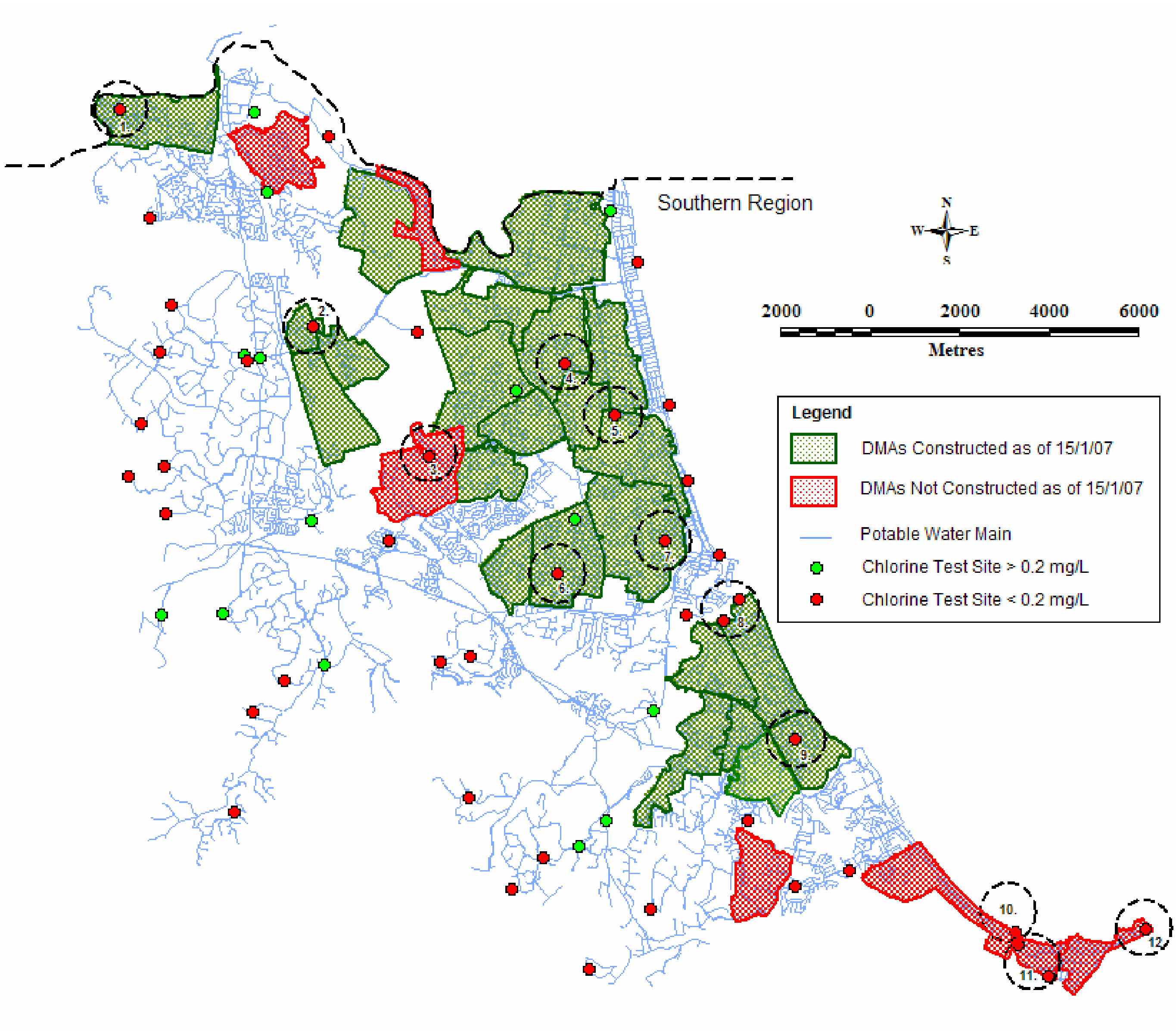


Figure 2.10: Chlorine concentrations relative to pressure reduction

2.6 PUBLIC HEALTH

Outbreaks of waterborne disease can effect large populations within a community quickly and effectively. Evidence suggests that drinking water can contribute to background rates of disease in non-outbreak situations. Hence, drinking water quality should be addressed to control waterborne diseases in the general community *(WHO, 2004)*.

Typical waterborne diseases are caused via organisms originating in the human or animal gut. Contamination of the DWDS by human or animal excreta and the micro-organisms contained within faeces are the most familiar and widespread public health risk associated with drinking water. Viruses, protozoa and bacteria are the most characteristic threat causing infection. These pathogens are commonly associated with mild gastroenteritis through to severe and sometimes fatal diarrhoea, cholera, dysentery, typhoid fever and hepatitis *(WHO, 2004)*.

In the last two centuries, pressure for wastewater disposal and clean water supply has been enhanced by rapid urbanisation and industrialisation. In ancient Roman times engineers went to great lengths to ensure sufficient quantity and quality of drinking water for major cities. During the 19th century, major epidemics of endemic typhoid and cholera infected Britain. It was William Budd (1856) and John Snow (1855) that provided outstanding evidence indicating water was a carrier for the diseases. William Budd demonstrated that the sewer was a continuation of the diseased gut and a vehicle for spreading typhoid, while John Snow compared three London water companies; one that used filtered Thames water, one a cleaner source of water from the Thames and another company that insisted using contaminated Thames water. As a result, in 1859 it became legal in Britain to filter water derived from rivers. This practice was quickly adopted throughout Europe.

When it first became apparent that there was a need to protect drinking water from faecal material, the existing methods to isolate organisms like *salmonella enterica serovar Typhi and Vibrio cholerae* were quite inadequate for large-scale drinking water supply. Therefore, surrogate indicators were needed, and those most obvious were the common flora of the gut, coliform organisms. The work of Alexander Houston (1917) and Doris Bardsley (1934) along with the help of others established the validity of using *Escherichia coli* (*E-coli*) as an indicator for faecal contamination *(ADWG, 2004)*. Typhoid and cholera are still prevalent today in many developing countries where sanitation and adequate water disposal is not available or accessible. Fortunately, incidences of these diseases are rare in Australia.

The DWDS is now well recognised as an efficient transportation mechanism for waterborne pathogens that can quickly transmit infectious disease. According to the ADWG *(2004)*, the probability of a waterborne disease occurring depends on:

- The incidence of the infection within the community (will determine the amount of the pathogen being excreted)
- The susceptibility of individuals and the relative level of immunity in the community
- The per capita intake of the contaminated water

- The infectious dose of the particular pathogen
- The virulence of the strain
- The concentration of the pathogenic organism in the water

Nuisance organisms that effect taste, odour and/or water appearance that do not cause disease, are also commonly found in the DWDS. These organisms are quite physiologically and morphologically diverse and include:

- Iron, manganese and sulphur bacteria
- Actinomycetes and fungi
- Procaryotic bacteria such a benthic and planktonic cyanobacteria (blue-green algae)
- Eurcaryotic organisms such as algae, crustacean and protozoa

These organisms are generally problematic in source waters, reservoirs and distribution systems where conditions support their growth. High nutrient levels, organic matter content and thermal stratification in source waters commonly promote the growth of algae such as cyanobacteria and fungi. Discolouration of the water can occur when blue-green algae is crushed by filters. This can also be exacerbated by micro-algae contained within filter beds, which increase turbidity. Similarly, iron and manganese oxidising bacteria are responsible for the tuberculate deposits and biofilm in pipes, which can discolour the water, stain laundry and upset food handling leading to poor aesthetic quality of the water and customer complaints, not to mention reduce the service capacity of the main *(Gray, 1999)*.

Chlorination of water supplies was adopted in the 20th century and clearly reduced the incidence of infectious disease. Disinfection through chlorination has evidently proven to destroy all bacterial pathogens and significantly reduce the quantity of viral and protozoan pathogens. Protozoans such as Cryptosporidium and Giardia cysts have proven to be somewhat more resistant to chlorine disinfection than bacteria and viruses. However they can be controlled if minimum chlorine levels are maintained within the system *(Sydney Catchment Authority, 2003)*. Today chlorine is predominantly used by water authorities around the world as a disinfectant for drinking-water systems.

Conversely, THMs are only found in waters disinfected with chlorine and are common DBPs known to have associated health risks. Common THM compounds have been mentioned in Section 2.3. THMs are known to be carcinogenic, however, the risk is generally low. Evidence has shown that long-term low-level exposure is linked to rectal, bladder and intestinal cancers. Although there is a carcinogenic risk of long-term THM exposure in water distributions systems, chlorine is the most effective mechanism for eradicating biological risks that are potentially much more dangerous *(Gray, 1999)*.

2.7 NATIONAL AND INTERNATIONAL STANDARDS

The ADWGs (2004) and the WHO Guidelines for Drinking Water Quality (2004) are the national and international guidelines that have determined the fate of drinking water quality in Australia today. GCW is also compliant with the Water Act 2000 which is a Queensland based legislation that governs the way water is used in Queensland. However, the Act does not specify the drinking water parameter values that have been identified in the ADWGs and the DSS. Typically, the ADWG were derived from the WHO guidelines. There are two common reasons why the values derived for chemicals in the ADWG are different from that identified in the WHO, these include:

1. The WHO uses an average adult weight of 60 kg as apposed to 70 kg adopted by the ADWG. This is to account for lighter body weight in developing countries. The ADWGs has adopted the higher average weight that is generally consistent with developed countries such as Canada. This can occasionally yield higher guideline values although the difference is usually insignificant given the safety factors used (a safety factor of 10 is considered a reasonable contribution by a given impurity in a water treatment chemical).

2. The WHO uses a risk assessment calculation for genotoxic compounds, where the value is set at a concentration that gives rise to the risk of cancer by one in 100,000 people. In contrast, the ADWG values consider the following:

    - A value based on a threshold effect calculation, with additional safety factor for carcinogenicity
    - Using the WHO risk assessment model to calculate a concentration where it could alter the risk of cancer by one in 1,000,000 people, if the water containing the compound at that concentration were consumed over a lifetime
    - The limit of determination described by the most common analytical procedure

A value for a parameter in the ADWGs will generally adopt the lower of the calculated value and the limit of determination to provide an adequate degree of protection. In Australia, the high expectation of consumers usually means that the protection commonly afforded by the risk of one in 1,000,000 people is adopted, and is generally based on the threshold calculation *(ADWG, 2004, p 6-9).*

2.7.1 THE AUSTRALIAN DRINKING WATER GUIDELINES

As specified in chapter 1, the ADWGs are GCW's external product specification for drinking water quality. In many parts of Australia, regulation requires water authorities to conform with the specifications outlined by these guidelines. However, in Queensland dinking water quality remains unregulated. Where regulation is not enforced, water producers generally consider compliance with these guidelines as properly diligent and legally prudent. GCW has taken a step further and created a set of internal product specifications (the DSS), which are more stringent than those provided by the ADWGs. These take into account the local and regional factors that influence water quality at this level. By adopting internal specifications, any water quality issue can be addressed before the national guidelines are breached. Table 2.1 in Section 2.4.1 compares the GCW's most commonly tested internal and external product specification parameters including values from the WHO guidelines. The stringent values adopted in the DSS indicate the importance of water quality to GGW in providing a high quality and safe product for its consumers.

The ADWGs discuss the issues associated with physical, chemical, microbial and radiological characteristics of water quality including pesticides, organic and inorganic compounds. Some of these issues concerning public health have been briefly mentioned in Section 2.6. For more detail regarding the derivation of guideline values see chapters 5-8 of the ADWGs (2004).

2.7.2 THE WORLD HEALTH ORGANISATION

The WHO (2004) advises water regulators at national, regional and local levels. The WHO recognises that guidelines must allow for economic, environmental, social and cultural circumstances. Therefore the Guidelines cover an extensive array of potential drinking-water constituents in order to meet the needs of countries around the world. Usually, only a few constituents will be of concern in any given situation. There are five key components that the

Guidelines outline for its approach in preventative management of drinking water supplies, these include:

- Health-based targets
- Assessment of the whole system (source to tape) to deliver Health-based targets
- Operational monitoring of system control measures
- Management plans for normal operation and incident situations
- Independent system surveillance to verify operating conditions

GCW have strategies in place to meet each of these requirements. The guidelines also provide a basis for assessing potentially hazardous water constituents in drinking water. Management must be prioritised for different parameters to protect and improve public health. Priority is expected to take the order of:

- Ensure water is microbiologically safe for consumers
- Control and manage key chemical contaminants known to have adverse health effects
- Address other contaminants

The ADWGs have identified the key contaminants relevant to Australia's geography and history of chemical use. Further, the DSS have adopted these priority levels by identifying the most indicative parameters for GCW's monitoring scheme and HACCP Plan.

The WHO has been the backbone for benchmarking Australian drinking water quality. It has allowed the development of a universal approach to safe drinking water.

2.8 CHAPTER SUMMARY

This chapter covered GCW's general water treatment practices and some of the fundamentals associated with drinking water quality. It presented insight to the issues associated with the poor chlorine residuals existing in the distribution network and provided indication of the performance of the southern region network. The performance of the system was evaluated against internal and external product specifications of the governing legislative requirements.

CHAPTER 3. REVIEW OF PREVIOUS ENGINEERING STUDIES

3.1 INTRODUCTION

An early introduction to water quality modelling in reticulation systems can be found in proceedings of the AWWA Distribution Systems Symposium Conference, 1986. Since then, vast improvements have been made to modelling capabilities and accuracy. Chapter 3 provides an informative overview of chlorine decay and optimisation studies that have previously been undertaken around the world. The aim of this chapter is to cover the main aspects of chorine decay and optimisation so the analysis and optimisation for this study can be representative for the Gold Coast system.

Chlorine modelling has not been undertaken for GCW in the past, so it is essential that the ideas and methodologies developed by previous engineering studies are recognised and understood. This will provide a foundation to develop the most suitable modelling methodology and develop the most efficient and effective booster location and design.

Since chlorine has been a major disinfecting agent for many decades, chlorine studies for distribution systems have been undertaken extensively around the world. Therefore, it is difficult to cover every aspect of such a widely studied field, so for the purpose of this project a detailed summary of four particular studies have been chosen. These studies are chosen on the basis that they cover four general phases of chlorine network modelling and design. They include; the distribution of chlorine through a network environment, the factors that effect chlorine decay, optimal selection of chlorine booster stations, and chlorine scheduling.

3.2 STUDY 1: CHLORINE RESIDUAL DISTRIBUTION IN MUNICIPAL WATER NETWORKS

3.2.1 DESCRIPTION

This study was undertaken by Hart, Wheeler and Daly (1992) for the Concord Department of Water Resources and presented at the NEWWA Computer Symposium in November 1992. The paper describes the results for determining the distribution of chlorine residual through the water network of Concord City, NH using two computer models, KY_NET (which is a modification of the KYPIPE network model) and NET (a water quality simulation model). The purpose for the study was to identify pipes within the network with chronically low chlorine residuals and propose appropriate corrective actions.

3.2.2 MODEL

The selection of the KYPIPE (steady state network model) program was chosen as it was recognized as the most reliable network analysis program in the American water industry at the time. The convergence characteristics associated with solution algorithm employed by KYPIPE proved to be paramount of any solution methodology. KY_NET was designed to use the data files generated by KYPIPE and the geometry data files as well as other operation inputs such as pipe reaction coefficients, relative roughness and water velocities. The KY_NET output was then used in the NET model, which traces water-borne materials and computes mixing regimes. Figure 3.1 illustrates the KY_NET and the NET flow chart.

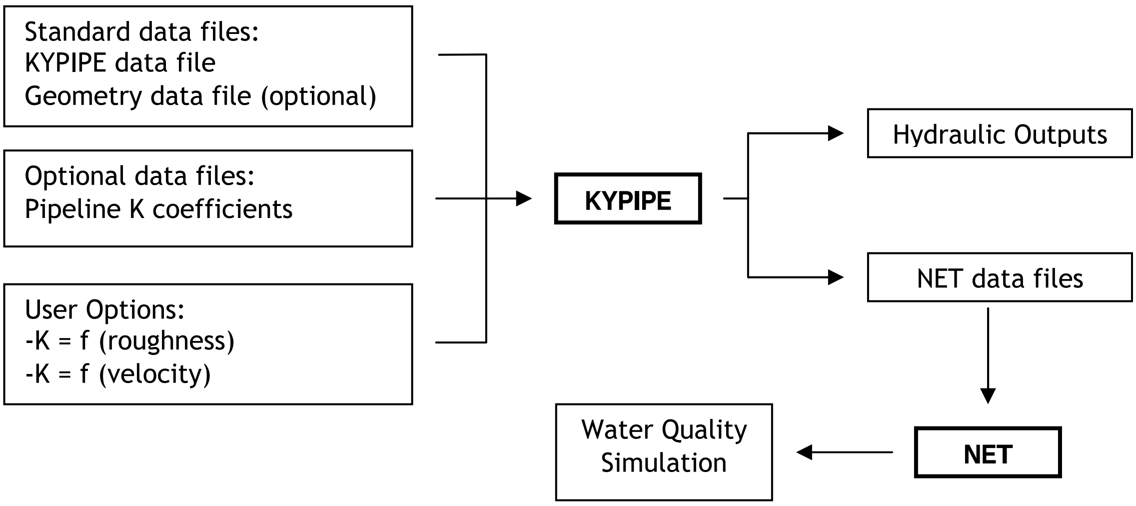


Figure 3.1: KY_NET and NET flow chart

3.2.3 STUDY APPROACH

Disinfection of the system is achieved by dosing with sodium hypochlorite. Water chlorine decay characteristics were evaluated by monitoring treated surface and well water under laboratory conditions. Chlorine decay coefficients were found to be very low with a decay rate of 0.00007/min for the surface water and 0.00003/min for the well water. Loss of chlorine residual in the network was suggested to be due to excessive storage times and the presence of high chlorine demand materials in the pipes. The decay rates from each sample generally followed the trend governed by equation 3.1.

$$C = C_0^{-kT} \qquad \textit{(Eqn. 3.1)}$$

where C_0 = chlorine concentration at time zero (mg/L)

k = the chlorine decay coefficient

T = time (hrs)

The hydraulic computer model was set up with demands created from billing records, development type and usage. An analysis was undertaken for conditions of average daily flow and maximum day demand at peak hour. Hydrant flow tests were conducted to calibrate the KY_NET model and provide additional hydraulic data for the analysis. The final calibrated model contained C (Hazen William friction coefficient) values between 25 and 130 depending on the age and size of the pipe. Chlorine decay coefficients were assigned to the pipes in the NET model using the equations 3.2 and 3.3.

K = *k* x (Default Roughness/Pipe Roughness) x Adjustment Coefficient *(Eqn. 3.2)*

K = *k* x (Default Velocity/Pipe Velocity) x Adjustment Coefficient *(Eqn. 3.3)*

Equation 3.3 relates to the network operating conditions that influences mixing and potential build up of chlorine demand materials. Initial default roughness and velocity coefficients were assumed to be 60 and 1.1 respectively.

Two scenarios were modelled. The first model adopted the above coefficients for chlorine decay while the second applied a global chlorine coefficient (0.00001/min) to all pipes to represent a maximum degree of improvement for a rehabilitation program.

3.2.4 RESULTS

The NET model demonstrated that the average age of the water in the network was 23.5 hours, equating to an approximate 4-9% chlorine loss. It was found that most chlorine decay was due to the chlorine demanding materials within the pipes rather than excessive storage times, although, this may not be true for the storage reservoirs which discharge unchlorinated water back into the network. For this trial, monitoring stations throughout the network did not indicate a reduction in chlorine residuals.

A Chlorine Distribution Curve (CDC) was generated to show how Concords network performed over the two scenarios. Both CDCs concluded that the water treatment plant could operate a lower chlorine dose whilst maintaining a good chlorine concentration throughout the network. Dosing chlorine in excess of 0.5 mg/L may not increase the distribution of chlorine throughout the network and field adjustments to the treatment plant supported this. It is was expected that a reduction in dose will also reduce the production of THMs in the network as well as the cost of disinfection.

3.2.5 SUMMARY

The investigation undertaken by Hart, Wheeler and Daly (1992) shows direct relevance to this study for the Gold Coast system. The model calibration methodology adopted here will be considered when undertaken calibration for the Gold Coast system. The selection of coefficients used for the study covered the basic principles for modelling chlorine distribution in water networks and the general approach is applicable to any water authority dosing with chlorine.

Many studies like this have been undertaken around the world and so the approach is fairly well known. In particular a study by Clark, Rossman and Graymen (1994) describes the mass transfer of chlorine in distribution systems while another study by Clark, Rossman and Wymer (1995) describes the regulatory implications associated with water quality modelling in distribution systems.

3.3 STUDY 2: BULK DECAY OF CHLORINE IN WATER DISTRIBUTION SYSTEMS

3.3.1 DESCRIPTION

This study quantifies the bulk chlorine decay coefficient that is necessary for mathematical models to determine chlorine concentration in a water distribution system. For the model to maintain its predictive capability, coefficients must be determined relative to seasonal and water treatment operational changes. This refers to independent variables such as temperature, TOC concentration, initial chlorine concentration and the number of rechlorinations in the network. The study was undertaken by Hallem et al, (2003) and presented in the Journal of Water Resources Planning and Management.

3.3.2 STUDY APPROACH

Over a two and half year period, the Melbourne Water Treatment Works (WTW), operated by Severn Trent Water carried out 148 chlorine decay tests. Melbourne WTW undertakes dissolved air flotation, rapid gravity sand filtration and granular activated carbon (GAC) prior to disinfection. To test the effects of initial chlorine concentration and TOC, samples were taken from the raw water (unchlorinated), while samples relative to temperature were taken from the GAC-treated water (this was the closest chlorine-free sample point to the final product). Samples on the effect of rechlorination were taken from the final water. The samples were dosed with hyperchlorite in the laboratory to give a chlorine concentration of 0.5 mg/L and incubated at a range of temperatures. TOC of the samples was also measured using the Schimadzu 5000 automated analyzer, pH was not considered since it did not change significantly throughout the WTW. Chlorine measurements were taken using the n,n-diethyl-p-phenylene diamine (DPD) colorimetric method. All samples were duplicated and gave an accuracy of $\pm$ 0.02 mg/L.

3.3.3 RESULTS

The bulk water (k_b) effects on initial chlorine concentration were measured. Temperature and TOC were stable for the length of each test (TOC 4.5 $\pm$ 0.2 mg/L, temp 16 $\pm$ 2^oC). The initial chlorine concentrations (C_o) varied from 0.22 to 2.06 mg/L. The results of a regression analysis comparing the relationships of this study and previous work found the R^2 value in the Melbourne water to be 0.67 with 99% significance. Similarly, decay rates associated with low and high initial chlorine concentrations were found to decrease a fast and slow respectively. This was because a small initial concentration would quickly react with spontaneous compounds before reducing to zero concentration.

Seven experiments were undertaken on Melbourne GAC water to examine the relationship between k_b and temperature. The Arrhenius equation was used to describe the way that temperature effects the chemical reactions of chlorine decay *(Powell et al, 2000)*.

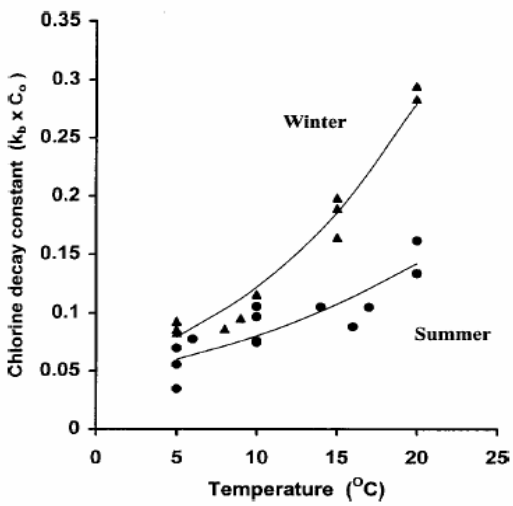
$$k_b = F \exp\left[\frac{-E}{R(T + 273)}\right] \qquad \textit{(Eqn. 3.4)}$$

where F = frequency factor

E = activation energy

R = radial gas constant (8.31 J/mol°C)

Analysis of the data set gave values for E/R = 6,616°C and $F = 3 \times 10^9$. Clear differentiation between the sample data was found by examining the product of ($k_b \times C_o$) for the summer and winter months. This was thought to be due to the varying efficiency of the treatment processes with season and/or the change in composition of the organic content in the source water. Figure 3.2 shows the impact of temperature on chlorine decay.



Where; k_b = bulk decay
C_o = chlorine concentration

Figure 3.2: Impact of temperature on chlorine decay corrected for initial chlorine concentration

TOC was measured as it was the most common method of detecting organic matter in potable water. The results indicated that only small amounts of chlorine decay occurred below a TOC value of 1.31 mg/L. Hence, it was expected that at lower TOC levels the slower organic compounds have a greater influence on decay. It was noted that the treatment processes would not remove the same proportion of slow and fast reacting TOC compounds; rather treatment generally leaves a larger proportion of slower reactants. The resulting decay would therefore be slower since less fast reacting compounds are present in the final product water. The power function is recognised as most representative of chlorine decay in bulk water for distribution systems and is most robust over TOC concentrations of 1.4 – 4.7 mg/L.

Four samples were taken from the final water at the Melbourne WTW and decay observed in one of the samples. All samples had an initial chlorine concentration. The samples were dosed with 1 mg/L when the chlorine fell to concentrations less than 0.05 mg/L and the decay observed. This was repeated 4 times. TOC and temperature were stable throughout the test. The results show a strong power relationship (R^2 > 99%) between bulk decay and the number of chlorinations (N). The exponents varied from –1.21 to –1.60 which were lower than the dilution results (-1.92). Field tests were also undertaken with samples taken downstream of the rechlorination sites. The samples were analysed in a similar fashion, the exponents were observed to range between –0.63 and –1.54, again lower than the dilution results. Field tests for other studies have also displayed the exponent or decay rate after rechlorination is less in the field with an average value of -1.14.

3.3.4 SUMMARY

This study provides a good indication of how chlorine decay is affected by initial chlorine concentration, TOC, temperature and rechlorination. These factors are experienced universally in a variety of ways. The results provide insight as to what might be seen if GCW were to undertake a similar analysis. By considering the variables used in this study, an informed decision on the decay coefficients can be made for the Gold Coast system.

Many studies like this have been undertaken internationally, such as those undertaken by Propato and Uber (2003), Gagnon et al (2004) and Sung, Huang and Wei (2005). These studies describe the bulk characteristics that effect chlorine decay in distribution systems.

3.4 STUDY 3: AN ALGORITHM TO OPTIMIZE BOOSTER CHLORINATION IN WATER DISTRIBUTION NETWORK

3.4.1 DESCRIPTION

The authors of this study, Harmant, Nace, Kiene (2000) identified that booster chlorination can be used as a strategic tool to maintain sufficient water quality throughout the distribution system. However, they recognized that the efficiency of such a tool is dependent on the selection of the applied dosage and booster locations. This study demonstrated the use of a quantitative algorithm developed by CIRSEE to optimise the location and number of potential booster chlorination sites for a distribution system in Cholet, France.

3.4.2 STUDY APPROACH

The Genetic Algorithm (GA) used in this study is based on publications from Lee and Deininger (1992) and operates to optimise booster location according to residence time and water demand. The Lee-Deininger approach was based on hydraulic simulations to determine the best coverage of the distribution system relative to consumption. Their method included taking water samples from network nodes that received (or covered) the highest quantity of water from other network nodes. The term cover was then used to define the potential water quality at a node based on the measurements taken from other nodes. Hence, *"node i covers node j if i receives sufficient water from node j" (Harmant, Nace and Kiene, 2000, pg. 3)* and therefore the water quality of the closest node upstream of the tested node can be assessed with certain assumptions. Therefore the water quality of node *j* can be deduced if the quality at *i* is known and covers node *i*. To exploit this throughout the entire network a matrix was created known as the "water fraction matrix" which provides the contribution of water at any node. Equation 3.5 gives the water fraction matrix where each term represents the flow contributions from all pathways from node *j* to node *i*.

$$[\ W(i,j)\ ] \text{ with } (i,j) \in \{1,\dots,n\}2 \qquad \textit{(Eqn. 3.5)}$$

A water fraction criterion was introduced which determine whether node *j* is covered or node by node *i*. Hence the water contribution from node *j* to *i* can be explained if:

$W(i,j) < c$, then $W_c(i,j) = 0$, else if $W(i,j) \geq c$, then $W_c(i,j) = 1$

A knowledge carrying matrix ($W_c(i,j)$) is built from this and used to obtain the solution in the optimisation problem. The optimisation problem is formulated in Equation 3.6.

$$Max \sum_{i} C_i * y_i \qquad \text{(Eqn. 3.6)}$$

The following constraints are applied so to impose a maximum number of sampling points:

$$Max \sum_{i=1}^{n} x_i \leq NS \qquad \text{(Eqn. 3.7)}$$

$$Max \sum_{i=1}^{n} W_c^{T}(i,\ j)x_i - y_j \geq 0 \qquad \text{(Eqn. 3.8)}$$

where C_i = the demand of the node (mg/L)

n = number of the nodes

x_i, y_i = 0 or 1

Equation 3.8 implies that control is set on the upstream node *i*, Harmant, Nace and Kiene (2000) indicated that control must be set on the downstream node *j* when considering booster chlorination. Therefore Equation 3.8 was adjusted as follows:

$$Max \sum_{i=1}^{n} W_c(i,\ j)x_i - y_j \geq 0 \qquad \text{(Eqn. 3.9)}$$

The reaction of water in a distribution system primarily takes place in the bulk water and/or between the pipe, hence, reactions are a function of residence time and pipe diameter *(Kiene, 1993)*. Because retention time varies in the network depending on the network connectivity, pipe size and demand, there are zones that will have poorer water quality and it is important to chlorinate these zones. Since chlorine decay is a function of retention time chlorine residual is said to decrease according to first-order kinetics, such that:

$$TC_2 = TC_1^{-kt} \qquad \textit{(Eqn. 3.10)}$$

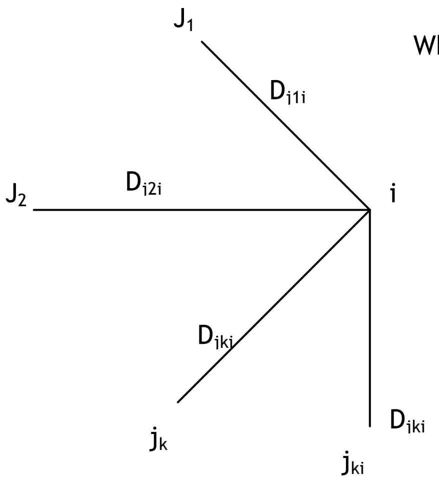
where T = retention time from node 1 to node 2 (hrs)

TC_1 = chlorine residual at node i (mg/L)

k = chlorine decay coefficient

**Note that Equation 3.10 is Equation 3.1 including retention time*

Figure 3.3 shows how the residual chlorine (TC_1) is a function of the flow arriving at i and the residual chlorine at nodes upstream of i.



Where ki = the number of preceding nodes of node I

Arc (i,j) and $d_{jki} > 0$

Figure 3.3: Chlorine residual at *i* as a function of chlorine residual at nodes upstream of *i*.

Finally, Equation 3.11 demonstrates the residual chlorine in node *i*.

$$TC_1 = \frac{\sum_{jk} TC_{jki}^{(-kT_{jki})*d_{jki}}}{\sum_{jk} d_{jk}} \qquad \text{(Eqn. 3.11)}$$

This equation provides a good representation of water quality and network consumption and was used in computer program to optimize booster rechlorination.

3.4.3 METHOLODOLOGY

In order to optimise the booster locations, 4 representative periods were selected from the hydraulic time step, 05:00, 09:00, 15:00 and 23:00 hrs, the tank flow patterns were analysed. A chlorine decay coefficient of 0.006 /min was used while a maximum free chlorine residual of 0.3 ppm and a minimum of 0.05 ppm was established for the optimisation. Firstly, the optimization for the booster chlorination program was undertaken for the 4 representative periods, each potential booster had a dose rate of 0.3 ppm. The algorithm then directly estimated the efficiency for the selected boosters based on the chlorine modelling. The efficiency was expressed as a percentage of free chlorine in the system having more than 0.05 ppm relative to the amount of water distributed.

3.4.4 RESULTS

The results of the study indicate that 2 booster chlorination stations have the greatest improvement for the city. They were shown to increase the total distributed water demand with a chlorine concentration greater than 0.05 ppm by 30% at 15:00 hrs; at 23:00 hrs 1 booster station had a maximum increase of 5%. More than 2 booster stations did not have a significant effect on the networks free chlorine, Figure 3.4 demonstrates this.

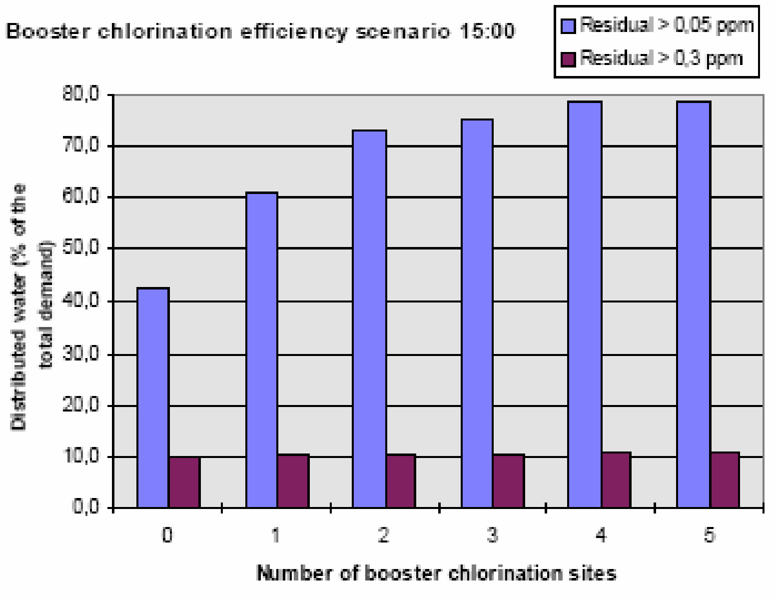


Figure 3.4: Booster chlorination efficiency

The most efficient booster station (station 1) was located at the outlet of a reservoir and effected approximately 18% of the distribution demand. The second booster station was located in the oldest part of the city where cast iron pipes were predominant and the residence times were high.

3.4.5 SUMMARY

On completion of the study the two booster stations were installed. Chlorine analysers located in the field confirmed the improvements of chlorine spread in the network. Harmant, Nace and Kiene (2000) commented that this approach may be particularly difficult for large water utilities.

This study provides a useful example for determining the location of chlorine booster stations within a reticulation system. Although the Gold Coast has a significantly larger network, the same principles can be applied. However, this approach is unlikely to be adopted due to the limited access to a GA optimisation model. It is anticipated that all physical characteristics of the network will be considered in addition to the modelling results when determining booster locations for the city.

Other parallel studies include those undertaken by Piriou et al (1999), which describes the optimisation of a disinfection strategy after the upgrade of a treatment plant, and a study undertaken by Harmant, Nace, and Villion (2001) describing the optimization of booster location and chlorine dosage. These studies provided valuable background information on booster chlorination.

3.5 STUDY 4: OPTIMAL SCHEDULING OF MULTIPLE CHLORINE SOURCES IN WATER DISTRIBUTION SYSTEMS

3.5.1 DESCRIPTION

This study provides a methodology to determine the optimal chlorine dosage at the chlorine source node to maintain a specified residual throughout the distribution system. The authors of the study, Munavalli and Mohan Kumar (2003) recognise that to achieve this in a dynamic network environment is a complex process, hence, a non-linear optimization problem is required to determine a chlorine dose relative to minimum and maximum constraints applied within the system. The study applies a GA approach to three sample systems to establish that the algorithm is an effective tool for evaluating optimal chlorine schedules.

3.5.2 STUDY APPROACH

The aim of this study is to minimize the concentrations of residual chlorine at distribution nodes violating the minimum chlorine constraints. This is achieved by using the GA to estimate the temporally varied source dosage at the source node. The formulation of the GA is not discussed here but can generally be described as a set of coded binary strings containing all decision variables, each of which have a set of maximum and minimum constraints of specified value. The solution technique of the GA includes three basic steps; random generation of an initial population, evaluation of fitness for each individual in the population considering the constraints, application of GA operators such as mutation, crossover, and reproduction to generate an improved population set. This is repeated for a specified number of generations. The ideology behind the mutation is that they enhance GA convergence characteristics.

A hydraulic water quality simulation model is required for the GA to function. The model adopts energy conservation equations (loop/path continuity) and flow conservation (node continuity). The former is solved by Tewarson-Chen adaptation of Newton-Raphson iterative technique and allows computation of dynamic flows and extended time simulation. The model is solved numerically by the la-gragian time-driven method, which is efficient and robust (Liou and Kroon, 1987).

The scheduling of the source dosage is assumed to be periodic such that the length of the simulation period is the same as the time at which concentration pattern starts to repeat itself. Optimal scheduling will ultimately determine a set of unknown variables within the dosage period at each source node. Equation 3.12 represents the total number of unknown source dosages.

$$Ncl = \sum_{s=1}^{Nsn} N(s) \qquad \text{(Eqn. 3.12)}$$

where Ncl = number of unknown source dosages

Nsn = number source nodes

N(s) = number of periodic source dosages applied at source *s*

These unknowns are estimated where no constraints are violated during the monitoring period while achieving the minimum allowable value. The decision variables are the source dosages at each source node over distinct time periods. If the maximum specified value for a decision variable is set to the higher constraint value then the maximum allowable constraint value is accounted for at the source node and therefore nodes downstream will only require the minimum constraint. The objective function is to achieve a minimum specified concentration without breaching the constraint, hence, it is formulated as the square difference between the minimum specified value and the computed chlorine concentration at all distribution nodes. Equation 3.13 illustrates the objective function.

$$E = \sum_{j=1}^{M} \sum_{k=1}^{N_j} \left[Cn_{jk} - C_{\min} \right]^2 \qquad \begin{array}{l} Cn_{jk} \geq C_{\min} \\ Cn_{jk} \leq C_{\max} \end{array} \qquad \text{(Eqn. 3.13)}$$

where M = number of monitoring nodes

N_j = number of monitoring times at node j within the period

C_{max} = specified maximum chlorine concentration (mg/L)

C_{min} = specified minimum chlorine concentration (mg/L)

A solution in which the objective function is minimised (higher fitness) will survive and proceed to the next generation in the GA process. Penalties are also employed to encourage the GA to eliminate maximum and minimum constraint violations.

3.5.3 MODEL APPLICATION AND RESULTS

The model was applied to three distribution systems. The third was applied to demonstrate the nonlinear reaction kinetics of chlorine. The network study area contained only one main supply source (concentration type) and one booster chlorine source (mass rate type). Four 6-hour chlorination periods were assigned to each source while the upper and lower constraints adopted were 2.0 and 0.2 mg/L respectively. A repetitive nodal concentration was observed after the second day in the model simulation run, so optimal scheduling runs were chosen to span a 3-day period.

Figure 3.5 expresses the chlorine concentration at the booster chlorination source (Node 9) and the most downstream distribution node (node 2), which maintains the minimum allowable concentration. These results were obtained from the final GA runs.

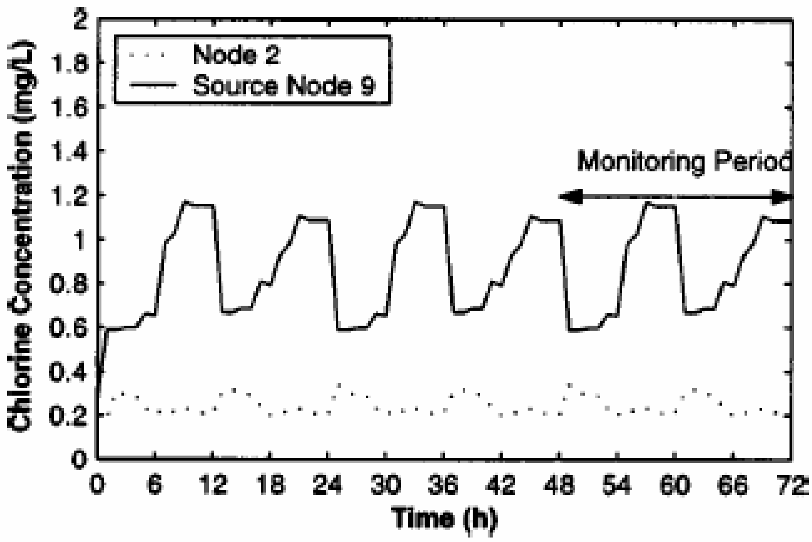


Figure 3.5: Chlorine concentrations at the booster source and at the most distant point downstream

The above figure demonstrates that the GA can effectively obtain the optimal chlorine dosage whilst maintain a specified residual anywhere in the network.

3.5.4 SUMMARY

Munavalli and Mohan Kumar (2003) indicate that booster chlorination is necessary to maintain uniform chlorine concentrations throughout a reticulation system. Within a reasonable number of evaluations the GA proved to provide a near-optimal solution for both linear and nonlinear chlorine kinetics.

The GA approach could effectively be applied to Gold Coast Water's distribution system and may be utilised at a later stage to verify the solutions recommended in this report. The optimisation technique will not be employed for the modelling in this project as it is not within the capabilities of the network model chosen for the study.

Prasad, Walters, and Savic. (2004) also provide similar information regarding the optimal scheduling of chlorine for distribution systems .

3.6 CHAPTER SUMMARY

Chapter 3 provided some detailed examples of engineering studies involving chlorine distribution in water networks and optimisation of chlorine booster location and scheduling. The optimisation techniques discussed will not be employed for this project since the access to GA software is limited within council. However, the GA approach may be employed at a later stage to verify the recommended solutions depending on allocated funding.

CHAPTER 4.　MODEL DESCRIPTION

4.1 INTRODUCTION

The objective of this chapter is to describe in detail the network model that will be used to analyse the hydraulics and chlorine distribution for the project study area. The model will be used to determine a near optimal solution to the existing disinfection problem.

4.2 NETWORK MODEL DESCRIPTION

The chosen hydraulic modelling software for this project is "H_2ONET Analyser", developed by MWH Soft, Inc. (1996-2003). The modelling suite was first used by GCW in 2001 and has been the chosen software package for network analysis. H_2ONET Analyser is a steady state water distribution modelling, analysis and design software integrated with an AutoCAD interface. H_2ONET has inbuilt modules which can perform comprehensive hydraulic and water quality modelling tasks, fire flow analysis, energy management, real time simulation and control and unidirectional flushing.

4.2.1 MODEL SETUP

GCW's hydraulic network model was developed by Cardno MBK (Brisbane) and was last updated in October 2006. The model is made up of elements including pipes, nodes, pumps, valves, emitters and tanks (reservoirs), each have been allocated particular attributes based on Gold Coast City Councils GIS system, MapInfo. Among these elements, there are different types that can be chosen to enable particular hydraulic task to be undertaken. For example, valves including pressure-reducing valves, pressure-sustaining valves, pressure breaker valves, flow control valves among others are available as well as fixed head and variable head tanks, and pump design point curves, exponential 3-point curves and multi-point curves.

The basic model inputs for nodes are elevation, water demand and initial water quality, while the basic outputs include hydraulic head, pressure and water quality. Similarly required pipe inputs are length, diameter and roughness with outputs of flow, velocity and head loss. Tanks inputs include elevation, top and bottom water level and diameter with outputs including flow, tank level and tank head. Pump information required include specified head and flow for design point curves, and complete pump curves where more than a single operating point is needed. Pump outputs include upstream and downstream pressure, flow, Net Positive Suction Head (NPSH), head loss and cavitation index. Much of the element input data is obtained from GCW's GIS database.

As mentioned in Chapter 1, the model nodes or junctions are allocated demand in equivalent tenement based on the IDM. Consumption patterns are also assigned to the nodes based on the development type of the corresponding land parcel. The 6 diurnal patterns for the 7 development types are established in GCW's Desired Standards of Service and were imported into the model. The diurnal patterns for MD and MDMM scenarios are shown in Figures 4.1 and 4.2 respectively (*Gold Coast Water_4, 2006*). Note that Commercial and Public sectors come under a single pattern.

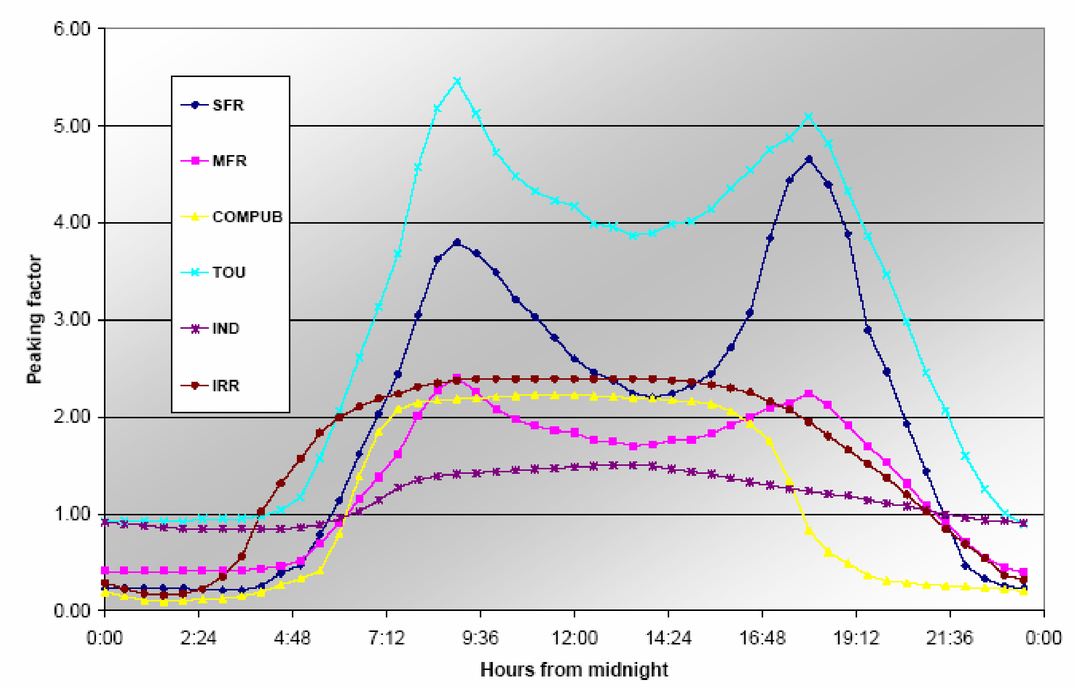


Figure 4.1: Diurnal patterns for maximum day water demands*

*SFR = Single Family Residential, MFR = Multi-Family Residential, COMPUB = Commercial & Public, TOU = Tourist, IND = Industrial and IRR =Irrigation.

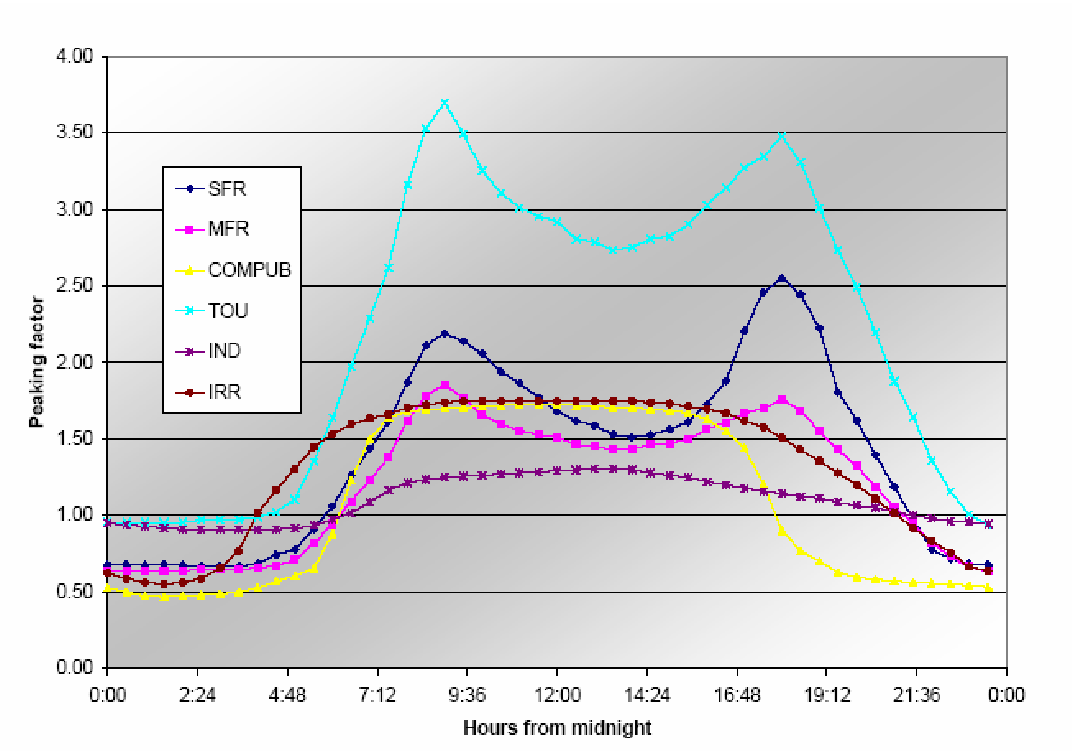
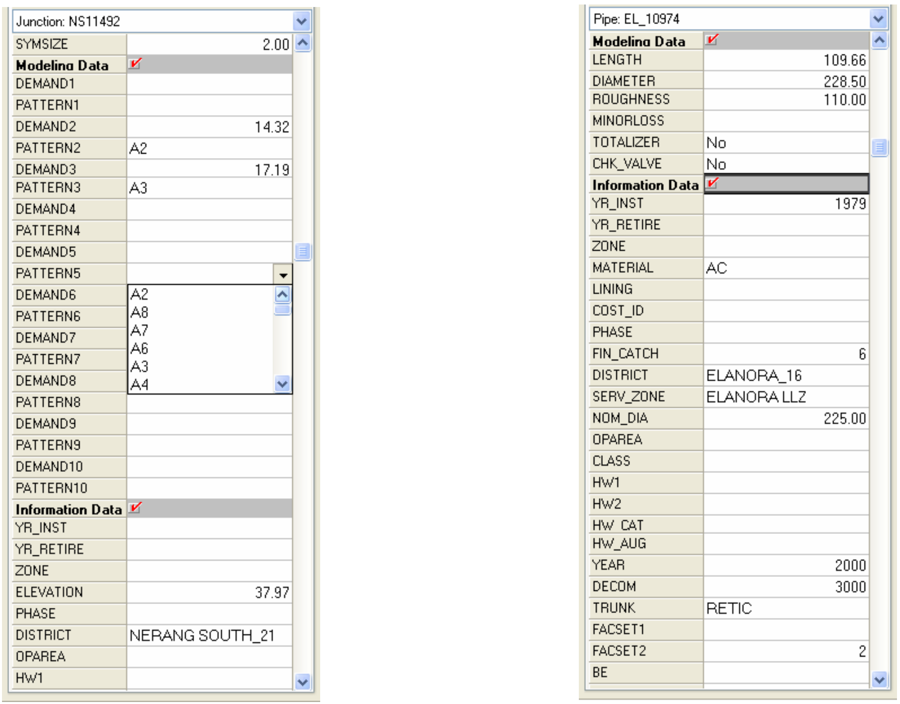


Figure 4.2: Diurnal patterns for mean day maximum month water demands*

To achieve the MD and MDMM diurnal pattern, an average peaking factor (PF) of approximately 1.9 and 1.45 is applied to the AD demands respectively. The peaking factors for maximum day demand and maximum hour demand have been determined for each development type by equating the percentage of peak rate of use relative to average rate of use *(Austin City Connection, 2001)*.

A H_2ONET screen shot shown in Figure 4.3 (a) illustrates how the model demands and patterns are assigned for each node in the junction attribute table. Note that it is possible to have more than one development type demand assigned to a single junction. The pipe attribute table is also shown in Figure 4.3 (b).

*SFR = Single Family Residential, MFR = Multi-Family Residential, COMPUB = Commercial & Public, TOU = Tourist, IND = Industrial and IRR =Irrigation.



(a) (b)

Figure 4.3: Screen shot of junction (a) and pipe (b) element attribute tables

The hydraulic calculations for the H_2ONET model are set up in the simulation options in Run Manager. This is where the flow units (i.e. L/s), head loss equations and characteristics including diffusivity, viscosity and vapour pressure are defined. To demonstrate this, another H_2ONET screen shot is illustrated in Figure 4.4. Other simulation options can be specified in the Demand, Quality and Energy tabs.

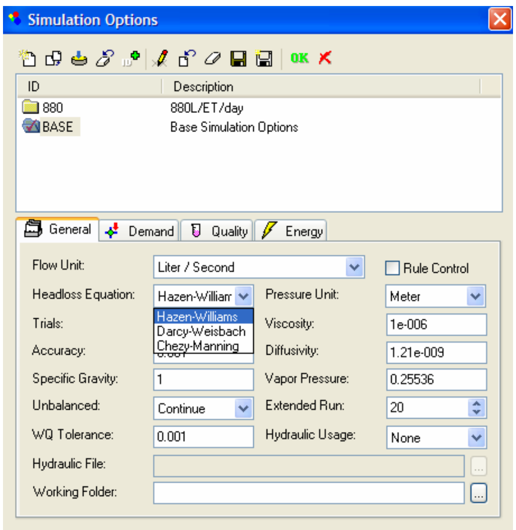


Figure 4.4: Simulation options within H_2ONET Run Manager

GCW adopt the Hazen-Williams (HW) head loss equation for all model simulations, the corresponding friction coefficient (C) is used as a pipe roughness parameter as shown in Figure 4.4 (b) under the modelling data heading. Note that a lower C value equates to higher friction or head loss according to the HW head loss formula shown in Equation 4.1 *(Chadwick, Morfett & Borthwick, 2004)*.

$$h_f = \frac{6.78}{D^{1.165}} \left(\frac{v}{C} \right)^{1.85} \qquad \text{(Eqn. 4.1)}$$

where h_f = head loss due to friction (m)

D = pipe diameter (m)

v = average pipe velocity (m/s)

Hazen-Williams (HW) head loss equation generally applies to smooth turbulent flow only and cannot be used for fluids other than water. Minor losses are caused by added turbulence in the system and occur at bends and fittings. However, in GCW's network model, minor losses are only applied to valves. In contrast, all 100 mm mains are given a reduced HW C coefficient of 100 (ultimately this depends on the pipe material i.e. C=130 for PVC pipes), this is to account for additional minor and pipe friction loss that may occur in old mains within the system. The model uses Equation 4.2 to compute the minor loss through a valve *(MWH Soft, 1996-2003)*.

$$h_v = \frac{0.0252 \quad KQ^2}{D^4} \qquad \text{(Eqn. 4.2)}$$

where h_v = Head loss through a valve

K = minor loss coefficient

Q = flow rate through the main (L/s)

The model boundary conditions are governed by the top and bottom water levels of the reservoirs and treatment plants. The model uses Equations 4.3 to calculate the change in reservoir level *(MWH Soft, 1996-2003)*.

$$\Delta y = \frac{Q}{A} \Delta t \qquad \textit{(Eqn. 4.3)}$$

where Δy = change in water level (m)

Q = flow rate in/out of tank (L/s)

A = cross-sectional area of the Tank (m^2)

Δt = time interval (sec)

4.2.2 HYDRAULIC SIMULATION MODEL

H_2ONET uses an extended period hydraulic simulator to solve a set of equations for each tank in the model. The process is based on the hybrid method *(Hamam and Brameller, 1971)* or the gradient algorithm *(Todini and Pilati, 1897)*. The equations are:

$$\frac{\partial y_s}{\partial t} = \frac{Q_s}{A_s} \qquad \text{(Conservation of water volume at a storage reservoir)} \qquad \textit{(Eqn. 4.4)}$$

$$Q_s = \sum_i Q_{is} - \sum_j Q_{sj} \qquad \textit{(Eqn. 4.5)}$$

$$h_s = E_s + y_s \qquad \textit{(Eqn. 4.6)}$$

The following set of equations are solved for each node (k) and each link (between *i* and *j*):

$$h_i - h_j = f(Q_{ij}) \qquad \text{(Energy loss or gain due to flow within a link)} \qquad \textit{(Eqn. 4.7)}$$

$$\sum_i Q_{ik} - \sum_i Q_{kj} - Q_k = 0 \qquad \textit{(Eqn. 4.8)}$$

Where the unknown quantities are: y_s = height of water stored at node *s* (m)

Q_s = flow storage at node *s* (L/s)

Q_{ij} = flow in link connecting *i* and *j* (L/s)

h_i = hydraulic grade line elevation at node *i* (elevation head + pressure head) (m)

and the known constants are: A_s = cross-sectional area of storage nodes (m^2)

E_s = elevation of node *s* (m)

Q_k = flow consumed (+) or supplied (-) at node *k* (L/s)

$f(.)$ = functional relationship between head loss and flow in a link

Equations 4.5 to 4.8 represent the conservation of water volume at pipe junctions. Hydraulic balancing occurs when Equations 4.7 and 4.8 are solved for Q_{ij} and h_i using 4.6 as a boundary condition and the modified Newton-Raphson iterative technique. The iterations finish and a steady state hydraulic solution is achieved when the flow rates between two iterations reaches a specified accuracy.

The H_2ONET hydraulic time step generally adopted by GCW is half hour periods. However, this can be altered to suit the modelling task. Further, the duration of the simulation may change depending on the modelling task, run times include 24, 72 and 120 hr durations.

4.2.3 WATER QUALITY MODEL

The dynamic water quality simulator in H_2ONET tracks the movement of a dissolved substance through the network over time via three processes. These processes are based on conservation of mass and reaction kinetics and include advection in pipes, mixing at nodes and reservoirs, and the kinetic reaction mechanism.

A substance dissolved in the bulk water will travel through a length of pipe at the same average velocity as the water and react with its surroundings at a specified rate. Rossman et al (1993), and Rossman and Boulos (1996) expressed that longitudinal mixing is generally not important under network operating conditions, meaning, there is no intermixing occurring between adjacent parcels of water flowing through the pipe. Advective transport is represented by Equation 4.9 *(MWH Soft, 1996-2003)*.

$$\frac{\partial C_i}{\partial t} = -v_i \frac{\partial C_i}{\partial x_s} + r(C_i) \qquad \text{(Eqn. 4.9)}$$

where C_i = concentration in pipe *i* as a function of distance *x* and time *t* (mg/L)

v_i = average flow velocity in pipe *i* (m/s)

r = rate of reaction as a fraction of concentration

Mixing is taken to be complete and instantaneous at the junction receiving flow from two or more pipes. Hence, the concentration leaving the junction is equivalent to the flow-weighted sum of concentrations from inflowing pipes. For a particular junction *k* the equation is:

$$C_{i|x=0} = \frac{\sum_{j \varepsilon I_k} Q_j C_{j|x=L_j} + Q_{k,ext} C_{k,ext}}{\sum_{j \varepsilon I_k} Q_j + Q_{k,ext}} \qquad \text{(Eqn. 4.10)}$$

where I = link with flow leaving node *k*

$C_{i|x=0}$ = concentration at start of link *i*

I_k = set of links with flow into *k*

$C_{j|x=L_j}$ = concentration at end of link *i*

L_j = length of link *j*

Q_j = flow into link j (L/s)

$Q_{k,ext}$ = external source flow entering the network at junction *k*

$C_{k,ext}$ = concentration of external flow entering junction *k*

Similarly the model can simulate four different mixing regimes including:

- Complete mixing
- Two Compartment mixing
- FIFO (first-in-first-out) plug flow; and
- LIFO (last-in-first-out) plug flow

During fill and draw conditions (a process by which most reservoirs operate), it is assumed that instantaneous and complete mixing occurs with water already in the tank. Complete mixing suggests that the concentration is a blend of the existing water in the reservoir and the water entering the tank. At the same time the concentration of the water in the tank is shifting. Equation 4.11 demonstrates complete mixing *(MWH Soft, 1996-2003)*.

$$\frac{\partial (V_s C_s)}{\partial t} = \sum_{i \varepsilon I_s} Q_i C_{i|x=L_i} - \sum_{j \varepsilon o_s} Q_j C_s + r(C_s) \quad \textit{(Eqn. 4.11)}$$

where V_s = volume of storage at time t

C_s = concentration within the storage tank

I_s = set of links providing flow into the storage facility

O_s = set of links withdrawing flow from the storage facility

H_2ONET can monitor water quality reactions as a substance passes through the system. The predominant reactions occur within the bulk water and the boundary layer at the pipe wall. According to Equation 4.12 an instantaneous rate of reaction (*R* in mass/volume/time) is assumed to be concentration-dependent. The bulk reaction is given in Equation 4.12 *(MWH Soft, 1996-2003)*.

$$R = k_b C^n \quad \textit{(Eqn. 4.12)}$$

where k_b = bulk reaction rate coefficient and has units of concentration raised to the (*1-n*) power divided by time

C^n = reactant concentration (mass/volume) (mg/L)

n = reaction order

Pipe wall reactions can significantly alter the water quality at the pipe wall interface. Corrosion of pipe walls, deposition of organic and inorganic compounds, sedimentation of particulates and micro-organism growth, formation of scale and slime layers, and leaching of pipe materials all influence chlorine reactions at the boundary layer, and therefore water quality. The reactions at the pipe wall are directly related to the concentration in the bulk flow, and are expressed by Equation 4.13 *(MWH Soft, 1996-2003)*.

$$R = \left(\frac{A}{V}\right) k_w C^n \qquad \textit{(Eqn. 4.13)}$$

where k_w = wall reaction coefficient

$\frac{A}{V}$ = surface area per unit volume (m/L)

In summary, there are three coefficients that are required by H_2ONET which describe reactions within reservoirs and pipes, these include k_b, k_w and k_f. The latter is a mass transfer coefficient and is calculated automatically by H_2ONET. The wall coefficient correlates to pipe age and temperature. As metal pipes age, encrustation and tuburculation of corrosion products increase, restricting the flow, the Hazen-Williams C coefficient is reduced. Evidence suggests that as a pipes roughness increases so does the reactivity of its wall with chlorine. H_2ONET allows a pipes k_w factor to be a function of its roughness, such that equations 4.14 is applied and specific to Hazen-Williams head loss formula. Other equations apply if Darcy-Weisbach or Chezy-manning head loss formulas are used.

$$k_w = F / C \qquad \textit{(Eqn. 4.14)}$$

where F = wall reaction – pipe roughness coefficient

Typical values for k_b ("Global Bulk" in H_2ONET) and k_w ("Global Wall" in H_2ONET) are –1.0 and -0.5 respectively, where the negative sign indicates decay *(MWH Soft, 1996-2003)*. These values are typical of chlorine decay and have been used widely around the world. The values that will be adopted in the Gold Coast model will be based on field tests undertaken at various reservoir sites if appropriate. This is discussed later in Chapter 6.

4.3 CHAPTER SUMMARY

The fundamental mathematics of the H_2ONET hydraulic model and the water quality model have been described. These are the two frameworks required to model the movement of chlorine through the distribution system. For these models to accurately represent the existing network and future scenarios, it is imperative that the model is well calibrated for the study area. Detail on the study area will be discussed in the next chapter.

CHAPTER 5. STUDY AREA

5.1 STUDY AREA

The Gold Coast water distribution system is essentially divided through the middle of the city at the Nerang River and joins the ocean in the east at Surfers Paradise, making up the northern and southern water supply regions. The southern region water network is the chosen study area for the project and has previously been shown in Chapter 2. Figure 2.10 shows the location of the 59 water quality monitoring stations with respect to 30 permanent DMAs within the study area.

The southern region currently receives water from both Molendinar and Mudgeeraba WPPs. Molendinar, however, only supplies Nerang South and Worongary WSDs while Mudgeeraba feeds the remaining districts. Appendix B provides a schematic of how the southern region operates showing the direction of flow and how the system will operate when the desalinisation is activated. It also shows the interconnectivity of each WSD and WSZ within the network.

The southern region water supply system contains approximately 15,000 pipes, 14,600 nodes, 45 reservoirs at 35 reservoir sites and 73 pumps in 34 pump stations. Table 5.1 provides details on the number of properties and the water consumption data for each planning horizon until PSD (2056) for each WSD in the southern region. This information has been obtained from Gold Coast Water's Infrastructure Demand Model. Demands were initially obtained in ET where they were converted to ML/day using GCW's average day demand distribution factor of 880 litres/ET/day.

Table 5.1: Water supply district consumption data

Water Supply District	Number of Properties	2006 Demand (ML/d)	2011 Demand (ML/d)	2021 Demand (ML/d)	PSD Demand (ML/d)
Nerang South	10131	9.807	10.60	11.75	15.60
Gilston	1219	1.373	1.557	1.662	2.170
Worongary	13587	23.82	24.84	29.56	34.87
Mudgeeraba	4799	4.864	5.259	5.905	8.139
Robina	13346	13.53	15.56	17.88	27.32
Burleigh	9282	10.22	10.39	11.17	14.22
Reedy Creek	8952	7.293	7.974	9.335	14.68
Elanora	5175	5.770	6.357	7.754	8.964
Currumbin Waters	9787	8.938	9.202	9.929	12.54
Currumbin	3496	4.404	4.731	5.910	7.267
Coolangatta	1163	3.563	3.736	4.596	5.198
Total	**80.937**	**93.61**	**100.2**	**115.4**	**150.9**

The data in Table 5.1 was used to set up GCW's hydraulic network model. Figure 5.1 presents a screen shot of the H_2ONET hydraulic model of the study area.

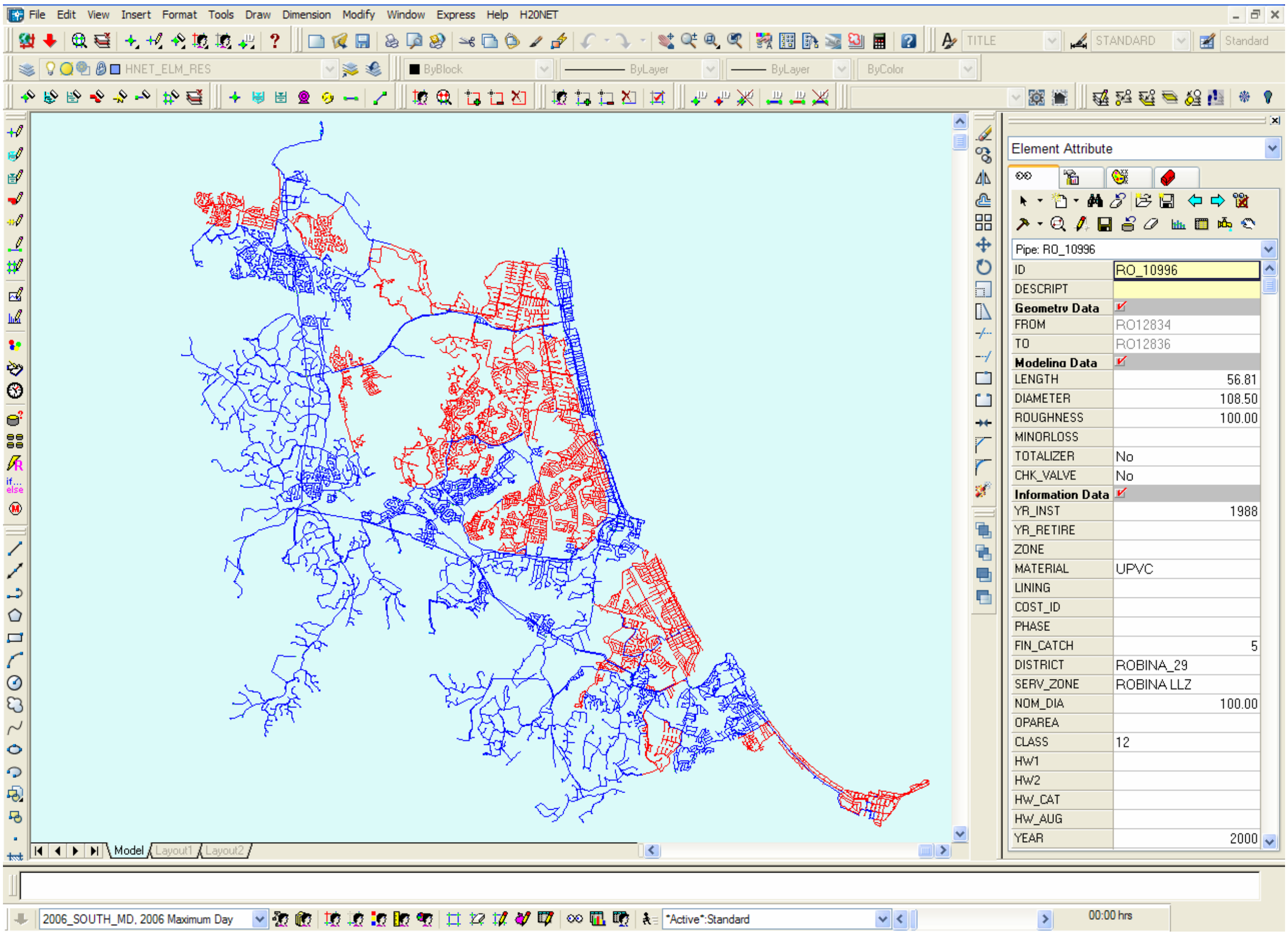


Figure 5.1: H_2ONET Screen shot of the southern region hydraulic network model

It is anticipated that a chlorine optimisation study will be undertaken for the northern region (Phase 2) following the completion and success of the southern region (Phase 1) study. The southern region was designated as Phase 1 of the 2-phase project as it is somewhat smaller geographically and simpler hydraulically with fewer pump stations and less complicated water supply zones. Similarly the only current chlorination for the southern region comes from Mudgeeraba and Molendinar WPPs, where as the north has two existing booster facilities. For these reasons, it was decided to make the large-scale calibration procedure less complex and less time consuming. The strict time frame for the project (final project report to be delivered by November 2007) was also a factor why the southern region was chosen as opposed to the northern region.

CHAPTER 6.　MODEL PREPARATION

6.1 INTRODUCTION

Chapter 6 will discuss model preparation. In order for the model to be an accurate tool in planning and optimisation it must be calibrated to actual field measurements and verified. This is mandatory as part of the model preparation process. A sensitivity analysis of model global parameters is also undertaken to demonstrate the sensitivity of model pressure relative to flow.

6.2 MODEL PREPARATION

To achieve the most accurate results and solutions, it is imperative to have a model that contains the most recent infrastructure. Therefore, before the model can be calibrated, it must be updated to illustrate the most recent changes in the network. The primary change the network underwent recently is the implementation of the DMAs for the PLMP. The closure of boundary valves has caused the redirection of flow influencing the pressure in the system. Hence, all existing permanent DMAs were entered into the model database by creating a domain, adding new fields (“DMA_ID” and “DMA STATUS”) in the pipe attribute table and querying all elements with the new fields into a query set. Similarly, boundary pipes were categorised into an additional query with a closed control set and applied to all planning horizons. The permanent DMAs in the model are presented as the red elements in Figure 5.1 in Chapter 5. Further, the model was also updated to capture the recent changes including:

- 06-07/07-08 Capital Works augmentations
- PLMP augmentations scheduled for 07-08
- Reservoir and pump operation
- Desalinisation pipeline alignment and network off takes (this will only be taken into account for the optimisation model runs to represent future operation of the network)

6.3 CALIBRATION/VALIDATION METHODOLOGY

The methodology adopted for the network model calibration involves matching model flow and pressure with actual field data as best as possible. It was initially planned to undertake calibration and validation in two phases, where Phase 1 would focus on matching model hydraulic data while Phase 2 would consider refining chlorine decay coefficients. However, the ambiguity of the DPD chlorine field test method meant that Phase 2 calibration could not be undertaken due to the unknown time of the test for samples in GCW's historical water quality database. The model provides 'real-time' data so it is impossible to accurately match this chlorine data. However, a comparative assessment will be made between model and field chlorine residuals and discussed in Section 6.5.2 as Phase 2 of the validation process.

Phase 1 of calibration involves altering the model flows and boundary conditions to match those determined in the field for a particular day and calibrating against hydraulic head. It is vital that the model is updated and representative of the existing network when undertaking calibration.

The following steps were undertaken to obtain the relevant field data for both calibration and validation:

1. Pressure loggers were installed at specified locations on the trunk main system. Undertaken by GCW Operations and Maintenance staff. Figure 6.1 (a) and (b) show the Greenspan PS310 loggers (pressure range 0 – 100 m) used for the data collection (installed 30/05/07). Loggers were installed for a minimum period of two weeks to obtain daily diurnal pressure patterns, readings were taken at 10-minute intervals. The loggers were retrieved from the network on the 14/06/07 and 15/06/07. The pressure graphs for the loggers are shown in Appendix C-3

(a)

(b)

Figure 6.1: Trunk main pressure loggers

2. Extract reservoir (% full) and pump (reservoir level controls) data from the existing SCADA system (RADTEL) at Molendinar WPP for the southern region between 31/05/07 and 15/06/07. A summary of the initial reservoir levels for the 31/05/07 are provided in Appendix C-2.
3. Extract PRV information (upstream/downstream pressure and flow) corresponding to existing permanent DMAs within the southern region; extracted from the available resources at Molendinar WPP.

A concurrent reservoir, pump and PRV field data set is crucial in matching actual data with the model output. Data obtained on the 31st may 2007 was chosen for model calibration as it represented the majority of the field locations with a more comprehensive data set.

4. Field data was organised for 31/05/07 to be compatible with model output; this was so model and field comparison could be undertaken quickly and efficiently.
5. Collect Gold Coast's water consumption data for the 31/05/07 (138.89 ML/d - extracted from the GCW's database). Apply this to the model by changing the global demand multiplier to manipulate the AD demand pattern. The calculation is as follows:

	Global Demand Factor	= Field/Model x 100
		= 138.89/221.13 x 100
		= 62.8%
Hence;	Global Demand Multiplier	= 0.628

6. Apply field head patterns to Molendinar and Mudgeeraba reservoirs using fixed head pattern reservoirs in the model.
7. Apply the initial levels to all the remaining reservoirs in the study area where data are available. Similarly, apply all the pump operating levels.
8. Apply the field flow at the PRV locations to the model by adjusting the demand pattern within the DMA. Do this by altering the main demand for the DMA by dividing it by the field flow pattern until a good correlation (±5%) is made. *Note: only those DMAs (PRVs) with a data record (shown in Appendix C-1) could be used and the flow altered in the model.* A graph matching model flow to field flow for Jura Parade DMA supply feed (total

difference of 1.26%) is provided in Appendix C-4 as an example. The remaining figures will not be shown in the Appendices.

9. Run the model for a single day with the new global demand factor and calibrate the trunk system. Tweak the pipe friction coefficient (Hazen-Williams C) according to pipe age and material to increase correlation between field and model pressure where appropriate. (Note: field flow were not available for the trunk system due to the major work required for meter installations and their associated cost)

If the difference between field and model pressures are less than ±10% for the trunk system and reservoirs, the reticulation system can then also be calibrated where required. Calibration of downstream distribution system can be undertaken by adopting the following:

10. Adjust pipe roughness coefficients (Hazen-Williams C) to match actual to model results, making changes according to pipe age and material.
11. Continuously run the model until an acceptable error (±10%) is achieved between the actual data and model output.

Figure 6.2 illustrates the location of the 12 loggers and 28 PRV chambers where pressure and flow data were collected within the study area. Appendix C-1 provides a table with the address of all test sites. However, not all sites provided accurate or available data for the chosen calibration period, these are identified in the table.

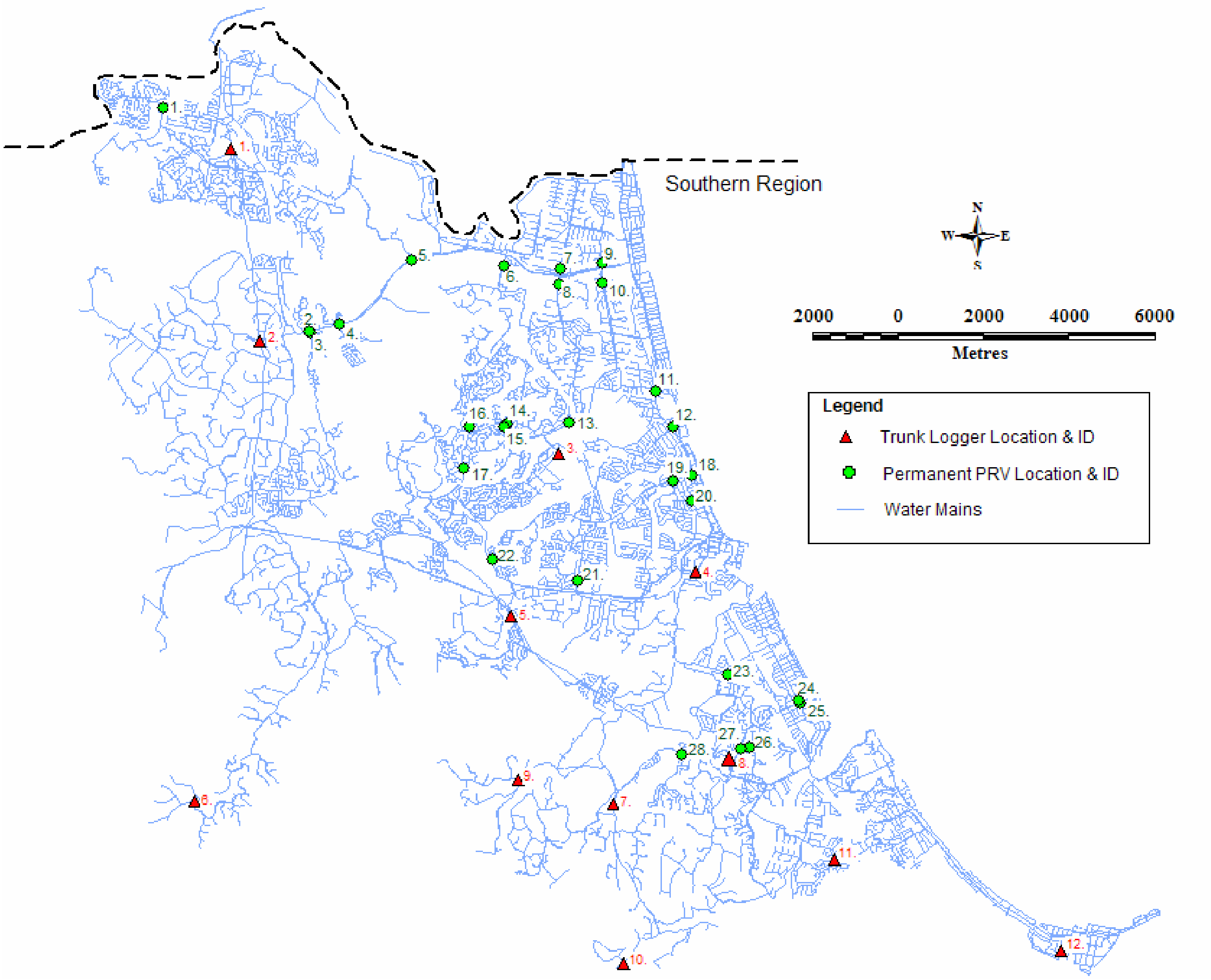


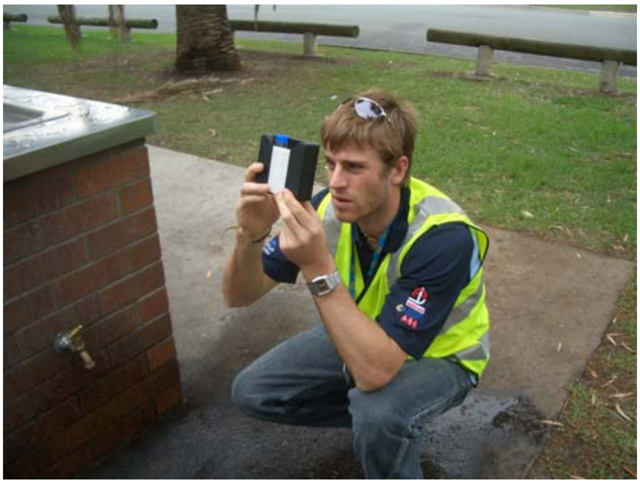
Figure 6.2: Trunk main pressure logger and DMA PRV logger locations

Collated field chlorine residuals consist of an average of all samples taken between July 2006 to February 2007 from GCW's historical water quality database, and chlorine data collected on the 7th and 8th June 2007. The model residuals will be deduced using the model from Phase 1 and simulated over a 3 day period. The following steps were undertaken as a comparative assessment for Phase 2 of the validation.

12. Calculate the average chlorine residual for the available data between July 2006 and February 2007. Test all water quality HACCP test sites for free chlorine using the DPD test (undertaken on 7-8/06/07)
13. Run the model from Phase 1 adopting the same boundary conditions with the same fixed head reservoir patterns at Molendinar and Mudgeeraba. Utilise the bulk and wall chlorine decay coefficients determined in Section 6.3.1 and compare the results.

Figure 6.3 (a) and (b) show a DPD chlorine test being undertaken at Murlong Crescent, Palm Beach (HACCP site – tap on BBQ in park).

(a)



(b)

Figure 6.3: DPD chlorine test undertaken at Murlong Crescent, Palm Beach

The chlorine data including both the average (July 06 – Feb 07) data and the sampled data (7-8/06/07) are included in Appendix D-1 and D-2.

6.3.1 DETERMINATION OF CHLORINE DECAY COEFFICIENTS

Prior to any chlorine modelling the global bulk and global wall decay coefficients must be determined. This will improve the accuracy of the chlorine distribution within the model for the Gold Coast system. Field investigations were undertaken to determine these coefficients.

6.3.1.1 METHODOLOGY

Two types of investigations were carried out, the methodology is described below.

1. Measure chlorine concentration from the inlet and outlet of two major reservoirs that receive relatively high chlorine concentrations (typically Molendinar reservoir 4 and Mudgeeraba reservoirs 1 and 2) and calculate the change in concentration across them. Note: data may be obtained from the WPP SCADA system. Determine the bulk decay

coefficient using first order kinetic law by applying Equation 3.1 ($C = C_0^{-k_bT}$; $-k_b = \frac{Log\frac{C}{C_o}}{T}$), where T is the residence time in the reservoir and is determined from daily flow rates.

2. Select three reservoirs and dose sodium hypochlorite solution (10%). The procedure follows:
   - Dose chlorine relative to reservoir volume to achieve approximately 1 mg/L and measure the concentration after initial mixing.
   - Flush a hydrant downstream of the reservoir until a chlorine residual is measured.
   - After a given time period take a second chlorine measurement from the reservoir and from the hydrant. The bulk coefficient of decay can be determine using the reservoir readings and Equation 3.1 while the wall coefficient can theoretically be calculated using the readings from the hydrant and Equation 3.1.

Ideally the investigation should be conducted on reservoirs containing a variety of configurations including single inlet/outlet, separate inlet/outlet and one in a pumped HLZ to demonstrate their effects in determining the decay coefficients. Similarly, it is favourable to undertake the experiment in chlorinated and unchlorinated areas of the network to demonstrate how the coefficients vary relative to pre-existing chlorine (similar to a booster scenario) and in the absence of chlorine. This would ultimately allow a better approximation of k_b and k_w, unfortunately these conditions were difficult to achieve.

All selected reservoirs for investigation 2 were located in small HLZs fed by booster pumps to avoid affecting a large number of customers. These reservoirs were also favourable as they require less chlorine for their small size and generally have longer retention times. Further, the inflow to the smaller HLZ reservoirs could be prevented during the course of the experiments to avoid any changes in reservoir volume and chlorine concentration. It is unlikely that any of the selected reservoirs will contain a pre-existing chlorine residual due to the current low levels within the network. The operating conditions of the chosen reservoirs are described in Table 6.1.

The following assumptions are made for both investigations:

1. When taking chlorine measurements from the reservoir to determine the bulk reaction coefficient, the wall reaction decay was considered negligible ($k_w << k_b$).
2. When taking chlorine measurements from the hydrant (i.e. the pipe) to determine the wall reaction coefficient, the bulk reaction decay was considered negligible ($k_b << k_w$).

Assumption 1 above is made based on the small ratio of the reservoir wetted perimeter to reservoir volume. Assumption 2 is based on the ideology behind H_2ONETs water quality model where Rossman et al (1993), and Rossman and Boulos (1996) expressed longitudinal mixing in pipes is not applicable i.e. bulk reaction is negligable. Also, it is virtually impossible to determine the actual advective transport of a chemical in a pipe under field conditions due to fluctuations in flow, and the volume of water to wetted perimeter ratio on the selected systems (all 100 mm mains). The assumptions are fair.

6.3.1.2 RESULTS

The results of the above investigations are summarised in Table 6.1. The dosing volumes were derived using the sample dose calculation demonstrated in Section 2.3.2 of Chapter 2, to achieve a desired initial concentration of 1 mg/L.

Table 6.1: Bulk and wall reaction factor coefficients

Investigation	Reservoir	Capacity (ML)	Condition	Volume Dosed (L)	Initial Concentration (mg/L)	Final Concentration (mg/L)	Residence Time (hrs)	Decay Coefficient (k)
1	Molendinar 4	30	Separate Inlet/Outlet, chlorine residual	Flow Paced Dosing	1.22	0.8	Not Available	k_b= N/A
	Mudgeeraba Reservoirs	6.6	Separate Inlet/Outlet, chlorine residual	Flow Paced Dosing	1.15	0.85	0.378	k_b= -0.35

Table 6.1 cont'd

Investigation	Reservoir	Capacity (ML)	Condition	Volume Dosed (L)	Initial Concentration (mg/L)	Final Concentration (mg/L)	Residence Time (hrs)	Decay Coefficient (k)
2	Windermere, Mudgeeraba (HLZ)	0.29	Pumped Separate Inlet/Outlet, no chlorine residual	3.0	Reservoir mixing was not effective No results			- k_b = N/A
								- k_w = N/A
	Aylesham Dr, Bonogin (HLZ)	0.12	Pumped Single Inlet/Outlet, no chlorine residual	0.9	Reservoir mixing was not effective No results			- k_b = N/A
								- k_w = N/A
	Glenrowan Dr, Gilston (HLZ)	0.022	Pumped, Single Inlet/Outlet, no chlorine residual	0.2	0.8	0.5	0.383	k_b = -0.53
					0.65	0.5	0.417	k_w = -0.27

Glenrowan Drive HLZ reservoir (22 kL) is presented in Figure 6.4. Flushing of the downstream hydrant is presented in Figure 6.5 for further illustration.

Figure 6.4: Glenrowan Dr HLZ reservoir

Figure 6.5: Hydrant being flushed downstream of Glenrowan Dr HLZ reservoir

6.3.1.3 DISCUSSION

The residence time for Molendinar reservoir 4 was unavailable as the flows were only calculated for the treatment plant as a whole which provides Southport West and Molendinar reservoirs together, hence it was not possible to calculate exact flow contributions between the two systems to determine the bulk chlorine decay coefficient.

The chlorine data for Mudgeeraba reservoir was collected from the treatment plant SCADA system. The concentration drop across the reservoir could be calculated as the system only has two reservoirs which feed directly from the clear water tanks at the plant. Chlorine is monitored at the outlet of the clear water tanks and at a pump station a kilometre down stream of the reservoirs. The 1068 meters of 850 mm main is assumed to have negligible wall decay due to the volume of water passing through the main on a daily basis. An average chlorine concentration was chosen for a period between 2am and 7am on the 13/05/07 where a constant flow leaving the reservoirs of 600 L/s was provided. The constant flow rate through the reservoirs allowed the residence time to be determined by calculating the average volume of water in the system between the upstream and downstream test points between 2am and 7am. The calculation of residence time is illustrated below:

Q = m^3/s

0.6 = [(Total Area of 1068 m of 850 mm main)+(Average Volume of Mudgeeraba reservoirs 1 and 2)]/Time (sec)

Time (sec) = (712.98 + 105.3) / 0.6

= 1363.8 sec

= 0.379 hrs

The bulk decay coefficient can be calculated using the decay formula described above.

k_b = log (1.15/0.85)/time (hrs)

k_b = 0.131/0.379

k_b = 0.347

Chlorine mixing within Windermere and Aylesham Drive reservoirs appeared to be incomplete over the duration of the test causing only partial dispersion of the chlorine within the reservoirs. This provided misleading concentrations in the grab samples. Windermere in particular experienced very high chlorine concentrations at the hydrant which indicated mixing at the reservoir did not occur successfully. Since a stable chlorine reading was not achieved at Aylesham Drive reservoir, the hydrant downstream was not opened.

The test undertaken on Glenrowan Drive was the only trial which achieved the anticipated outcome. The results show the wall decay factor to be approximately half the bulk decay factor, which supports the trend given by the default chlorine decay coefficients provided in *MWH Soft (1996-2003)* H_2ONET user guide.

Investigation 2 did not prove to work well when the volume of the reservoir became rather large. It is supposed that longer mixing times are required to achieve a uniform concentration within the reservoir.

The 3 trials did not generate the desired outcome with only two of the five tests producing a bulk decay coefficient and one of three tests producing a calculated wall decay coefficient. Since the results are somewhat vague, the default chlorine decay coefficients provided by *MWH Soft (1996-2003)* will be utilised. These are of -1 and -0.5 for the bulk and wall respectively.

6.4 CALIBRATION

Results associated with Phase 1 of the calibration are discussed below.

6.4.1 PHASE 1

Table 6.2 presents the results and accuracy of the model pressure with reference to logger and PRV data. The logger numbers correspond to those shown on Figure 6.2.

Table 6.2: Logger and PRV pressure comparison - model and field calibration results

Logger Map Number	Location Description	Field Data Available	Difference in Model and Field Upstream Pressure (% Ave for 24 hrs)
1	17 Pappas Way Highland Park	YES	2.6
2	Off Nash Rd, upstream or Worongary reservoir connection	NO	-
3	University Dr, Bond Univsity Entrance, Southport	YES	3.6
4	End of Page St, Before connection into Burleigh reservoirs	YES	6.1 (logger incorrect)
5	Upstream of Reedy Creek LLZ connection	YES	28.4 (logger incorrect)
6	Aylesham Dr, Bonogin	YES	3.4
7	Upstream of Tullebudgera-Connection Rd PS	YES	18.6 (logger incorrect)
8	Just Upstream of Elanora Reservoirs	YES	38.6 (operating condition incorrect)
9	Monday Dr, upstream of Myall Mundi Reservoir	YES	13.5 (elevation difference)
10	Currumbin Creek Rd, Currumbin Waters	YES	5.8
11	Bienvenue Dr, Currumbin Waters	NO	-
12	Stapylton Street	NO	-
Average Model Error			**11.23%**

Table 6.2 cont'd

PRV Map Number	Chamber Name	Field Data Available	Difference in Model and Field Upstream Pressure (% Ave for 24 hrs)
1	Winderadeen Crt	NO	-
2	Jura Parade	YES	0.5
3	Bourton Rd	YES	2.2
4	Gooding Dr	NO	-
5	Palm Meadows Dr	YES	0.4
6	Fairway Dr	YES	1.4
7	Rio Vista Boulevard North (Feed 1)	YES	0.8
8	Rio Vista Boulevard South	YES	0.2
9	Rio Vista Boulevard North (Feed 2)	YES	0.1
10	Sunshine Boulevard (Feed 1)	YES	2.0
11	Sunshine Boulevard (Feed 2)	YES	0.8
12	Gold Coast Highway	YES	1.2
13	Ben Lexcen Place	YES	0.8
14	Ron Penhaligon North	YES	0.9
15	Ron Penhaligon South (Feed 1)	YES	2.9
16	Robina Parkway	YES	1.2
17	Ron Penhaligon South (Feed 2)	YES	2.2
18	Gold Coast Highway	NO	-
19	Deodar Dr	NO	-
20	Deodar Dr	YES	2.1
21	Mattocks Rd	YES	6.8
22	Varsity Sound	YES	6.0
23	Nineteenth Ave	NO	-
24	Phillipine Parade North	YES	0.5
25	Phillipine Parade South	YES	0.8
26	Doubleview Dr	NO	-
27	Doubleview Dr	NO	-
28	Guineas Creek Rd	NO	-
Average Model Error			**1.69%**

These results are represented graphically in Appendix C-5. It was made apparent after reviewing the graphs and consulting with the appropriate field staff that loggers 4, 5 and 7 were miss calculated making the data redundant, nevertheless the graphs have been included in Appendix C-5.

Model and field results associated with Logger 8 also demonstrated substantial error. However, this was not deemed to be a model inaccuracy as the model pattern shows a similar diurnal response. It was established that the difference in pressure is due to the field operating conditions of the Elanora reservoirs (they fill by bleeding a valve), which causes an increase in the upstream pressure. Similarly, the error associated with logger 9 is assumed to be predominantly due to the elevation difference between the model node and field hydrant where the logger was located. The patterns shows a well correlated diurnal trend which indicate that model flows are fairly similar for the area (model and field flows could not be directly coordinated as there are no DMAs located up or downstream). Due to this uncertainty pipe friction coefficients were not altered.

It is evident that there is generally a good relationship between the model and field diurnal patterns for the trunk system. The error associated with logger 8 is due to field specific operations and can therefore be disregarded. A total average error for the trunk loggers calculated to 5.76%. This appears relatively accurate given that the model flows in the trunk could not be coordinated with those in the field. However, the reservoirs demonstrated to be operating correctly. Due to the uncertainties associated with some of the loggers, the trunk main pipe coefficients can not be modified to improve model head loss.

The relationship between the model and field results at PRV locations demonstrated very high accuracy with an average model error of 1.69%. This lies within the GCW's desired $\pm$10% model accuracy threshold and therefore the pipe friction coefficients will not be adjusted. This level of accuracy is not surprising given the flows through the system are operating at 62.8% of AD conditions, where head loss due to friction is almost negligible and pressures more or less static. During low demand periods such as this, it is very difficult to calibrate a network model. Therefore, it can be assumed that the model is accurate and adequately predicts field pressure throughout the network. Ideally the calibration should be undertaken in peak demand period where model error is potentially much higher.

Model efficiency can be tested statistically to verify the behaviour of network pressure. The correlation coefficient (R^2) is a useful tool to assess the competency of the data, it indicates the degree of association between the model output and field measurements. The R^2 value has a maximum of 1 which represents 100% correlation, from which a value of 0.9 and above demonstrates a very strong correlation while a value below 0.4 indicates a poor correlation. An spreadsheet analysis was conducted on model and field pressure data. Figure 6.6 is provided as an example and illustrates the scatter plot and the R^2 value for Fairway Drive PRV data set.

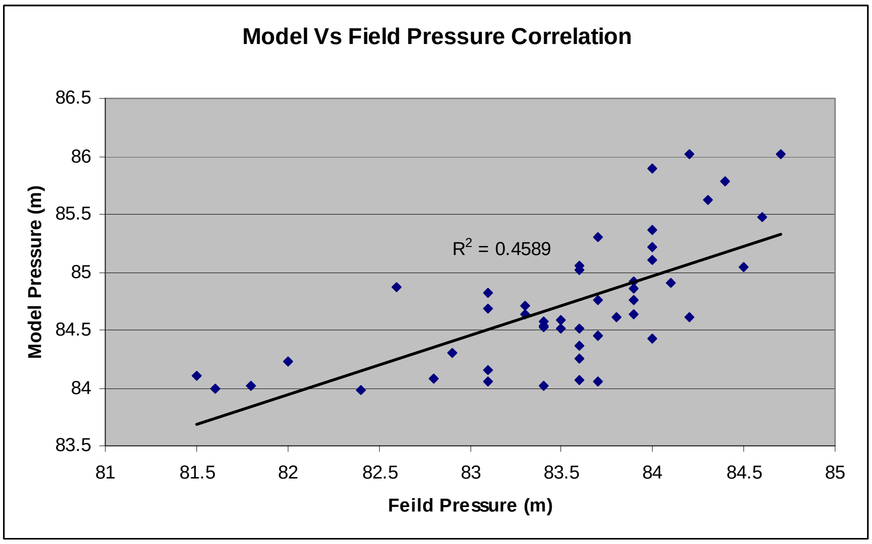


Figure 6.6: Correlation plot for model and field pressure for Fairway Drive PRV

The figure above shows a relatively poor correlation between the two data sets. However, this is fair given that the flow is not constant at each point in time. Having only 48 data points may also minimise the accuracy of the correlation. A Kolmogorov-Smirnov Test can be conducted to confirm this correlation. This test determines whether two data sets differ significantly and makes no assumptions about the distribution of the data (that is - it is non-parametric and distribution free). Fairway Drive data set was entered into the online KS-test, the results are displayed in Appendix E. A cumulative fraction plot of the data is provided in Figure 6.7. The cumulative fraction for any number of x, is the fraction of the data that is strictly smaller than x. It provides a visual indication of the relative distribution of the data (*Kolmogorov-Smirnov Test, 2006*).

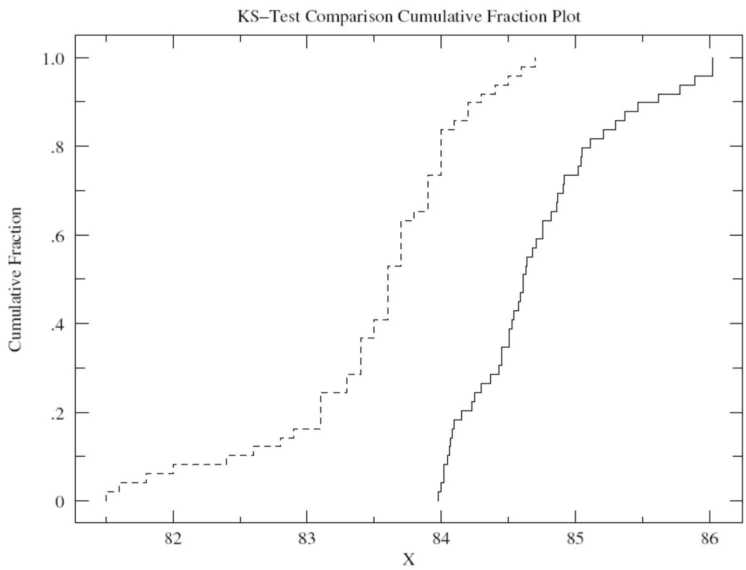


Figure 6.7: Cumulative Fraction Plot produced by the online KS-Test.

Figure 6.7 shows that for most values of any *x*, the fraction of the model pressure (line on the right) that is strictly less than *x* is clearly less than the fraction of the field pressure that is less than *x*. This demonstrates that the model pressures are larger than the field pressures for the same cumulative fraction. The model pressure and field pressures are significantly different but normally distributed, meaning the model has poor efficiency but a related diurnal patterns. Although, the difference in pressure between the two data sets is relatively minor, which, from an operational perspective, demonstrates high model performance, statistically, it does not show to be true.

Given the above findings, and from an operational and engineering perspective, the model can be considered an indicative tool and sufficiently accurate for the purpose of network planning and design. The majority of the model diurnal pressure patterns are analogous and lie within a few metres of the field. Statistically the model has demonstrated to be inefficient, however, for the purpose of this study we have assumed the model is operationally sound based on ±10% model error. Further statistical analysis and refinement of the model leaves opportunity for later work as it is very difficult to calibrate the model under such low demands currently experienced in the system. It would be most favourable to undertake network calibration during peak day conditions where model and field discrepancies are more likely to occur. The model will be validated with an alternative date set to verify the results provided in the calibration run.

6.5 VALIDATION

The validation procedure adopts the same methodology undertaken for the calibration. Data from the 9/06/07 will be deployed to validate Phase 1; hydraulic pressure. Rather than comparing all field 40 sites, strategic selection of 3 loggers and 5 PRVs across 4 Water Supply Districts will be used to compare model and field pressure. Phase 2 of the validation will compare average chlorine data collected between July 2006 and February 2007 and on the 7th and 8th of June 2007.

6.5.1 PHASE 1

Validation results for Phase 1 accurately supported those achieved for calibration. The accuracies between the model and field are demonstrated in Table 6.3.

Table 6.3: Logger and PRV pressure comparison - validation model and field results

Logger Map Number	Location Description	Field Data Available	Difference in Model and Field Upstream Pressure (% Ave for 24 hrs)
1	17 Pappas Way Highland Park	YES	5.485
6	Aylesham Dr, Bonogin	YES	3.396
9	Monday Dr, upstream of Myall Mundi Reservoir	YES	14.705
Average Model Error			**7.86%**
PRV Map Number	**Chamber Name**	**Field Data Available**	**Difference in Model and Field Upstream Pressure (% Ave for 24 hrs)**
2	Jura Parade	YES	4.446
8	Rio Vista Boulevard South	YES	4.654
16	Robina Parkway	YES	0.737
19	Deodar Dr	YES	1.341
25	Philippine Parade South	YES	0.482
Average Model Error			**2.33%**

These results are presented graphically in Appendix C-6. It is evident from these results that the selected sites chosen for validation closely match the corresponding calibration results. Therefore, it is fair to assume that the remaining sites would also have sufficiently accurate pressures and diurnal patterns. The model has shown to be hydraulically representative based on the above validation.

6.5.2 PHASE 2

As discussed previously, collected chlorine data can only be used as a general comparison against the model output. Since chlorine in a network fluctuates over time depending on demand, the opportunity to use the field data (where the time is unknown) to further increase the accuracy of the model is limited. Further, the DPD chlorine test procedure is an approximation and therefore can not be used to fine tune the model decay parameters.

Figures 6.8 and 6.9 graphically represent the results for the Phase 2 chlorine model run. The figures illustrate the change in chlorine concentration in the network between maximum and minimum hour demand. The model was run for a 3 day simulation to minimise the effect of the initial condition (Cl^- = 0.3 mg/L). Figure 6.8 shows the chlorine concentration at 18:00 hours (max hr – peak demand) on the second day while Figure 6.9 shows the concentration at 2:00 hours (min hr – min demand) on the third day. The two figures demonstrate how the chlorine concentration changes during the day as the demands fluctuate i.e. low flows at min hour will correspond to poor chlorine distribution, high flows at peak or maximum hour will correspond to higher chlorine distribution.

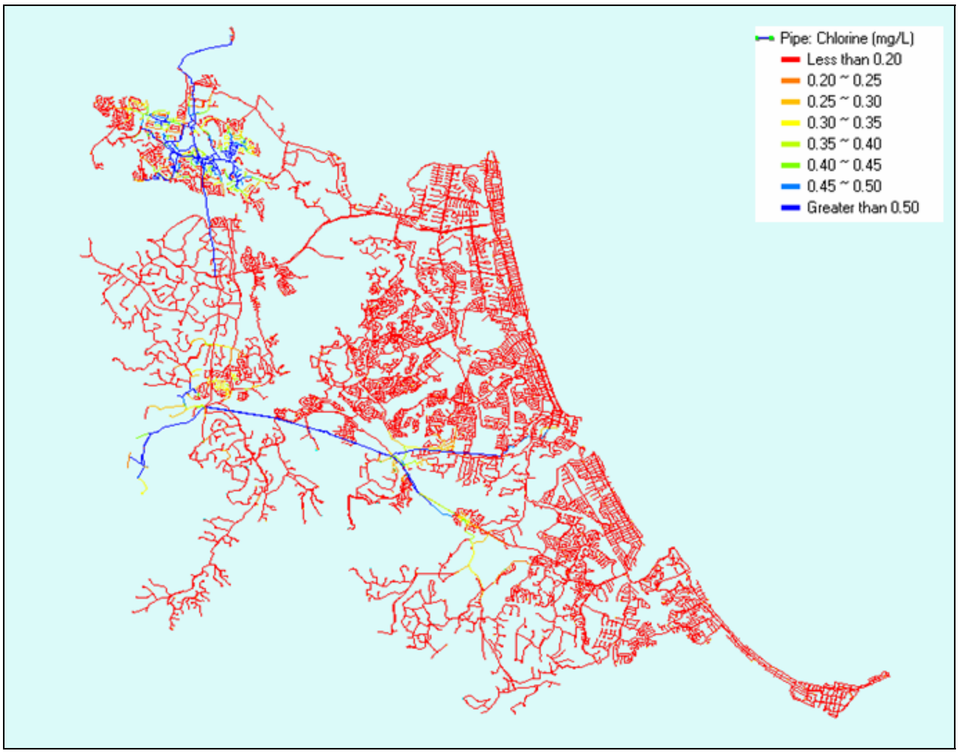


Figure 6.8: Chlorine concentration in the network at 42:00 hrs (max hr day 2) of a 3-day model simulation

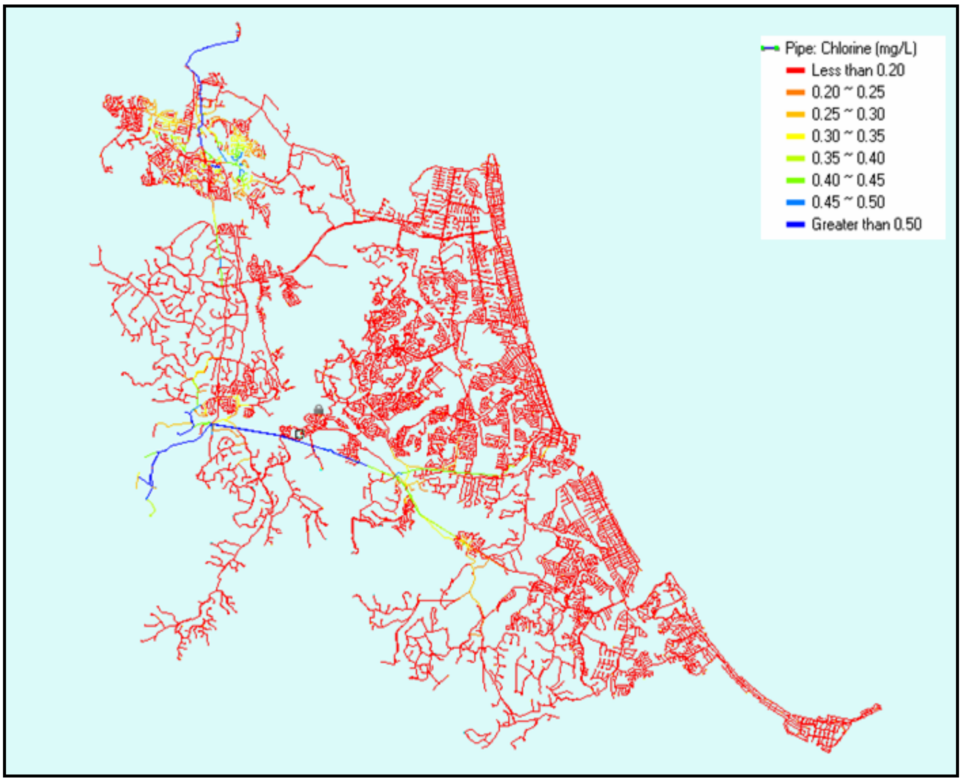


Figure 6.9: Chlorine concentration in the network at 50:00 hrs (min hr day 3) of a 3-day model simulation

Figures 6.8 and 6.9 clearly demonstrate that that the distribution of chlorine depends on the time of day and the demands during that time. Therefore is important that when comparing field data that the time of the sample and simulation are synchronized. The table containing the results for Phase 2 is presented in Appendix C-7.

The chlorine concentrations given from the network model show to be moderately indicative of the field results. Where the field concentrations measured over 0.3 mg/L the model appeared to provide reasonably similar concentrations, however, as the field readings fell below this concentration the model generally did not show the presence of a residual. The DPD test is also less accurate at low chlorine concentrations where the presence of oxidised manganese can incur false readings of residual chlorine. Given the uncertainty of the field results it can only assumed that the model is correct in representing chlorine movement through the network given that it has proved to be hydraulically accurate.

6.6 SENSITIVITY ANALYSIS

A sensitivity analysis was conducted to determine the sensitivity of the model output against a varied Global Demand Factor (GDF) and model flows rates. The GDF was altered $\pm$10% from that used in calibration (0.628) run with the flow rates at each PRV the same as the field, and the pressure compared. Similarly, the sensitivity related to network flow rates was also examined by changing the field flows back to a standard average day demand (221.13 ML/d). The analysis was undertaken for two PRVs (Jura Parade DMA supply feed & Deodar Dr DMA supply feed) located in two separate Water Supply Districts (Worongary and Burleigh) to determine the influence on network pressure. Figure 6.10 and 6.11 show the diurnal pressure patterns for each set of factors tested.

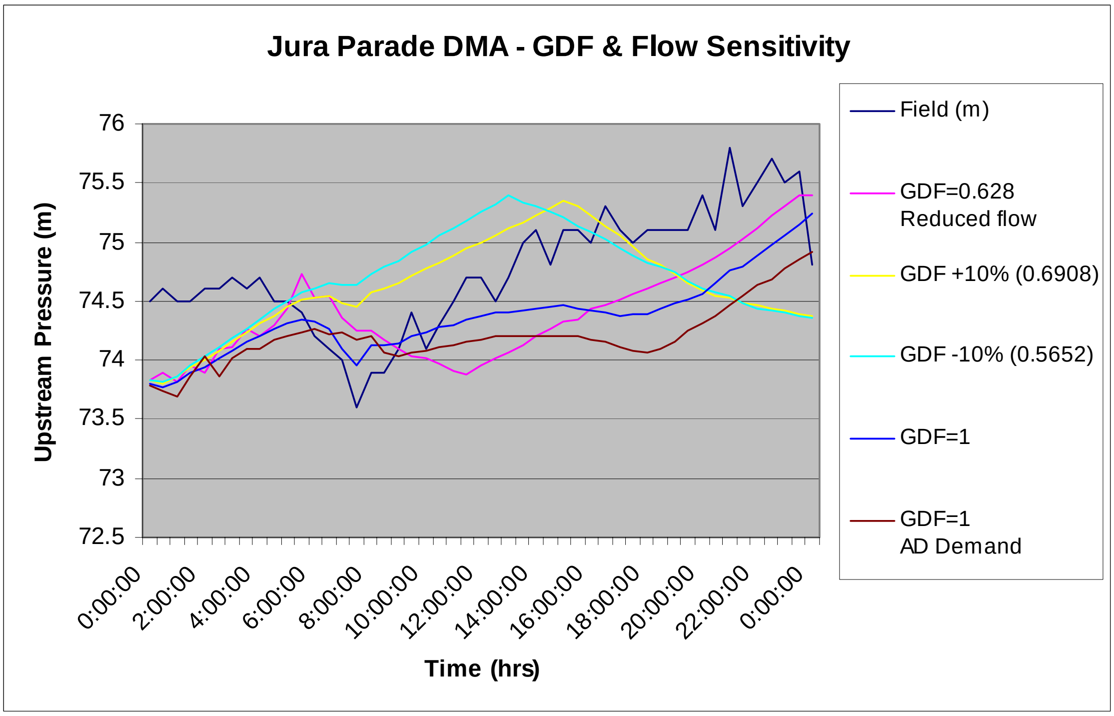


Figure 6.10: Jura Parade DMA supply feed - GDF and flow sensitivity

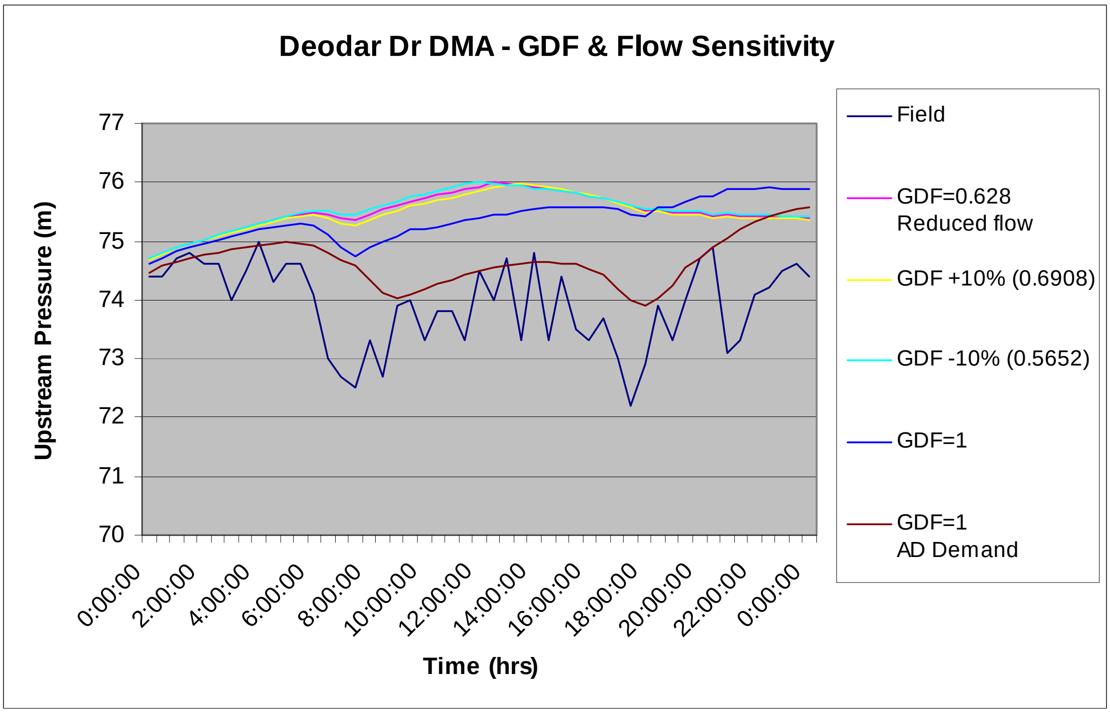


Figure 6.11: Deodar Dr DMA supply feed - GDF and flow sensitivity

It is evident that varying the GDF by $\pm$10% or by changing it back to the average day number of 1.0 does not significantly influence the pressures at these locations. The pressure difference between the GDF=1 and that caused by the flow increase of GDF=1 + AD demand is small, although the later demonstrates a better diurnal pattern. It appears that the GDF has more of an influence on the overall diurnal pressure pattern rather than by altering the flow patterns at each PRV. This is understandable given that the GDF effects the total model flow where as individual DMA flows are only have a localised affect on pressure. The total flow difference for Worongary and Burleigh Water Supply Districts from the AD non-reduced scenario to the reduced flow scenario is approximately 58% and 53% respectively. Although this appears to be a reasonable reduction in flow for both districts, the flows are still well below the capacity of the mains which supply each district (designed on MDMM). Therefore, there is no substantial friction loss through the mains to cause significant reduction in downstream pressure for these areas. Since flows during an AD demand scenario and a reduced scenario are so low, it is very difficult to calibrate the model, especially given that larger model errors are more likely to occur at higher flow rates.

The effects of temperature on chlorine decay are evident (*Koechling et al., 1997*). A sensitivity analysis regarding decay coefficients of chlorine relative to temperature would be valuable in demonstrating the seasonal distribution of the disinfectant. Since the validation runs have demonstrated extremely poor distribution of chlorine (based on 20^oC), and given that an increase in temperature would reduce the delivery of the disinfectant (this also applies to chloramine), the analysis was not undertaken. The effects of two different decay coefficients at 20^oC are seen later in Chapter 7 with the simulation of chlorine and chloramine.

6.7 CHAPTER SUMMARY

Chapter 6 discussed the preparation of the H_2ONET model including calibration, validation and a sensitivity analysis of model parameters. It was found that the model did not require adjustment of pipe friction coefficients to manipulate network pressure since the model was within $\pm$10% accuracy. Further, the flows were so low for the calibration period that any adjustment made to the friction coefficients could not be justified. Friction losses were almost negligible and pressure were nearly static, hence, calibration became more or less a model validation. The model is assumed hydraulically representative for low demand scenarios.

Given the unreliability of the chlorine data utilised for the second phase of the validation, it is assumed that the model, being hydraulically representative, is also chemically representative. Although statistically the models efficiency shows to be performing poorly, from an operation perspective it is sufficient for the purpose of the study.

CHAPTER 7. NETWORK ANALYSIS AND OPTIMISATION

7.1 INTRODUCTION

Chapter 7 provides the methodology and results for a near optimal disinfection solution. Although chlorine is the main disinfectant for the design, chloramine will also be modelled and considered as an alternative option.

7.2 NETWORK CONDITIONS

The conditions adopted for the initial network analysis and optimisation model runs include:

- Activation of Tugun Desalinisation Plant, pipeline and off takes;
- Chlorine injection at Mudgeeraba WPP (1.22 mg/L), Molendinar WPP (1.22 mg/L) and the Desalinisation Plant (1.5 mg/L);
- Adopt Average Day Reduced Scenario (worst case) if possible – Otherwise adopt a normal Average Day Demand Scenario;
- Run a 2 day model simulation for initial model runs. Run a 3 day simulation for optimisations runs, examine chlorine concentration at max hour (66:00 hrs) day 3 to alleviate initial chlorine condition;
- Use decay coefficients:
    - Chlorine bulk decay = -1, wall decay = -0.5
    - Chloramine bulk decay = -0.25, wall decay = -0.125 (discussed in Section 7.3)
- Initial conditions;
    - Cl- conc. = 0.3 mg/L at all nodes
    - Cl- conc. =1.22 mg/L at Mudgeeraba and Molendinar WPP and 1.5 mg/L at the Desalinisation Plant
    - PLMP boundary pipes closed at time 0:00 hrs
- Design constraints;
    - Cl- residual within DSS criteria (0.2 mg/L ≤ Cl^- ≤ 1.5 mg/L) in 200 mm mains and above - assume a detectable residual downstream

- Optimisation scenarios;
    - 2007 AD Demand – MH day 3 (it is assumed that this single scenario will suffice, other planning horizons beyond 2007 will only increase in demand and therefore increase chlorine distribution)

7.3 INITIAL NETWORK ANALYSIS

Different scenarios were modelled incorporating the new southern regional operating strategy to provide an indication of how residual chlorine is affected by the system changes and flow variations. This will also facilitate to determine whether worst case scenario can be adopted for the optimisation runs. The following scenarios were modelled using the network conditions provided above.

- 2007 Reduced AD Demand – worst case scenario (discussed in Section 7.3.1)
- 2007 AD Demand
- 2007 MD Demand
- 2021 AD Demand
- 2021 MD Demand

Figures 7.1 - 7.5 show a snapshot of chlorine residual in the network under the above demand scenarios. It is important to recognise that chlorine movement through the network corresponds directly with flow and therefore varies throughout the day. Hence, the following figures do not represent a total or an accumulative chlorine concentration within the network during a 24 hour period. In those areas fed by alternating pump and reservoir heads, chlorine concentration will be governed by the flow through the pump (pump operating schedule). The selected times were chosen to represent the general spread of chlorine in the network and also to demonstrate that distribution of chlorine for varies demands.

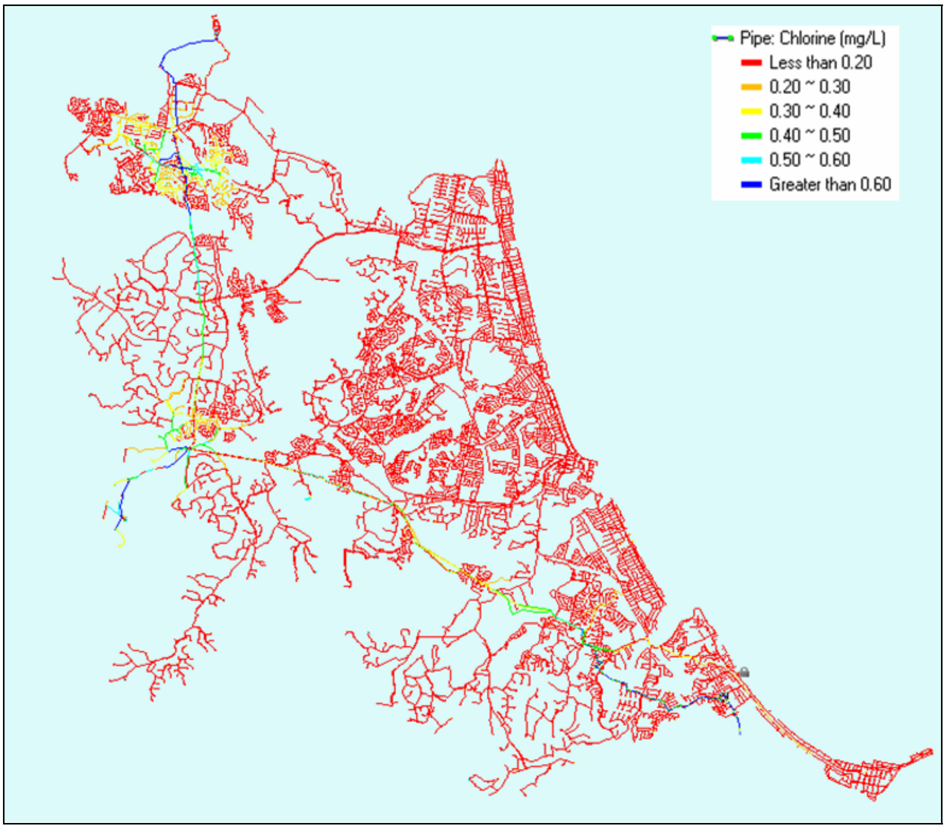


Figure 7.1: 2007 Reduced AD Demand, GDF=0.79, Chlorine at 44:30 hrs (8:30pm day 2)

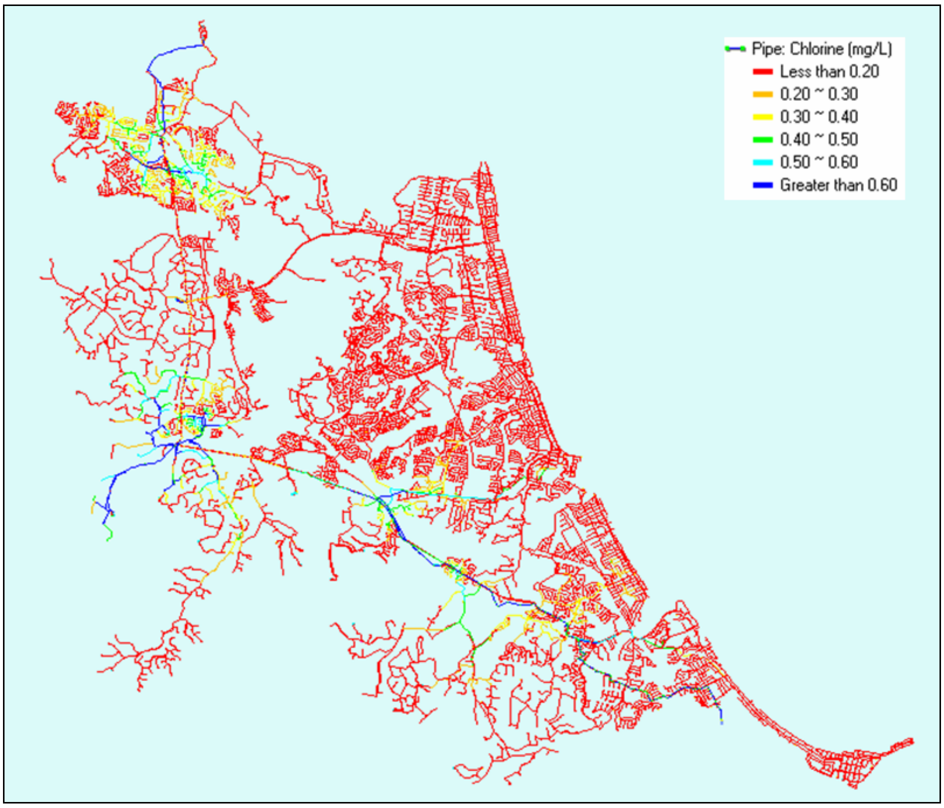


Figure 7.2: 2007 AD Demand, GDF = 1, Chlorine at 44:30 hrs (8:30pm day 2)

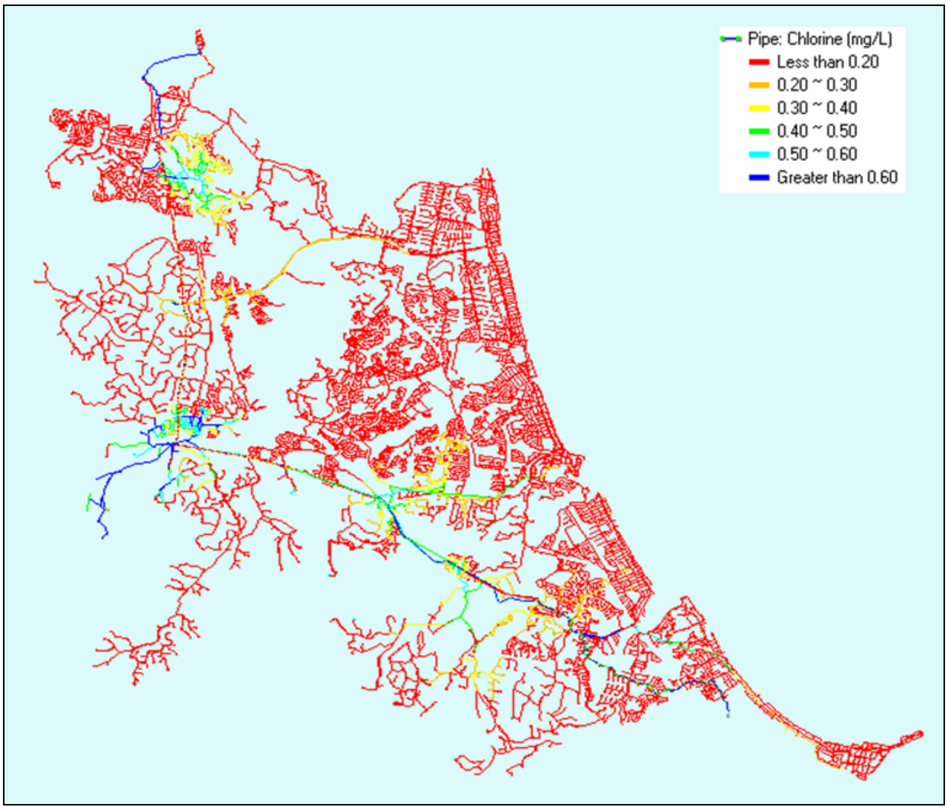


Figure 7.3: 2007 MD Demand, GDF = 1, Chlorine at 40:00 hrs (4pm day 2)

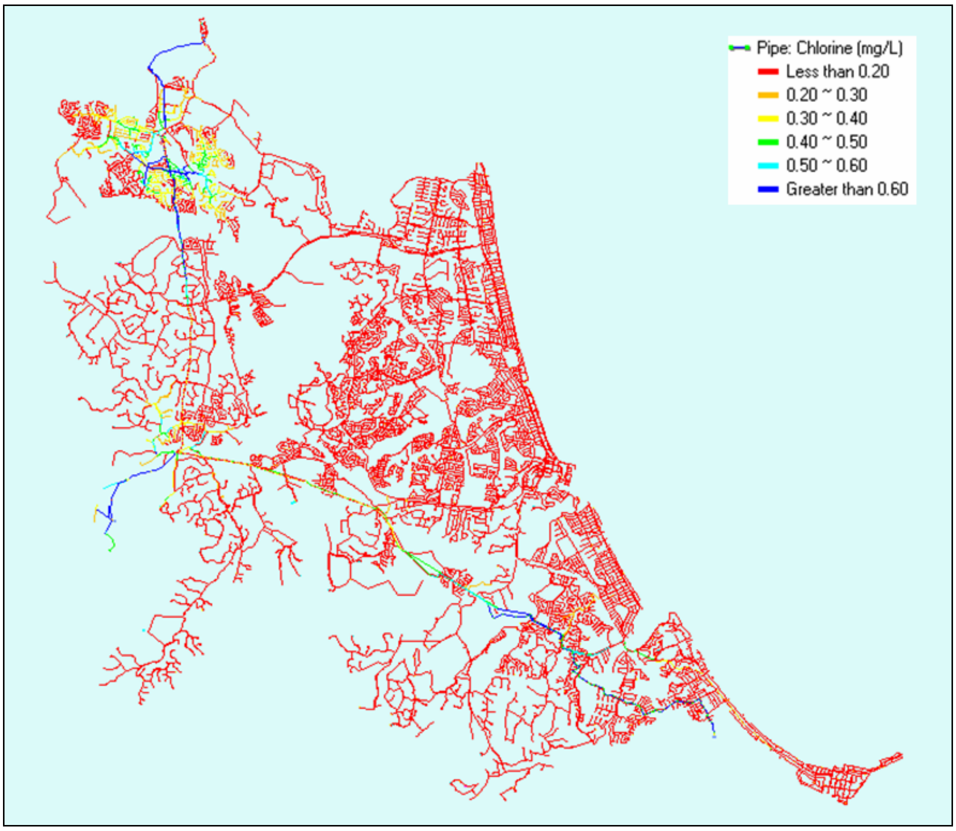


Figure 7.4: 2021 AD Demand, GDF = 1, Chlorine at 44:30 hrs (8:30pm day 2)

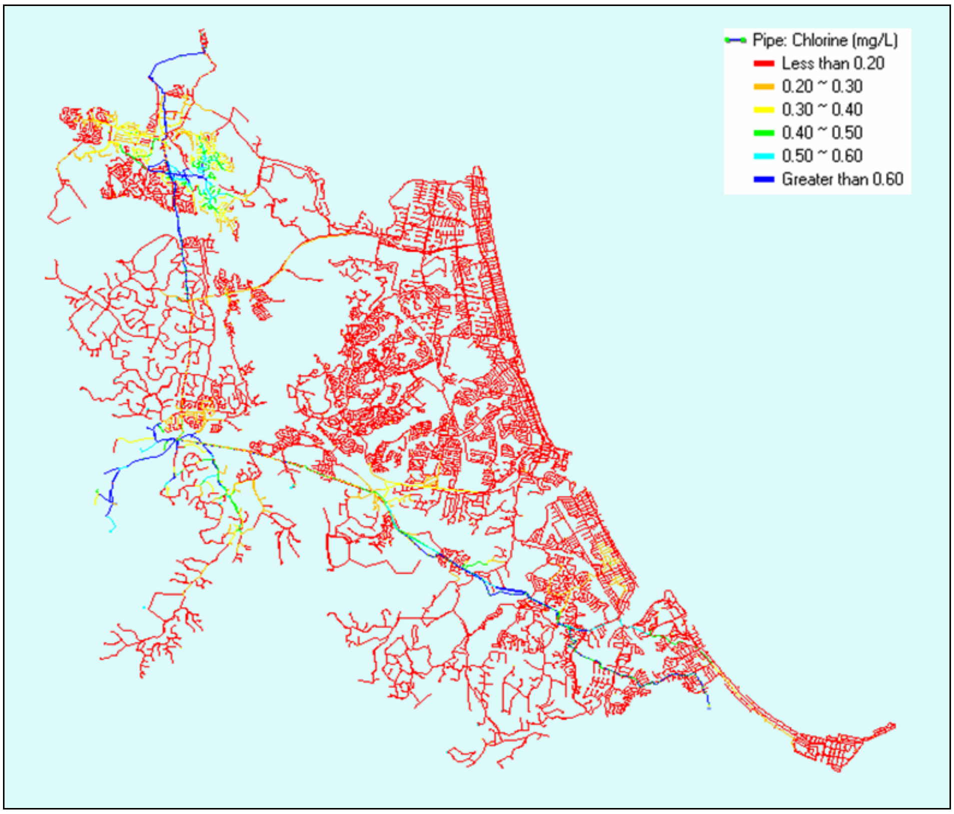


Figure 7.5: 2021 MD Demand, GDF = 1, Chlorine at 40:00 hrs (4pm day 2)

Since chlorine dissipates extremely quickly within the network, it seems relevant to compare Figures 7.1 to 7.5 with what might be expected if chloramine was applied to the system at equal concentrations. Given that chloramine is much longer lasting but less potent as a disinfecting agent, it is likely that chloramine, if adopted for Gold Coast's distribution system, will need to be applied in higher concentrations to reach the required disinfection. Nevertheless, for comparative purposes, the same concentrations will be applied at the source nodes.

Although chloramine reacts more readily with NOM than chlorine, it decays much slower with an approximate reduction of 0.2%/hr at 20 degrees Celsius. A chloramine decay study undertaken on Lake Mendota, Madison, Wisconsin identified a decay of 0.2%/hr at a pH of 7, while at a pH of 8 the decay reduced to 0.11%/hr (*AWWA, 2004*). The average temperature in the lake during the summer month of July is about 23 degrees Celsius (*Lake Mendota, 2006*). Since GCW maintains comparable temperatures within its distribution system at a pH around 7.2, it can be assumed that chloramine will decay at a similar rate of 0.2%/hr.

The global bulk and global wall decay coefficients can be calculated so chloramine distribution can be modelled. The following working concludes the chloramine decay coefficients:

From the model; Initial chlorine Conc. = 0.3 mg/L

Chlorine decay = 0.004028 mg/L/hr

Chlorine decay = 0.004028/0.3*100 = 1.34%/hr

This is equivalent in the model to a global bulk decay of -1 and a global wall decay of -0.5.

Given; Chloramine decay = 0.2%/hr

Thus; 1.34%/hr = -1 (bulk decay coefficient)

0.2%/hr = (-1/1.34)*0.2 = -0.1489 (bulk decay coefficient)

Therefore the bulk decay of chloramine is approximately -0.15. Assuming that the wall decay is roughly 50% of the bulk decay, we equate: k_w = -0.075.

Considering this as a rather subjective approach, bias can be minimised adopting a Factor of Safety (FOS). This will account for excess chloramine decay associated to the oxidation with NOM. A FOS of 10% for the bulk and 5% for the wall will be applied, giving the following coefficients for chloramine:

Global bulk decay k_b = -0.25

Global wall decay k_w = -0.125

Figures 7.6 – 7.10 show chloramine residual in the network under the same demand scenarios provided in Figures 7.1 to 7.5.

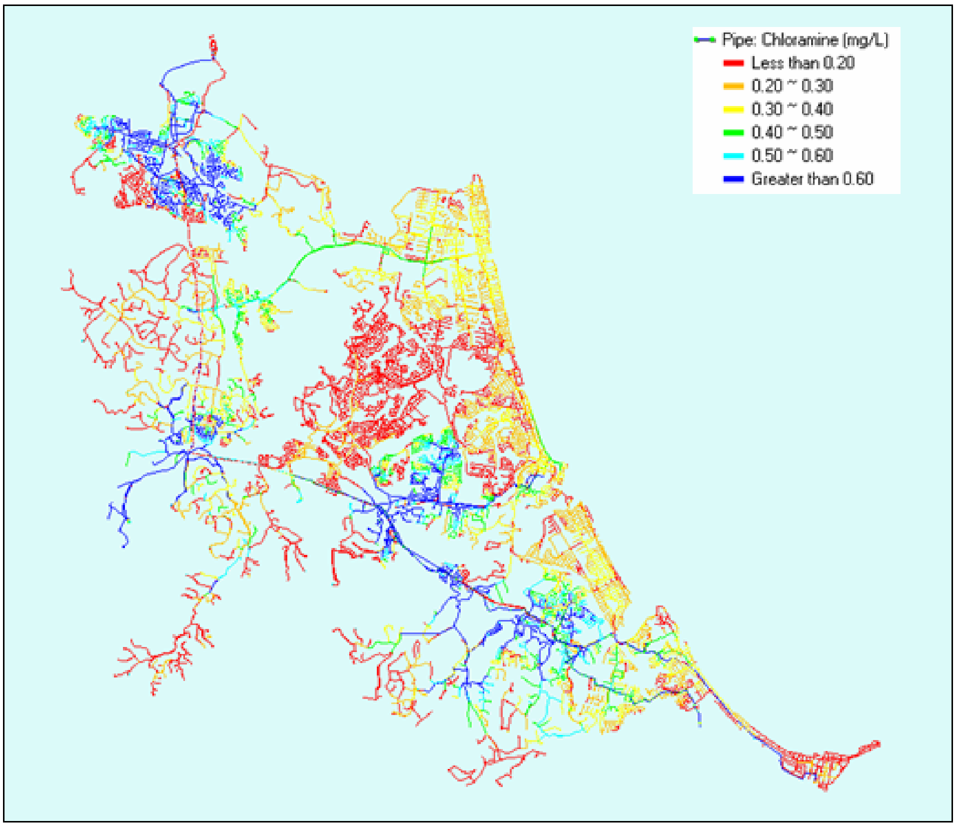


Figure 7.6: 2007 Reduced AD Demand, GDF=0.79, Chloramine at 44:30 hrs (8:30pm day 2)

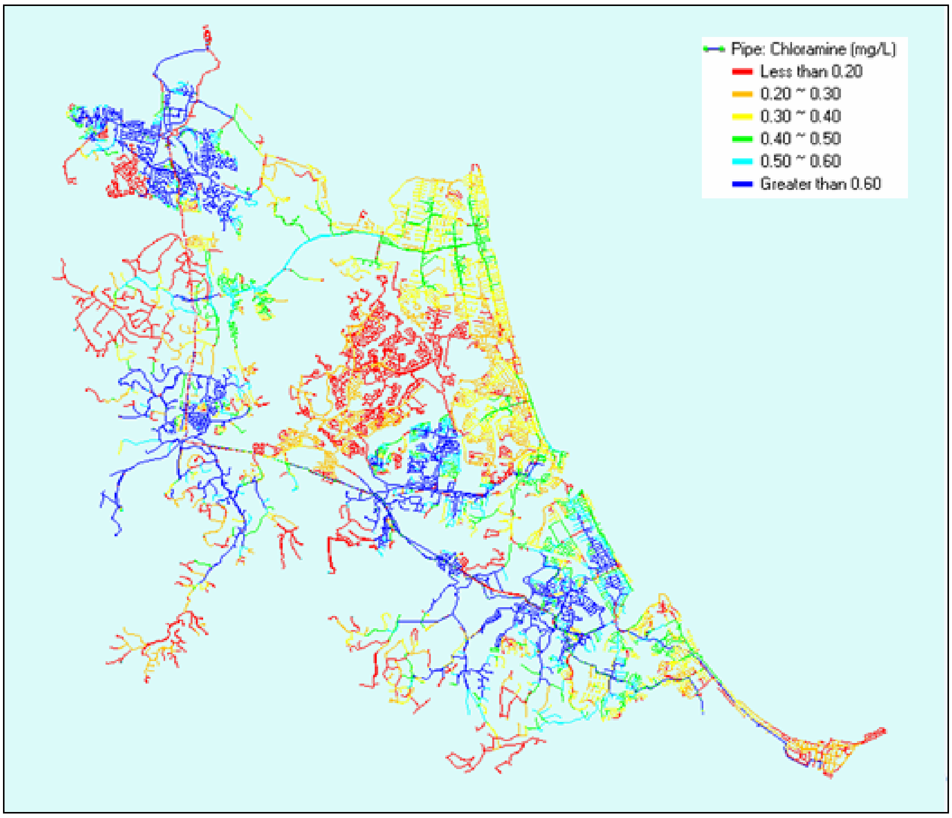


Figure 7.7: 2007 AD Demand, GDF = 1, Chloramine at 44:30 hrs (8:30pm day 2)

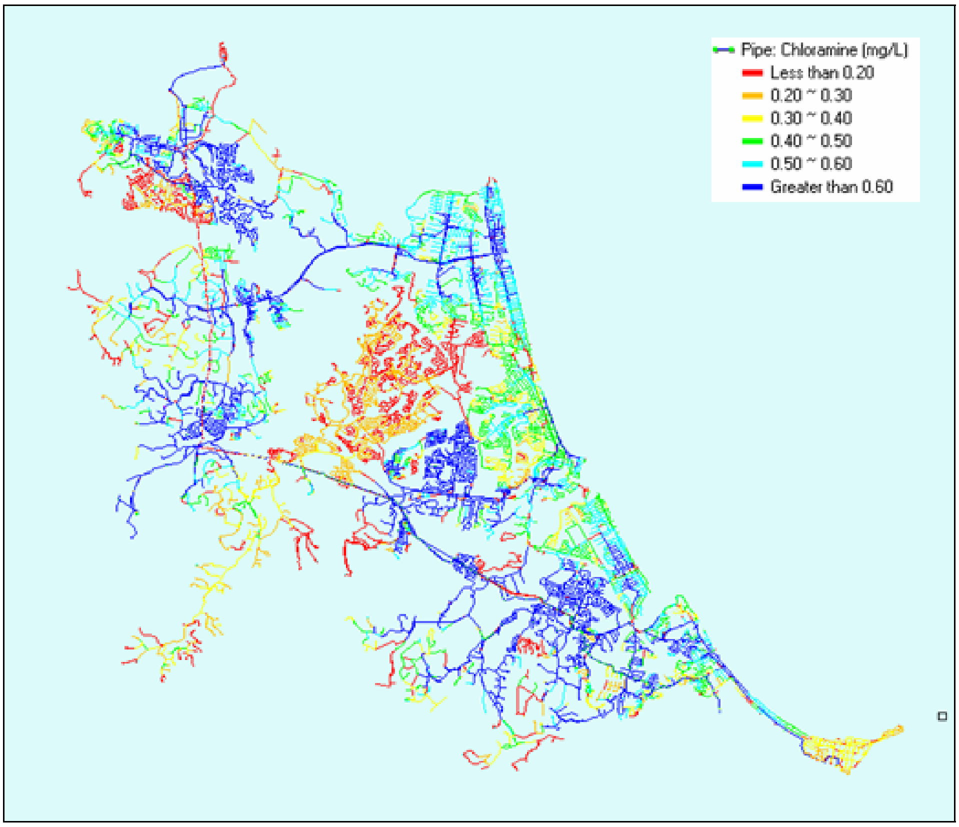


Figure 7.8: 2007 MD Demand, GDF = 1, Chloramine at 40:00 hrs (4pm day 2)

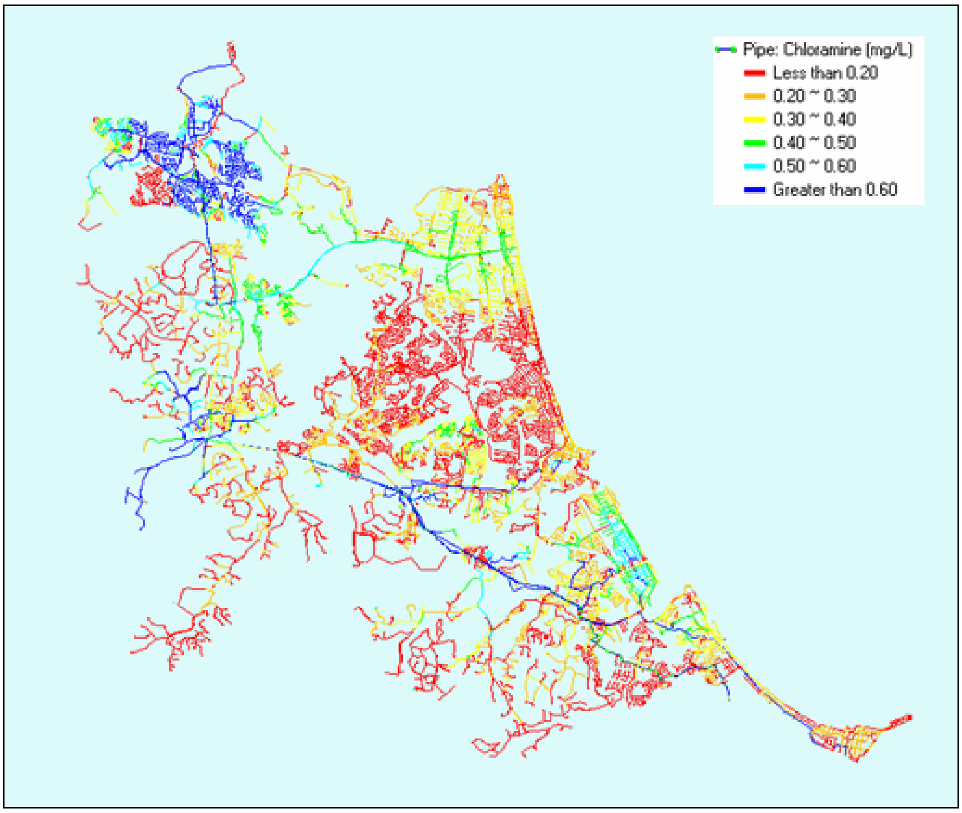


Figure 7.9: 2021 AD Demand, GDF = 1, Chloramine at 44:30 hrs (8:30pm day 2)

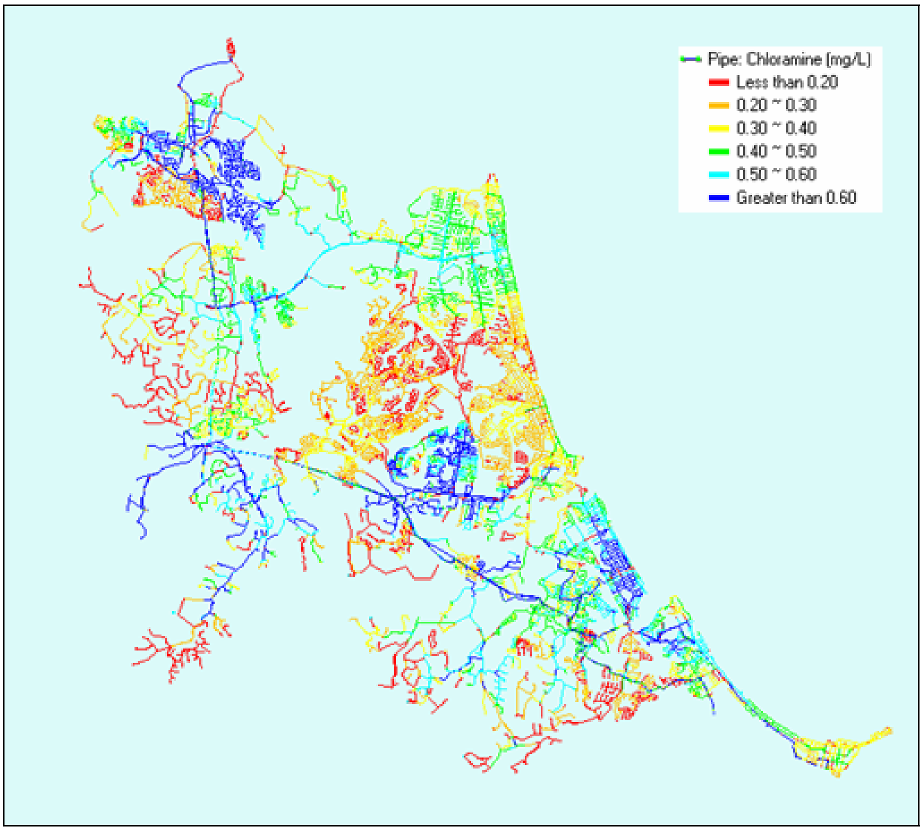


Figure 7.10: 2021 MD Demand, GDF = 1, Chloramine at 40:00 hrs (4pm day 2)

Comparing corresponding Figures 7.1 to 7.10 above, it is clear that chloramine is far more sustainable throughout the network. Table 7.1 summates the percentage of pipes with a chlorine/chloramine residual above 0.2 mg/L at the given times. The substantial difference between AD and MD scenarios is largely due to the effects of higher flows during MD scenario, also because chlorine supply to the Nerang South Water Supply District is influenced primarily by the operation of the Molendinar booster pumps at 44:00hrs and 44:30hrs.

Table 7.1: Distribution of chlorine and chloramine residual for a variety of planning scenarios

Demand Scenario	Time of Simulation (hrs)	% of all Pipes ≥ 0.2 mg/L		% Increase [(1)/(2)*100]
		Chlorine (1)	Chloramine (2)	
2007 Reduced AD	44:30:00	10.2	52.5	514.7
2007 AD	44:30:00	15.5	73.1	471.6
2007 MD	40:00:00	13.1	81.8	624.4
2021 AD	44:30:00	9.5	56.3	592.6
2021 MD	40:00:00	12.4	83.1	670.2

7.3.1 CALCULATION OF REDUCED AVERAGE DAY DEMAND

To gain a representative average day demand pattern for the model, it is critical to examine the existing historical water consumption data for Gold Coast City. Currently, GCW use an average day demand figure of 880 L/ET/d which equates to 221.13 ML/d for the entire city (deduced from the model). This demand pattern was developed by MWH (2006) based on the IDM as discussed in previous chapters. However, since the continued application of water restrictions over the past decade it appears 880 L/ET/d is rather conservative and possibly does not accurately represent the existing average day demand pattern for the city. This is evident in the lack of residual chlorine in the distribution system. Therefore, it would seem suitable to adopt a demand pattern that is representative of recent average day demands, especially when modelling the chlorine system which is dependent on network retention time. Figure 7.11 illustrates the demands for the Gold Coast over 7 years from 2000 to 2007 including comments on recent water restriction implementation.

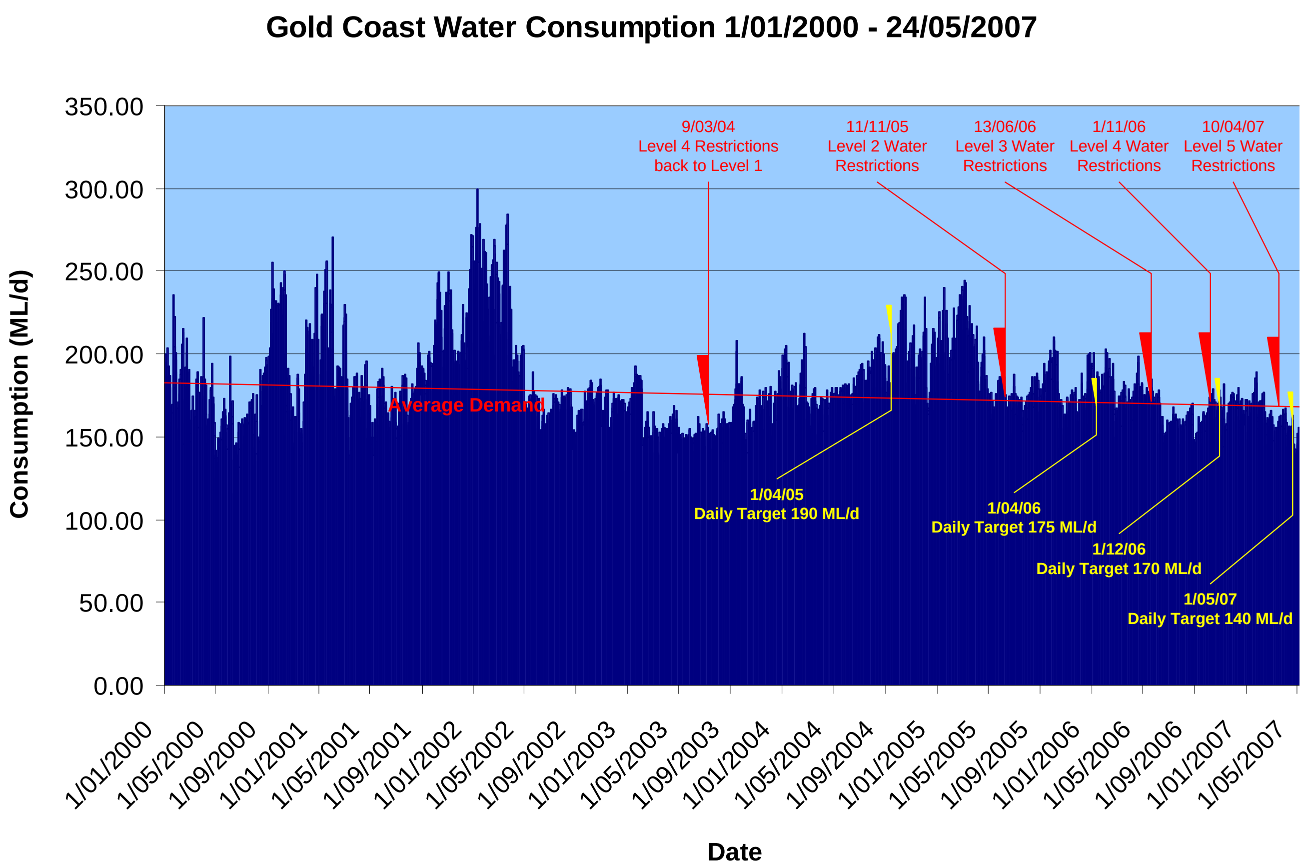


Figure 7.11: Water consumption graph for the Gold Coast

Similarly Table 7.2 provides seasonal and annual averages for Gold Coast water consumption during the same period (all water consumption data was obtained internally from GCW's historical records).

Table 7.2: Seasonal and annual average water consumption figures for Gold Coast City

Year	Season	Seasonal Average (ML/d)	Annual Average (ML/d)
2000	Summer 1999-2000	182.94	174.93
	Autumn	160.78	
	Winter	159.63	
	Spring	190.73	
2001	Summer 2000-01	200.82	185.77
	Autumn	168.00	
	Winter	168.82	
	Spring	199.09	
2002	Summer 2001-02	234.76	184.38
	Autumn	195.78	
	Winter	162.47	
	Spring	165.89	
2003	Summer 2002-03	163.48	157.55
	Autumn	149.41	
	Winter	150.87	
	Spring	164.10	
2004	Summer 2003-04	175.18	182.91
	Autumn	168.70	
	Winter	186.11	
	Spring	194.65	
2005	Summer 2004-05	201.58	183.33
	Autumn	189.85	
	Winter	171.25	
	Spring	175.17	
2006	Summer 2005-06	178.48	166.49
	Autumn	172.54	
	Winter	158.03	
	Spring	161.50	
2007	Summer 2006-07	164.79	156.48
	Autumn	150.35	
Total Average			**173.98**

According to the available and calculated data presented in Figure 7.11 and Table 7.2, the average annual consumption calculated for the entire city is approximately 174 ML/d. This converts to a 21.0% (174/221.13*100) reduction in the models existing average day GDF. This can be applied to the models GDF to alter the average day consumption accordingly.

7.4 NETWORK OPTIMISATION

It would seem appropriate to design the disinfection system based on the worst case scenario as mentioned above. However, under a reduced AD demand scenario the flows through the system are such that the water age in many parts of the network exceeds the length of the 3-day model run time. Therefore, by reducing water age to meet the design constraints and based on the sensitivity analysis, the optimisation runs will use the standard 2007 AD demand scenario and a GDF of 1.

The methodology undertaken for the optimisation engaged a trial and error approach based on achieving the DSS minimum chlorine criteria. Reservoirs were the first choice for booster chlorination due to their mixing capabilities, network injection was only considered in areas where network operations provide no reservoir alternative. Design runs will be undertaken for both chlorine and chloramine.

7.4.1 OPTION 1 – CHLORINE

Chlorine optimisation runs for the study area were undertaken adopting the network conditions described in Section 7.2. Numerous simulations concluded that a total of 14 source nodes are required to satisfy approximately 81% (at max hr - 66:00 hrs) of the distribution system $\geq$ 200 mm with a chlorine residual $\geq$ 0.2 mg/L, while 83.8% contained a residual $\geq$ 0.15 mg/L and 86.6% contained a residual $\geq$ 0.1 mg/L. Many of the remaining pipes were either closed (i.e. DMA or WSD boundary pipes) or had little/no demand at the end node. To increase the number of pipes with a residual, above 90%, an estimated 5 more booster stations are required. Table 7.3 provides the details for the 14 source nodes required (3 x existing and 11 booster facilities).

Figure 7.12 and 7.13 displays the geographical location of the booster stations and the chlorine concentration at 66:00 hrs in the network after dosing at the given concentration provided in Table 7.3. Note that although some of the network appears not to have chlorine residual at 66:00 hrs, they may have a residual $\geq$ 0.2 mg/L at other times during the day.

Table 7.3: Booster facility details for Option 1 - Chlorine

Station #	Location	Injection Type	Concentration Required at outlet (mg/L)	Comment
1	Molendinar WPP	Clear Water Tank	1.5	Existing (increase Molendinar dose from 1.22 to 1.5 mg/L)
2	Mudgeeraba WPP	Clear Water Tank	1.22	
3	Tugun Desalinisation Plant	Storage Reservoir	1.5	Activated ~ 10/08
4	Nerang South LLZ Reservoir 1	Reservoir (W07)	0.55	Required to service Highland Park ILZ
5	Worongary LLZ Reservoir 1	Reservoir (WG01)	1.3	Required to Service Worongary and Gilston LLZ
6	Clover Hill Reservoir 2, Robina LLZ	New 30 ML Overflow Reservoir	1.5	Required to Service Robina LLZ
7	86 Glenmore Rd, Bonogin	Network (300 mm main)	1.3	Required to Service Bonogin zones
8	5 Asperia Street, Reedy Ck	Network (750 mm main)	0.6	Injected into the Desal off take to boost chlorine levels to 1.3 from 0.6 mg/L. Required to service Reedy Ck
9	Burleigh Head s LLZ Reservoir	Reservoir (BU04)	1.3	Required to Service Burleigh Heads LLZ
10	213 Tallebudgera Connection Rd, Tallebudgera Pump Station	Network (500 mm main)	1.3	Inject just downstream of the Tallebudgera PS to service zones in Currumbin Waters
11	Elanora LLZ Reservoir 2	Reservoir (EL02)	0.8	Required to Service Elanora LLZ
12	132 Guiness Creek Rd, Currumbin Waters	Network (375 mm main)	0.7	Required to Service Currumbin Waters LLZ
13	Currumbin LLZ Reservoir	Reservoir (CU01)	0.9	Required to Service Currumbin LLZ
14	Coolangatta LLZ Reservoir	Reservoir (CO01)	0.7	Required to Service Coolangatta LLZ

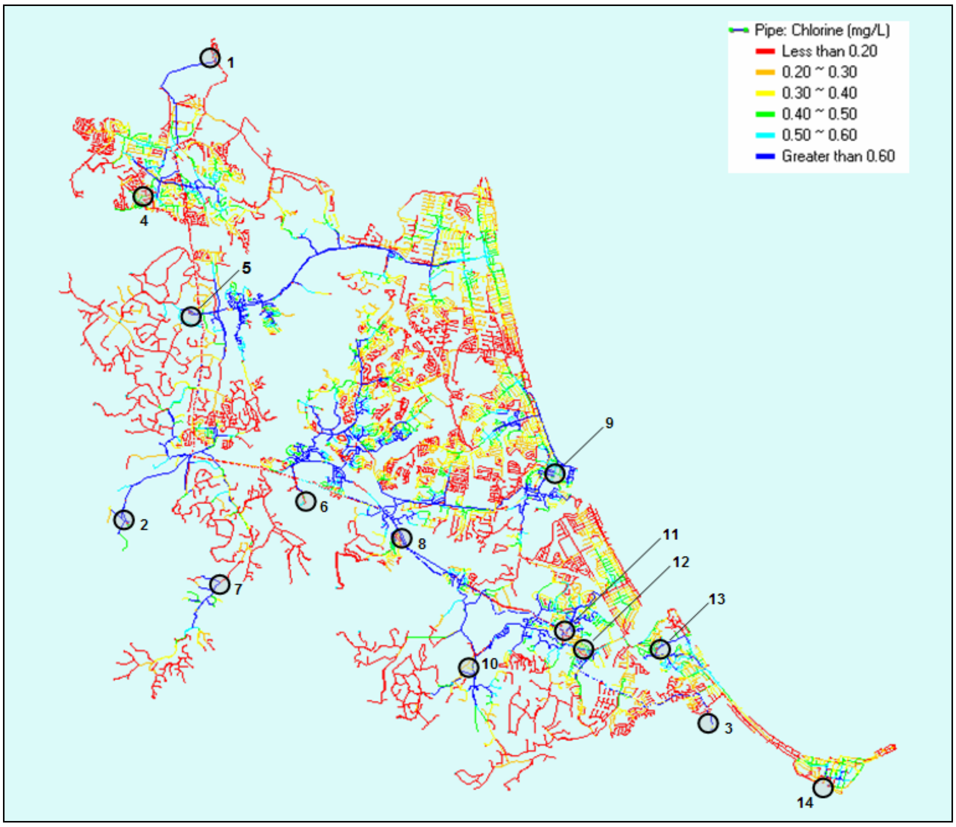


Figure 7.12: 2007 AD Demand - Cl$^-$ source nodes and network conc. at 66:00 hrs (all distribution mains)

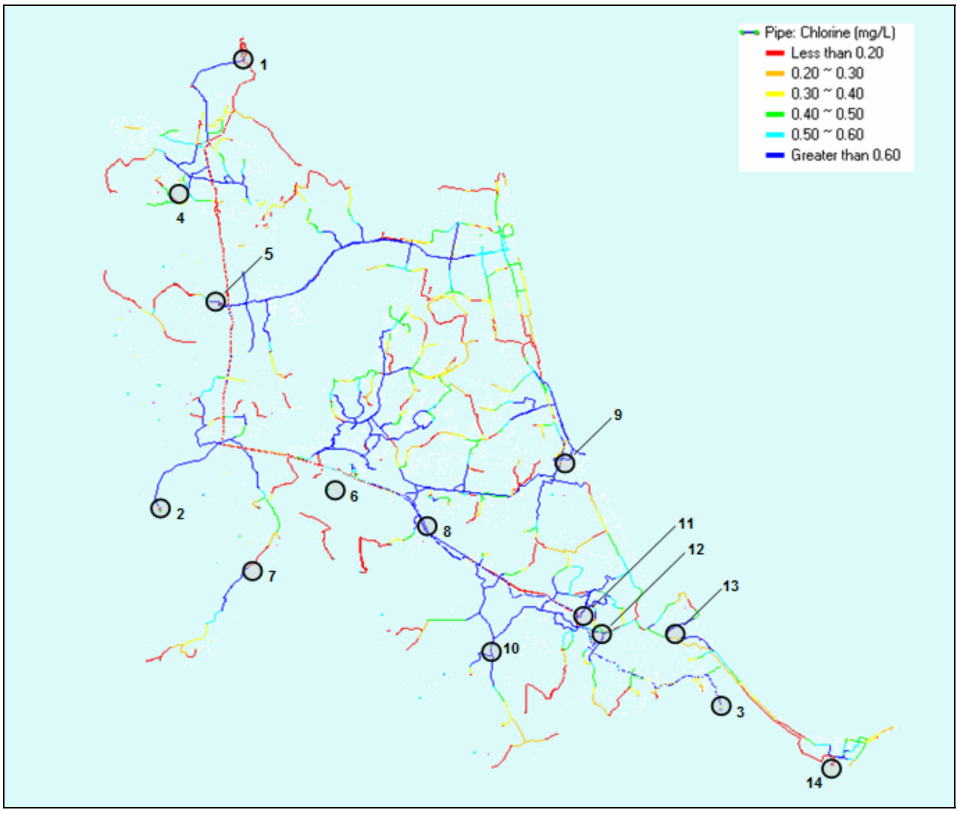


Figure 7.13: 2007 AD Demand - Cl$^-$ source nodes and network conc. at 66:00 hrs (mains ≥ 200 mm)

The chlorine booster locations were selected based on achieving the largest spread of chlorine downstream of the source node. Most of these locations were at major reservoirs with only four boosters requiring direct injection into the network. Concentrations were toggled to achieve the furthest detectable chlorine residual $\geq$ 0.2 mg/L, noting that higher dosage does not always cause an increased residual for low demand areas.

The analysis indicated that the source node concentration at Molendinar WPP required increasing from 1.22 mg/L to 1.5 mg/L to boost residuals in Nerang South Water Supply District. Increasing the dose at Mudgeeraba WPP to 1.5 mg/L does not substantially influence the residual downstream in Mudgeeraba LLZ or Bonogin and does not alleviate the need for a booster facility on Glenmore Rd, hence the dose was maintained at 1.22 mg/L.

The proposed booster station 4 at Nerang South LLZ reservoir (W07) is required to predominantly provide disinfectant residual for highland Park ILZ. Although an ILZ reservoir exist for the zone, it was chosen to dose at the LLZ reservoir to also boost chlorine levels in the LLZ. Renfrew Drive Pump Station, adjacent W07, feeds the ILZ reservoir (W01) and the surrounding network in the zone. Since the flows into the W01 are much higher than those leaving the reservoir, the chlorine residual in the surrounding mains tend to be higher if dosed from W07. Further, W01 is fed via a single 325 m long 300 mm inlet/outlet main so it is likely that most of the chlorine decay would occur in this main limiting the extent of the chlorine movement. It was concluded that a dose of 0.55 mg/L at W07 is sufficient to service most of the surrounding ILZ and LLZ zones during average day demands. It was difficult to maintain a residual on Gilston Rd due to low demands for the area.

Proposed booster 5 and 6 at Worongary (WG01) and Clover Hill (overflow) are necessary for the downstream service zones. Similarly, boosters 9 and 11 at Burleigh (BU04) and Elanora (EL02) are also required. WG01 and BU04 both require a flow paced dose of 1.3 mg/L while Clover Hill overflow reservoir requires 1.5 mg/L and Elanora 0.8 mg/L to reach the extremities of the network $\geq$ 200 mm. Reservoirs were selected for their mixing capabilities and also because they are the point source of supply for the downstream network.

Booster 7 is located adjacent 87 Glenmore Rd where residual just under the minimum is achieved from Mudgeeraba chlorination. At this location, a booster dosing at 1.3 mg/L allows a chlorine residual just below 0.2 mg/L at the inlet to the Aylesham Dr HLZ reservoir (BM01) in Bonogin South. Booster 8 is intended to elevate the residuals in Reedy Creek. The booster is proposed to inject directly into the Reedy Creek off take which, at which there is an existing residual of 0.6

mg/L coming from the desalinisation plant. A further 0.6 mg/L is required to achieve a residual downstream in the 300 mm main on Christine Avenue and the 225 on Tallebudgera Creek Rd.

Booster station 10 is required at the downstream side of Tallebudgera Connection Rd pump station to service Currumbin Waters LLZ. A dose of 1.3 mg/L services most of the zones however the 200 mm main up to Westminster and Trees Rd reservoirs still contains residuals slightly under the DSS criteria. Increasing the dose rate higher does not influence the residual due to the very small flows passing through the mains during an average day demand. Without further dosing it is difficult to service these areas to achieve the DSS criteria. Similarly, a dose of 0.7 mg/L at booster 12 is required to service mains on Galleon Way and Currumbin Creek Rd.

Lastly, booster 13 at Currumbin LLZ reservoir (CU01) and booster 14 at Coolangatta LLZ reservoir (CO01) are required to service Currumbin and Coolangatta LLZ's respectively. CU01 requires a dose of 0.9 mg/L and CO01 requires 0.7 mg/L to satisfy all mains $\geq$ 200 mm with the minimum DDS chlorine residual.

The number of booster stations needed for approximately 81% of the network $\geq$ 200 mm to comply with the DSS minimum chlorine criteria would not appear economically viable. The Gold Coast water distribution system is not designed in a way that sustains chlorine, any option that involves chlorination is expected to contain a large number of chlorine booster facilities. No pipe augmentation or valve arrangements were necessary that would significantly increase the distribution of chlorine. An alternative disinfection option is therefore required.

7.4.2 OPTION 2 – CHLORAMINE

After undertaking the initial network analysis and comparing the extent of chlorine and chloramine movement through the network, chloramine would appear to be a suitable alternative option for primary disinfection. The chloramine analysis is heavily dependant on the decay coefficients that have been assumed.

The model indicates that a single chloramine booster facility is required to achieve satisfactory disinfection for the study area under AD demand conditions. The booster station is necessary to service Robina LLZ within the DSS criteria (assuming the DSS minimum chlorine criteria also applies to chloramine; the same dose rates for chloramine were used at the treatment plants).

Clover Hill overflow reservoir is the recommended location for the new booster facility. Table 7.4 provides the details.

Table 7.4: Booster facility details for Option 2 - Chloramine

Station #	Location	Injection Type	Concentration Required at outlet (mg/L)	Comment
1	Molendinar WPP	Clear Water Tank	1.22	Existing
2	Mudgeeraba WPP	Clear Water Tank	1.22	
3	Tugun Desalinisation Plant	Storage Reservoir	1.5	Activated ~ 10/08
4	Clover Hill Reservoir 2, Robina LLZ	New 30 ML Overflow Reservoir	0.6	Required to Service Robina LLZ

Disinfecting with chloramine presents a much better outcome, where, with a single booster 83.3% of pipes $\geq$ 200 mm contained a residual $\geq$ 0.2 mg/L, while 92.7% contained a residual $\geq$ 0.15 mg/L and 98.1% contained a residual $\geq$ 0.1 mg/L at 66:00 hrs. Additionally, 79.6% of all pipes in the network contained a residual $\geq$ 0.2 mg/L, 86.6% contained a residual $\geq$ 0.15 mg/L and 95.6% contained a residual $\geq$ 0.1 mg/L.

Figures 7.14 and 7.15 display the chloramine concentration at 66:00 hrs in the network after dosing at the given concentration provided in Table 7.4. The geographical location of each chloramine source node is also shown in the figures.

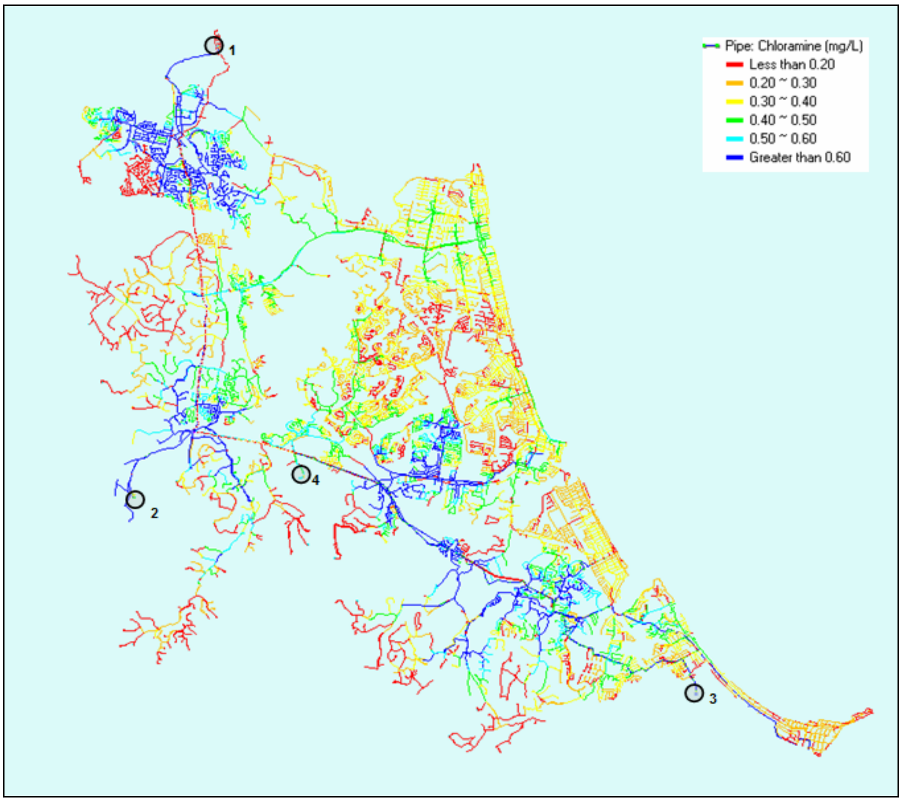


Figure 7.14: 2007 AD Demand - chloramine source nodes and network conc. at 66:00 hrs (all distribution mains)

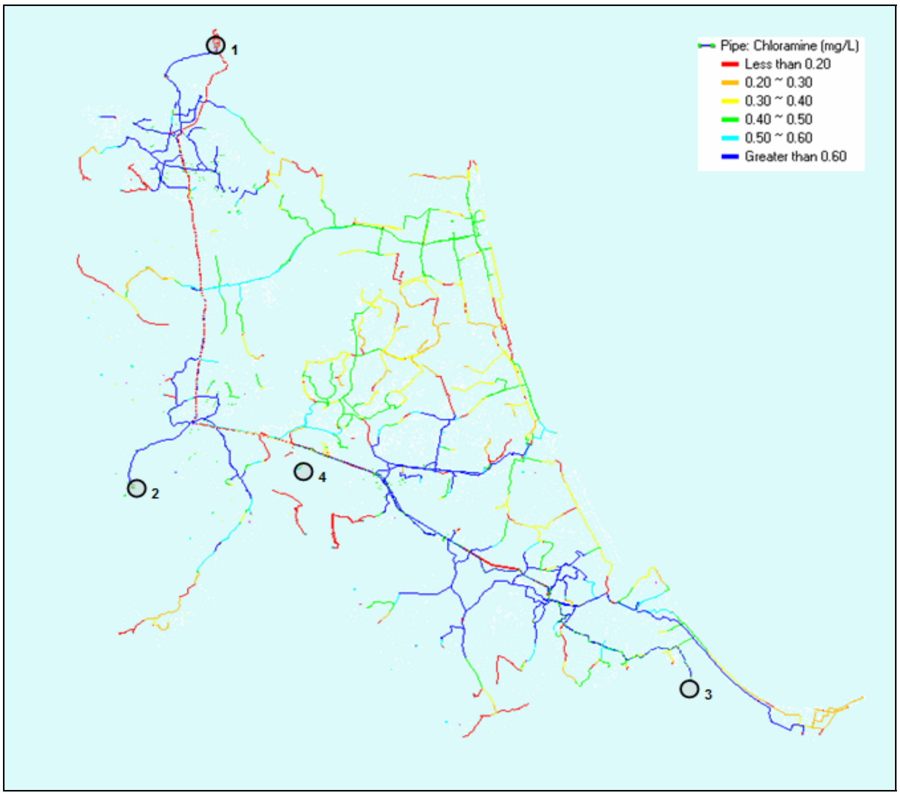


Figure 7.15: 2007 AD Demand - chloramine source nodes and network conc. at 66:00 hrs (mains ≥ 200 mm)

Table 7.5 summarises the residuals within the distribution system for Options 1 and 2. It is clear that Option 2 is noticeably superior in regards to maintaining a network residual.

Table 7.5: Summary of pipes satisfying the DSS disinfectant residual for Options 1 and 2

Option	Disinfectant	Boosters Required	Main Size	Pipes Satisfying Disinfectant Residual Design Criteria (%)		
				≥ 0.2 mg/L	≥ 0.15 mg/L	≥ 0.1 mg/L
1	Chlorine	11	≥ 200 mm	81.0	83.8	86.6
			all	62.5	70.4	78.2
2	Chloramine	1	≥ 200 mm	83.3	92.7	98.1
			all	79.6	86.6	95.6

It was not possible to obtain a residual above 0.2 mg/L in 100% of mains ≥ 200 mm due to the connectivity of the system and associated small demands. There were also no augmentations or feasible valving alterations that would greatly impact the distribution of either disinfectant through the network.

7.5 ECOMONOMIC ANALYSIS

A cost-effective analysis can be undertaken for Options 1 and 2 based on the initial site capital costs and the ongoing volume of chemical required. Approximate site capital costs have been obtained/broken down from raw subcontract costs and tender estimates provided for a gauging station and a combined chloramination/pump booster station. GCW's current bulk purchase price for sodium hypochlorite is $0.191/L. The approximate unit cost for anhydrous ammonia required for chloramine was provided by Orica Specialty Chemicals at a rate of $2.80/kg; this unit rate is likely to change depending on the quantity purchased and how often it is purchased.

7.5.1 CAPITAL COSTS

The initial capital costs associated with setting up a booster facility are estimated in Table 7.6. An approximate value of $100,000.00 is expected for each booster station.

Table 7.6: Estimated capital costs for Options 1 and 2

Item	Description	Total Cost
1	**General**	
1.3	Project Managament & Administration	$10,000.00
1.5	Design and Pre-Manufacture Activities	$2,500.00
1.6	As Constructed Drawings	$3,000.00
	Subtotal	**$15,500.00**
2	**Chemical Handling Area**	
2.1	Slab	$5,000.00
2.2	Blockwork	$2,000.00
2.3	Brickwork and Roofing	$5,000.00
2.4	Fencing	$2,000.00
2.5	Fixtures and Finishing	$1,000.00
2.6	Other materials	$2,000.00
	Subtotal	**$17,000.00**
3	**Chemical Storage and Dosing**	
3.1	Chemical Storage and Handling	$15,000.00
3.2	Chemical Dosing System	$15,000.00
	Subtotal	**$30,000.00**
4	**Site Works**	
4.1	Landscaping and Grassing	$1,500.00
	Subtotal	**$1,500.00**
5	**Electrical and Telemetry Works**	
5.1	Outflow Meter	$12,000.00
5.2	Telemetry Works	$4,000.00
5.3	Power (Field Conduits, Electrical Wiring, Switchboard)	$6,000.00
	Contingencies (10%)	$2,200.00
	Subtotal	**$24,200.00**
	Subtotal Per Site	**$88,200.00**
	10% GST	**$8,820.00**
	Total Per Site	**$97,020.00**
	Subtotal - Option 1 (11 Sites)	**$1,067,220.00**
	Subtotal - Option 2 (1 Site)	**$97,020.00**
	Training Workshop	$4,000.00
	Operation and Maintenance Manuals	$1,300.00
	Subtotal	**$5,300.00**
	10% GST	**$530.00**
	Total Combined Sites	**$5,830.00**
	Total - Option 1	**$1,073,050.00**
	Total - Option 2	**$102,850.00**

7.5.2 CHEMICAL COSTS

Monochloramine (NH2Cl) has a chlorine nitrogen (Cl:N) ratio of approximately 2:1, by weight, and 1:1, by molecule. However a preferred ratio chlorine to Ammonia is 3:1 as it appears to produce better tasting water (*White, 1999*). Therefore the quantity of ammonia per unit of sodium hypochlorite can easily be calculated in order to create monochloramine. The total volume of sodium hypochlorite can be calculated for each option and booster station using the sample calculation provided in Section 2.3.2.

The calculations for station 1, Option 1 base on 10% sodium hypochlorite solution are as follows:

1.5 mg/L x 86.73 ML/day x 1000 000 L x 1 g/1000 mg = **130089.9 g/day**

130089.9 g/day x 1 L/100 g = **1300.9 L/day NaOCL**

Alternatively; $C_1V_1 = C_2V_2$

100 000 mg/L x Y Litres NaOCL = 1.5 mg/L x 86 730 000 L

Y Litres NaOCL = (1.5 * 86 730 000)/100 000 = **1300.9 L/day NaOCL**

Hence; 1300.9 L/day x 0.191 = **$248.47 /L**

Tables 7.7 and 7.8 are indicative of the ongoing chemical costs likely to be experienced for Options 1 and 2 respectively. Note that the flows associated with the chlorine dosing at Molendinar include those distributed between both the Northern Region and the Southern Region, and takes into account the operation of the Desalinisation Plant. Costs provided for Molendinar, Mudgeeraba and the Desalinisation Plant do not include pre-chlorination costs.

Table 7.7: Chemical costs for Option 1

Option	Station #	NaOCl Dose (mg/L)	Average Flow from Source Node (L/s)	Total Flow (ML/day)	NaOCl Required (g/day)	NaOCl Volume (L/day)	Cost Cl-@ $0.191/L
1	1	1.5	1003.78	86.73	130089.9	1300.90	$248.47
	2	1.22	1048.93	90.63	110565.6	1105.66	$211.18
	3	1.5	1411.05	121.91	182872.1	1828.72	$349.29
	4	0.55	3.73	0.32	177.2	1.77	$0.34
	5	1.3	303.02	26.18	34035.2	340.35	$65.01
	6	1.5	139.68	12.07	18102.5	181.03	$34.58
	7	1.3	7.91	0.68	888.5	8.88	$1.70
	8	0.6	516.97	44.67	26799.7	268.00	$51.19
	9	1.3	101.97	8.81	11453.3	114.53	$21.88
	10	1.3	23.01	1.99	2584.5	25.84	$4.94
	11	0.8	66.18	5.72	4574.4	45.74	$8.74
	12	0.7	39.68	3.43	2399.8	24.00	$4.58
	13	0.9	46.92	4.05	3648.5	36.48	$6.97
	14	0.7	41.22	3.56	2493.0	24.93	$4.76
	Total Cost / Day						**$1,013.61**
	Annual Cost						**$369,966.48**

Ammonia necessary for Option 2 can be calculated based on the 3:1 ratio. An example calculation for station 1 is as follows:

Sodium hypochlorite	**= 105806.4 g/day**
	= 105.806 kg/day
3:1 ratio	= 105.806 kg/day x 1/3 = **35.27 kg/day**
Ammonia	= 35.27 kg/day x $2.80 /kg = **$98.75 /day**

Table 7.8: Chemical costs for Option 2

Option	Station #	NaOCl Dose (mg/L)	Average Flow from Source Node (L/s)	Total Flow (ML/day)	NaOCl Required (g/day)	NaOCl Volume (L/day)	Cost Cl- @ $0.191/L	Cost NH3 @ ~$2.80/kg	Total Cost
2	1	1.22	1003.78	86.73	105806.4	1058.06	$202.09	$98.75	$300.84
	2	1.22	1048.93	90.63	110565.6	1105.66	$211.18	$103.19	$314.37
	3	1.5	1411.05	121.91	182872.1	1828.72	$349.29	$170.68	$519.97
	4	0.6	139.68	12.07	7241.0	72.41	$13.83	$6.76	$20.59
	Total Cost / Day								**$1,155.77**
	Annual Cost								**$421,857.06**

7.6 CHAPTER SUMMARY

A summary of costs are presented below in Table 7.9, including maintenance costs.

Table 7.9: Summary of Costs for Options 1 and 2

Option	Initial Capital Cost	Annual Chemical Cost	Ongoing Annual Maintenance Cost ($5,000/site)	Total Cost for 1st Year	Total Cost for 2nd Year
1	$1,073,050.00	$369,966.48	$55,000.00	$1,498,016.48	$424,966.48
2	$102,850.00	$421,857.06	$5,000.00	$529,707.06	$426,857.06

It is clear that Option 1 has much higher initial capital costs. Although, Option 2 has higher chemical costs due to the addition of anhydrous ammonia, the ongoing costs are evened out by the maintenance costs required for Option 1. These results indicate that Option 2 is slightly

($1,890/yr) more expensive beyond the first year of operation, but it would take more than 500 years to equal the overall initial cost for Option 1. However, maintenance costs are subject to change as the likelihood of something going wrong or breaking down with 11 rechlorination facilities is much more prominent than that with a single booster. Hence, Option 2 may ultimately have cheaper ongoing costs. Option 2 has demonstrated to be operationally more sustainable and also economically more practical.

CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

This study has investigated options to mitigate the residual chlorine deficiency existing within GCW's potable water reticulation system. The options were determined using GCW's hydraulic network model and designed according the water quality criteria set out in the Desired Standards of Service.

In order to achieve accurate results, the hydraulic model required calibration and validation of model parameters. An initial calibration run was undertaken where the field and model pressures were compared. It appeared that the model was highly accurate in simulating field pressure during low demand periods where friction losses were almost negligible and network pressure approximately static. Therefore, pipe friction coefficients did not require adjusting and the model was believed to be hydraulically accurate to model low demand scenarios.

The model could not be calibrated chemically. Although low demand conditions are ideal for chlorine calibration (worse case), the quality of the field data was not sufficient to undertake the task. Instead, a comparison between model and field chlorine data was carried out where it was found that a field reading of 0.3 mg/L and above generally correlated to a similar chlorine residual in the model. Given, the model proved to be hydraulically representative it was assumed that it be also chemically indicative.

A sensitivity analysis indicated that altering the Global Demand Factor below 1 seemed to slightly disturb the diurnal pressure pattern within the network. Small adjustments to the parameter proved not to effect pressure noticeably. This outcome was thought to be due to the minimised flow rates occurring in these scenarios. It is expected that larger variations in pressure would occur if the analysis was undertaken during MD and MDMM conditions where head loss in the network is more substantial.

Initial network analysis found that chloramine maintained a residual throughout the network with an approximate 500% increase over chlorine distribution during AD and AD reduced conditions. Chloramine was then considered as an alternative option to chlorine in the optimisation runs.

Optimisation modelling indicated that from the existing 3 disinfectant source nodes in the system a further 11 booster sources were required for chlorine (Option 1) to achieve a residual equal to or greater than 0.2 mg/L in more than 80% of 200 mm diameter mains and above. Conversely, chloramine (Option 2) required only 1 booster to achieve the same outcome. There were no pipe augmentations or valving arrangement recognized for either option that enhanced the chemical distribution. The cost-benefit analysis indicated that Option 1 has a much higher initial capital costs but less ongoing costs. The addition of anhydrous ammonia in Option 2 indicated that ongoing cost would are rather significant, however, both options demonstrated to have substantial ongoing costs.

8.2 RECOMMENDATIONS

Hydraulic and water quality modelling have indicated that chloramine is a more sustainable disinfectant for GCW's potable water network. However, it is recommended that the use of chloramine be further investigated in order to make a solid decision on the appropriate disinfectant for Gold Coast City's distribution Network. The suitability and operating regime of chloramine may be dependent on or change based on the following areas recommended for further investigation.

- Chloramine reaction with source water constituents
- Seasonal variations and chloramine decay (temperature and flow)
- Reservoir operating levels
- Pump operating schedules

It is also recommended that calibration be undertaken during peak day conditions or in summer months where network flows are higher and model discrepancies are more likely to be identified. Further, it is suggested that widespread field testing be undertaken to determine the actual chemical decay coefficients of chlorine, which are representative of the Gold Coast network.

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APPENDIX A

Project Specification

UNIVERSITY OF SOUTHERN QUEENSLAND
FACULTY OF ENGINEERING AND SURVEYING

ENG 4111/2 RESEARCH PROJECT

UNDERTAKEN BY:	Michael Chamberlain
TOPIC:	Optimisation of Gold Coast City's Chlorine Dosing System for the Southern Region Distribution Network
SUPERVISORS:	USQ: Ken Moore and Vasanthadevi Aravinthan GCW: Uttam Saha and Arran Canning
ENROLMENT:	ENG4111 – S1, EXT, 2007; ENG4112 – S2, EXT, 2007
SPONSORSHIP:	Gold Coast Water, Gold Coast City Council
PROJECT AIM:	To optimise Gold Coast Water's disinfection system. The project will specifically aim to remedy areas of inadequate disinfection within southern region distribution network and provide recommendations for an optimal disinfection treatment.
PROGRAMME:	To be completed October, 2007

1. Describe Gold Coast Water's town water reticulation system and water treatment practices and compare these against national and international standards.

2. Undertake a literature review to identify any previous studies undertaken in regard to the optimisation of disinfection treatments in drinking water supplies. Briefly describe each of these studies and comment on the suitability of each study technique for the reticulation system of the Gold Coast.

3. Identify an appropriate hydraulic and/or water quality model which could be used to simulate the effect of various disinfection treatments and/or pipeline reconfigurations on Gold Coast Water's reticulation network. Provide a detailed description of a suitable model, including its theoretical basis, structure, embodied physical processes, main parameters, data requirements and principal outputs.

4. Identify a suitable study area within the reticulation system and calibrate the model to this area. Validate the model against an independent data set and undertake a sensitivity analysis to assess the effect of errors in parameter values on the model output.

5. Apply the model to identify areas of inadequate disinfection within the study area. Use the model to simulate various alternative treatments and/or pipeline reconfigurations of the network in a bid to remedy these deficiencies.

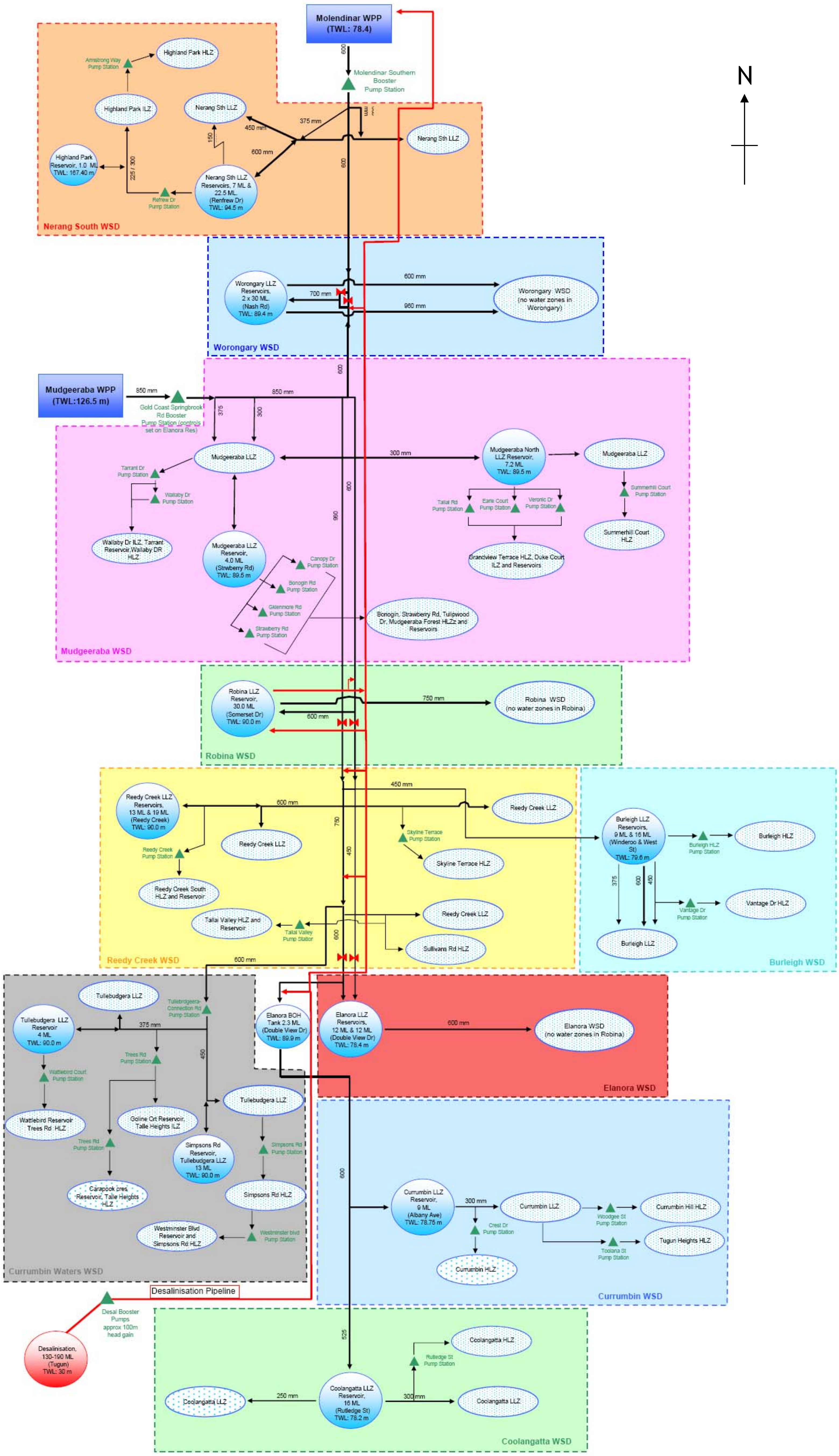
6. Undertake an economic analysis incorporating each of these alternatives and provide recommendations on an optimal disinfection treatment and/or pipeline reconfiguration strategy based on the results.

AGREED:

STUDENT	Michael Chamberlain	
SUPERVISORS		
USQ	Ken Moore KGMoore 28/3/07	Vasanthadevi Aravinthan V Aravin
GCW	Uttam Saha USaha	Arran Canning

APPENDIX B

SOUTHERN REGION WATER SUPPLY SCHEMATIC - EXISTING SYSTEM



APPENDIX C

C-1 DATA LOCATIONS

Table C-1: Logger and PRV locations

Trunk Logger Number	Pipe Size (mm)	Location Description	Data Aquired	Comments
1	600	17 Pappas Way Highland Park	Yes	
2	600	Off Nash Rd, upstream or Worongary reservoir connection	NO	didn't activate
3	375	University Dr, Bond Univsity Entrance, Southport	Yes	
4	450	End of Page St, Before connection into Burleigh reservoirs	Yes	Incorrect data
5	750	Upstream of Reedy Creek LLZ connection	Yes	Incorrect data
6	225	Aylesham Dr, Bonogin	Yes	
7	600	Upstream of Tullebudgera-Connection Rd PS	Yes	Incorrect data
8	450	Just Upstream of Elanora Reservoirs	Yes	
9	600	Monday Dr, upstream of Myall Mundi Reservoir	Yes	
10	225	Currumbin Creek Rd, Currumbin Waters	2 days only	Chamber flooded
11	300	Bienvenue Dr, Currumbin Waters	NO	Didn't activate
12	525	Stapylton Street	NO	Recorded Temp only
PRV Map Number	**Chamber Name**	**Chamber Location**	**Data Aquired**	**Comments**
1	Winderadeen Crt	1 Winderadeen Crt	NO	No Record
2	Jura Parade	2 Jura Parade	YES	
3	Bourton Rd	2 Bourton Rd	YES	
4	Gooding Dr	Corner Gooding Dr & Jondique Ave	NO	No Record
5	Palm Meadows Dr	Corner Gooding Dr & Palm Meadow Dr	YES	
6	Fairway Dr	13 Fairway Dr	YES	
7	Rio Vista Boulevard North	167 Rio Vista Blvd	YES	
8	Rio Vista Boulevard South	194 Rio Vista Blvd	YES	
9	Rio Vista Boulevard North	1 Rebecca Crt	YES	
10	Sunshine Boulevard	98 Sunshine Blvd	YES	
11	Sunshine Boulevard	3 Albicore St	YES	
12	Gold Coast Highway	2121 Gold Coast Hwy	YES	
13	Ben Lexcen Place	6 Ben Lexcen Place	YES	
14	Ron Penhaligon North	Corner of Cottesloe and Ron Penhaligon Way	YES	
15	Ron Penhaligon South	235 Ron Penhaligon Way	YES	
16	Robina Parkway	20 Gleniris Pl on Robina Parkway	YES	
17	Ron Penhaligon South	9/13 Federal Pl	YES	
18	Gold Coast Highway	Corner 1904 Gold Coast Highway and Christine Ave	NO	Waiting for installation of 300 mm main

19	Deodar Dr	On Deodar Dr at 30 Symonds Rd	YES	Waiting for installation of 300 mm main
20	Deodar Dr	33 Christine Ave	NO	
21	Mattocks Rd	27 Mattocks Rd	YES	
22	Varsity Sound	Corner Varsity Sound Ave and Coromandel Lane	YES	
23	Nineteenth Ave	Behind 36-38 Spundle St	NO	No Record
24	Phillipine Parade North	49 Palm Beach Ave	YES	
25	Phillipine Parade South	55 Phillipine Pde	YES	No Flow Recorded
26	Doubleview Dr	1 Chidlow Crt	NO	No Record
27	Doubleview Dr	44 Doubleview Dr	NO	Reading Error
28	Guineas Creek Rd	Roundabout at Corner Guiness Creek Rd and 19th Ave	NO	No Record

C-2 RESERVOIR DETAILS FOR CALIBRATION AND VALIDATION

Table C-2: Reservoir initial levels for calibration

Reservoir	Initial Level of Reservoir (% Full)	Max Level of Reservoir (m)	Initial Level of Reservoir (m)
Burleigh 4	72	6.31	4.5432
Canterbury Farms	Not available		
Carapook	44	3.8	1.672
Clover Hill	73	10	7.3
Coolangatta	79	10.2	8.058
Currumbin Valley	64	5.7	3.648
Currumbin LLZ	78	5.95	4.641
Duke Crt	62	3	1.86
Elanora 1	65	4.27	2.7755
Elanora 2	80	7.68	6.144
Elanora 3	80	7.66	6.128
Gilston	66	5.7	3.762
Goolabah	42	3.8	1.596
Grandview	68	2.7	1.836
Mol 4	55	Diurnal Pattern	
Mudg Clearwater	68	Diurnal Pattern	
Mudg North	75	9.5	7.125
Mudg Forest	46	4.88	2.2448
Myal Mundi	30	2.9	0.87
Nerang ILZ	74	4.85	3.589
Nerang South 1	78	9.2	7.176
Nerang South 2	79	9.5	7.505
Observatory	94	6	5.64
Pioneer downs	78	1.87	1.4586
Reedy Creek	71	10	7.1
Simpsons Rd	69	7.25	5.0025
Strawberry Rd	74	8.3	6.142
Tallai Hill estate	50	3.75	1.875
Tarrant Dr	31	8.9	2.759
Vantage Point	89	20	17.8
Westminster	85	4.33	3.6805
Windermere	63	3.6	2.268
Worongary 1	81	10	8.1
Worongary 2	85	10	8.5

C-3 TRUNK MAIN PRESSURE GRAPHS

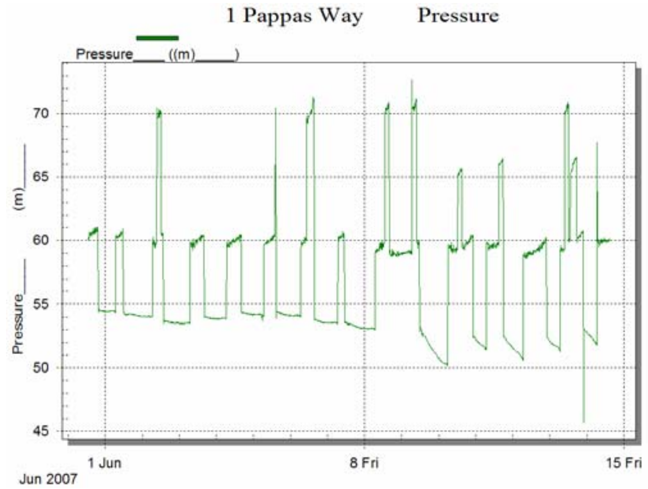


Figure C-3_1: Logger 1, Pappas Way pressure graph

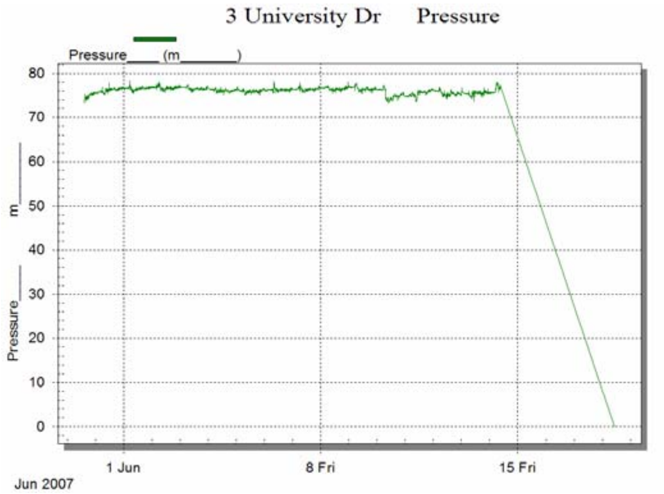


Figure C-3_2: Logger 3, University Dr pressure graph

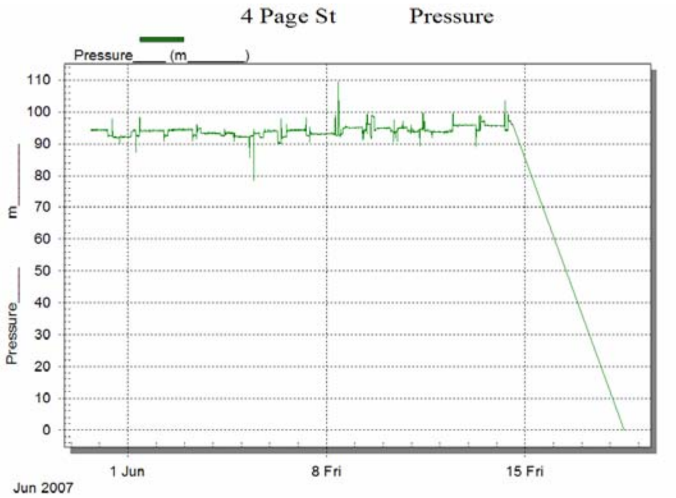


Figure C-3_3: Logger 4, Page St pressure graph

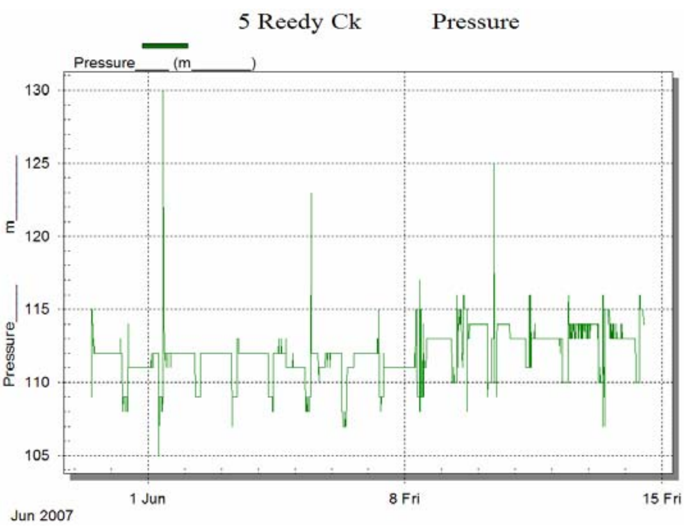


Figure C-3_4: Logger 5, Reedy Creek pressure graph

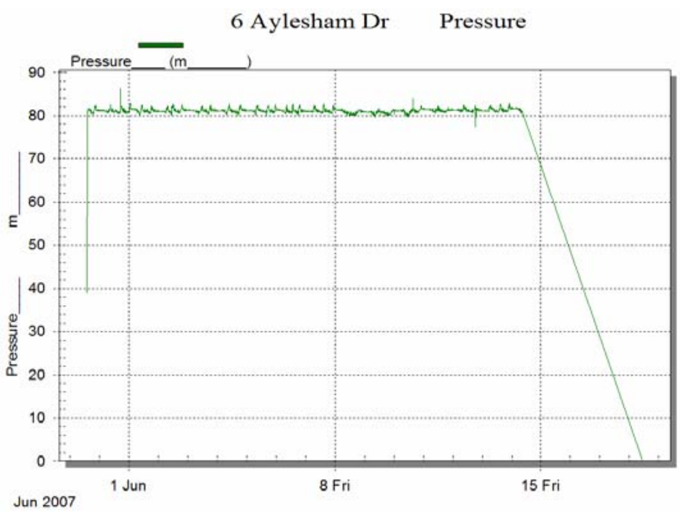


Figure C-3_5: Logger 6, Aylesham Dr pressure graph

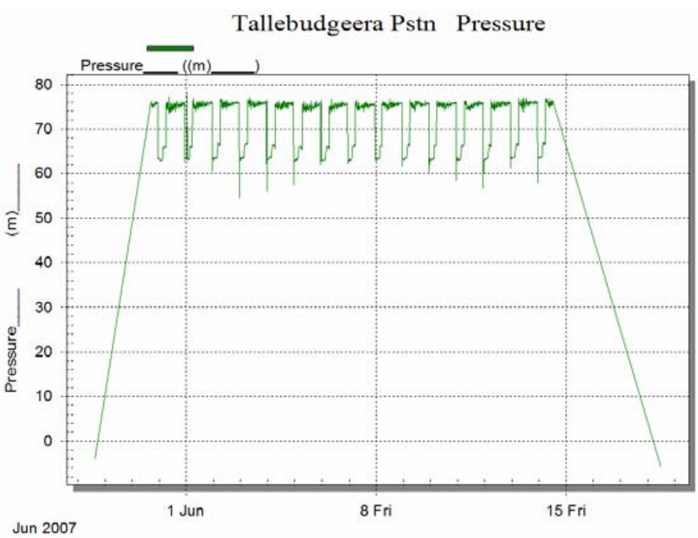


Figure C-3_6: Logger 7, Tallebudgera Pump Station pressure graph

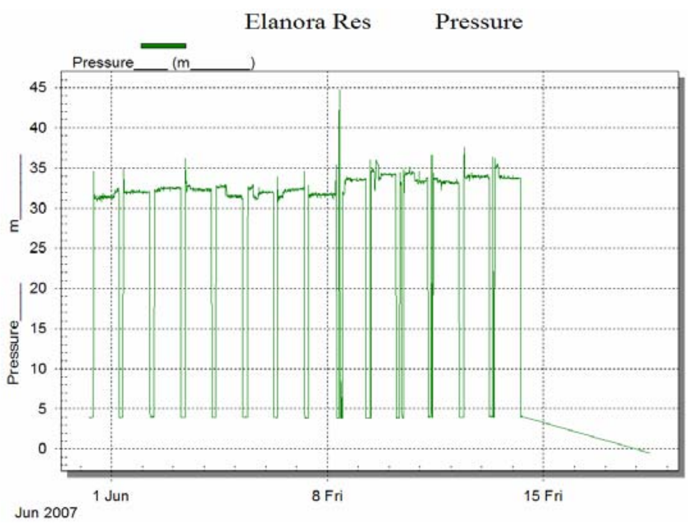


Figure C-3_7: Logger 8, Elanora Reservoir pressure graph

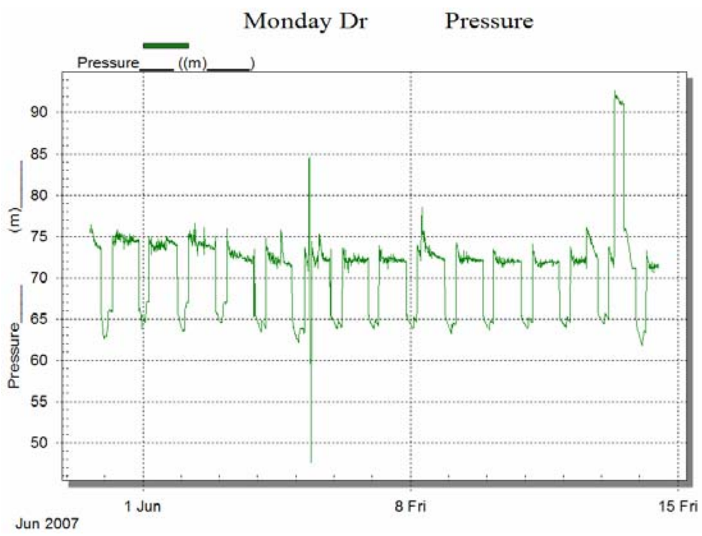


Figure C-3_8: Logger 9, Monday Dr pressure graph

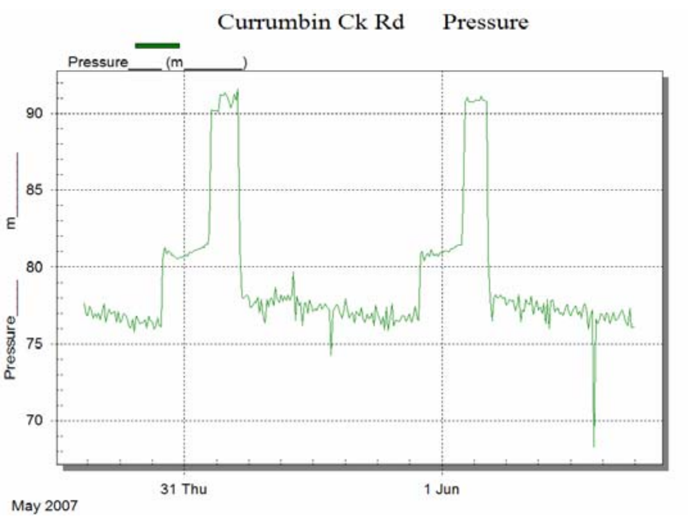


Figure C-3_9: Logger 10, Currumbin Creek Rd pressure graph

C-4 FIELD AND MODEL FLOW FOR JURA PARADE DMA SUPPLY FEED

The graph shows the correlation between the field and model flow after 3 iterations. A single iteration involves taking the % difference between the model flow and field flow and reducing the main demand pattern for the DMA by this difference. After 3 iterations a total difference of 1.26% was achieved (< 5%).

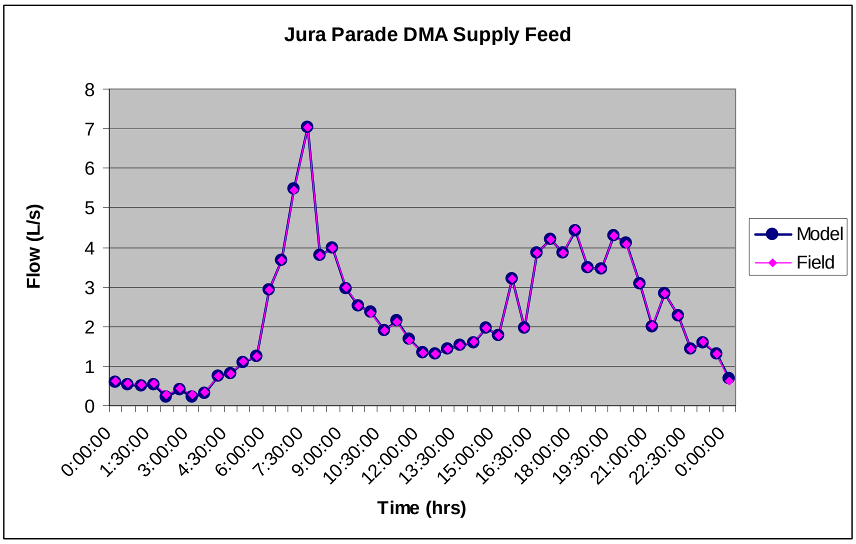


Figure C-4: Jura Parade model flow verse field flow

C-5 PHASE 1 CALIBRATION – LOGGER AND PRV PRESSURE GRAPHS FOR 31/05/07 (MODEL AND FIELD)

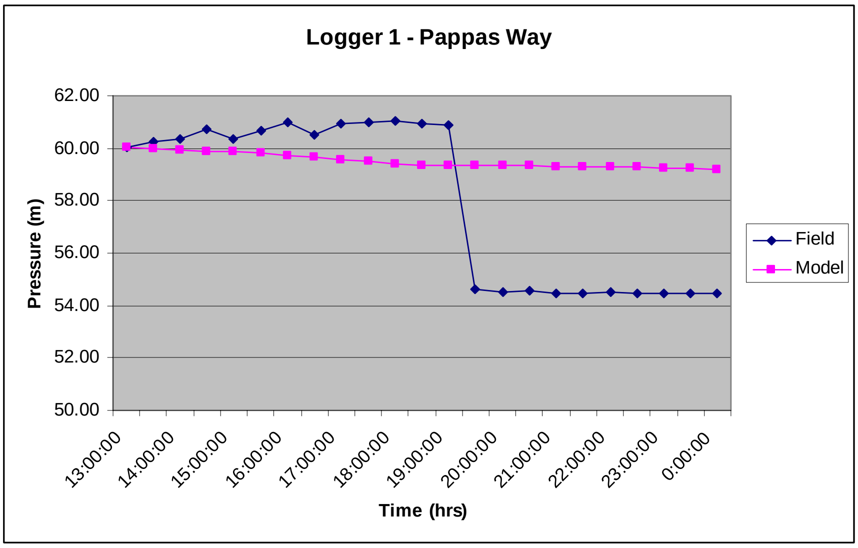


Figure C-5_1: Logger 1, Pappas Way model and field pressure graph - Calibration

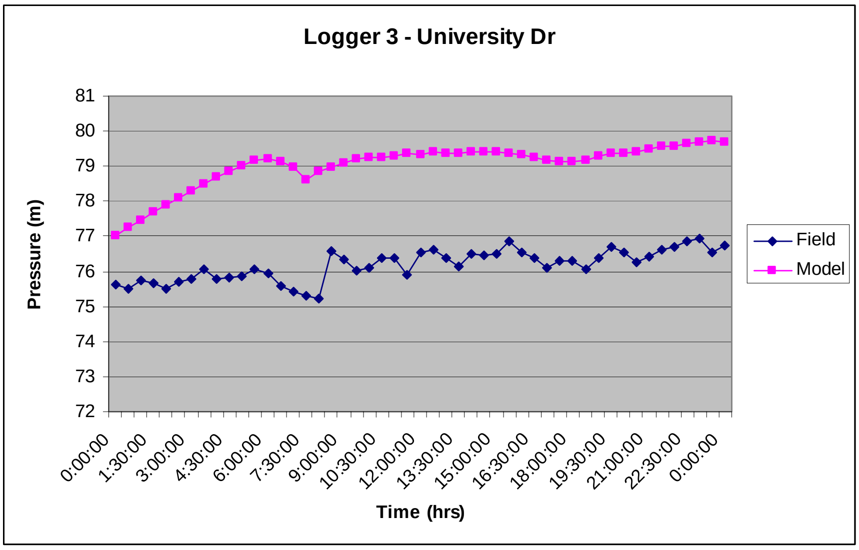


Figure C-5_2: Logger 3, University Dr model and field pressure graph - Calibration

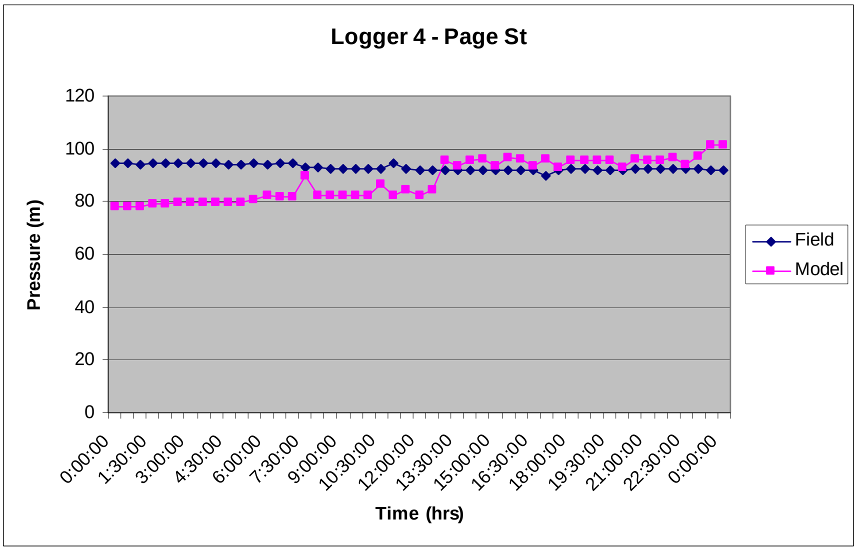


Figure C-5_3: Logger 4, Page St model and field pressure graph - Calibration

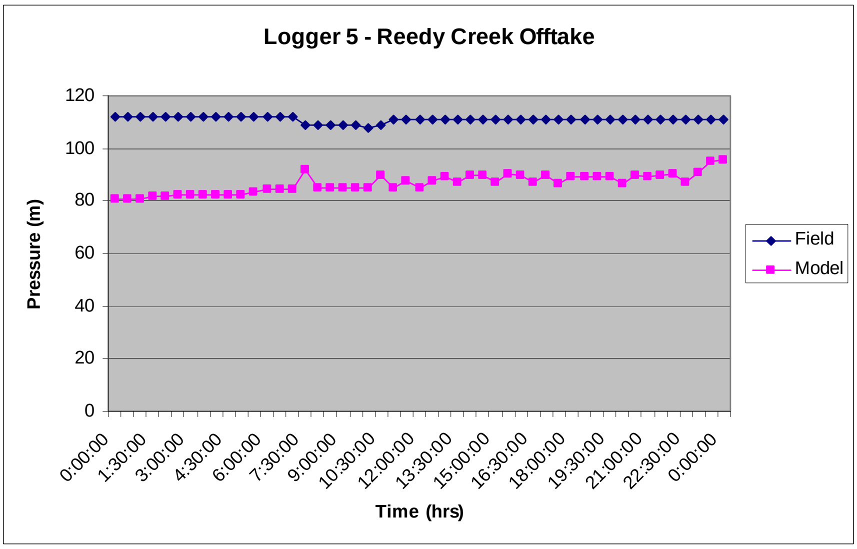


Figure C-5_4: Logger 5, Reedy Creek model and field pressure graph - Calibration

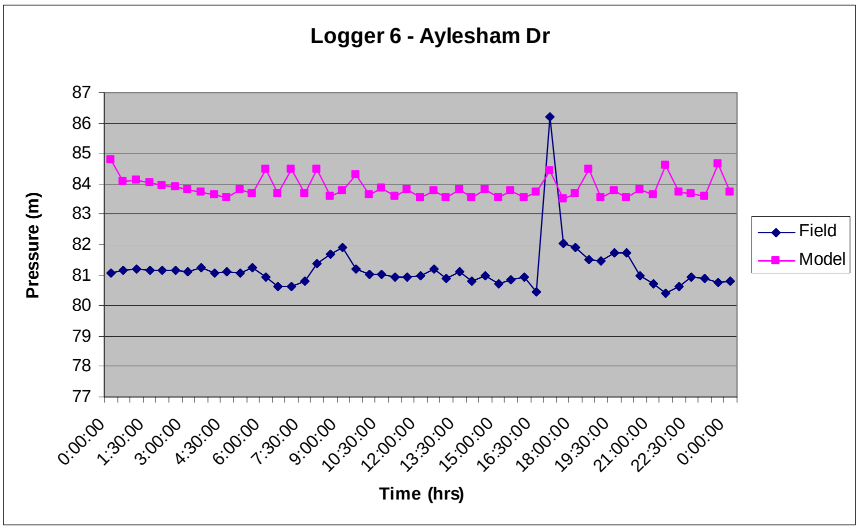


Figure C-5_5: Logger 6, Alyesham Dr model and field pressure graph - Calibration

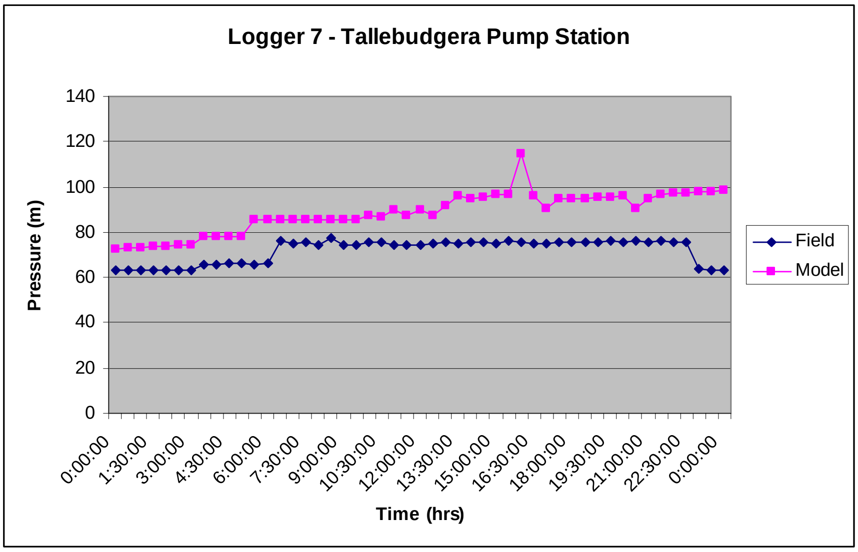


Figure C-5_6: Logger 7, Tallebudgera Pump Station model and field pressure graph - Calibration

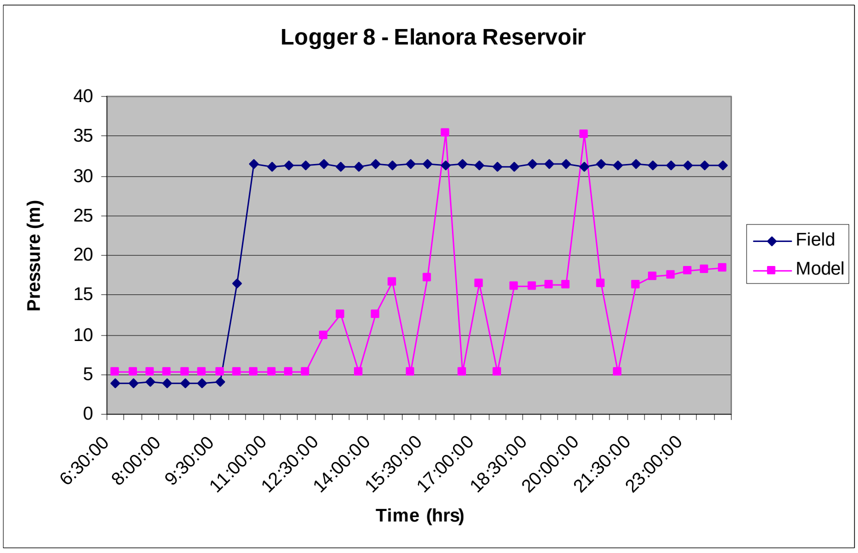


Figure C-5_7: Logger 8, Elanora Reservoir model and field pressure graph - Calibration

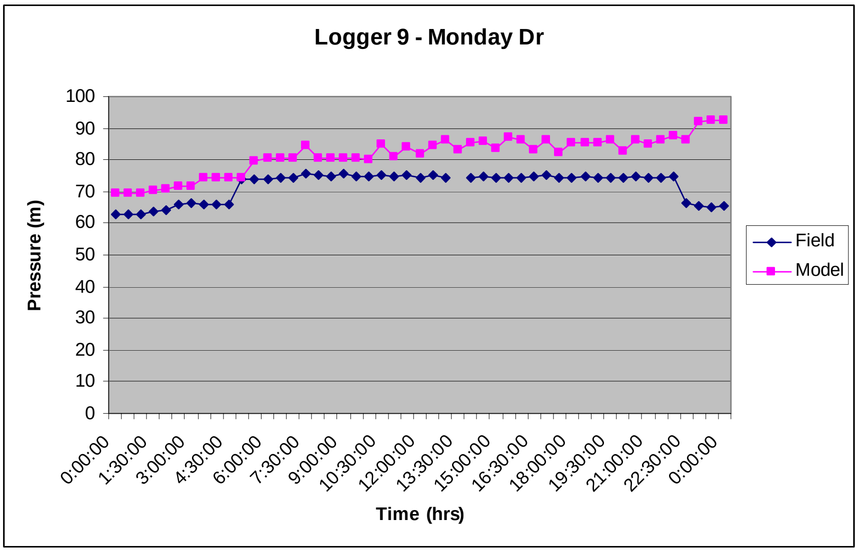


Figure C-5_8: Logger 9, Monday Dr model and field pressure graph - Calibration

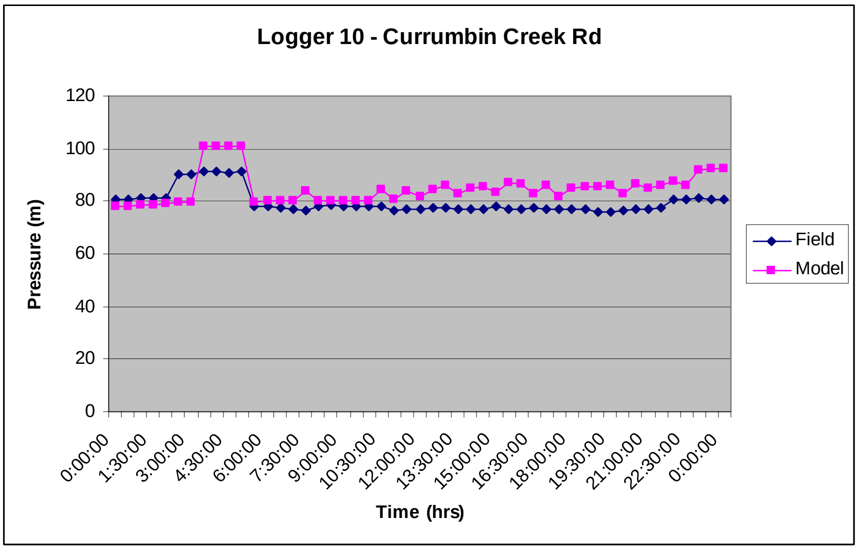


Figure C-5_9: Logger 10, Currumbin Creek Rd model and field pressure graph - Calibration

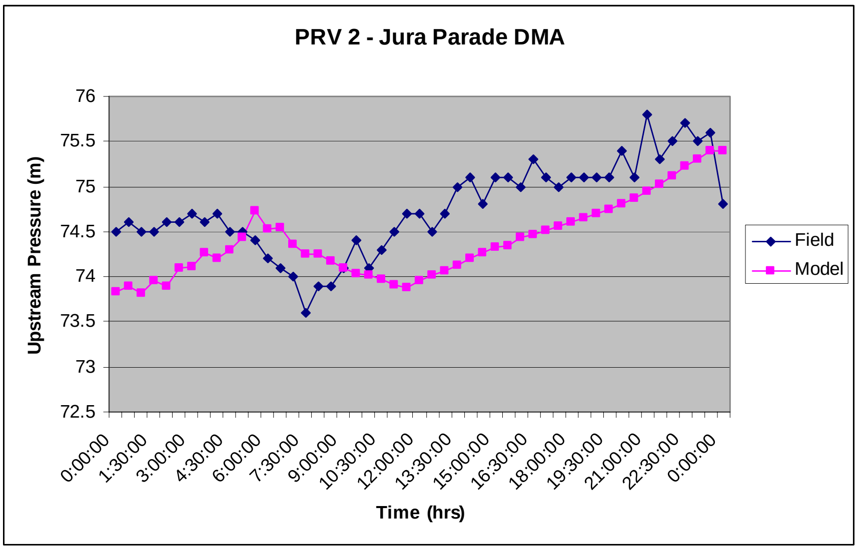


Figure C-5_10: PRV 2, Jura Parade model and field pressure graph - Calibration

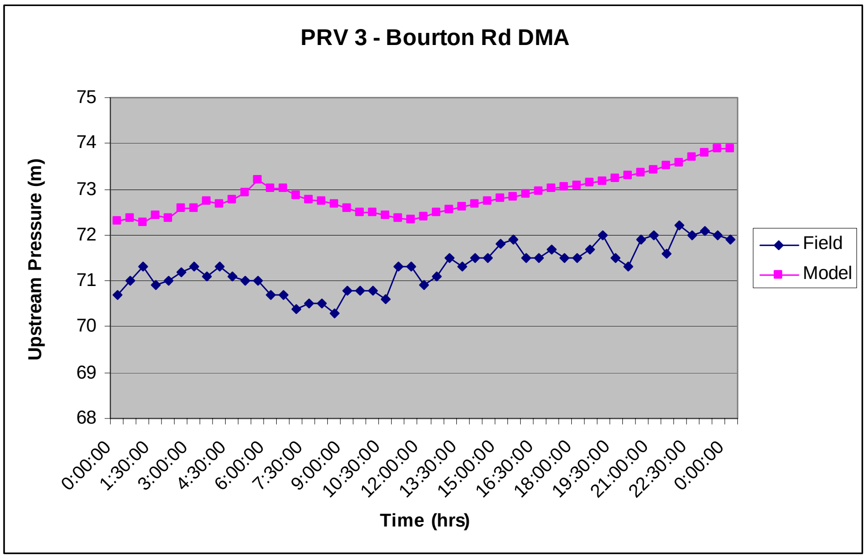


Figure C-5_11: PRV 3, Bourton Rd model and field pressure graph - Calibration

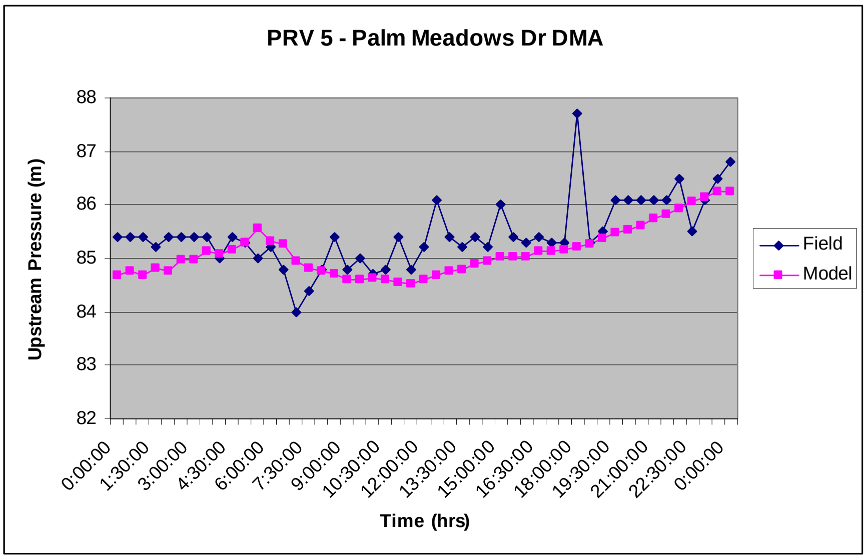


Figure C-5_12: PRV 5, Palm Meadows Dr model and field pressure graph - Calibration

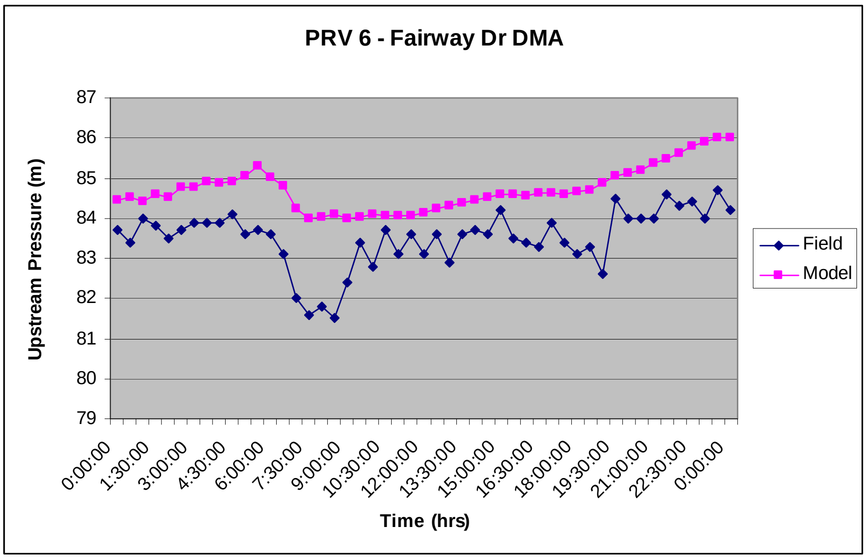


Figure C-5_13: PRV 6, Fairway Dr model and field pressure graph - Calibration

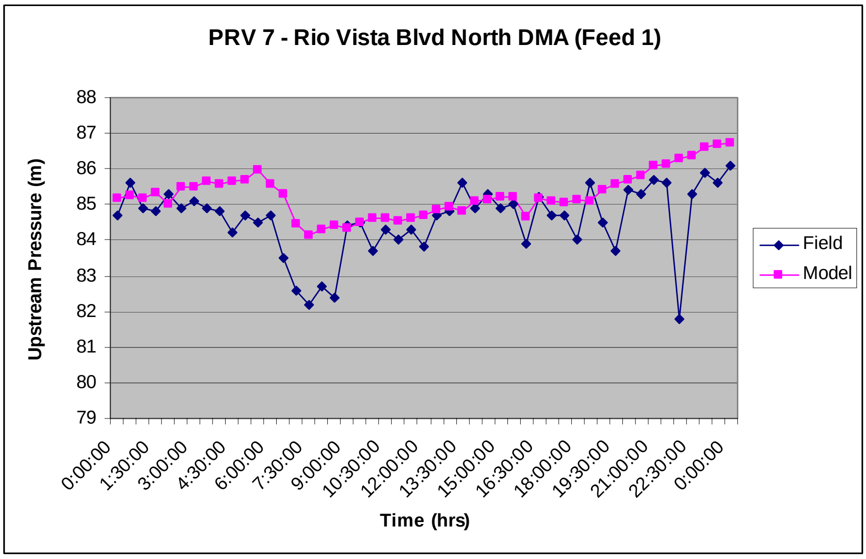


Figure C-5_14: PRV 7, Rio Vista Boulevard North (feed 1) model and field pressure graph - Calibration

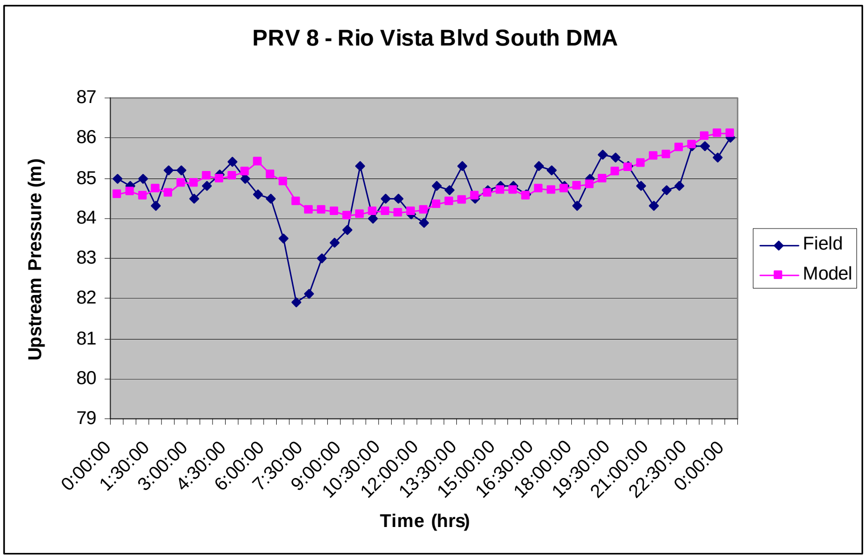


Figure C-5_15: PRV 8, Rio Vista Boulevard South model and field pressure graph - Calibration

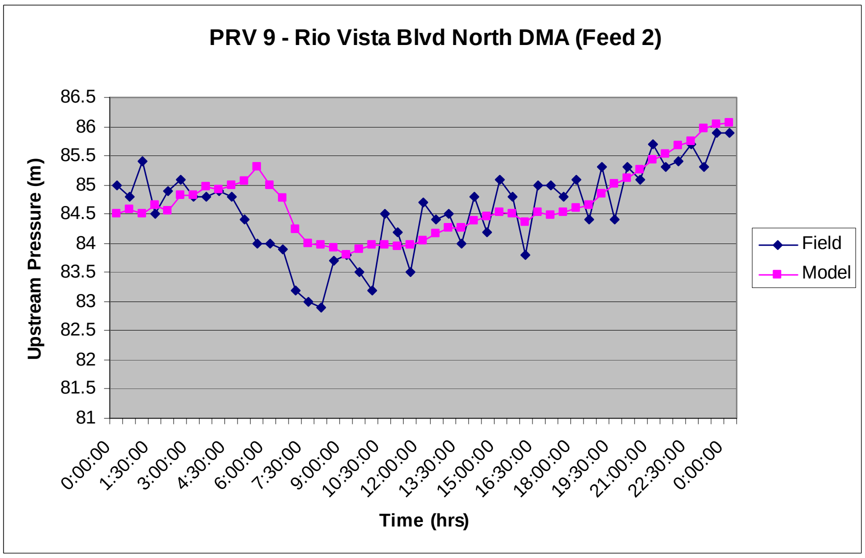


Figure C-5_16: PRV 9, Rio Vista Boulevard North (feed 2) model and field pressure graph - Calibration

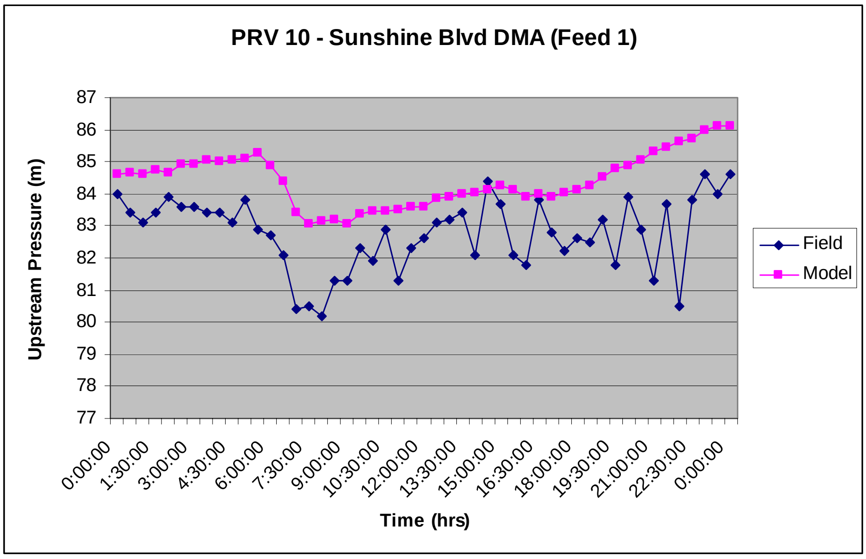


Figure C-5_17: PRV 10, Sunshine Boulevard (feed 1) model and field pressure graph - Calibration

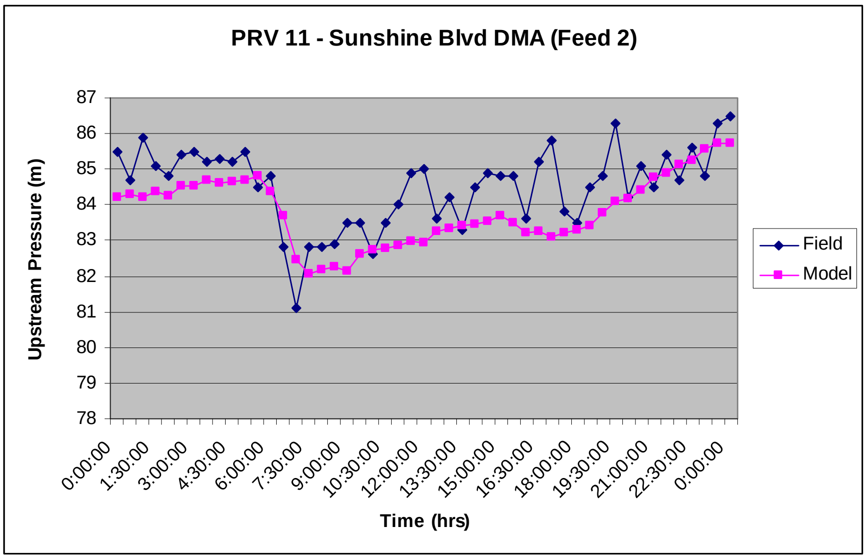


Figure C-5_18: PRV 11, Sunshine Boulevard (feed 2) model and field pressure graph - Calibration

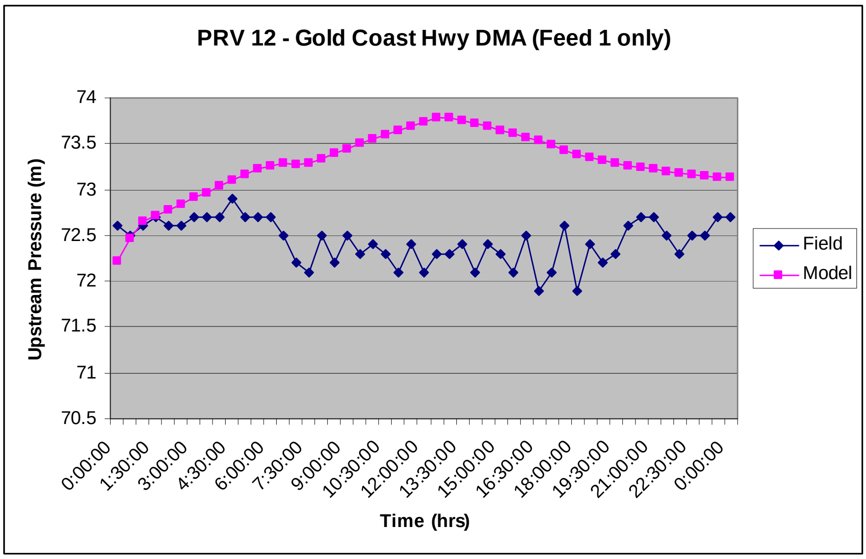


Figure C-5_19: PRV 12, Gold Coast Highway (feed 1) model and field pressure graph - Calibration

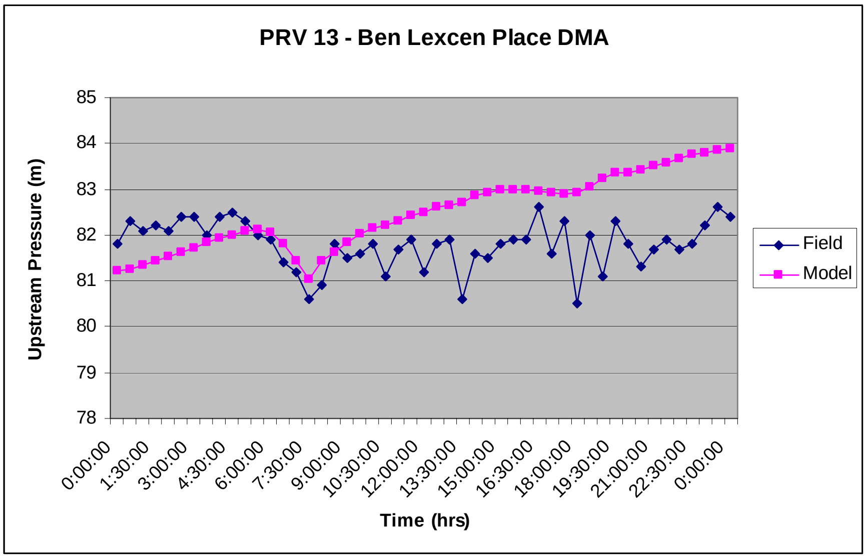


Figure C-5_20: PRV 13, Ben Lexcen Place model and field pressure graph - Calibration

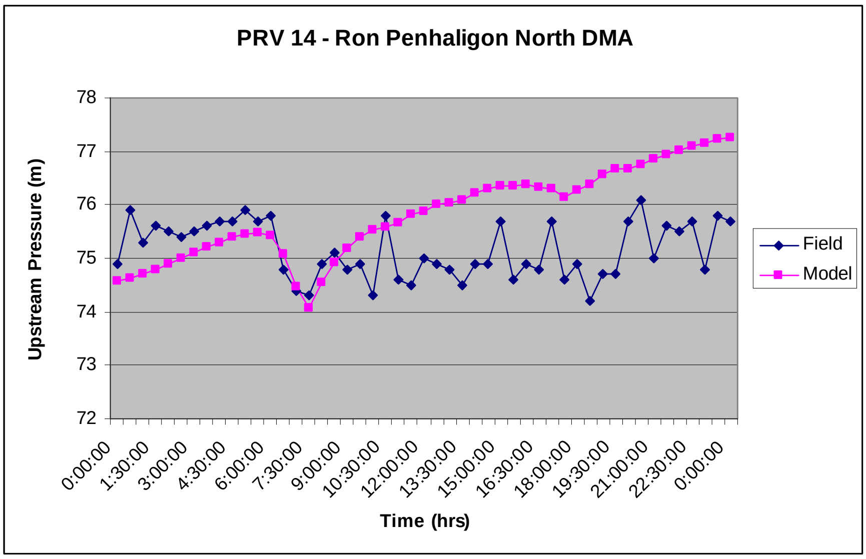


Figure C-5_21: PRV 14, Ron Penhaligon North model and field pressure graph - Calibration

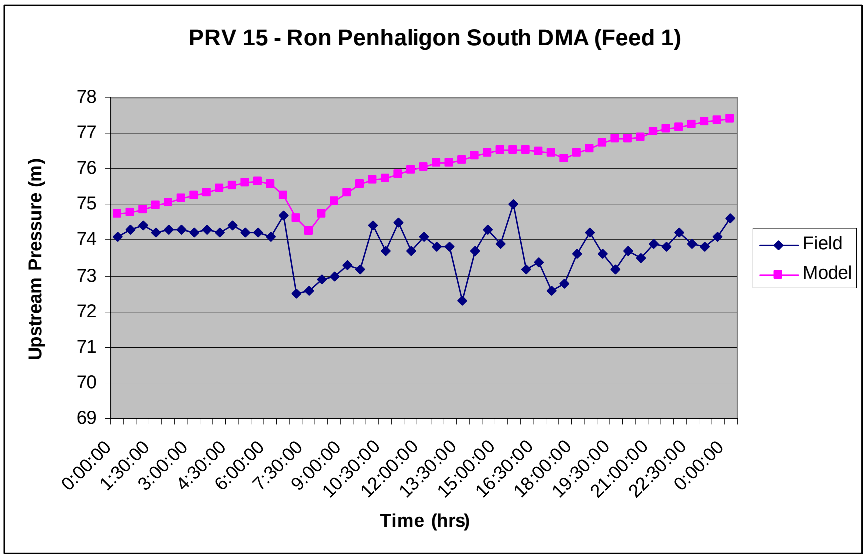


Figure C-5_22: PRV 15, Ron Penhaligon South (feed 1) model and field pressure graph - Calibration

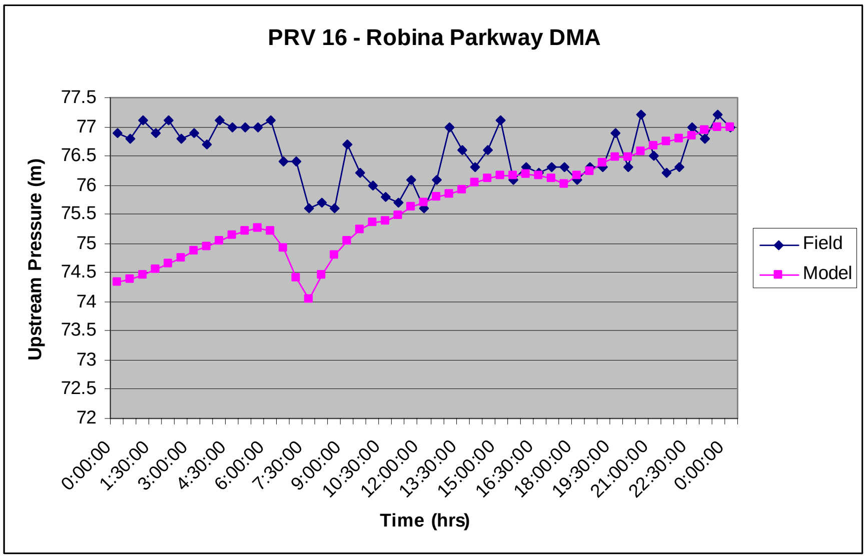


Figure C-5_23: PRV 16, Robina Parkway model and field pressure graph - Calibration

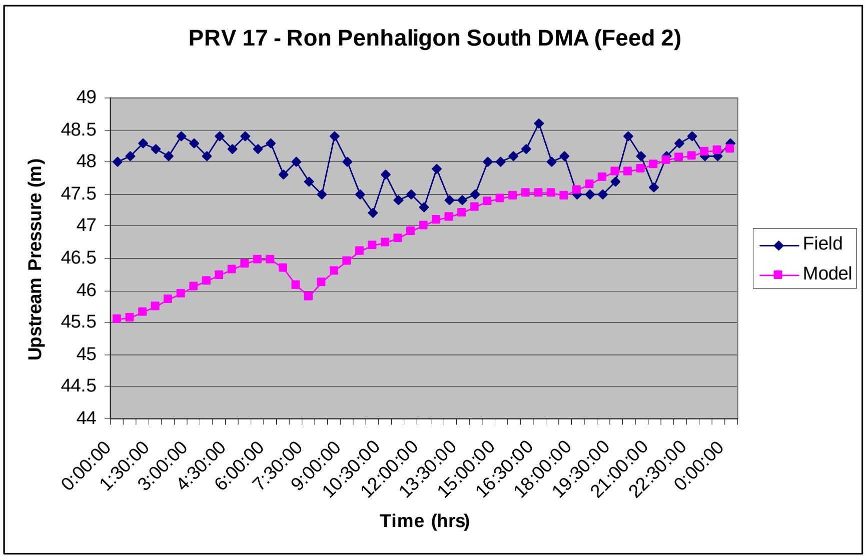


Figure C-5_24: PRV 17, Ron Penhaligon South (feed 2) model and field pressure graph - Calibration

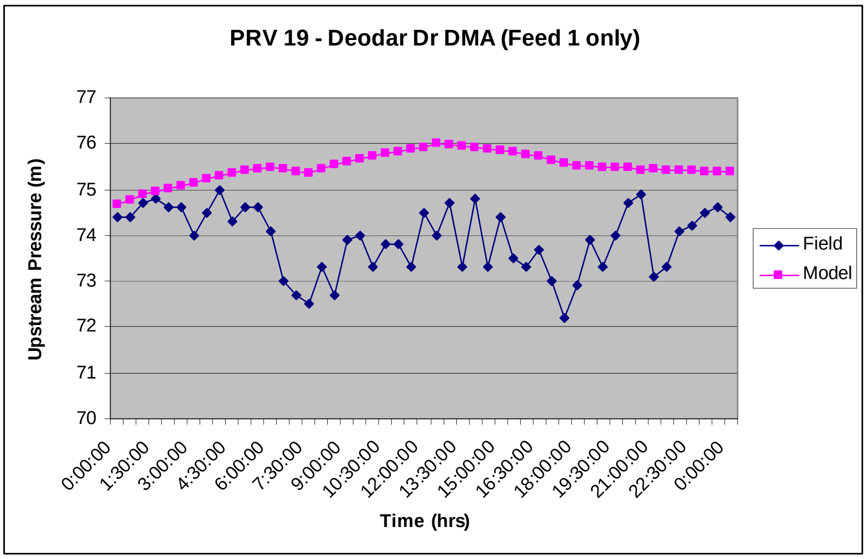


Figure C-5_25: PRV 19, Deodar Dr (feed 1) model and field pressure graph - Calibration

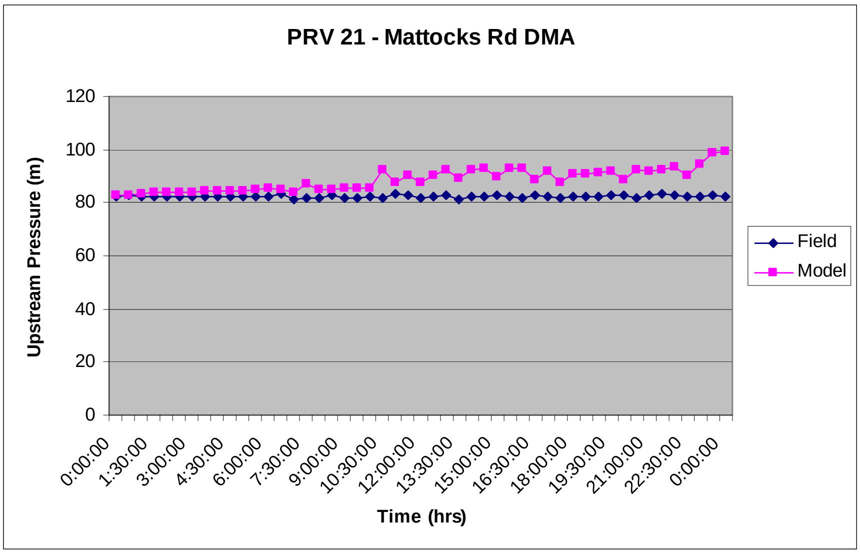


Figure C-5_26: PRV 21, Mattocks Rd model and field pressure graph - Calibration

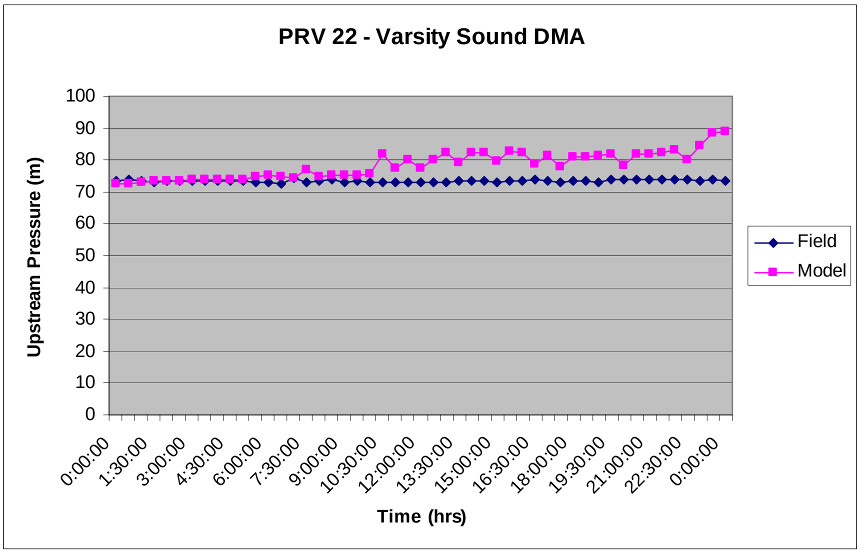


Figure C-5_27: PRV 22, Varsity Sound model and field pressure graph - Calibration

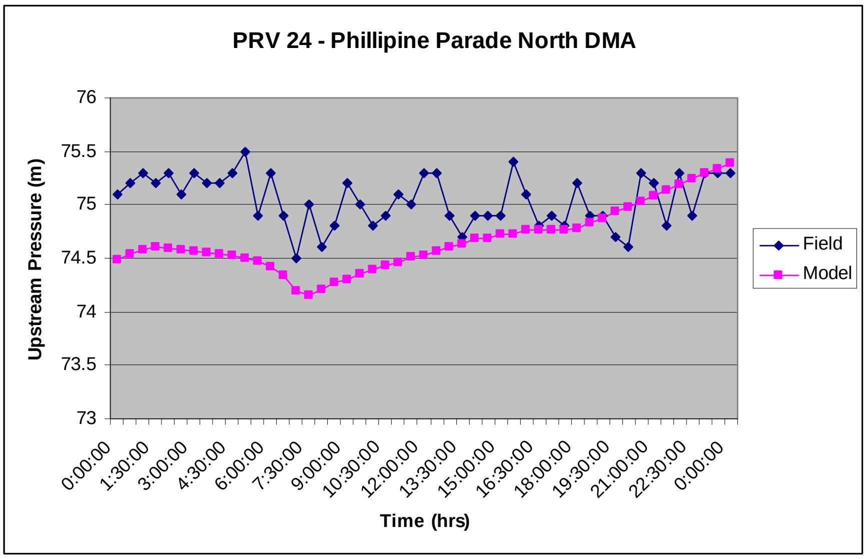


Figure C-5_28: PRV 24, Phillipine Parade North model and field pressure graph - Calibration

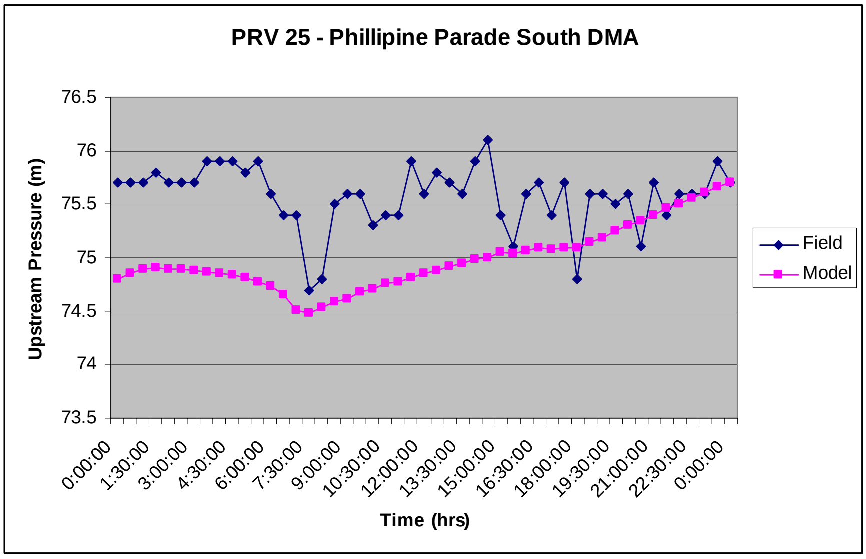


Figure C-5_29: PRV 25, Phillipine Parade South model and field pressure graph - Calibration

C-6 PHASE 1 VALIDATION - LOGGER AND PRV PRESSURE GRAPHS FOR 9/06/07 (MODEL AND FIELD)

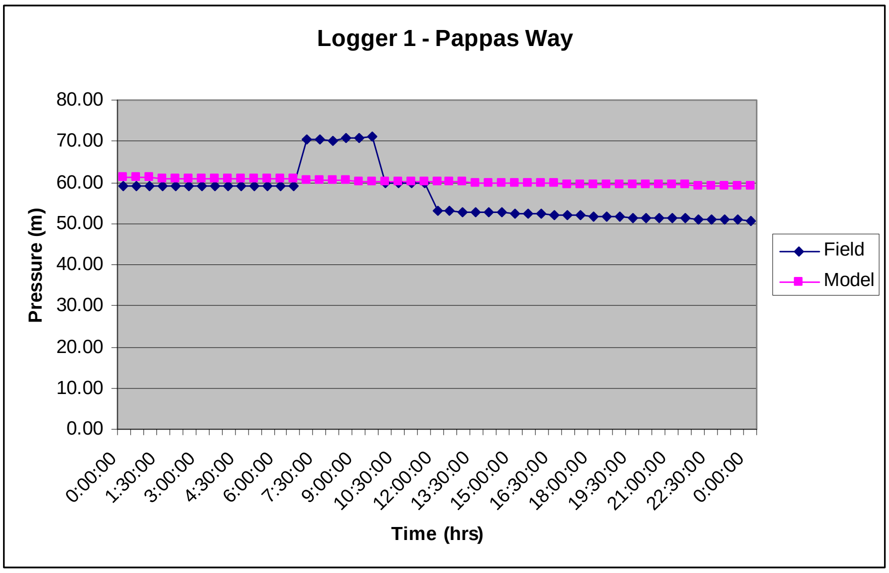


Figure C-6_1: Logger 1, Pappas Way model and field pressure graph - Validation

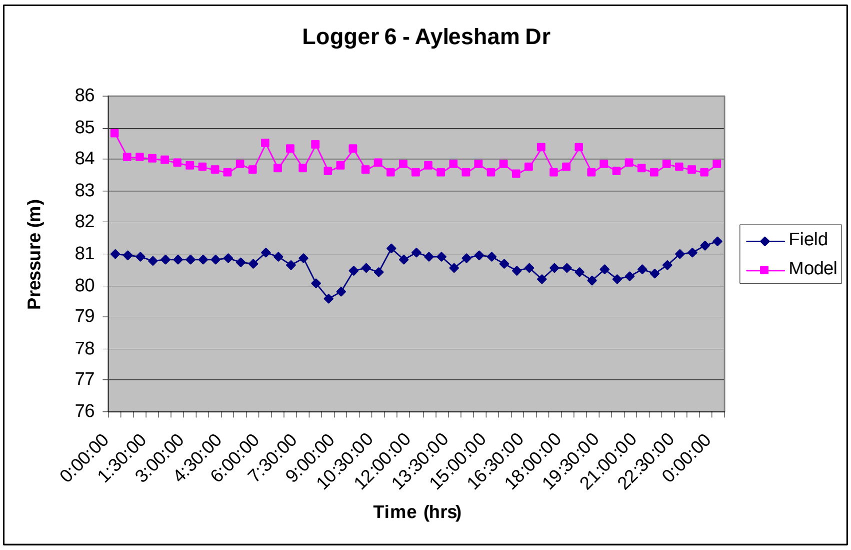


Figure C-6_2: Logger 6, Alyesham Dr model and field pressure graph - Validation

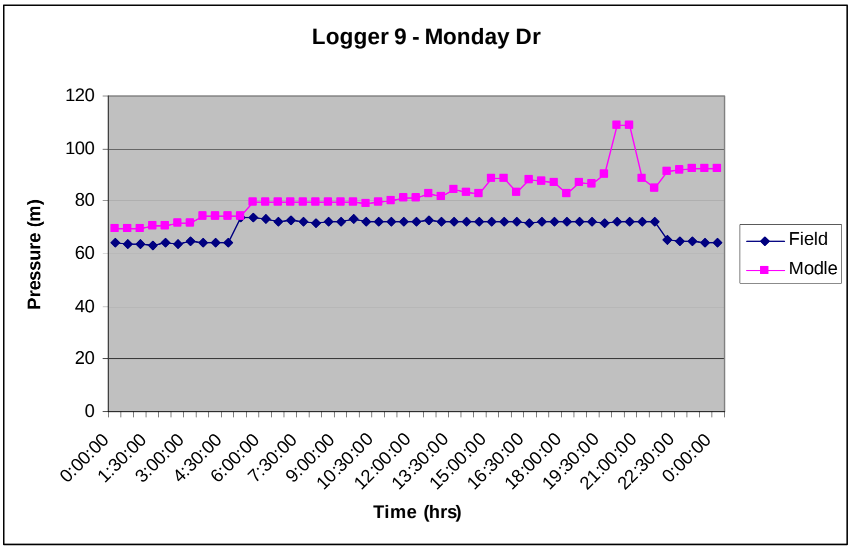


Figure C-6_3: Logger 9, Monday Dr model and field pressure graph - Validation

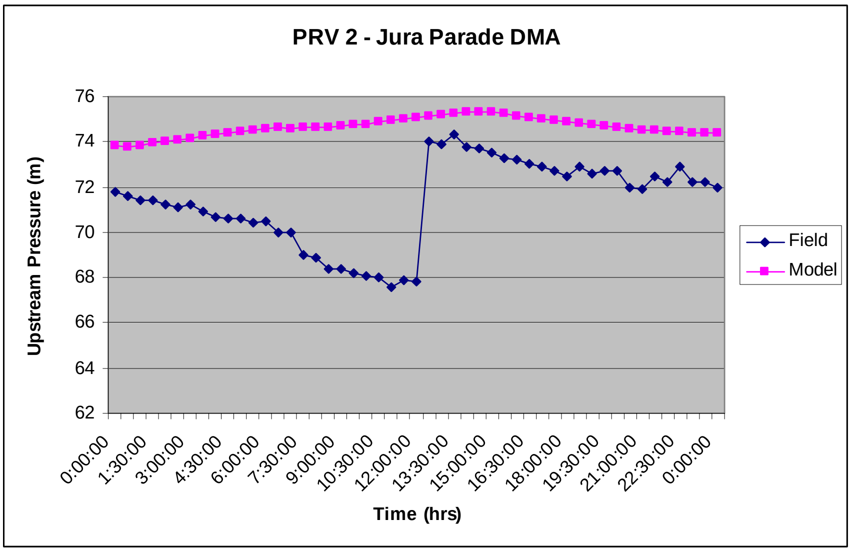


Figure C-6_4: PRV 2, Jura Parade model and field pressure graph - Validation

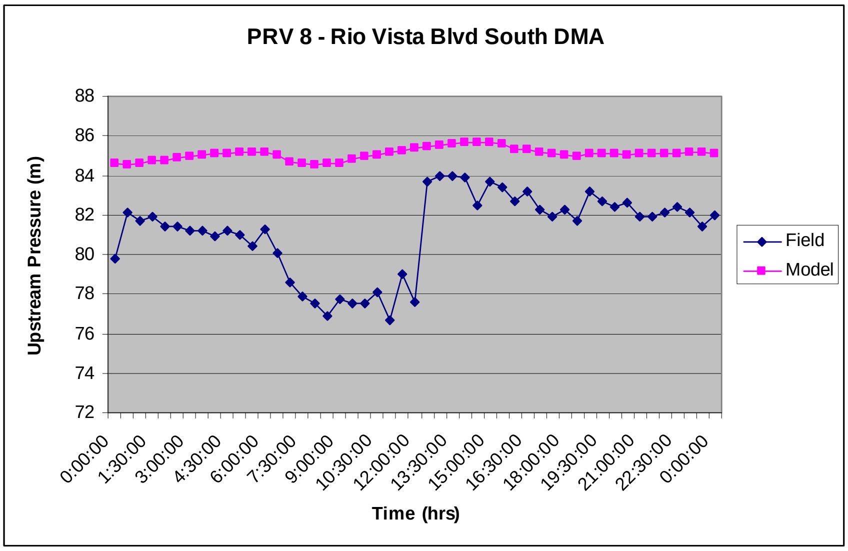


Figure C-6_5: PRV 8, Rio Vista Boulevard South model and field pressure graph - Validation

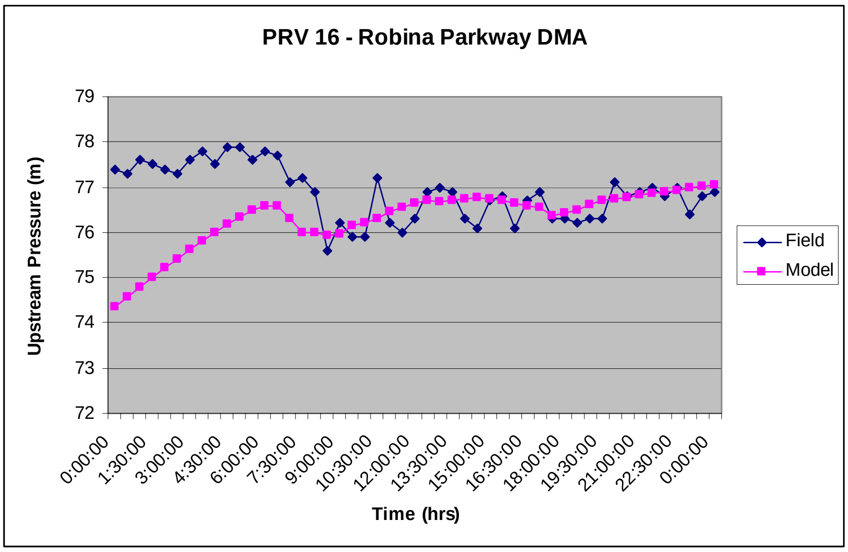


Figure C-6_6: PRV 16, Robina Parkway model and field pressure graph - Validation

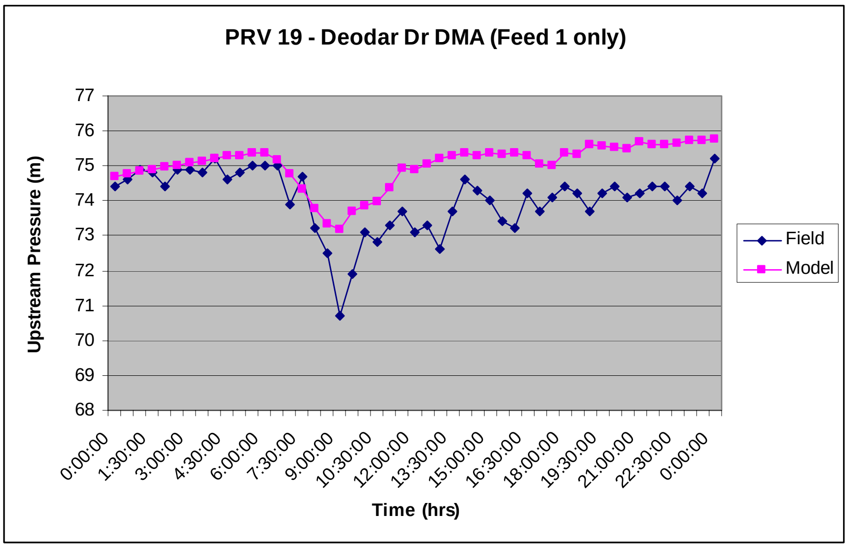


Figure C-6_7: PRV 19, Deodar Dr model and field pressure graph - Validation

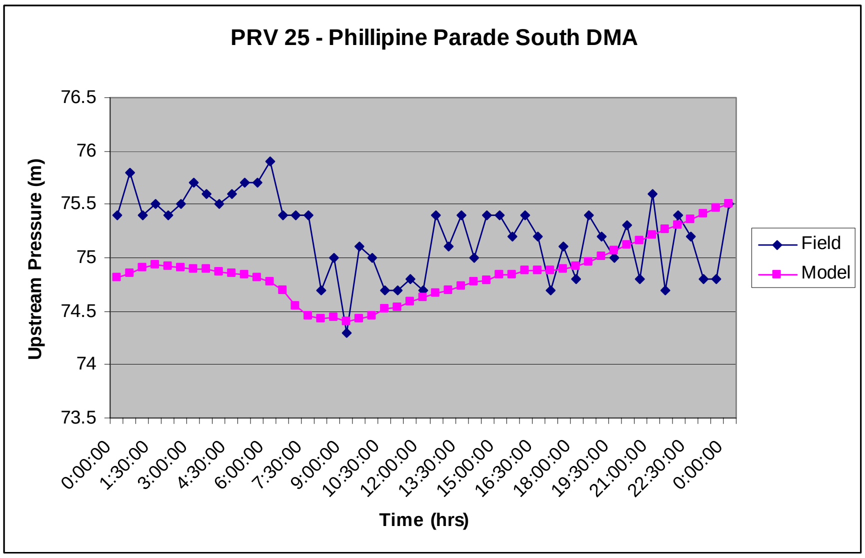


Figure C-6_8: PRV 25, Phillipine Parade South model and field pressure graph - Validation

C-7 PHASE 2 VALIDATION – FIELD AND MODEL CHLORINE COMPARISON

Table C-7: Model and field chlorine comparison

HACCP Test Site	Average Cl- Residual 06/06 - 02/07 (mg/L)	Sample Taken 7/06/07		Sample Taken 8/06/08		Model Cl- Residual Concentration			
		Conc. (mg/L)	Time	Conc. (mg/L)	Time	@ Sample Time (mg/L)	Min (mg/L)	max (mg/L)	Ave (mg/L)
BARDEN RIDGE RD									
BIENVENUE DR	0.11	0.00	03:15:00			0.00	0.00	0.00	0.00
BINYA AV	0.1	0.00	14:35:00			0.00	0.00	0.00	0.00
BRIDGMAN DR	0.1			0.00	15:00:00	0.00	0.00	0.00	0.00
BURLEIGH HEADS SLSC	0.14	0.15	13:10:00			0.00	0.00	0.00	0.00
BURLIEGH ST	0.14	0.00	13:00:00			0.00	0.00	0.00	0.00
CARAPOOK CT	0.1			0.00	14:00:00	0.00	0.00	0.00	0.00
CARRARA STADIUM	0.14	0.00	09:45:00			0.00	0.00	0.13	0.06
CHELTENHAM DR	0.17	0.00	11:35:00			0.00	0.00	0.00	0.00
COOLANGATTA RD									
CORALCOAST DR	0.1						0.00	0.00	0.00
DARYL RADNELL DR	0.16			0.00	11:50:00	0.00	0.00	0.00	0.00
DUKE CT	0.11			0.00	11:00:00	0.04	0.00	0.22	0.04
DUROBBY DR	0.1			0.00	14:15:00	0.00	0.00	0.00	0.00
ESPLANADE	0.13	0.00	12:30:00			0.00	0.00	0.00	0.00
GALLEON WY	0.13	0.00	15:30:00			0.05	0.04	0.09	0.05
GARDENIA DR	0.1			0.00	16:25:00	0.00	0.00	0.00	0.00
GOLD COAST HWY	0.21	0.20	10:25:00			0.00	0.00	0.00	0.00
GOLINE CRT	0.1			0.00	13:55:00	0.04	0.00	0.10	0.03
GRANDVIEW TCE				0.00	22:45:00	0.00	0.00	0.00	0.00

HARDYS RD	0.5			0.25	16:10:00	0.61	0.04	0.74	0.21
HARRY MILLS DR	0.17			0.00	11:30:00	0.00	0.00	0.00	0.00
HEATHER STREET	0.37	0.45	16:35:00			0.05	0.05	0.06	0.05
JOHN ROGERS RD	0.6	0.85	Ave				0.11	1.10	0.83
KURRAWA SLSC	0.17	0.25	10:35:00			0.00	0.00	0.00	0.00
LAGUNA AVE	0.19	0.00	13:45:00			0.05	0.02	0.05	0.03
LAMBOR DR	0.19	0.25	17:15:00			0.06	0.05	0.08	0.06
MACKELLAR DR	0.15						0.00	0.17	0.05
MARRIOT WY	0.1			0.00	08:55:00	0.00	0.00	0.00	0.00
MERRIMAC WWTP				0.00	09:35:00	0.00	0.00	0.00	0.00
MIRREEN DR	0.1	0.20	15:00:00			0.00	0.00	0.00	0.00
MURLONG CRES	0.1	0.00	13:30:00			0.00	0.00	0.00	0.00
NASH RD (A)	0.3			0.40	10:00:00	0.17	0.05	0.20	0.15
NASH RD (B)	0.1			0.25	10:10:00	0.05	0.04	0.06	0.05
NASH RD (C)	0.43			0.00	09:50:00	0.38	0.01	0.82	0.34
NIELSENS RD	0.23	0.15	08:55:00			0.07	0.02	0.78	0.33
NOBBY BEACH SLSC	0.13	0.00	10:45:00			0.00	0.00	0.00	0.00
NORTHWESTERN CT	0.2	0.00	11:55:00			0.01	0.01	0.04	0.03
PACIFIC PDE	0.1	0.00	14:00:00			0.00	0.00	0.00	0.00
RECREATION DR	0.2	0.40	09:10:00			0.20	0.01	0.35	0.14
RIO VISTA BLVD	0.13	0.00	11:10:00			0.00	0.00	0.00	0.00
ROBINA RESERVOIR	0.2			0.15	16:00:00	0.15	0.15	0.19	0.16
RON PENHALIGAN WY	0.26						0.00	0.00	0.00
SAUNDERS DR	0.13			0.00	16:45:00	0.02	0.01	0.10	0.40
SEMISH CRT	0.16			0.00	04:35:00	0.00	0.00	0.00	0.00
SOMERSET DR	0.41			0.40	11:55:00	0.31	0.00	0.39	0.13
SONIA ST	0.17	0.00	11:00:00			0.00	0.00	0.00	0.00
SOUTHERN SKIES AVE	0.1			0.00	15:30:00	0.00	0.00	0.00	0.00
TALLAI RD	0.14			0.00	10:50:00	0.04	0.01	0.05	0.03
TALLEBUDGERA CONNECTION RD	0.35	0.00	16:00:00			0.11	0.00	0.29	0.18
TALLEBUDGERA RECREATION CAMP	0.12	0.00	13:20:00			0.00	0.00	0.00	0.00
TARBERT CL	0.19			0.20	09:10:00	0.05	0.02	0.09	0.07
THURSDAY DR	0.16			0.00	13:40:00	0.00	0.00	0.00	0.00
TREES RD	0.35	0.00	16:00:00			0.13	0.05	0.21	0.13
TWEED TCE	0.1	0.00	02:20:00			0.00	0.00	0.00	0.00
VANTAGE POINT DR	0.13						0.00	0.00	0.00
WALLABY DR	0.44			0.50	16:50:00	0.39	0.02	0.39	0.07
WESTMINSTER BLVD	0.16						0.00	0.00	0.00
YODELAY DR	0.17	0.00	16:50:00			0.12	0.02	0.12	0.06

APPENDIX D

D-1 HISTORICAL CHLORINE CONCENTRATION RECORD (JUNE 2006 TO FEB 2007)

Table D-1: Historical chlorine concentrations for 59 test sites in the southern region

Analysis Group	Registration Date	Sample Description	Test Results	Estimated Value	
PWDS - A Run	4/07/2006	Bienvenue Drive, Currumbin (A1 C01)	< 0.2	0.1	
PWDS - A Run	2/08/2006	Bienvenue Drive, Currumbin (A1 C01)	< 0.2	0.1	
PWDS - A Run	6/09/2006	Bienvenue Drive, Currumbin (A1 C01)	< 0.2	0.1	
PWDS - A Run	4/10/2006	Bienvenue Drive, Currumbin (A1 C01)	< 0.2	0.1	
PWDS - A Run	1/11/2006	Bienvenue Drive, Currumbin (A1 C01)	< 0.2	0.1	
PWDS - A Run	29/11/2006	Bienvenue Drive, Currumbin (A1 C01)	< 0.2	0.1	
PWDS - A Run	3/01/2007	Bienvenue Drive, Currumbin (A1 C01)	< 0.2	0.1	
PWDS - A Run	30/01/2007	Bienvenue Drive, Currumbin (A1 C01)	0.2	0.2	0.11
PWDS - B Run	12/07/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - B Run	9/08/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - B Run	14/09/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - B Run	10/10/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - B Run	8/11/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - B Run	6/12/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - B Run	10/01/2007	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - D Run	3/07/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - D Run	26/07/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - D Run	23/08/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - D Run	26/09/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - D Run	25/10/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - D Run	22/11/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - D Run	19/12/2006	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	
PWDS - D Run	24/01/2007	Binya Avenue, Coolangatta (B1/D1CO01)	< 0.2	0.1	0.10
PWDS - C Run	19/07/2006	Bridgeman Drive, Stephens (C1 SS01)			
PWDS - C Run	16/08/2006	Bridgeman Drive, Stephens (C1 SS01)	< 0.2	0.1	
PWDS - C Run	20/09/2006	Bridgeman Drive, Stephens (C1 SS01)	< 0.2	0.1	
PWDS - C Run	18/10/2006	Bridgeman Drive, Stephens (C1 SS01)	< 0.2	0.1	
PWDS - C Run	15/11/2006	Bridgeman Drive, Stephens (C1 SS01)	< 0.2	0.1	
PWDS - C Run	13/12/2006	Bridgeman Drive, Stephens (C1 SS01)	< 0.2	0.1	0.10
PWDS - A Run	4/07/2006	Burleigh SLSC, Burleigh Heads (A1 BU04)	< 0.2 T	0.1	
PWDS - A Run	1/08/2006	Burleigh SLSC, Burleigh Heads (A1 BU04)	< 0.2	0.1	
PWDS - A Run	5/09/2006	Burleigh SLSC, Burleigh Heads (A1 BU04)	< 0.2	0.1	
PWDS - A Run	2/10/2006	Burleigh SLSC, Burleigh Heads (A1 BU04)	0.2	0.2	
PWDS - A Run	31/10/2006	Burleigh SLSC, Burleigh Heads (A1 BU04)	< 0.2	0.1	
PWDS - A Run	28/11/2006	Burleigh SLSC, Burleigh Heads (A1 BU04)	0.3	0.3	
PWDS - A Run	2/01/2007	Burleigh SLSC, Burleigh Heads (A1 BU04)	< 0.2	0.1	
PWDS - A Run	30/01/2007	Burleigh SLSC, Burleigh Heads (A1 BU04)	< 0.2	0.1	0.14
PWDS - D Run	3/07/2006	Burleigh Street, Burleigh Waters (D1 BU04)	< 0.2	0.1	
PWDS - D Run	26/07/2006	Burleigh Street, Burleigh Waters (D1 BU04)	< 0.2	0.1	
PWDS - D Run	23/08/2006	Burleigh Street, Burleigh Waters (D1 BU04)	0.2	0.2	
PWDS - D Run	26/09/2006	Burleigh Street, Burleigh Waters (D1 BU04)	0.2	0.2	
PWDS - D Run	25/10/2006	Burleigh Street, Burleigh Waters (D1 BU04)	< 0.2	0.1	
PWDS - D Run	22/11/2006	Burleigh Street, Burleigh Waters (D1 BU04)	< 0.2	0.1	
PWDS - D Run	19/12/2006	Burleigh Street, Burleigh Waters (D1 BU04)	0.2	0.2	
PWDS - D Run	24/01/2007	Burleigh Street, Burleigh Waters (D1 BU04)	< 0.2	0.1	0.14
PWDS - C Run	19/07/2006	Carapook Court, Tallebudgera (C1 T05)	< 0.2	0.1	0.10

PWDS - C Run	16/08/2006	Carapook Court, Tallebudgera (C1 T05)	< 0.2	0.1	
PWDS - C Run	20/09/2006	Carapook Court, Tallebudgera (C1 T05)	< 0.2	0.1	
PWDS - C Run	18/10/2006	Carapook Court, Tallebudgera (C1 T05)	< 0.2	0.1	
PWDS - C Run	15/11/2006	Carapook Court, Tallebudgera (C1 T05)	< 0.2	0.1	
PWDS - C Run	13/12/2006	Carapook Court, Tallebudgera (C1 T05)	< 0.2	0.1	
PWDS - C Run	17/01/2007	Carapook Court, Tallebudgera (C1 T05)	< 0.2	0.1	
PWDS - A Run	4/07/2006	Carrara Stadium, Carrara (A1 W02)	< 0.2	0.1	
PWDS - A Run	1/08/2006	Carrara Stadium, Carrara (A1 W02)	0.3	0.3	
PWDS - A Run	5/09/2006	Carrara Stadium, Carrara (A1 W02)	< 0.2	0.1	
PWDS - A Run	2/10/2006	Carrara Stadium, Carrara (A1 W02)	< 0.2	0.1	
PWDS - A Run	31/10/2006	Carrara Stadium, Carrara (A1 W02)	0.2	0.2	
PWDS - A Run	28/11/2006	Carrara Stadium, Carrara (A1 W02)	< 0.2	0.1	
PWDS - A Run	2/01/2007	Carrara Stadium, Carrara (A1 W02)	< 0.2	0.1	
PWDS - A Run	30/01/2007	Carrara Stadium, Carrara (A1 W02)	< 0.2	0.1	0.14
PWDS - D Run	3/07/2006	Cheltenham Drive, Robina (D1 SS03)			
PWDS - D Run	26/07/2006	Cheltenham Drive, Robina (D1 SS03)	0.2	0.2	
PWDS - D Run	23/08/2006	Cheltenham Drive, Robina (D1 SS03)	0.2	0.2	
PWDS - D Run	26/09/2006	Cheltenham Drive, Robina (D1 SS03)	0.2	0.2	
PWDS - D Run	25/10/2006	Cheltenham Drive, Robina (D1 SS03)	0.3	0.3	
PWDS - D Run	22/11/2006	Cheltenham Drive, Robina (D1 SS03)	< 0.2	0.1	
PWDS - D Run	19/12/2006	Cheltenham Drive, Robina (D1 SS03)	< 0.2	0.1	
PWDS - D Run	24/01/2007	Cheltenham Drive, Robina (D1 SS03)	< 0.2	0.1	0.17
PWDS - A Run	4/07/2006	Clover Hill Drive, Mudgeeraba (A3 SS5)			
PWDS - A Run	2/08/2006	Clover Hill Drive, Mudgeeraba (A3 SS5)	0.2	0.2	
PWDS - A Run	6/09/2006	Clover Hill Drive, Mudgeeraba (A3 SS5)	0.2	0.2	0.20
PWDS - D Run	3/07/2006	Connection Road, Tallebudgera (D1 C01)	0.4	0.4	
PWDS - D Run	26/07/2006	Connection Road, Tallebudgera (D1 C01)	0.3	0.3	
PWDS - D Run	23/08/2006	Connection Road, Tallebudgera (D1 C01)	0.5	0.5	
PWDS - D Run	26/09/2006	Connection Road, Tallebudgera (D1 C01)	0.3	0.3	
PWDS - D Run	25/10/2006	Connection Road, Tallebudgera (D1 C01)	0.2	0.2	
PWDS - D Run	22/11/2006	Connection Road, Tallebudgera (D1 C01)	0.4	0.4	
PWDS - D Run	19/12/2006	Connection Road, Tallebudgera (D1 C01)	0.3	0.3	
PWDS - D Run	24/01/2007	Connection Road, Tallebudgera (D1 C01)	0.4	0.4	0.35
PWDS - A Run	4/07/2006	Coral Coast Drive, Tallai (A1 W06)	< 0.2	0.1	
PWDS - A Run	2/08/2006	Coral Coast Drive, Tallai (A1 W06)	< 0.2	0.1	
PWDS - A Run	6/09/2006	Coral Coast Drive, Tallai (A1 W06)	< 0.2	0.1	
PWDS - A Run	4/10/2006	Coral Coast Drive, Tallai (A1 W06)	< 0.2	0.1	
PWDS - A Run	1/11/2006	Coral Coast Drive, Tallai (A1 W06)	< 0.2	0.1	
PWDS - A Run	29/11/2006	Coral Coast Drive, Tallai (A1 W06)	< 0.2	0.1	
PWDS - A Run	3/01/2007	Coral Coast Drive, Tallai (A1 W06)	< 0.2	0.1	
PWDS - A Run	30/01/2007	Coral Coast Drive, Tallai (A1 W06)			0.10
PWDS - D Run	24/07/2006	Daryl Radnell Drive, Worongary (D1 W05)	< 0.2	0.1	
PWDS - D Run	21/08/2006	Daryl Radnell Drive, Worongary (D1 W05)	0.3	0.3	
PWDS - D Run	26/09/2006	Daryl Radnell Drive, Worongary (D1 W05)	0.3	0.3	
PWDS - D Run	24/10/2006	Daryl Radnell Drive, Worongary (D1 W05)	< 0.2	0.1	
PWDS - D Run	21/11/2006	Daryl Radnell Drive, Worongary (D1 W05)	< 0.2	0.1	
PWDS - D Run	18/12/2006	Daryl Radnell Drive, Worongary (D1 W05)	< 0.2	0.1	
PWDS - D Run	23/01/2007	Daryl Radnell Drive, Worongary (D1 W05)	< 0.2	0.1	0.16
PWDS - C Run	18/07/2006	Duke Court, Tallai (C1 MU02)	< 0.2	0.1	0.11
PWDS - C Run	14/08/2006	Duke Court, Tallai (C1 MU02)	< 0.2	0.1	
PWDS - C Run	19/09/2006	Duke Court, Tallai (C1 MU02)	< 0.2	0.1	
PWDS - C Run	17/10/2006	Duke Court, Tallai (C1 MU02)	< 0.2	0.1	
PWDS - C Run	14/11/2006	Duke Court, Tallai (C1 MU02)	0.2	0.2	

PWDS - C Run	12/12/2006	Duke Court, Tallai (C1 MU02)	< 0.2	0.1	
PWDS - C Run	16/01/2007	Duke Court, Tallai (C1 MU02)	< 0.2	0.1	
PWDS - B Run	12/07/2006	Durobby Drive, Currumbin Valley (B1 T07)	< 0.2	0.1	
PWDS - B Run	9/08/2006	Durobby Drive, Currumbin Valley (B1 T07)	< 0.2 T	0.1	
PWDS - B Run	14/09/2006	Durobby Drive, Currumbin Valley (B1 T07)	< 0.2	0.1	
PWDS - B Run	10/10/2006	Durobby Drive, Currumbin Valley (B1 T07)	< 0.2	0.1	
PWDS - B Run	8/11/2006	Durobby Drive, Currumbin Valley (B1 T07)	< 0.2	0.1	
PWDS - B Run	6/12/2006	Durobby Drive, Currumbin Valley (B1 T07)	< 0.2	0.1	
PWDS - B Run	10/01/2007	Durobby Drive, Currumbin Valley (B1 T07)	< 0.2	0.1	0.10
PWDS - B Run	12/07/2006	Galleon Way, Currumbin Waters (B1 C01)	0.2	0.2	
PWDS - B Run	9/08/2006	Galleon Way, Currumbin Waters (B1 C01)	< 0.2	0.1	
PWDS - B Run	14/09/2006	Galleon Way, Currumbin Waters (B1 C01)	< 0.2	0.1	
PWDS - B Run	10/10/2006	Galleon Way, Currumbin Waters (B1 C01)	< 0.2	0.1	
PWDS - B Run	8/11/2006	Galleon Way, Currumbin Waters (B1 C01)	< 0.2	0.1	
PWDS - B Run	6/12/2006	Galleon Way, Currumbin Waters (B1 C01)	0.2	0.2	
PWDS - B Run	10/01/2007	Galleon Way, Currumbin Waters (B1 C01)	< 0.2	0.1	0.13
PWDS - D Run	24/07/2006	Gardenia Drive, Bonogin (D1 BM01)	< 0.2	0.1	
PWDS - D Run	21/08/2006	Gardenia Drive, Bonogin (D1 BM01)	< 0.2	0.1	
PWDS - D Run	26/09/2006	Gardenia Drive, Bonogin (D1 BM01)	< 0.2	0.1	
PWDS - D Run	24/10/2006	Gardenia Drive, Bonogin (D1 BM01)			
PWDS - D Run	21/11/2006	Gardenia Drive, Bonogin (D1 BM01)	< 0.2	0.1	
PWDS - D Run	18/12/2006	Gardenia Drive, Bonogin (D1 BM01)	< 0.2	0.1	
PWDS - D Run	23/01/2007	Gardenia Drive, Bonogin (D1 BM01)	< 0.2	0.1	0.10
PWDS - A Run	6/09/2006	Hardys Road, Mudgeeraba (A1 MU14)	0.7	0.7	
PWDS - A Run	4/10/2006	Hardys Road, Mudgeeraba (A1 MU14)	0.5	0.5	
PWDS - A Run	1/11/2006	Hardys Road, Mudgeeraba (A1 MU14)	0.4	0.4	
PWDS - A Run	29/11/2006	Hardys Road, Mudgeeraba (A1 MU14)	0.4	0.4	
PWDS - A Run	3/01/2007	Hardys Road, Mudgeeraba (A1 MU14)	0.5	0.5	
PWDS - A Run	30/01/2007	Hardys Road, Mudgeeraba (A1 MU14)	0.2	0.2	0.50
PWDS - C Run	18/07/2006	Harry Mills Drive, Worongary (C1 MU14)	0.3	0.3	
PWDS - C Run	14/08/2006	Harry Mills Drive, Worongary (C1 MU14)	< 0.2	0.1	
PWDS - C Run	19/09/2006	Harry Mills Drive, Worongary (C1 MU14)	0.3	0.3	
PWDS - C Run	17/10/2006	Harry Mills Drive, Worongary (C1 MU14)	< 0.2	0.1	
PWDS - C Run	14/11/2006	Harry Mills Drive, Worongary (C1 MU14)	0.2	0.2	
PWDS - C Run	12/12/2006	Harry Mills Drive, Worongary (C1 MU14)	< 0.2	0.1	
PWDS - C Run	16/01/2007	Harry Mills Drive, Worongary (C1 MU14)	< 0.2	0.1	0.17
PWDS - C Run	19/07/2006	Heather Street, Andrews (C1 C01)	0.5	0.5	
PWDS - C Run	16/08/2006	Heather Street, Andrews (C1 C01)	0.7	0.7	
PWDS - C Run	20/09/2006	Heather Street, Andrews (C1 C01)	0.5	0.5	
PWDS - C Run	18/10/2006	Heather Street, Andrews (C1 C01)	0.3	0.3	
PWDS - C Run	15/11/2006	Heather Street, Andrews (C1 C01)	< 0.2	0.1	
PWDS - C Run	13/12/2006	Heather Street, Andrews (C1 C01)	0.2	0.2	
PWDS - C Run	17/01/2007	Heather Street, Andrews (C1 C01)	0.3	0.3	0.37
PWDS - D Run	24/07/2006	John Rogers Road, Mudgeeraba (D1 MU14)		0	
PWDS - D Run	21/08/2006	John Rogers Road, Mudgeeraba (D1 MU14)	0.8	0.8	
PWDS - D Run	26/09/2006	John Rogers Road, Mudgeeraba (D1 MU14)	0.6	0.6	
PWDS - D Run	24/10/2006	John Rogers Road, Mudgeeraba (D1 MU14)	1	1	0.60
PWDS - A Run	4/07/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	< 0.2 T	0.1	0.17
PWDS - A Run	1/08/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	0.2	0.2	
PWDS - A Run	5/09/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	0.2	0.2	
PWDS - A Run	2/10/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	< 0.2	0.1	
PWDS - A Run	31/10/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	< 0.2	0.1	
PWDS - A Run	28/11/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	0.2	0.2	

PWDS - A Run	2/01/2007	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	< 0.2	0.1	
PWDS - A Run	30/01/2007	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	< 0.2	0.1	
PWDS - C Run	18/07/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	0.3	0.3	
PWDS - C Run	14/08/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	0.3	0.3	
PWDS - C Run	19/09/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	0.3	0.3	
PWDS - C Run	17/10/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	< 0.2	0.1	
PWDS - C Run	14/11/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	0.2	0.2	
PWDS - C Run	12/12/2006	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	< 0.2	0.1	
PWDS - C Run	16/01/2007	Kurrawa SLSC, Broadbeach (A2/C2 WG01)	< 0.2	0.1	
PWDS - C Run	19/07/2006	Laguna Avenue, Palm Beach (C1 EL01)	0.3	0.3	
PWDS - C Run	16/08/2006	Laguna Avenue, Palm Beach (C1 EL01)	< 0.2	0.1	
PWDS - C Run	20/09/2006	Laguna Avenue, Palm Beach (C1 EL01)	0.3	0.3	
PWDS - C Run	18/10/2006	Laguna Avenue, Palm Beach (C1 EL01)	< 0.2	0.1	
PWDS - C Run	15/11/2006	Laguna Avenue, Palm Beach (C1 EL01)	0.2	0.2	
PWDS - C Run	13/12/2006	Laguna Avenue, Palm Beach (C1 EL01)	0.2	0.2	
PWDS - C Run	17/01/2007	Laguna Avenue, Palm Beach (C1 EL01)	< 0.2	0.1	0.19
PWDS - A Run	4/10/2006	Lambor Drive, Mudgeeraba (A3 SS5)	0.3	0.3	
PWDS - A Run	1/11/2006	Lambor Drive, Mudgeeraba (A3 SS5)	< 0.2	0.1	
PWDS - A Run	29/11/2006	Lambor Drive, Mudgeeraba (A3 SS5)	0.3	0.3	
PWDS - A Run	3/01/2007	Lambor Drive, Mudgeeraba (A3 SS5)	< 0.2	0.05	0.19
PWDS - D Run	24/10/2006	Mackellar Drive, Nerang (D1 W02)	< 0.2	0.1	
PWDS - D Run	21/11/2006	Mackellar Drive, Nerang (D1 W02)	0.3	0.3	
PWDS - D Run	18/12/2006	Mackellar Drive, Nerang (D1 W02)	< 0.2	0.1	
PWDS - D Run	23/01/2007	Mackellar Drive, Nerang (D1 W02)	< 0.2	0.1	0.15
PWDS - A Run	4/07/2006	Marriot Way, Nerang (A1 W01)	< 0.2	0.1	
PWDS - A Run	1/08/2006	Marriot Way, Nerang (A1 W01)	< 0.2	0.1	
PWDS - A Run	5/09/2006	Marriot Way, Nerang (A1 W01)	< 0.2	0.1	0.10
PWDS - D Run	26/09/2006	Mireen Drive, Tugun (B1/D1 CU01)	< 0.2	0.1	
PWDS - D Run	25/10/2006	Mireen Drive, Tugun (B1/D1 CU01)	< 0.2	0.1	
PWDS - D Run	22/11/2006	Mireen Drive, Tugun (B1/D1 CU01)	< 0.2	0.1	
PWDS - D Run	19/12/2006	Mireen Drive, Tugun (B1/D1 CU01)	< 0.2	0.1	
PWDS - D Run	24/01/2007	Mireen Drive, Tugun (B1/D1 CU01)	< 0.2	0.1	0.10
PWDS - A Run	4/07/2006	Murlong Crescent, Palm Beach (A1 EL01)	< 0.2	0.1	
PWDS - A Run	2/08/2006	Murlong Crescent, Palm Beach (A1 EL01)	< 0.2	0.1	
PWDS - A Run	3/01/2007	Murlong Crescent, Palm Beach (A1 EL01)	< 0.2	0.1	
PWDS - A Run	30/01/2007	Murlong Crescent, Palm Beach (A1 EL01)	< 0.2	0.1	0.10
PWDS - C Run	17/10/2006	Nash Road A, Worongary (C2 WG01a)	0.4	0.4	
PWDS - C Run	14/11/2006	Nash Road A, Worongary (C2 WG01a)	0.3	0.3	
PWDS - C Run	12/12/2006	Nash Road A, Worongary (C2 WG01a)	0.2	0.2	
PWDS - C Run	16/01/2007	Nash Road A, Worongary (C2 WG01a)	0.3	0.3	0.30
PWDS - C Run	17/10/2006	Nash Road B, Worongary (C2 WG2)			
PWDS - C Run	14/11/2006	Nash Road B, Worongary (C2 WG2)			
PWDS - C Run	12/12/2006	Nash Road B, Worongary (C2 WG2)	< 0.2	0.1	
PWDS - C Run	16/01/2007	Nash Road B, Worongary (C2 WG2)			0.10
PWDS - C Run	18/07/2006	Nash Road, Worongary (C2 WG2)	0.5	0.5	
PWDS - C Run	14/08/2006	Nash Road, Worongary (C2 WG2)	0.3	0.3	
PWDS - C Run	19/09/2006	Nash Road, Worongary (C2 WG2)	0.5	0.5	0.43
PWDS - C Run	18/07/2006	Neilsens Road, Carrara (C1 W02)	0.2	0.2	0.23
PWDS - C Run	14/08/2006	Neilsens Road, Carrara (C1 W02)	0.7	0.7	
PWDS - C Run	19/09/2006	Neilsens Road, Carrara (C1 W02)	< 0.2	0.1	
PWDS - C Run	17/10/2006	Neilsens Road, Carrara (C1 W02)	0.3	0.3	
PWDS - C Run	14/11/2006	Neilsens Road, Carrara (C1 W02)	< 0.2	0.1	
PWDS - C Run	12/12/2006	Neilsens Road, Carrara (C1 W02)	< 0.2	0.1	

PWDS - C Run	16/01/2007	Neilsens Road, Carrara (C1 W02)	< 0.2	0.1	
PWDS - B Run	11/07/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	< 0.2	0.1	
PWDS - B Run	7/08/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	< 0.2	0.1	
PWDS - B Run	12/09/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	0.2	0.2	
PWDS - B Run	9/10/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	< 0.2	0.1	
PWDS - B Run	7/11/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	< 0.2	0.1	
PWDS - B Run	5/12/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	< 0.2	0.1	
PWDS - B Run	8/01/2007	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	0.2	0.2	
PWDS - D Run	3/07/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	< 0.2	0.1	
PWDS - D Run	26/07/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	< 0.2	0.1	
PWDS - D Run	23/08/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	0.2	0.2	
PWDS - D Run	26/09/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	0.2	0.2	
PWDS - D Run	25/10/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	< 0.2	0.1	
PWDS - D Run	22/11/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	< 0.2	0.1	
PWDS - D Run	19/12/2006	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	0.2	0.2	
PWDS - D Run	24/01/2007	Nobby Beach SLSC, Nobby Beach (B2/D1 WG01)	< 0.2	0.1	0.13
PWDS - A Run	4/07/2006	North Western Court, Robina (A1 SS03)	0.3	0.3	
PWDS - A Run	2/08/2006	North Western Court, Robina (A1 SS03)	0.2	0.2	
PWDS - A Run	6/09/2006	North Western Court, Robina (A1 SS03)	0.2	0.2	
PWDS - A Run	4/10/2006	North Western Court, Robina (A1 SS03)	0.3	0.3	
PWDS - A Run	1/11/2006	North Western Court, Robina (A1 SS03)	< 0.2	0.1	
PWDS - A Run	29/11/2006	North Western Court, Robina (A1 SS03)	0.2	0.2	
PWDS - A Run	3/01/2007	North Western Court, Robina (A1 SS03)	0.2	0.2	
PWDS - A Run	30/01/2007	North Western Court, Robina (A1 SS03)	< 0.2	0.1	0.20
PWDS - C Run	19/07/2006	Observatory Drive, Reedy Creek (C3 SS6)	0.2	0.2	
PWDS - C Run	16/08/2006	Observatory Drive, Reedy Creek (C3 SS6)	< 0.2	0.1	
PWDS - C Run	20/09/2006	Observatory Drive, Reedy Creek (C3 SS6)	0.2	0.2	0.17
PWDS - A Run	4/10/2006	Pacific Parade, Bilinga (A1/C1 AU01)	< 0.2	0.1	
PWDS - A Run	1/11/2006	Pacific Parade, Bilinga (A1/C1 AU01)	< 0.2	0.1	
PWDS - A Run	29/11/2006	Pacific Parade, Bilinga (A1/C1 AU01)	< 0.2	0.1	
PWDS - A Run	3/01/2007	Pacific Parade, Bilinga (A1/C1 AU01)	< 0.2	0.1	
PWDS - A Run	30/01/2007	Pacific Parade, Bilinga (A1/C1 AU01)	< 0.2	0.1	
PWDS - C Run	18/10/2006	Pacific Parade, Bilinga (A1/C1 AU01)	< 0.2	0.1	
PWDS - C Run	15/11/2006	Pacific Parade, Bilinga (A1/C1 AU01)	< 0.2	0.1	
PWDS - C Run	13/12/2006	Pacific Parade, Bilinga (A1/C1 AU01)	< 0.2	0.1	
PWDS - C Run	17/01/2007	Pacific Parade, Bilinga (A1/C1 AU01)	< 0.2	0.1	0.10
PWDS - B Run	11/07/2006	Recreation Drive, Nerang (B1 W02)	0.3	0.3	
PWDS - B Run	7/08/2006	Recreation Drive, Nerang (B1 W02)	0.2	0.2	
PWDS - B Run	12/09/2006	Recreation Drive, Nerang (B1 W02)	0.2	0.2	
PWDS - B Run	9/10/2006	Recreation Drive, Nerang (B1 W02)	0.2	0.2	
PWDS - B Run	7/11/2006	Recreation Drive, Nerang (B1 W02)	< 0.2	0.1	
PWDS - B Run	5/12/2006	Recreation Drive, Nerang (B1 W02)	< 0.2	0.1	
PWDS - B Run	8/01/2007	Recreation Drive, Nerang (B1 W02)	0.3	0.3	0.20
PWDS - D Run	3/07/2006	Rio Vista Boulevarde, Robina (D2 WG01)	< 0.2	0.1	
PWDS - D Run	26/07/2006	Rio Vista Boulevarde, Robina (D2 WG01)	< 0.2	0.1	
PWDS - D Run	23/08/2006	Rio Vista Boulevarde, Robina (D2 WG01)	< 0.2	0.1	
PWDS - D Run	26/09/2006	Rio Vista Boulevarde, Robina (D2 WG01)	0.2	0.2	
PWDS - D Run	25/10/2006	Rio Vista Boulevarde, Robina (D2 WG01)	0.2	0.2	
PWDS - D Run	22/11/2006	Rio Vista Boulevarde, Robina (D2 WG01)	< 0.2	0.1	
PWDS - D Run	19/12/2006	Rio Vista Boulevarde, Robina (D2 WG01)	< 0.2	0.1	
PWDS - D Run	24/01/2007	Rio Vista Boulevarde, Robina (D2 WG01)	< 0.2	0.1	0.13
PWDS - C Run	19/07/2006	Ron Penhaligan Way, Robina (C1 SS03)	0.3	0.3	0.26
PWDS - C Run	16/08/2006	Ron Penhaligan Way, Robina (C1 SS03)	0.3	0.3	
PWDS - C Run	20/09/2006	Ron Penhaligan Way, Robina (C1 SS03)	0.4	0.4	

PWDS - C Run	18/10/2006	Ron Penhaligan Way, Robina (C1 SS03)	0.3	0.3	
PWDS - C Run	15/11/2006	Ron Penhaligan Way, Robina (C1 SS03)	< 0.2	0.1	
PWDS - C Run	13/12/2006	Ron Penhaligan Way, Robina (C1 SS03)	< 0.2	0.1	
PWDS - C Run	17/01/2007	Ron Penhaligan Way, Robina (C1 SS03)	0.3	0.3	
PWDS - B Run	12/07/2006	Saunders Drive, Bonogin (B1 MU08)	< 0.2	0.1	
PWDS - B Run	9/08/2006	Saunders Drive, Bonogin (B1 MU08)	< 0.2	0.1	
PWDS - B Run	14/09/2006	Saunders Drive, Bonogin (B1 MU08)	0.2	0.2	
PWDS - B Run	10/10/2006	Saunders Drive, Bonogin (B1 MU08)	< 0.2	0.1	
PWDS - B Run	8/11/2006	Saunders Drive, Bonogin (B1 MU08)	< 0.2	0.1	
PWDS - B Run	6/12/2006	Saunders Drive, Bonogin (B1 MU08)	0.2	0.2	
PWDS - B Run	10/01/2007	Saunders Drive, Bonogin (B1 MU08)	< 0.2 T	0.1	0.13
PWDS - C Run	18/07/2006	Sehmish Court, Bonogin (C1 MU06)	0.4	0.4	
PWDS - C Run	14/08/2006	Sehmish Court, Bonogin (C1 MU06)	< 0.2	0.1	
PWDS - C Run	19/09/2006	Sehmish Court, Bonogin (C1 MU06)	0.3	0.3	
PWDS - C Run	17/10/2006	Sehmish Court, Bonogin (C1 MU06)	< 0.2	0.1	
PWDS - C Run	14/11/2006	Sehmish Court, Bonogin (C1 MU06)	0.3	0.3	
PWDS - C Run	12/12/2006	Sehmish Court, Bonogin (C1 MU06)	< 0.2	0.1	
PWDS - C Run	16/01/2007	Sehmish Court, Bonogin (C1 MU06)	< 0.2	0.1	0.16
PWDS - B Run	12/07/2006	Somerset Drive, Mudgeeraba (B1 MU14)	0.6	0.6	
PWDS - B Run	9/08/2006	Somerset Drive, Mudgeeraba (B1 MU14)	0.5	0.5	
PWDS - B Run	14/09/2006	Somerset Drive, Mudgeeraba (B1 MU14)	0.5	0.5	
PWDS - B Run	10/10/2006	Somerset Drive, Mudgeeraba (B1 MU14)	0.4	0.4	
PWDS - B Run	8/11/2006	Somerset Drive, Mudgeeraba (B1 MU14)	0.3	0.3	
PWDS - B Run	6/12/2006	Somerset Drive, Mudgeeraba (B1 MU14)	0.4	0.4	
PWDS - B Run	10/01/2007	Somerset Drive, Mudgeeraba (B1 MU14)	0.2	0.2	0.41
PWDS - C Run	19/07/2006	Sonia Street, Miami (C1 BU04)	< 0.2	0.1	
PWDS - C Run	16/08/2006	Sonia Street, Miami (C1 BU04)	0.2	0.2	
PWDS - C Run	20/09/2006	Sonia Street, Miami (C1 BU04)	0.2	0.2	
PWDS - C Run	18/10/2006	Sonia Street, Miami (C1 BU04)	0.2	0.2	
PWDS - C Run	15/11/2006	Sonia Street, Miami (C1 BU04)	0.2	0.2	
PWDS - C Run	13/12/2006	Sonia Street, Miami (C1 BU04)	< 0.2	0.1	
PWDS - C Run	17/01/2007	Sonia Street, Miami (C1 BU04)	0.2	0.2	0.17
PWDS - C Run	18/10/2006	Southern Skies Avenue, Mudgeeraba (C3 SS6)	< 0.2	0.1	
PWDS - C Run	15/11/2006	Southern Skies Avenue, Mudgeeraba (C3 SS6)	< 0.2	0.1	
PWDS - C Run	13/12/2006	Southern Skies Avenue, Mudgeeraba (C3 SS6)	< 0.2	0.1	
PWDS - C Run	17/01/2007	Southern Skies Avenue, Mudgeeraba (C3 SS6)	< 0.2	0.1	0.10
PWDS - B Run	12/07/2006	Tallai Road, Tallai (B1 MU10)	< 0.2	0.1	
PWDS - B Run	9/08/2006	Tallai Road, Tallai (B1 MU10)	0.3	0.3	
PWDS - B Run	14/09/2006	Tallai Road, Tallai (B1 MU10)	0.2	0.2	
PWDS - B Run	10/10/2006	Tallai Road, Tallai (B1 MU10)	< 0.2	0.1	
PWDS - B Run	8/11/2006	Tallai Road, Tallai (B1 MU10)	< 0.2	0.1	
PWDS - B Run	6/12/2006	Tallai Road, Tallai (B1 MU10)	< 0.2	0.1	
PWDS - B Run	10/01/2007	Tallai Road, Tallai (B1 MU10)	< 0.2	0.1	0.14
PWDS - B Run	12/07/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	0.2	0.2	0.12
PWDS - B Run	9/08/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2 T	0.1	
PWDS - B Run	14/09/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2	0.1	
PWDS - B Run	10/10/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	0.2	0.2	
PWDS - B Run	8/11/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2	0.1	
PWDS - B Run	6/12/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2	0.1	
PWDS - B Run	10/01/2007	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2	0.1	

PWDS - D Run	3/07/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2	0.1	
PWDS - D Run	26/07/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2	0.1	
PWDS - D Run	23/08/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2 T	0.1	
PWDS - D Run	26/09/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2	0.1	
PWDS - D Run	25/10/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2	0.1	
PWDS - D Run	22/11/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2	0.1	
PWDS - D Run	19/12/2006	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	0.2	0.2	
PWDS - D Run	24/01/2007	Tallebudgera Rec. Camp, Palm Beach (B1/D1 EL01)	< 0.2	0.1	
PWDS - A Run	4/07/2006	Tarbet Court, Carrara (A1 WG01)	0.2	0.2	
PWDS - A Run	1/08/2006	Tarbet Court, Carrara (A1 WG01)	0.3	0.3	
PWDS - A Run	5/09/2006	Tarbet Court, Carrara (A1 WG01)	0.3	0.3	
PWDS - A Run	2/10/2006	Tarbet Court, Carrara (A1 WG01)	0.2	0.2	
PWDS - A Run	31/10/2006	Tarbet Court, Carrara (A1 WG01)	< 0.2	0.1	
PWDS - A Run	28/11/2006	Tarbet Court, Carrara (A1 WG01)	< 0.2	0.1	
PWDS - A Run	2/01/2007	Tarbet Court, Carrara (A1 WG01)	0.2	0.2	
PWDS - A Run	30/01/2007	Tarbet Court, Carrara (A1 WG01)	< 0.2	0.1	0.19
PWDS - B Run	11/07/2006	The Esplanade, Miami (B1 BU04)	< 0.2	0.1	
PWDS - B Run	7/08/2006	The Esplanade, Miami (B1 BU04)	0.3	0.3	
PWDS - B Run	12/09/2006	The Esplanade, Miami (B1 BU04)	< 0.2	0.1	
PWDS - B Run	9/10/2006	The Esplanade, Miami (B1 BU04)	< 0.2	0.1	
PWDS - B Run	7/11/2006	The Esplanade, Miami (B1 BU04)	< 0.2	0.1	
PWDS - B Run	5/12/2006	The Esplanade, Miami (B1 BU04)	< 0.2	0.1	
PWDS - B Run	8/01/2007	The Esplanade, Miami (B1 BU04)	< 0.2 T	0.1	0.13
PWDS - B Run	12/07/2006	Thursday Drive, Tallebudgera Valley (B1 T04)	0.3	0.3	
PWDS - B Run	9/08/2006	Thursday Drive, Tallebudgera Valley (B1 T04)	< 0.2 T	0.1	
PWDS - B Run	14/09/2006	Thursday Drive, Tallebudgera Valley (B1 T04)	0.3	0.3	
PWDS - B Run	10/10/2006	Thursday Drive, Tallebudgera Valley (B1 T04)	< 0.2	0.1	
PWDS - B Run	8/11/2006	Thursday Drive, Tallebudgera Valley (B1 T04)	< 0.2	0.1	
PWDS - B Run	6/12/2006	Thursday Drive, Tallebudgera Valley (B1 T04)	< 0.2	0.1	
PWDS - B Run	10/01/2007	Thursday Drive, Tallebudgera Valley (B1 T04)	< 0.2	0.1	0.16
PWDS - D Run	3/07/2006	Trees Road, Tallebudgera (D1 T01)	0.4	0.4	
PWDS - D Run	26/07/2006	Trees Road, Tallebudgera (D1 T01)	0.3	0.3	
PWDS - D Run	23/08/2006	Trees Road, Tallebudgera (D1 T01)	0.5	0.5	
PWDS - D Run	26/09/2006	Trees Road, Tallebudgera (D1 T01)	0.3	0.3	
PWDS - D Run	25/10/2006	Trees Road, Tallebudgera (D1 T01)	0.2	0.2	
PWDS - D Run	22/11/2006	Trees Road, Tallebudgera (D1 T01)	0.4	0.4	
PWDS - D Run	19/12/2006	Trees Road, Tallebudgera (D1 T01)	0.3	0.3	
PWDS - D Run	24/01/2007	Trees Road, Tallebudgera (D1 T01)	0.4	0.4	0.35
PWDS - A Run	4/07/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	0.10
PWDS - A Run	2/08/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - A Run	6/09/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - A Run	4/10/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - A Run	1/11/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - A Run	29/11/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - A Run	3/01/2007	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - A Run	30/01/2007	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - C Run	19/07/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - C Run	16/08/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - C Run	20/09/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - C Run	18/10/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	

PWDS - C Run	15/11/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - C Run	13/12/2006	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - C Run	17/01/2007	Tweed Terrace, Coolangatta (A1/C1 CO01)	< 0.2	0.1	
PWDS - A Run	4/07/2006	Vantage Point Drive, Burleigh Heads (A1 BH01)	< 0.2 T	0.1	
PWDS - A Run	2/08/2006	Vantage Point Drive, Burleigh Heads (A1 BH01)	< 0.2	0.1	
PWDS - A Run	6/09/2006	Vantage Point Drive, Burleigh Heads (A1 BH01)	0.2	0.2	
PWDS - A Run	4/10/2006	Vantage Point Drive, Burleigh Heads (A1 BH01)	< 0.2	0.1	
PWDS - A Run	1/11/2006	Vantage Point Drive, Burleigh Heads (A1 BH01)	< 0.2 T	0.1	
PWDS - A Run	29/11/2006	Vantage Point Drive, Burleigh Heads (A1 BH01)	< 0.2	0.1	
PWDS - A Run	3/01/2007	Vantage Point Drive, Burleigh Heads (A1 BH01)	0.2	0.2	
PWDS - A Run	30/01/2007	Vantage Point Drive, Burleigh Heads (A1 BH01)	< 0.2 T	0.1	0.13
PWDS - A Run	4/07/2006	Wallaby Drive, Mudgeeraba (A1 MU01)	0.6	0.6	
PWDS - A Run	2/08/2006	Wallaby Drive, Mudgeeraba (A1 MU01)	0.5	0.5	
PWDS - A Run	6/09/2006	Wallaby Drive, Mudgeeraba (A1 MU01)	0.7	0.7	
PWDS - A Run	4/10/2006	Wallaby Drive, Mudgeeraba (A1 MU01)	0.4	0.4	
PWDS - A Run	1/11/2006	Wallaby Drive, Mudgeeraba (A1 MU01)	0.4	0.4	
PWDS - A Run	29/11/2006	Wallaby Drive, Mudgeeraba (A1 MU01)	0.2	0.2	
PWDS - A Run	3/01/2007	Wallaby Drive, Mudgeeraba (A1 MU01)	0.3	0.3	
PWDS - A Run	30/01/2007	Wallaby Drive, Mudgeeraba (A1 MU01)	0.4	0.4	0.44
PWDS - A Run	4/07/2006	Westminster Blvd, Elanora (A1 C02)	< 0.2 T	0.1	
PWDS - A Run	2/08/2006	Westminster Blvd, Elanora (A1 C02)	< 0.2	0.1	
PWDS - A Run	6/09/2006	Westminster Blvd, Elanora (A1 C02)	0.2	0.2	
PWDS - A Run	4/10/2006	Westminster Blvd, Elanora (A1 C02)	0.2	0.2	
PWDS - A Run	1/11/2006	Westminster Blvd, Elanora (A1 C02)	0.2	0.2	
PWDS - A Run	29/11/2006	Westminster Blvd, Elanora (A1 C02)	0.2	0.2	
PWDS - A Run	3/01/2007	Westminster Blvd, Elanora (A1 C02)	0.2	0.2	
PWDS - A Run	30/01/2007	Westminster Blvd, Elanora (A1 C02)	< 0.2 T	0.1	0.16
PWDS - B Run	12/07/2006	Yodelay Drive, Stephens (B1 SS03)	0.2	0.2	
PWDS - B Run	9/08/2006	Yodelay Drive, Stephens (B1 SS03)	0.2	0.2	
PWDS - B Run	14/09/2006	Yodelay Drive, Stephens (B1 SS03)	0.4	0.4	
PWDS - B Run	10/10/2006	Yodelay Drive, Stephens (B1 SS03)	< 0.2	0.1	
PWDS - B Run	8/11/2006	Yodelay Drive, Stephens (B1 SS03)	< 0.2	0.1	
PWDS - B Run	6/12/2006	Yodelay Drive, Stephens (B1 SS03)	< 0.2	0.1	
PWDS - B Run	10/01/2007	Yodelay Drive, Stephens (B1 SS03)	< 0.2	0.1	0.17

District	# of Sample Sites
Nerang south	5
worongary	5
Gilston	6
Mudgeeraba	9
Robina	5
Burleigh heads	5
Elanora	3
Reedy Creek	7
Currumbin Waters	10
Currumbin	1
Coolangatta	3
Total	59

D-2 FIELD CHLORINE SAMPLES (07/06/07 – 08/06/09)

Table D-2: Field chlorine concentrations for 59 test sites in the southern region (07/06/07 – 08/06/07)

Site #	Sample_Site	Reservoir System	Location	Sample Taken 7/06/07		Sample Taken 8/06/08		Comment
				Conc. (mg/L)	Time	Conc. (mg/L)	Time	
..1	BARDEN RIDGE RD	REEDY CREEK						Couldn't Find Tap
..2	BIENVENUE DR	SIMPSONS ROAD	CLUBHOUSE INFORNT OF DISABLED PARK	0.00	03:15:00			
..3	BINYA AV	COOLOANAGATTA	WESTERN WALL CLUB HOUSE NEAR ROLLER DOOR	0.00	14:35:00			
..4	BRIDGMAN DR	REEDY CREEK	AT BASE OF OLD RESERVOIR			0.00	15:00:00	
..5	BURLEIGH HEADS SLSC	BURLEIGH	ON BBQ	0.15	13:10:00			
..6	BURLIEGH ST	BURLEIGH	IN PARK, NEXT TO TREE & PICNIC TABLE OPS. NO 79	0.00	13:00:00			
..7	CARAPOOK CT	SIMPSONS ROAD	NEXT TO LIGHT POLE AT END OF STREET			0.00	14:00:00	
..8	CARRARA STADIUM	NERANG SOUTH	RHS WHITE/GREEN OPERATIONS OFFICE	0.00	09:45:00			
..9	CHELTENHAM DR	REEDY CREEK	NORTHERN CORNER CRICKET CLUBHOUSE	0.00	11:35:00			
..10	COOLANGATTA RD							No Tap in Park - new development
..11	CORALCOAST DR	MOLENDINAR 3	INSIDE PRV HALFWAY UP STREET RHS					Couldn't Find tap
..12	DARYL RADNELL DR	MOLENDINAR 3	INSIDE RESERVOIR COMPOUND			0.00	11:50:00	
..13	DUKE CT	MUDGEERABA	ON RESERVOIR INSIDE COMPOUND			0.00	11:00:00	
..14	DUROBBY DR	SIMPSONS ROAD	LHS OF DRIVEWAY 41 DUROBBY DR, NEXT TO LIGHT POLE			0.00	14:15:00	
..15	ESPLANADE	BURLEIGH	BBQ /SHELTER NEAR NORTH SLSC	0.00	12:30:00			
..16	GALLEON WY	SIMPSONS ROAD	BEHIND CLUBHOUSE ON WALL NEXT TO BUBBLER NEAR LADIES TOILET	0.00	15:30:00			
..17	GARDENIA DR	MUDGEERABA	IN PRV PIT INFORNT OF 110 GARDENIA DR			0.00	16:25:00	
..18	GOLD COAST HWY	WORONGARY	IN PARK BETWEEN GARDENS AND GC HWY	0.20	10:25:00			
..19	GOLINE CRT	SIMPSONS ROAD	BASE OF RESERVOIR NEAR VALVE PIT			0.00	13:55:00	
..20	GRANDVIEW TCE	MUDGEERABA	NEAR RESERVOIR			0.00	22:45:00	
..21	HARDYS RD	MUDGEERABA	WATER CARRIER FILLING STATION			0.25	16:10:00	
..22	HARRY MILLS DR	MOLENDINAR 3	OUTSIDE 13 HARRYMILLS DR			0.00	11:30:00	
..23	HEATHER STREET	SIMPSONS ROAD	NE SIDE OF CORNER, NEAR LIGHT POLE AND VALVE	0.45	16:35:00			

			POST					
..24	JOHN ROGERS RD	MUDGEERABA	RHS OF WPP ENTRANCE. MARKED 'POTABLE WATER'	0.85	Ave			
..25	KURRAWA SLSC	WORONGARY	IN PARK SOUTHERN END SLSC AT FREE BOILING WATER SINK/SHELTER	0.25	10:35:00			
..26	LAGUNA AVE	ELANORA	N SIDE PLAY EQUIPMENT & BBQ	0.00	13:45:00			
..27	LAMBOR DR	MUDGEERABA	IN PARK AT END OF CHUTE ST	0.25	17:15:00			
..28	MACKELLAR DR	NERANG SOUTH	NEXT TO 7 MACKELLAR DR AT END OF EASEMENT					Couldn't Find Tap
..29	MARRIOT WY	NERANG SOUTH	INSIDE VALVE PIT AT RESERVOIR. ACCESS UP STEEP GRAVEL DRIVE			0.00	08:55:00	
..30	MERRIMAC WWTP	WORONGARY	RHS DRIVEWAY, IN MIDDLE OF FIRST BUILDING			0.00	09:35:00	
..31	MIRREEN DR	CURRUMBIN	LHS PARK NEAR FENCE	0.20	15:00:00			
..32	MURLONG CRES	ELANORA	BBQ FACILITY OPPOSITE #38	0.00	13:30:00			
..33	NASH RD (A)	WORONGARY	NORTH SIDE OF RESERVOIR. ON OUTLET			0.40	10:00:00	
..34	NASH RD (B)	WORONGARY	INSIDE BOMB SHELTER RESERVOIR WG2			0.25	10:10:00	
..35	NASH RD (C)	WORONGARY	DOWNHILL FROM THE RESERVOIRS			0.00	09:50:00	
..36	NIELSENS RD	NERANG SOUTH	UNDER MAILBOXES ON WEST SIDE OF PIZZA HUT	0.15	08:55:00			
..37	NOBBY BEACH SLSC	WORONGARY	ON BEACH SHOWER OR TAP ON BBQ NEAR HOT WATER TAP	0.00	10:45:00			
..38	NORTHWESTERN CT	REEDY CREEK	NEAR BBQ	0.00	11:55:00			
..39	PACIFIC PDE	CURRUMBIN	TAP ON BBQ SOUTH OF SLSC	0.00	14:00:00			
..40	RECREATION DR	NERANG SOUTH	EASTERN END ON CANTEEN - OPPOSITE END TO PLAY EQUIPMENT	0.40	09:10:00			
..41	RIO VISTA BLVD	WORONGARY	IN PARK LHS OF WALKWAY	0.00	11:10:00			
..42	ROBINA RESERVOIR	ROBINA				0.15	16:00:00	
..43	RON PENHALIGAN WY	REEDY CREEK	TAP ON NW SIDE OF COMMUNITY CENTRE					No Handle
..44	SAUNDERS DR	MUDGEERABA	IN PRV PIT OPPOSITE 28 SAUNDERS DR			0.00	16:45:00	
..45	SEMISH CRT	MUDGEERABA				0.00	04:35:00	
..46	SOMERSET DR	MUDGEERABA	IN MEN'S TOILET - JUNIOR RUGBY LEAGUE CLUBHOUSE NORTHERN END			0.40	11:55:00	
..47	SONIA ST	BURLEIGH	N END BBQ FACITLITY IN CATHERINE CRAWFORD PICNIC AREA	0.00	11:00:00			
..48	SOUTHERN SKIES AVE	REEDY CREEK	NORTHERN END OF PARK NEAR BBQ			0.00	15:30:00	
..49	TALLAI RD	MUDGEERABA	NEAR LAMP POST OUTSIDE 190 TALLAI RD			0.00	10:50:00	
..50	TALLEBUDGERA CONNECTION RD	SIMPSONS ROAD	UNDER POWER METER BOX ON PUMPSTATION BUILDING	0.00	16:00:00			
..51	TALLEBUDGERA RECREATIOn CAMP	ELANORA	BBQ NEAR PUBLIC TOILETS	0.00	13:20:00			

..52	TARBERT CL	WORONGARY	AT WASTEWATER PUMPSTATION W32			0.20	09:10:00	
..53	THURSDAY DR	REEDY CREEK	NEAR PUMP STATION LIDS			0.00	13:40:00	
..54	TREES RD	SIMPSONS ROAD	TAP INSIDE WATER PUMPSTATION ON OUTLET OF PUMP	0.00	16:00:00			
..55	TWEED TCE	COOLANGATTA	RHS BBQ NEAR CAPTAIN COOK MEMORIAL	0.00	02:20:00			
..56	VANTAGE POINT DR	BURLEIGH	BASE OF RESERVOIR. EASTERN SIDE.					No Flow out of Tap
..57	WALLABY DR	MUDGEERABA	LHS DRIVEWAY BASE OF RESERVOIR			0.50	16:50:00	
..58	WESTMINSTER BLVD	SIMPSONS ROAD	TAP INSIDE BUILDING UNDER STAIRS ON HIGH LEVEL PRESSURE SYS					Couldn't Find Tap
..59	YODELAY DR	REEDY CREEK	ON BBQ	0.00	16:50:00			

APPENDIX E

E-1 KOLMOGOROV-SMIRNOV TEST

Data Set 1: Model

Items in Data Set 1:

84.0 84.0 84.0 84.0 84.0 84.1 84.1 84.1 84.1 84.2 84.2 84.2 84.3 84.4 84.4 84.5 84.5 84.5 84.5 84.5 84.5 84.6 84.6 84.6 84.6 84.6 84.6 84.7 84.7 84.8 84.8 84.8 84.9 84.9 84.9 84.9 85.0 85.0 85.0 85.1 85.2 85.3 85.4 85.5 85.6 85.8 85.9 86.0 86.0

- Mean = 84.71
- 95% confidence interval for actual Mean: 84.56 thru 84.87
- Standard Deviation = 0.550
- Median = 84.61
- Average Absolute Deviation from Median = 0.420
- KS says it's unlikely this data is normally distributed: *P*= 0.08 where the normal distribution has mean= 84.82 and sdev= 0.5495
- KS says it's unlikely this data is log normally distributed: *P*= 0.08 where the log normal distribution has geometric mean= 84.82 and multiplicative sdev= 1.006

Data Set 2: Field

Items in Data Set 2:

81.5 81.6 81.8 82.0 82.4 82.6 82.8 82.9 83.1 83.1 83.1 83.1 83.3 83.3 83.4 83.4 83.4 83.4 83.5 83.5 83.6 83.6 83.6 83.6 83.6 83.6 83.7 83.7 83.7 83.7 83.7 83.8 83.9 83.9 83.9 83.9 84.0 84.0 84.0 84.0 84.0 84.1 84.2 84.2 84.3 84.4 84.5 84.6 84.7

- Mean = 83.50
- 95% confidence interval for actual Mean: 83.30 thru 83.71
- Standard Deviation = 0.723
- Median = 83.60
- Average Absolute Deviation from Median = 0.508
- KS says it's unlikely this data is normally distributed: *P*= 0.02 where the normal distribution has mean= 83.40 and sdev= 0.8789

KS says it's unlikely this data is log normally distributed: *P*= 0.02 where the log normal distribution has geometric mean= 83.40 and multiplicative sdev= 1.011