University of Southern Queensland

Faculty of Engineering and Surveying

Analysis of Catchment G in Oakey's Sewer Network

A dissertation submitted by

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Courses ENG4111 and 4112 Research Project

Towards the degree of

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Abstract

Oakey is a small town located 25km west of Toowoomba and has a population of approximately 4000 people. Current and future development of urban areas in Oakey is leading to a greater strain on the current sewer network. A large amount of the network was installed in the mid sixties. Council is looking at ways of managing the current network to satisfy future demand requirements.

This text aims to model a catchment in the town's sewer network and provide recommendations on current and future work requirements.

Modelling and analysing the chosen catchment has shown that even though on paper sections of the network are not ideal, the current flows in the area analysed, are well below the design capacity, hence the network functions with minimal problems. The main area of interest in this catchment was the limit storage capacity of the pump station.

Future upgrades to the chosen catchment will include the possible construction of a new pump station. This document gives recommendations on this.

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ENG4111 Research Project Part 1 &
ENG4112 Research Project Part 2

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Signature

Date

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Nomenclature

ADWF	Average Dry Weather Flow
PWWF	Peak Wet Weather Flow
I/I	Infiltration and Inflow

CHAPTER 1 - INTRODUCTION

1.1. Introduction

Sewer networks are an integral part of the Councils services. In many cases people are unaware of there existence and like water grids just expect water to flow down the drain as easily as it comes out of the tap.

1.2. Project Aim

The aim of this project is to develop a model of a catchment in Oakey's sewer network. Once the model has been developed an analysis will be undertaken under current and future demand situations. From this analysis recommendations will be made on any upgrades that maybe required.

1.3. Project Outcomes

Once this study has been undertaken the collected catchment information will provide much needed data for recommendations on new infrastructure requirements as well as assisting Council's management of the area in respect to current and future development.

This analysis is divided up into two parts. The first part is the capacity under current loading of the network. The second part is an analysis of a possible future situation of the catchment. Much of vacant land that surrounds the catchment has great development potential and is slowly being developed with much emphasis being placed on access to Council services, i.e. sewer and water.

The analysis once complete will provide recommendations for the design of a new pump station. The new pump station will be required to pump directly to the Council's treatment plant located only a short distance from the current pumping facilities. The current pump station feeds into a much larger catchment in the town sewer network.

1.4. Background

The town of Oakey is located approximately 30 minutes west of Toowoomba and has a permanent population of approximately 4000 people. The town contains one of the two sewer areas in the Shire of Jondaryan, with the other area being Westbrook.

The sewer network in Oakey begun construction in the mid sixties, with the majority of the town's network constructed after the houses and much of the infrastructure was in place. This poses many problems with the network, and any alterations or repairs that are required, have great effect on the surrounding residents.

One of the first tasks that needed to be completed before the commencement of this study was the gathering of data about the network in general. Over semester three with the assistance of a Council surveyor much of the network was surveyed. Our main interests were in the exact locations of manholes and pump stations, surface and invert levels of the sewer manholes and mains, and to confirm asset data on pipe diameters and materials. Other data that had to be gathered was exact pump models, specifications and operating limits.

In the 2004/2005 financial year Council began upgrading the existing pump stations. Until this the pump stations were self contained units using floats to control the levels in the pump wells. Thus the only method of obtaining flows was from pump hour meters read on a daily basis. This gave Council limited feedback on the pump stations with no other information such as well levels, the number of pump starts, or even faults and overflow events being available. In the event that one of the pump stations reached its alarm level a red flashing light would be triggered. Thus there was a requirement for this to be reported to avoid an overflow event occurring.

This method of management was deemed not acceptable and the upgrading and installation of real time monitoring systems was undertaken. The current system in place is Citect SCADA-C. Currently all of the water reservoirs and sewerage pump station apart from pump station G are connected to this system. The most recent pump station to be connected to the telemetry was pump station J, this pump station serves a small industrial area with very small flows.

The pump station and catchment area of this study is the final pump station requiring upgrading to the new system. The upgrade of this pump station has been delayed til a study could be under taken into the requirements of any new infrastructure.

1.5. Dissertation Overview

Chapter 2 covers background information about the study area.

Chapter 3 contain the methodology for the project as well as calculations for ADWF and PWWF.

Chapter 4 covers information regarding the software and how the model was setup and verified.

Chapter 5 contains the results obtained from the analysis of the catchment.

Chapter 6 covers recommendations that have been made for additional infrastructure that is required.

Chapter 7 contains conclusion on what was undertaken during the project. Also covered is any future work that may result from the undertaking of this project.

CHAPTER 2 – DESCRIPTION OF STUDY AREA

2.1. Introduction

The majority of the sewer network in Oakey was constructed in the mid nineteen sixties. Over time many modification have been made to network to deal with increased population growth. Many of these modifications were only undertaken when they were deemed to be necessary and not as suggested in previous studies of the network. This was done in an attempt to reduce costs but to still maintain a high level of service.

Since a study was undertaken in 1994 the majority of the Shire of Jondaryan has been experiencing drought conditions. As the current dam levels decrease, there is a need to conserve water, currently Oakey is connected to Toowoomba’s water supply, and therefore are required to adhere to the same level of water restriction. It can be seen that as the level of water restrictions rise the amount of water consumed also decreases, with the decrease in water consumption and increase in grey water reuse this leads to a reduction in sewage flows. This reduction in sewage flows aloud Council to put back any non-essential modifications to infrastructure.

2.2. Focus Area

Oakey is located 25km by road east of Toowoomba in Jondaryan Shire. Oakey is one of many major community centres located with in the Shire and is also the location of the current Council office's and depot. As at 30 June 2005 the estimated population of the Shire was 14,329 persons. This estimate was obtained from the Office of Economic and Statistical Research and is based on current growth trend and Australian Bureau of Statistics Data. A plan of the Shire can be seen in Figure 2-1.

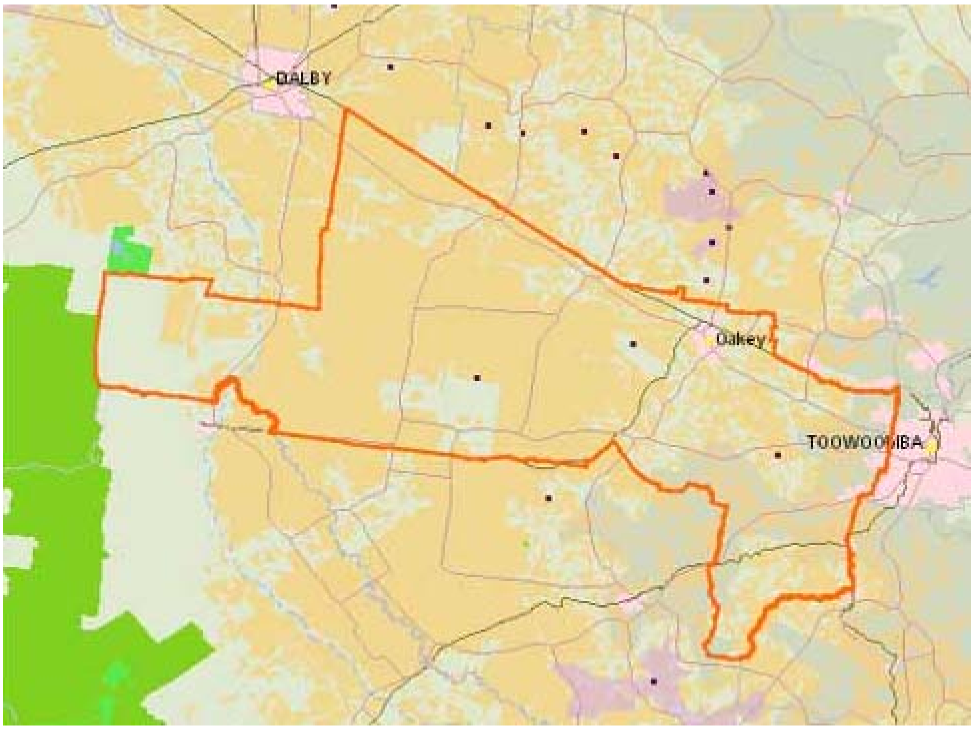

Figure 2-1 Map of Jondaryan Shire, Obtained from Council's Website.

The town of Oakey has an estimated population of 4000 persons. Oakey's sewer network consists of seven pump stations, four of these, pump into other sections of the network, with the three larger of the pump stations pumping directly to the treatment plant. An overall plan of Oakey's Sewer Network can be seen in Figure 2-2.

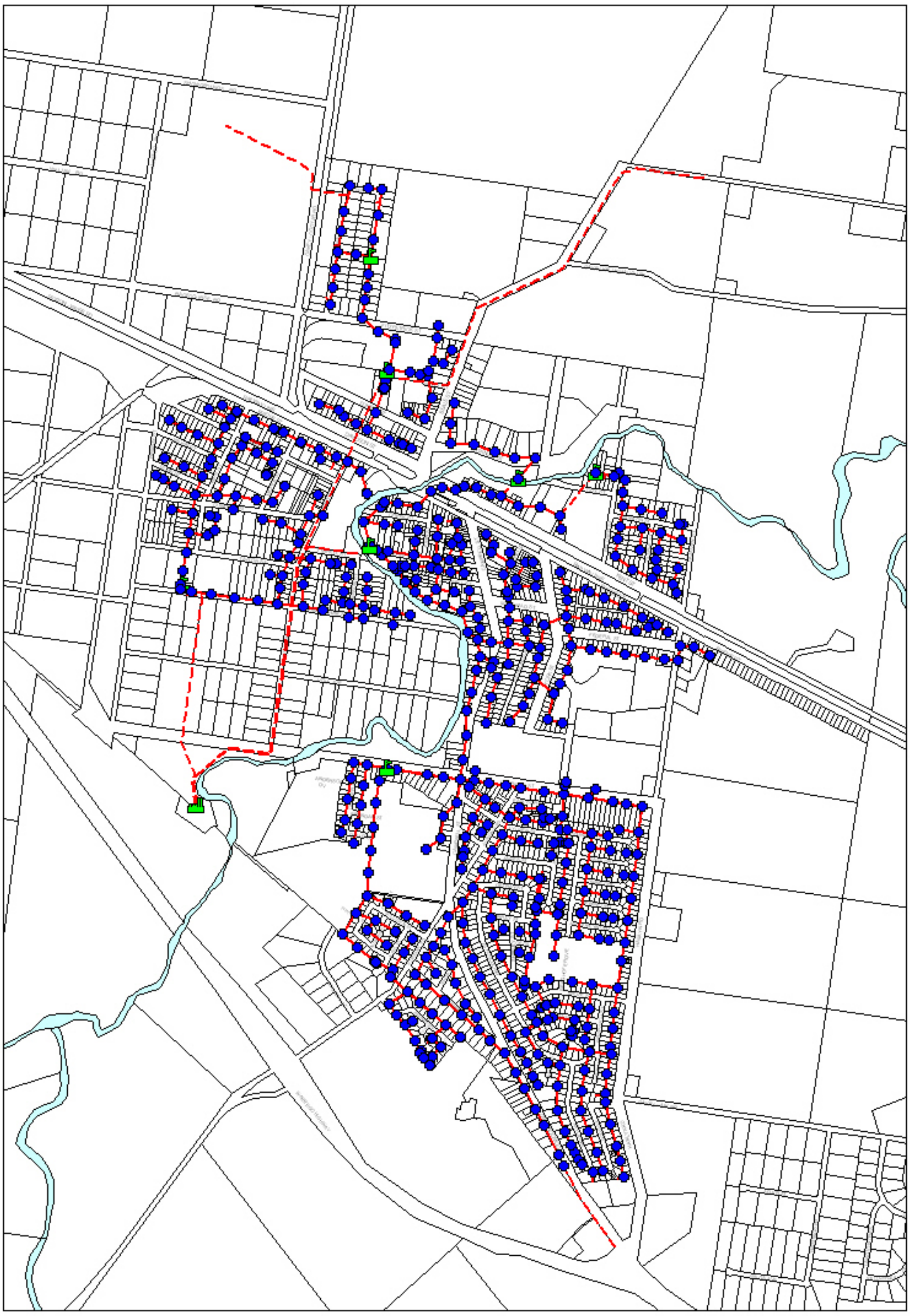

Figure 2-2 Oakey's current sewer network

2.3. Focus Area – Sewer Pump Station G

The focus area for this study is the catchment draining to sewerage pump station G. The original pump station was constructed to serve only the area surrounding Cooper Avenue; this is shown in Figure 2-3. From records in the mid seventies this pump station was filled in and the manhole located directly west was converted to the current pump station, this aloud the area of King and Milligan Streets and then the Aruma Drive area to be connected to sewer. This can be seen in Figure 2-4. In the 06/07 and 07/08 Council is looking to upgrade the current pump station.

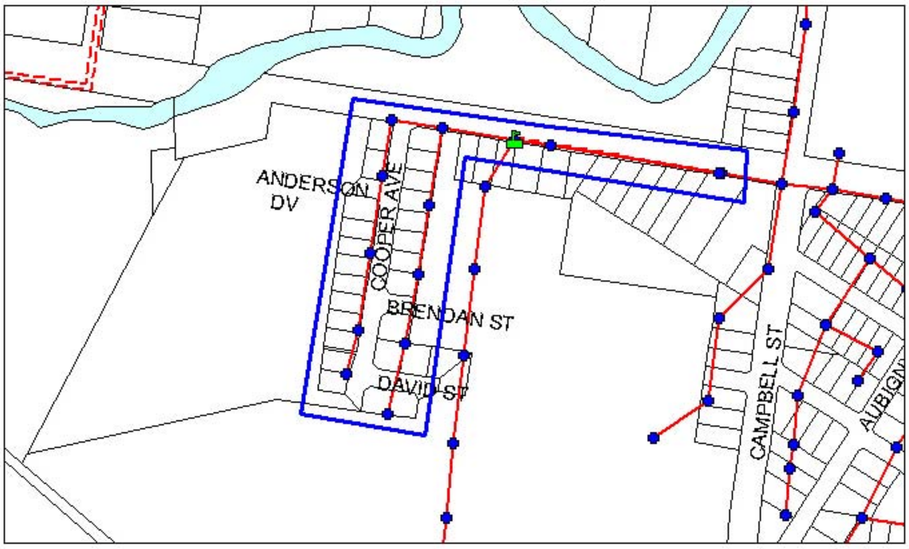

Figure 2-3 Initial sewer area

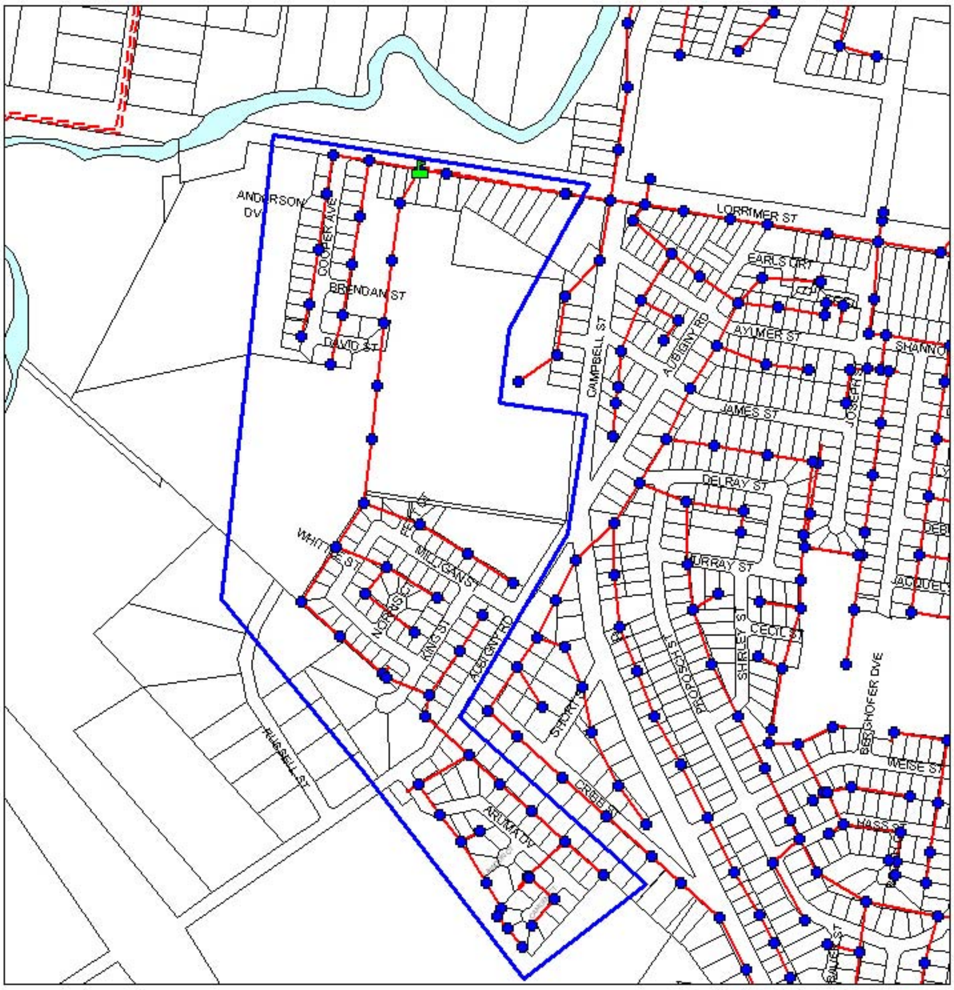

Figure 2-4 Overview of the study area

The current pump station as indicated in Figure 2-4, is only of a temporary nature. Currently no records are held of the exact details of the pump station but from field survey the following details were obtained. The pump station has a depth of 5.0m, a diameter of 1.2m and is fitted with a single Flygt CP3102.180 submersible pump, the performance curve for this pump can be found in Appendix G. The pump is controlled by a simple system of floats. Once the pump well contains a set amount of liquid the pump starts and then stops once another float is triggered. Currently this is the last pump station in Oakey not connected to the telemetry system. Because of this the flows obtained for modelling purposes were estimated from pumping rates and records of total hours pumped as read each day by the treatment plant operator.

When the upgrade of the pump station is undertaken it will enable real time monitoring of pump well levels, daily runtime, daily number of starts as well as faults and warnings. Under the current system there are no records of the number of starts, or warnings/ faults unless a visual failure has occurred and raw sewage in seen flowing from the overflow system into the local creek.

A brief analysis was undertaken of the current data on the network, primarily manhole locations, pipe material, pipe diameter, surface and invert levels. It was found that over time with the updating of the Council's GIS that many of the manhole locations were not completely correct and the only levels that were available were from very old as-constructed plans that had been converted from State Datum to AHD. The data regarding the diameter and pipe materials could be utilised thus reducing the required amount of survey.

The main survey was undertaken using a dumpy level and staff. This was under taken by Leonard Knight, Council Technical Officer and the author. The survey process was hindered mainly by the fact that many of the manholes were difficult to locate. Some of the problem encountered related to limit access, but many of the manholes needed to be bought to the surface, as over time they have become buried. A labelled map of the study area can be found in Appendix B.

2.4. Condition Assessment

An assessment of the condition of the sewer mains in the catchment was undertaken. From this it was found that the majority of the gravity mains present in the catchment were in serviceable condition. One area of special interest is the road crossing from the Aruma Drive section to the Milligan, King Streets area. This can be seen in Figure 2-5.

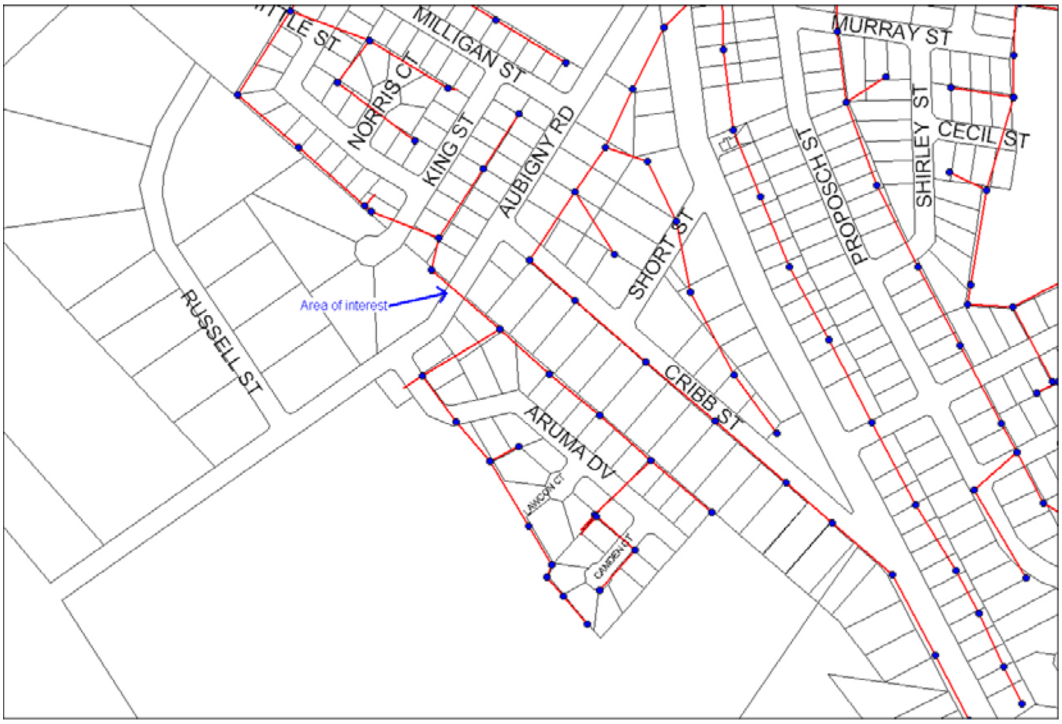

Figure 2-5 GIS map showing area of interest

At this location it was found that a large section of main had been broken out. Records at Council show current stormwater infrastructure crossing the sewer main at this point. An image of this is shown in figure 2-6. This was one of the few major problems found from CCTV survey of the lines. Other problems encountered were in regards to root masses with the lines, these roots were removed during the cleaning required to complete the CCTV inspection.

Figure 2-6 Image from camera work of the broken section of main

The most major problem encountered in the inspection of the catchment was silt and debris build ups within the lines. Over time there have been very few breaks and blockages of this section of network. Thus it is assumed that the silt and debris had only a small effect on the serviceability of the network and only was found to be a problem when an attempt to conduct CCTV inspection was undertaken. It should be noted that on the second attempt to inspect the network via CCTV inspection in mid October 2006 the remaining silt and debris were removed.

2.5. Summary

The proposal to upgrade the current infrastructure has lead to the need for this section of the network to be analysed. Because of the need for accurate locations and levels, this lead to a good opportunity for updating of the details of the entire Oakey sewer network.

From the condition assessment it can be concluded that there are only a few problems with the sewer mains in this section of the network.

CHAPTER 3 –METHODOLOGY

3.1. Introduction

The process of analysing sewer networks is set out in publications such as Department of Natural Resources & Mines, Planning Guidelines for Water Supply and Sewerage 2005. This document gave the direction for how the network was to be analysed.

The first sections of this chapter cover a brief background of what was used with remainder covering the methodology of the work undertaken.

3.2. Review of Previous Study

The previous study was undertaken in 1994 by Sinclair Knight Merz. This study mainly concentrated on the overall network and not individual catchments. Because of this there is very little information given on the section of network being analysed. The results given in this study gave an ADWF at the pump station and with the use of a peaking factor of 5 they also gave an approximate PWWF at this point in the system. From this study similar results were also found for the volume of sewage that the pump station could store before an overflow event occurred.

The main difference between this study and the previous study is in the analysis of each main in the catchment to determine the capacities at current and future flows.

3.3. Current Guidelines

Current Department of Natural Resources & Mines Planning Guidelines for Water Supply and Sewerage set out requirements for network modelling. This formed the basis for the method of modelling that was adopted. The only difference was due to the lack of diurnal patterns and accurate I/I information the model was run under a constant peaking factor for each of the PWWF cases

3.4. Details of Software Used

The software that was used for the modelling of this section of the network was Haestad Methods SewerCAD. The program can be run as an add-on to AutoCAD or as a stand alone sewer modelling program as was used in this case. Apart from just analysing network the package has a built in design tool, with this you can enter the design parameters, surface levels and outlet level and use a built-in design function to obtain pipe grades and levels. This system uses a large database of pipes and materials to select from, with it being left to the operator to set a limited selection to be used in design.

The program has a built in numerical model, which users both direct and standard step gradually varied flow models (SewerCAD v5 Users Guide) to analyse the gravity network. Flows calculated by the model can be closely approximated by use of Manning's Equation.

$$Q = \frac{1}{n} \cdot A \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$$ Manning's Equation

Where: Q=Discharge (m^3/s)

n=Manning's roughness

A=Flow area (m^2)

R=Hydraulic Radius (m)

S=Friction slope (m/m)

Because flow in a sewer main is part full flow, proportional discharge, velocity and depth ratios need to be applied to obtain actual discharge, velocity and depth values.

For the purpose of this model the design capacity was at 75% full depth of flow.

3.5. Physical Data Collection

The collection of all the physical data for the network was undertaken with the assistance of one of Council's Technical Officer's who assisted in the levelling, locating and gathering of data about the catchment.

The Shire's detail plans and GIS system where analysed to give approximate positions of all sewer mains that were required for analysis, the next step was to locate and level manholes and pipe inverts.

The levelling was undertaken with a dumpy level and permanent survey marks located around the town to gain the surface levels of the manholes in the network. We then opened each manhole inspected the pipe configuration and measured the depths to the inverts.

3.6. Flow Data Collection

An approximation of the flows through the pump station was obtained from runtimes of the pump in the well. With a known capacity of 10.2L/s obtained from draw down tests undertaken an approximate value can be calculated for daily flows. This was then averaged out to give the ADWF. The ADWF was then divided up amongst the 132 residential connections in the catchment. The flows calculated are well below those recommended for sewer design. The low flows discovered can be attributed the current water usage which is found to be much lower then in previous years due to a higher level water restrictions and increased grey water reuse. The EP for each connection was given by Council as 2.8 persons per house; this figure was taken from adjusted Census data. The ADWF used in the current situation model are displayed in Table 3.1

EP per house	ADWF (L/c.d)	Total per Connection (L/d)
2.8	146	408

Table 3.1 Current Situation Sewer Flows

No significant rainfall event has occurred in the given time span, hence the PWWF will need to be approximated using methods obtained from section 5.2.2 of the Department of Natural Resources and Mines publication, “Planning Guidelines for Water Supply and Sewerage”. For this the historical Queensland approach was used.

Thus:

PWWF = 5 x ADWF or C_1 x ADWF, which ever is larger.

$C_1 = 15 \times (EP)^{-0.1587}$ with C_1 being greater then 3.5

Note: EP in the equation being the total equivalent population in the catchment gravitating to a pump station.

In this case all properties draining to the pump station are residential. Thus an equivalent population can be calculated from the number of sewer connections (132) and the average 2.8 persons per house given by Council. Shown in the Table 3.2 is the C_1 peaking factor and the calculated PWWF for the current situation.

C_1 Peaking Factor	Calculated PWWF (L/c.d)
5.9	861.4

Table 3.2 Peaking factor and PWWF

I/I, Infiltration and inflow have been assessed in the past with it being found that the amount of I/I is minimal for the smaller catchments. A part of past assessment was smoke testing of the southern section of the town's sewer network in 2004. From this testing it was found that only one property had the stormwater from the roof connected to the sewer lines via a manhole located on the site. This has since been rectified. Figure 3-1 below shows the runtimes of the pump station located in Lorrimer Street Oakey, approximately 30mm of rainfall was received in early December this corresponds with the peak runtime.

The average shown on the chart excludes runtime from periods of rain hence allowing the ADWF as previously estimated.

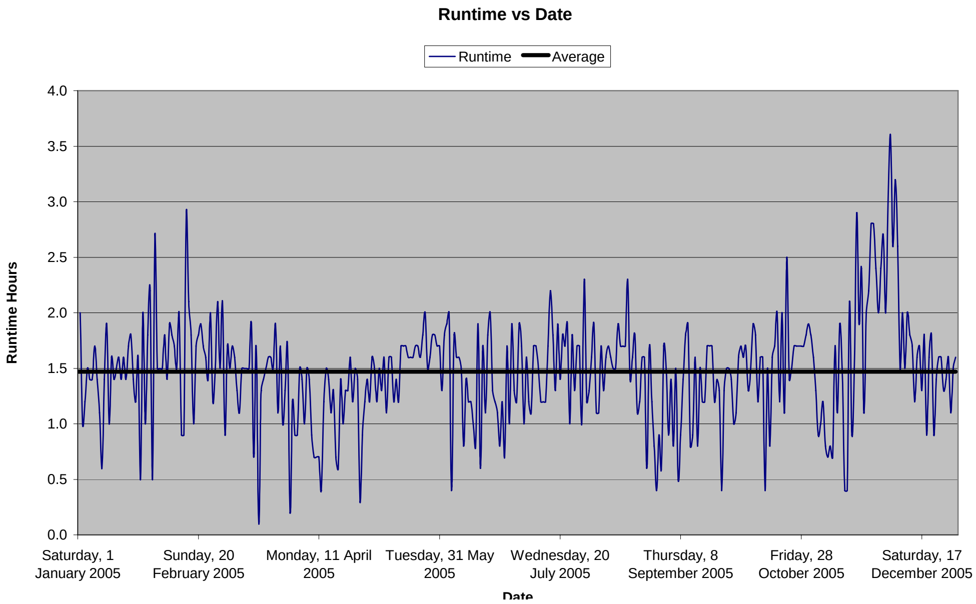

Figure 3-1 Runtime Vs Date for 2005

The extra flow on days of rainfall is used to approximate I/I for this section of the network work. This was completed by taking the total flow for the peak event and removing the ADWF leaving an approximated I/I. The I/I rate of 0.26L/s.km of sewer was found. After further analysis the conclusion was drawn that this only represents the inflow caused by the rain event and with no significant rain events to increase the current low moisture content of surrounding soil it will be very difficult to approximate any ground water infiltration.

This is the main reason that the peaking factor method was used to determine the PWWF.

3.7. Determine Future Demand

The level of future demand was assessed in regard to the current Council planning scheme. Much of the current open space is zoned residential, an approximate number of future house connections has been determined for these areas. This gives the maximum number of future residences that can be connected to the current catchment area. The extra loading was then applied to the model to see the affects on current infrastructure. The future demand here neglects time frame as the life span of the major infrastructure elements i.e. pump stations, can be up to 100 years. This 100years is based on the pump well, as over time there will be a need to replace pumps and control systems as they wear out and technology levels increase.

The Council's Planning Scheme 2005 shows undeveloped areas currently zoned as residential, these are the areas of main interest. Other areas that are currently zoned as rural residential may also provide some future sewer flows. At present only the two rural residential lots bordering the current sewer area have been connect to sewer, thus removing the requirement for onsite treatment. Figure 3-2 below shows the zoning of the areas surrounding the current catchment.

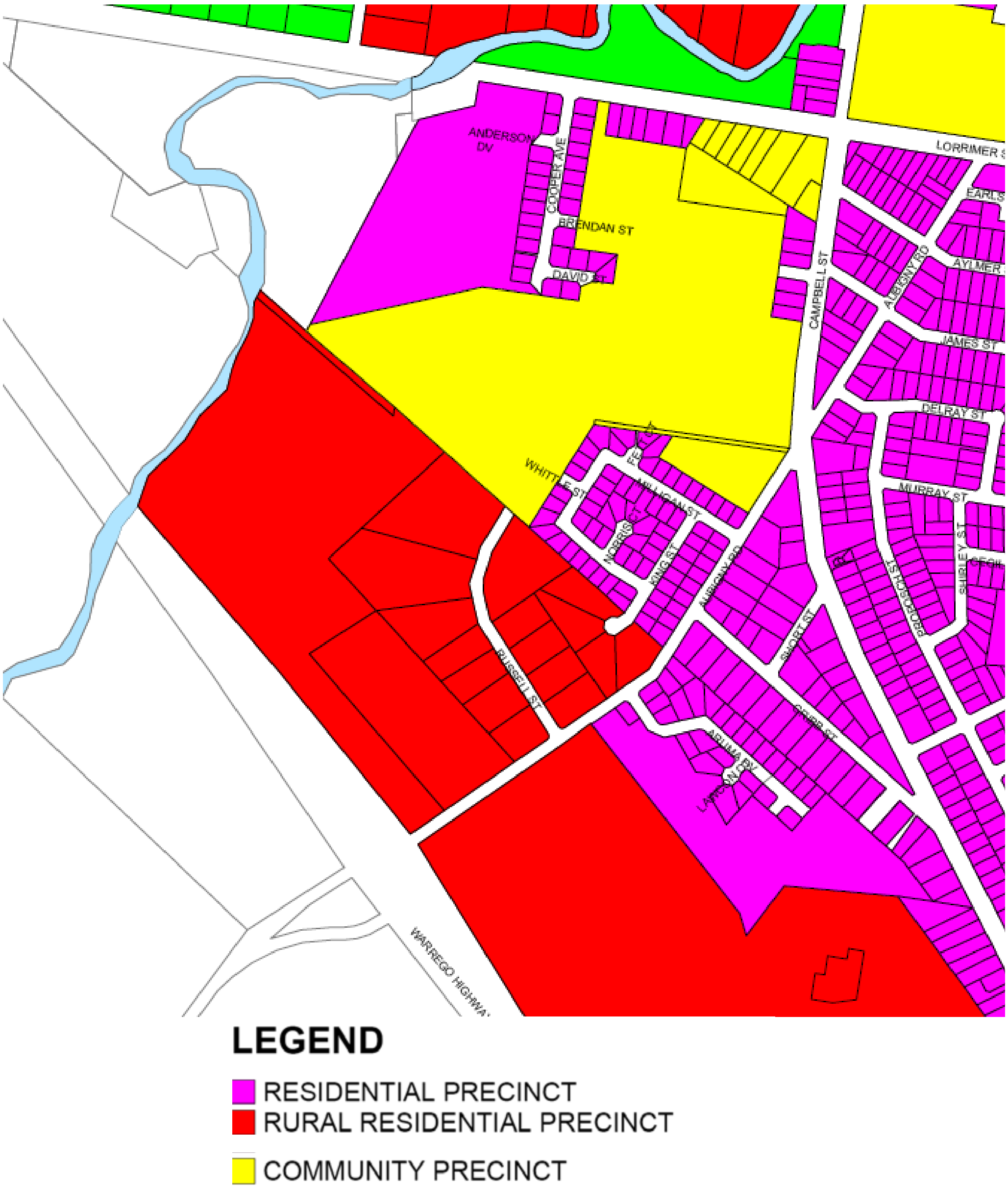

Figure 3-2 Current zoning, Obtained from Jondaryan Shire Council Planning Scheme 2004.

The approximate number of possible new allotments was calculated roughly from the area available. In the residential section containing Cooper Avenue and Anderson Drive it was calculated that another 55 allotments could be placed in the areas west of Cooper Avenue and bordered on the other side by Oakey Creek. The other area of interest for future development was that south of Aruma Drive. In this area it was calculated that another 40 allotments could be added in the remaining residential space. This is before a material change of use would be required to rezone the rural residential section that borders the highway.

The extra connections that would be generated by these developments were then added to the model. For the area a joining Cooper Avenue they were divided between manhole G5 and G11, in the Aruma Drive section the additional connections were divided between G39, G42 and G51. The location of any future network connections will be determined once an application is presented to Council. The current locations are used to show the effects of extra flow through the catchment.

As we are unsure as to the extent of the current drought condition, the future case was analysis under an EP loading of 225L/c.d (Nominated value for Queensland from the range 150-275L/c.d) this value is almost twice the current loading but is assumed to depict the past scenario in which no water restrictions were in place. Due to an adjustment in population the C_1 peaking factor was recalculated, the new factor for the future case is 5.35. The model was then run under both the future loading and the current loading rates with the results of this being discussed in chapter 5.

3.8. Assessment of the Condition of Sewer Mains

The assessment of the sewer mains will be undertaken from CCTV work completed on the section of the network. The lines in question will then be assessed for structural problems, condition of joints, roots and cracking. Once the problems are found, any major serviceability and structural problems will be attended to and a management strategy will be determined for lines in the area assessed. At the conclusion of this study only sections of the network had been inspected thus only a brief overview of what was found has been given in chapter 2. The remainder of the assessment will be undertaken once the remaining inspections have been completed. This information will then be entered into Council's GIS system and a needs analysis of the entire network will then be undertaken to determine any repairs that are required.

3.9. Input of Data into a SewerCAD Model

This is a simple process, an AutoCAD background is created from the GIS database, on this all the manholes are drawn in their correct locations. This AutoCAD file is then imported into SewerCAD where the manholes, lines and pump stations where put into place. The exact levels and material data are then keyed in for each element. The load count for each manhole is then entered and once the analysis is ready the peaking factor is inputted and the model analysed.

3.10. Perform a Network Analysis for Current and Future Demand Situation

With the model entered into the computer, an analysis can be run under steady state as well as extend period scenarios. To undertake the analysis of the network the ADWF will be applied and then factored to give the PWWF. From the analysis any problems with the network will be determined. At this step it will also give the ultimate capacity of each of the mains and the excess capacity at 75% depth of flow.

3.11. Recommendations of Required Works

From the analysis of the current situation and future situations, recommendations will be able to be made for any alterations needed for the network to perform to an acceptable level in its current state. As the current guidelines leave much of this to what Council requires, the main governing factors used for the recommendations of future modification are the requirement by Council for 4-6hours of storage capacity outside of the operating limits and the WSAA Sewerage Code requirement for pump station starts per set period of time.

3.12. Summary

The methodology of this study is simular but still varied from that given in the guidelines. The main reason for this was because of the time constraints as well as availability of accurate data in regard to flows and diurnal patterns of the catchment.

CHAPTER 4 – FORMULATION OF THE SEWERCAD MODEL

4.1. Introduction

SewerCAD offers many different methods of entering data for the model. The main method used is this case is detailed below. Once the model is developed, verification of the model is required to ensure accuracy of the outputs.

4.2. Variables Used in the Formulations of the Model

In the development of the model the mains variables that had to be set were those of pipe roughness, after some analysis of the values given by the SewerCAD package and a comparison with those values available in other text books such as *Wastewater Collection System Modelling and Design 1st Edition* the author made the decision to accept the nominated pipe roughness values. These values along with other manhole and pipe data can be seen in Appendix C.

4.3. Method of Data Input

The main method of data input into the SewerCAD model was via a graphical user interface. Figure 4-1 shows the current versions interface.

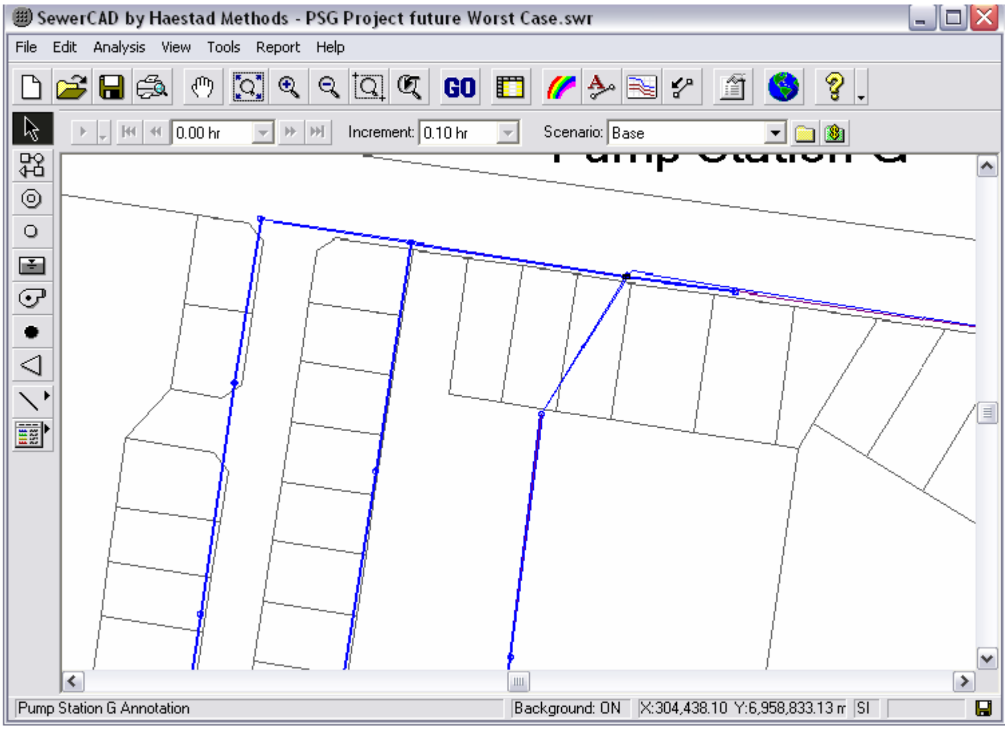

Figure 4-1, SewerCAD interface

4.3.1 Project Setup

The first setup to inputting the data was to setup the project in this the user is required to define general information about the project and a save directory. This information is then used later in the documentation produced by the program. As well as general information input user can also select the friction method to be used for gravity and pressure pipe flows. For this analysis Manning's equation was used for gravity flow and Hazen-Williams formula for pressure pipes.

4.3.2 Element Locations and Input

Once the project is setup, the user then has to define each element. This is usually general data such as pipe and manhole diameters. Because the majority of the network is 150mm diameter asbestos cement mains and all manhole are of the same diameter these values can be set as this point to avoid the editing these values of every element later.

After the element data has been inputted the user can then import a background file, in this case an AutoCAD drawing in DXF format. With the background in place the user can then begin to place elements at the required locations.

4.3.3 Element Data

After each element has been placed in the model, the individual data can be inputted. There are two methods in which this data can be entered. The first method is by selecting the element, Figure 4-2 shows an example of the properties of a manhole in the system where users can add the required information.

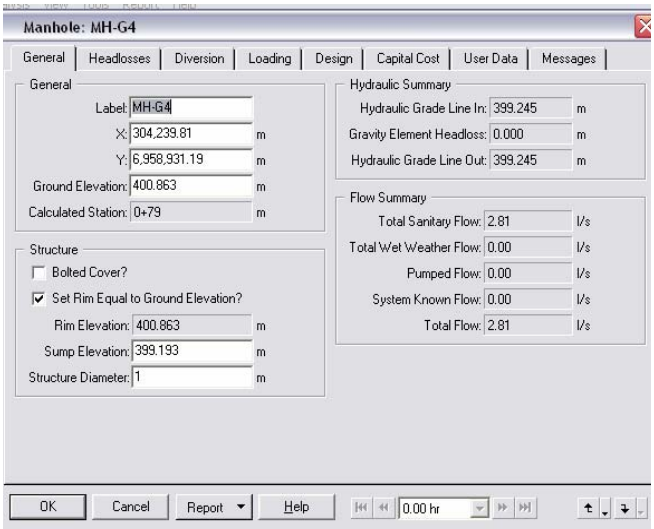

Figure 4-2, Example of manhole data input

The other method of data entry is using the table manager, if the data that required to be entered is in the same order as the table that is displayed in the program it is a very simple process of pasting data into the model.

In most cases it was found that each elements data needed to be entered by the properties display, this was a time consuming process. Once all data is entered only future modification will be needed to be added, with no requirement to re-enter current information for each of the different loading conditions.

4.4. Verification of Current Loading

This was undertaken by running an extended period simulation of the model. The main difference between this and a steady state simulation is the fact that it takes into account hydraulic retention times and pump start and stop limits. By running the model under the ADWF flow it was found that the number of pump starts was very high, but once the total runtime was calculated and total volume pumped was compared to 2005 data its was found that the model closely represents what is actually happening.

The reason for the high number of pump starts is due to the limit range that it operates under. Currently there is only approximately 0.8m of sewage in the well at the time the pump starts, this equates to $0.9m^3$ or 900L. At the measured pumping rate of 10.2L/s it was found to take only one and a half minutes to empty the well. The average daily runtime for the pump station is 1.47hours from this it can be seen that the pump starts around 60 times each day. Thus matching the high number of pump starts from the model.

The flows through the pump station were expected to match the ADWF from 2005 data. The main reason for this is that the flows entered into the model were taken from the 2005 data. Thus the confirmation of this showed that the loading had been entered correctly into the model.

4.5. Sensitivity Analysis

The sensitivity of the network to increased flows has been assessed under the future unrestricted loading case. The load that is used is at the lower end of the recommended values, the reason for this is that water consumption in times of no water restriction was estimated at 200L/c.d with the present average between 150L/c.d to 160L/c.d.

From the results shown in Appendix F all of the mains had at least 5L/s excess capacity at a 75% depth of flow.

4.6. Summary

Once the model was developed verification of this model was undertaken, it was found that under the ADWF the model appeared to depict the happenings in the network.

CHAPTER 5 – CURRENT AND FUTURE SITUATIONS

5.1. Results – Current Situation

From the analysis undertaken it was found that the network has sufficient capacity for current PWWF. The only weak point in the network is the pump station located in Lorrimer Street. In the event that a failure occurs within the pump station the well will only be able to hold approximately 10 minutes worth of PWWF and under the ADWF it was found to be able to contain approximately 58 minutes of flow. This falls very short of Councils requirement of 4-6hours ADWF capacity from the upper operating limit. In this case, since the pump station is located close to the treatment plant and any equipment required in dealing with a failure, only 4hours of capacity is required. To obtain these time figures it was assumed that the pump will fail at the same point as it is going to start hence giving the maximum level of sewage in the well. Once the pump had failed it was assumed that sewage continued to fill the well until it reached the overflow level this is the amount of time that has been shown above.

The reason for the current problem with storage capacity is due to the nature of the surrounding terrain, the small diameter of the pump well and current overflow system.

The overflow is currently located at the lowest manhole in the network manhole, G5, the overflow structure and long section showing from the pump station to manhole G6 can be seen in Figure 5-1 and 5-2 respectively. Due to this once the level in the pump well reaches a RL of 399.514m sewage will begin to back up in this manhole and once the level in the pump well exceeds a RL of 399.543m it will overflow to Oakey Creek. Because of this it leaves the near 5m deep pump well with a usable capacity of 1.6m, allowing for a total volume of 1.8m^3 of sewage to be stored for pumping. Appendix B contains a labelled map of the study area.

Figure 5-1 Current overflow structure and outlet

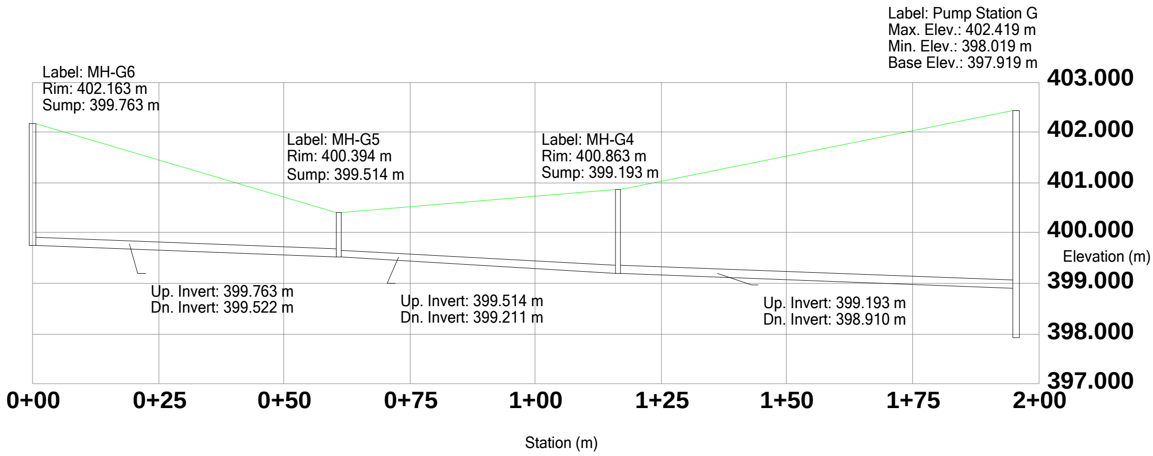

Figure 5-2 Long section showing the line from the pump station to manhole G6

With a capacity of 10.2L/s the current pump is easily able to cope with the current PWWF. The model shows that a problem will only occur in the case that some part of the pumping infrastructure fails. This does not take into account any blockages or failures through out the gravity network. Appendix D shows the outputs from the model under the current PWWF loading case. By examining the average velocity for each of the lines, it was seen that the majority of the short lines failed to meet the self cleansing velocity of 0.6m/s as given in the current guidelines. The longer lines were also on the limit or just above the self cleansing velocity, this account for the amount of silt found in the lines during cleaning. Periodic flushing in the further maybe required, but as these lines have never been cleaned in the past the extent of problems cause by the low velocities is minimal.

5.2. Results – Future Situation

The future model was run under both current and an assumed future load of 225L/c.d using the same peaking factor of 5.35 which is adjusted for the extra population.

5.2.1 Future Situations – Current Load

The current loading with the extra future connection lead to very similar results to the current situation. The only main difference was a marginal increase in velocities with the majority of lines meeting the self cleaning velocity this can be seen in the outputs for this situation in Appendix E.

The PWWF at the pump station was found to be 5.83L/s, this is still well below the pumps capacity and hence unless a failure occurs in the pump station there is little chance of problems with the network. This is also confirmed by the excess design capacity in the lines with the minimum excess capacity at 6.33L/s. A minor increase in the excess capacity can be seen, this is due to a very small increase in flow and hence a chance in velocity.

For this case the amount of time between a pump failure and an overflow event is also decreased slightly, for the PWWF only 6.5minutes of flow can be contained from the upper pump operating limit. In the case of the ADWF

34minutes of flow can be contained. Both times are well short of current requirements.

5.2.2 Future Situations – Assumed Future Loading

The assumed future load was applied to the every connection in the network this gave a total PWWF at the pump station of 8.83L/s this is still below the current pumps capacity. From this it can be seen that only the holding capacity of the pump well is a problem. Appendix F contains the outputs for each pipe in the network under this loading case. Velocities in this case are much closer to that required for the network to be self cleansing. Also there is a small reduction in the minimum excess capacity from the previous case, from 6.33L/s to 5.5L/s.

Under this case the amount of time before an overflow event in the case of pump failure was also assessed, the amounts of time calculated are as follows, for the PWWF, 4.3minutes and the ADWF, 23minutes.

5.3. Summary

The results were as expected from this section of the network with the main problem being the lack of capacity in the current pump well. Velocities have also been found as close to if not below the 0.6m/s self cleansing velocity limit this can mainly be attributed to the flatness of the surrounding terrain which limits the grades on the mains.

CHAPTER 6 – RECOMMENDATIONS

6.1. Introduction

The recommendations made are divided into two sections, the first section being short term. This section covers any requirement for the network to function adequately to current Council requirements. The second section covers more long term recommendations; these entail ideals for additional infrastructure to deal with both current and future demand.

6.2. Short Term Recommendations

From the analysis of the current loading on the network it was found that the only weak point was the capacity of the pump well.

Option 1

The simplest method of reducing the chance of an overflow event occurring is to increase the pump well capacity. The main problem with the increasing of capacity of the pump well is in regard to the current depth of the well and the limited range before an overflow event occurs. The additional capacity required in dealing with current loading and the Council requirement of 4hours storage capacity is 8.5m^3. Since the well has an operating range of only 1.6m, a second well would be required for the additional capacity. The second well would need to fall within the operating range of the current pump station, this would reduce the chance of an overflow event because if the second well was above the overflow limit, an overflow event would occur before the extra capacity was utilised. With the current overflow being 2.9m below ground level at the pump station, any new storage tanks would be required to have any usable capacity between this level and the inlet level of the pump. The main reason for this was that a 8.5m^3 well is required within this 1.6m deep, but for this to occur a well 4.7m deep with a surface area of 2.8m^2. is required, this would be expensive to construct due to it being required to be located on the footpath next to the existing pump station.

Option 2

The second option for an increase in capacity would be to install a well at the overflow. Hence in the event that the pump failed and the main well overflowed the sewage would be collected in a holding tank. Yet again problems were encountered with this solution, although being cheaper to install, there is no method of feeding the sewage back into the pump station once problems have been fixed. Because of this the level in this well would require monitoring and once an overflow event has occurred the well would need to be pumped out. Generally this would be by a truck similar to what happens with septic tanks.

Option 3

The final and most viable option for a solution would be the construction of a completely new pump station, this pump station could be located in the park opposite the site. Figure 6-1 shows a view of the neighbouring park, a good location for a new pump station.

Figure 6-1, Neighbouring Park

The main aspects that would require further analysis in this situation would be flood levels. In the background of Figure 6-1 you can see Oakey Creek and with a site visit it was confirmed that just outside the road reserve there was a large drop in ground level. This would aid in the construction of a new pump station by insuring the inlet is as near to the top of the well as possible. This allows for more usable capacity in the well before any sewage begins to back up in the lines.

The basic requirements of this well are as follows:

The required storage capacity of well for the current situation is a minimum $12.5m^3$ this includes a pump operating range of $4m^3$ or 1.5hours of ADWF. This 1.5hours of storage is shorter than the maximum 3hours between pump starts as set for other pumps station to control odour problems. This also accounts for an expected lower flow during the night and the middle of the day due to the effect of diurnal patterns.

A dual pump setup should be installed in the well; this will allow for the pump station to still function if one pump fails or is on maintenance.

The pumps required have not been sized due to unknown pump head requirement. It is proposed that this pump station pump directly to Council's treatment plant located on the other side of the creek further west on Lorrimer St. Pending survey of the area for the new pump station and outlet level at the treatment plant, pumps can be sized appropriately. The required flow for the current situation would be at least 3.8L/s as this is the current PWWF and pump must be able to cope without an overflow event occurring, or pump starts per hour exceeding manufacture and Council requirements.

6.3. Long Term Recommendations

In the short term recommendation it was seen that the best method of dealing with the problem of limited capacity was to construct a completely new pump station. As further development occurs the capacity of this pump station would also become a problem. Therefore in the case that another pump station is constructed it is proposed that it occurs is sections. The first stage of construction would be the replacement of the existing pump station with a prefabricated pump station from a manufacture such as Flygt. Council has already had experience with this style of pump stations in Westbrook.

The size of this main well would be required to handle an operating range of $9m^3$ this was taken from 1.5hours of ADWF under the future loadings of 225L/c.d. the extra storage capacity is not critical to the future well design. The main reason for this is that a well can be constructed to handle current flows, but in the case of future development additional storage wells can be constructed to feed back into the main well. This method has been utilised also in Westbrook at Council's pump station N and M. This method allows for a lower upfront cost with the ability for expansion if required in the future and in the event that the area does not become fully developed Council has not spent money that could be better used elsewhere. The costs involved with this addition storage could form part of head works charges incurred on new developments

6.4. Summary

Under the current situation the best option is to construct a new pump station with a minimum capacity of $12.5m^3$ in the park land opposite the current pump station, with this pumping directly to the Council treatment plant.

As the flows in the network increase the recommendations for the long term come into practice. Hence this short term solution would then grow as more development occurred, with additional storage being added as required.

CHAPTER 7 –CONCLUSIONS

7.1. Conclusions

This project gives a good insight into what is currently occurring in the catchment of the sewer network that was analysed. Although some section are not ideal on paper, once modelled it was found that the capacities of these lines are well with in the requirements with the main problem existing at the current pump station. This study also confirms initial thoughts about the networks capacity and the future requirements for upgrading and replacing infrastructure.

Although sections of this study will need to be taken further, the current information and results obtained will assist in giving direction for the future management of the catchment.

The remainder of the condition assessment was not undertaken due to a second attempted at the CCTV inspection being required. This is expected to be completed by mid November. With the analysis of information obtained undertaken shortly after.

7.2. Future Work

There are many more aspect in regard to the analysis undertaken here that will form further work. The main areas of interest are in the development of diurnal patterns as well as a full assessment of the infiltration and inflow in the network, this assessment will require significant rainfall events to provide sufficient data for analysis.

The next step after this project is to undertake an analysis of the remainder of the sewer network. This will help to determine any current or possible future problems.

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Appendices

APPENDIX A

Project Specification

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

ENG4111/4112 Research Project
PROJECT SPECIFICATION

FOR:	**James Andrew Hooper**
TOPIC:	Analysis of a catchment in Oakey's sewer network
SUPERVISORS:	Dr Ernest Yoong (Semester 1) Dr Vasantha Aravinthan (Semester 2) Philip Boshoff, Manager Water and Sewerage, Jondaryan Shire Council.
ENROLEMENT:	ENG 4111 – S1, D, 2006; ENG 4112 – S2, D, 2006
PROJECT AIM:	Develop a model of one of Oakey's sewer catchments; assess the physical condition of the current system from camera footage, verify the model against the actual flows within the catchment. Assess the I/I and PWWF. Analyse the capacity of the selected catchments' sewer system for current and future situations. Determine weaknesses in the system, by conducting a needs analysis and a sensitivity analysis and developing suggestions for a management strategy and make recommendations of any modifications that maybe required to meet current and future demand levels. Catchment size: 132 house connections covering approximately 35 hectares.
SPONSORSHIP:	Jondaryan Shire Council
PROGRAMME:	**Issue A, 16th March 2006**

1. Flow data collection – physical and flow from records, develop peak flow from analysis of flow data for all of Oakey's sewerage pump stations. Determine future demand by assessing future development with regards to current Council planning scheme.

2. Conduct spot check to confirm data records
3. Assess the condition of the sewer mains
4. Input data into a SewerCAD model
5. Perform a network analysis for current situation and future demand situation
6. Perform a sensitivity analysis on the model
7. Verification of model
8. System capacity integrity
9. Needs analysis/ strategy → asset management
10. Make recommendations for any modifications to existing infrastructure to meet current and future demand

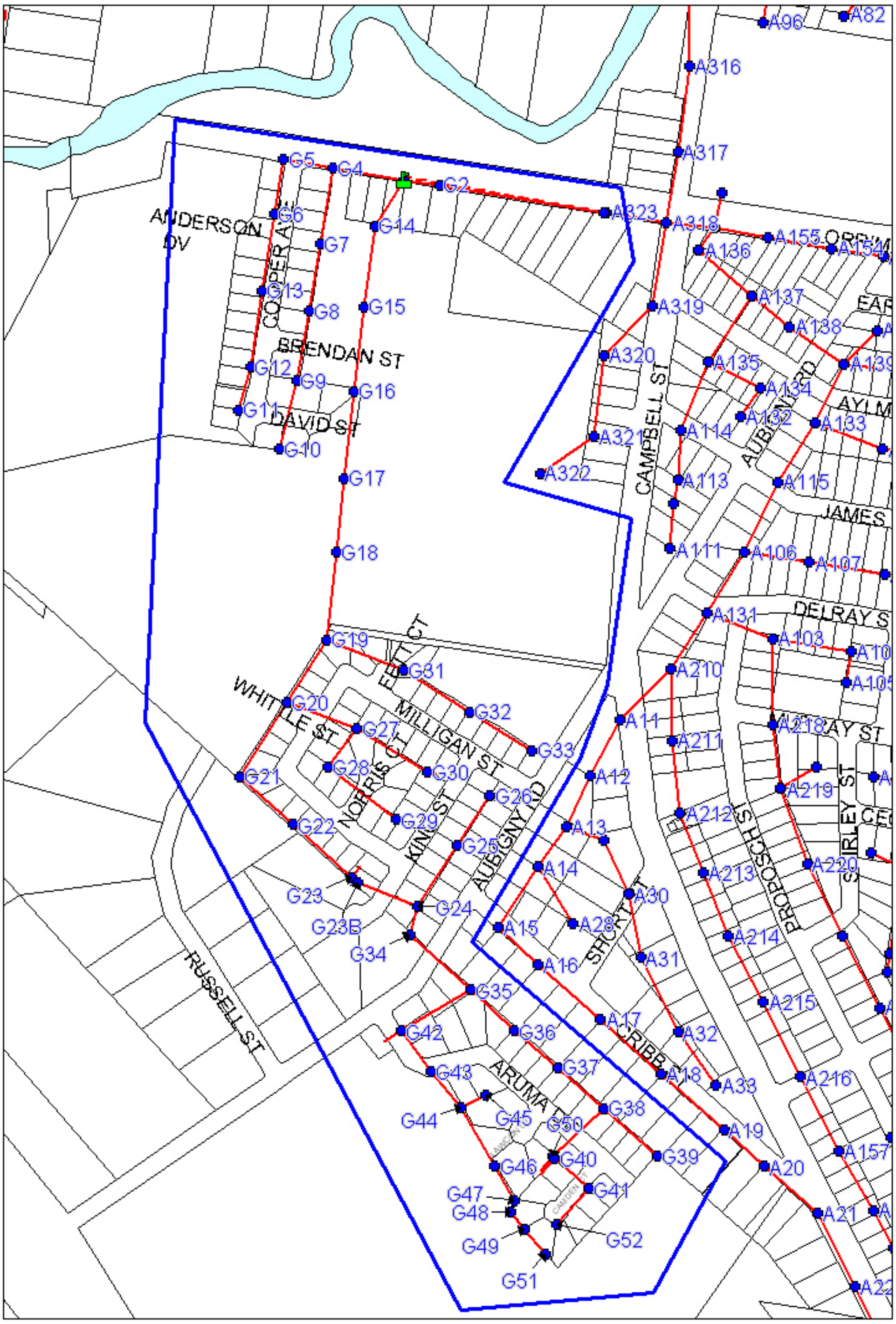
Agreed:

____________ (Student) ____________, ____________ (Supervisors)

___/ ___/ ___ ___/ ___/ ___ ___/ ___/ ___

APPENDIX B

Labelled Map of Study Area


ANDERSON DV
COOPER AVE
BRENDAN ST
DAVID ST
CAMPBELL ST
AUBIGNY RD
JAMES
DELRAY ST
FETT CT
WHITTLE ST
MILLIGAN ST
NORRIS CT
KING ST
AUBIGNY RD
SHORT ST
CRIBB
ARUMA
RUSSELL ST
PROPOSCH ST
SHIRLEY ST
G2 G4 G5 G6 G7 G8 G9 G10 G11 G12 G13 G14 G15 G16 G17 G18 G19 G20 G21 G22 G23 G23B G24 G25 G26 G27 G28 G29 G30 G31 G32 G33 G34 G35 G36 G37 G38 G39 G40 G41 G42 G43 G44 G45 G46 G47 G48 G49 G50 G51 G52
A11 A12 A13 A14 A15 A16 A17 A18 A19 A20 A21 A28 A30 A31 A32 A33 A82 A96 A103 A105 A106 A107 A111 A113 A114 A115 A131 A132 A133 A134 A135 A136 A137 A138 A154 A155 A157 A210 A211 A212 A213 A214 A215 A216 A218 A219 A220 A316 A317 A318 A319 A320 A321 A322 A323

APPENDIX C

Manhole and Pipe Data

US Manhole	DS Manhole	US SL (m)	US IL (m)	DS SL (m)	DS IL (m)	Slope %	Length (m)	Material	Mannings n	Section Size
MH-G8	MH-G7	402.447	399.847	402.630	399.740	0.143	75	Asbestos Cement	0.011	150 mm
MH-G4	Pump Station G	400.863	399.193	402.419	398.910	0.357	79	Asbestos Cement	0.011	150 mm
MH-G6	MH-G5	402.163	399.763	400.394	399.522	0.395	61	Asbestos Cement	0.011	150 mm
MH-G12	MH-G13	402.378	400.558	402.229	400.200	0.424	85	Asbestos Cement	0.011	150 mm
MH-G17	MH-G16	403.329	400.639	403.231	400.221	0.436	96	Asbestos Cement	0.011	150 mm
MH-G20	MH-G19	403.738	401.948	403.816	401.576	0.463	80	Asbestos Cement	0.011	150 mm
MH-G10	MH-G9	402.703	400.633	402.765	400.265	0.473	78	Asbestos Cement	0.011	150 mm
MH-G13	MH-G6	402.229	400.183	402.163	399.763	0.491	86	Asbestos Cement	0.011	150 mm
MH-G19	MH-G18	403.816	401.566	403.352	401.111	0.499	91	Asbestos Cement	0.011	150 mm
MH-G28	MH-G27	404.955	403.417	405.048	403.142	0.505	55	Asbestos Cement	0.011	150 mm
MH-G2	Pump Station G	402.753	401.589	402.419	401.129	0.501	92	Asbestos Cement	0.011	150 mm
MH-G16	MH-G15	403.231	400.203	402.797	399.727	0.503	95	Asbestos Cement	0.011	150 mm
MH-G21	MH-G20	403.511	402.451	403.738	401.958	0.505	98	Asbestos Cement	0.011	150 mm
MH-G18	MH-G17	403.352	401.086	403.329	400.648	0.500	88	Asbestos Cement	0.011	150 mm
MH-G9	MH-G8	402.765	400.265	402.447	399.867	0.513	78	Asbestos Cement	0.011	150 mm
MH-G26	MH-G25	406.784	405.532	406.741	405.193	0.527	64	Asbestos Cement	0.011	150 mm
MH-G25	MH-G24	406.741	405.159	407.058	404.717	0.543	81	Asbestos Cement	0.011	150 mm
MH-G5	MH-G4	400.394	399.514	400.863	399.211	0.548	55	Asbestos Cement	0.011	150 mm
MH-G15	MH-G14	402.797	399.717	402.675	399.215	0.560	90	Asbestos Cement	0.011	150 mm
MH-G24	MH-G23b	407.058	404.698	406.071	404.279	0.589	71	Asbestos Cement	0.011	150 mm
MH-G7	MH-G4	402.630	399.730	400.863	399.200	0.625	85	Asbestos Cement	0.011	150 mm
MH-G42	MH-G35	409.002	407.202	409.096	406.626	0.644	89	Asbestos Cement	0.011	150 mm
MH-G27	MH-G20	405.048	403.118	403.738	402.543	0.695	83	Asbestos Cement	0.011	150 mm
MH-G14	Pump Station G	402.675	399.195	402.419	398.750	0.754	59	Asbestos Cement	0.011	150 mm
MH-G32	MH-G31	405.874	404.404	405.048	403.618	0.912	86	Asbestos Cement	0.011	150 mm
MH-G33	MH-G32	406.633	405.143	405.874	404.404	0.907	81	Asbestos Cement	0.011	150 mm
MH-G50	MH-G38	416.589	414.180	414.638	413.468	0.920	77	Asbestos Cement	0.011	150 mm
MH-G48	MH-G47	416.955	415.605	416.455	415.505	0.758	13	PVC	0.010	150 mm
MH-G23b	MH-G23	406.071	404.251	406.001	404.176	0.937	8	Asbestos Cement	0.011	150 mm
MH-G23	MH-G22	406.001	404.171	404.857	403.337	0.955	87	Asbestos Cement	0.011	150 mm
MH-G22	MH-G21	404.857	403.337	403.511	402.471	1.091	79	Asbestos Cement	0.011	150 mm
MH-G45	MH-G44	412.894	410.719	412.824	410.374	1.097	31	Asbestos Cement	0.011	150 mm
MH-G11	MH-G12	402.234	401.094	402.378	400.558	1.099	49	Asbestos Cement	0.011	150 mm
MH-G29	MH-G28	406.061	404.531	404.955	403.435	1.175	93	Asbestos Cement	0.011	150 mm
MH-G31	MH-G19	405.048	403.588	403.816	402.291	1.422	91	Asbestos Cement	0.011	150 mm
MH-G30	MH-G27	405.956	404.456	405.048	403.120	1.474	91	Asbestos Cement	0.011	150 mm
MH-G35	MH-G34	409.096	406.616	406.477	405.277	1.495	90	Asbestos Cement	0.011	150 mm
MH-G34	MH-G24	406.477	405.277	407.058	404.717	1.736	32	Asbestos Cement	0.011	150 mm
MH-G46	MH-G44	414.398	413.248	412.824	411.924	1.787	74	Asbestos Cement	0.011	150 mm
MH-G52	MH-G41	419.981	418.538	419.120	417.688	1.614	53	PVC	0.010	150 mm
MH-G44	MH-G43	412.824	410.374	410.840	409.280	2.114	52	Asbestos Cement	0.011	150 mm
MH-G37	MH-G36	412.209	411.009	410.630	409.430	2.494	63	Asbestos Cement	0.011	150 mm
MH-G36	MH-G35	410.630	409.430	409.096	407.236	3.322	66	Asbestos Cement	0.011	150 mm
MH-G38	MH-G37	414.638	413.450	412.209	411.009	3.619	67	Asbestos Cement	0.011	150 mm
MH-G43	MH-G42	410.840	409.240	409.002	407.202	3.601	57	Asbestos Cement	0.011	150 mm
MH-G49	MH-G48	418.315	416.555	416.955	415.705	3.407	25	PVC	0.010	150 mm
MH-G39	MH-G38	417.975	416.614	414.638	413.468	3.990	79	PVC	0.010	150 mm
MH-G40	MH-G50	416.640	414.300	416.589	414.180	4.706	3	PVC	0.010	150 mm
MH-G47	MH-G46	416.455	415.405	414.398	413.298	4.740	44	PVC	0.010	150 mm
MH-G41	MH-G40	419.120	416.817	416.640	414.350	4.900	50	PVC	0.010	150 mm
MH-G51	MH-G49	420.585	419.005	418.315	416.655	6.537	36	PVC	0.010	150 mm

APPENDIX D

Current PWWF Situation Outputs

Upstream Node	Downstream Node	Total Flow (l/s)	Design Capacity (l/s)	Excess Full Capacity (l/s)	Excess Design Capacity (l/s)	Average Velocity (m/s)	Full Capacity (l/s)
MH-G8	MH-G7	0.17	6.48	6.94	6.31	0.16	7.10
MH-G17	MH-G16	2.99	11.31	9.40	8.31	0.56	12.40
MH-G16	MH-G15	3.05	12.14	10.26	9.09	0.59	13.31
MH-G19	MH-G18	2.99	12.10	10.27	9.11	0.59	13.27
MH-G18	MH-G17	2.99	12.10	10.28	9.11	0.59	13.27
MH-G20	MH-G19	2.50	11.65	10.28	9.15	0.54	12.78
MH-G4	Pump Station G	0.69	10.23	10.53	9.54	0.34	11.22
MH-G15	MH-G14	3.05	12.81	11.00	9.76	0.62	14.05
MH-G21	MH-G20	1.80	12.17	11.54	10.37	0.51	13.34
MH-G6	MH-G5	0.28	10.76	11.52	10.48	0.27	11.80
MH-G12	MH-G13	0.08	11.14	12.14	11.06	0.19	12.22
MH-G24	MH-G23b	1.44	13.14	12.97	11.70	0.51	14.41
MH-G10	MH-G9	0.03	11.78	12.89	11.75	0.14	12.91
MH-G13	MH-G6	0.19	12.00	12.97	11.81	0.26	13.16
MH-G14	Pump Station G	3.05	14.87	13.26	11.82	0.69	16.31
MH-G28	MH-G27	0.22	12.16	13.12	11.94	0.27	13.34
MH-G9	MH-G8	0.11	12.26	13.34	12.15	0.22	13.45
MH-G5	MH-G4	0.33	12.67	13.57	12.34	0.32	13.90
MH-G26	MH-G25	0.06	12.43	13.57	12.37	0.18	13.63
MH-G25	MH-G24	0.22	12.61	13.61	12.39	0.28	13.83
MH-G2	Pump Station G	0.06	13.26	14.49	13.21	0.19	14.54
MH-G7	MH-G4	0.25	13.53	14.59	13.28	0.31	14.84
MH-G42	MH-G35	0.25	13.74	14.82	13.49	0.31	15.07
MH-G27	MH-G20	0.53	14.28	15.13	13.75	0.40	15.66
MH-G23b	MH-G23	1.53	16.58	16.66	15.05	0.61	18.18
MH-G23	MH-G22	1.55	16.74	16.80	15.18	0.61	18.35
MH-G22	MH-G21	1.72	17.89	17.90	16.17	0.66	19.62
MH-G50	MH-G38	0.25	16.42	17.76	16.17	0.35	18.01
MH-G32	MH-G31	0.17	16.35	17.77	16.19	0.31	17.93
MH-G33	MH-G32	0.03	16.31	17.86	16.28	0.18	17.89
MH-G48	MH-G47	0.06	16.39	17.92	16.34	0.22	17.98
MH-G45	MH-G44	0.06	17.93	19.61	17.88	0.23	19.67
MH-G11	MH-G12	0.03	17.95	19.66	17.93	0.19	19.69
MH-G29	MH-G28	0.06	18.56	20.30	18.51	0.24	20.36
MH-G35	MH-G34	1.00	20.94	21.96	19.94	0.63	22.96
MH-G31	MH-G19	0.31	20.42	22.09	20.11	0.43	22.39
MH-G30	MH-G27	0.08	20.79	22.71	20.70	0.30	22.79
MH-G34	MH-G24	1.00	22.56	23.74	21.56	0.66	24.74
MH-G46	MH-G44	0.08	22.89	25.02	22.80	0.32	25.10
MH-G52	MH-G41	0.03	23.93	26.22	23.90	0.23	26.24
MH-G44	MH-G43	0.22	24.89	27.08	24.67	0.45	27.30
MH-G37	MH-G36	0.44	27.04	29.21	26.60	0.59	29.66
MH-G36	MH-G35	0.53	31.21	33.69	30.68	0.69	34.22
MH-G38	MH-G37	0.36	32.57	35.36	32.21	0.63	35.72
MH-G43	MH-G42	0.22	32.49	35.41	32.27	0.54	35.63
MH-G49	MH-G48	0.03	34.76	38.10	34.74	0.30	38.12
MH-G39	MH-G38	0.00	37.62	41.26	37.62	0.00	41.26
MH-G40	MH-G50	0.19	40.86	44.61	40.66	0.61	44.81
MH-G47	MH-G46	0.06	41.01	44.91	40.95	0.42	44.97
MH-G41	MH-G40	0.11	41.69	45.61	41.58	0.52	45.72
MH-G51	MH-G49	0.00	48.15	52.81	48.15	0.00	52.81

APPENDIX E

Future Situation Current PWWF Loading Outputs

Upstream Node	Downstream Node	Total Flow (l/s)	Design Capacity (l/s)	Excess Full Capacity (l/s)	Excess Design Capacity (l/s)	Average Velocity (m/s)	Full Capacity (l/s)
MH-G8	MH-G7	0.15	6.48	6.95	6.33	0.16	7.10
MH-G17	MH-G16	3.72	11.31	8.68	7.59	0.59	12.40
MH-G4	Pump Station G	2.01	10.23	9.21	8.22	0.47	11.22
MH-G16	MH-G15	3.77	12.14	9.54	8.37	0.63	13.31
MH-G19	MH-G18	3.72	12.10	9.55	8.38	0.62	13.27
MH-G20	MH-G19	3.27	11.65	9.51	8.38	0.59	12.78
MH-G18	MH-G17	3.72	12.10	9.55	8.38	0.62	13.27
MH-G15	MH-G14	3.77	12.81	10.28	9.04	0.65	14.05
MH-G21	MH-G20	2.64	12.17	10.70	9.53	0.57	13.34
MH-G6	MH-G5	0.93	10.76	10.87	9.83	0.39	11.80
MH-G12	MH-G13	0.75	11.14	11.47	10.39	0.37	12.22
MH-G24	MH-G23b	2.31	13.14	12.10	10.83	0.58	14.41
MH-G5	MH-G4	1.68	12.67	12.21	10.99	0.52	13.90
MH-G14	Pump Station G	3.77	14.87	12.54	11.1	0.73	16.31
MH-G13	MH-G6	0.85	12.00	12.31	11.15	0.41	13.16
MH-G10	MH-G9	0.03	11.78	12.89	11.75	0.14	12.91
MH-G28	MH-G27	0.20	12.16	13.14	11.96	0.27	13.34
MH-G9	MH-G8	0.10	12.26	13.35	12.16	0.22	13.45
MH-G26	MH-G25	0.05	12.43	13.58	12.38	0.18	13.63
MH-G25	MH-G24	0.20	12.61	13.63	12.41	0.27	13.83
MH-G42	MH-G35	0.98	13.74	14.09	12.76	0.47	15.07
MH-G2	Pump Station G	0.05	13.26	14.49	13.21	0.19	14.54
MH-G7	MH-G4	0.23	13.53	14.61	13.31	0.30	14.84
MH-G27	MH-G20	0.48	14.28	15.18	13.8	0.39	15.66
MH-G23b	MH-G23	2.39	16.58	15.79	14.19	0.69	18.18
MH-G23	MH-G22	2.41	16.74	15.94	14.32	0.70	18.35
MH-G22	MH-G21	2.56	17.89	17.05	15.32	0.74	19.62
MH-G48	MH-G47	0.43	16.39	17.55	15.97	0.41	17.98
MH-G50	MH-G38	0.23	16.42	17.78	16.2	0.34	18.01
MH-G32	MH-G31	0.15	16.35	17.78	16.2	0.30	17.93
MH-G33	MH-G32	0.03	16.31	17.86	16.28	0.17	17.89
MH-G11	MH-G12	0.70	17.95	18.98	17.25	0.51	19.69
MH-G45	MH-G44	0.05	17.93	19.62	17.88	0.23	19.67
MH-G29	MH-G28	0.05	18.56	20.31	18.51	0.23	20.36
MH-G35	MH-G34	1.91	20.94	21.05	19.03	0.76	22.96
MH-G31	MH-G19	0.28	20.42	22.12	20.14	0.42	22.39
MH-G34	MH-G24	1.91	22.56	22.83	20.65	0.80	24.74
MH-G30	MH-G27	0.08	20.79	22.72	20.71	0.29	22.79
MH-G46	MH-G44	0.45	22.89	24.65	22.43	0.53	25.10
MH-G52	MH-G41	0.03	23.93	26.22	23.91	0.23	26.24
MH-G44	MH-G43	0.58	24.89	26.72	24.32	0.60	27.30
MH-G37	MH-G36	0.65	27.04	29.00	26.39	0.66	29.66
MH-G36	MH-G35	0.73	31.21	33.49	30.48	0.76	34.22
MH-G43	MH-G42	0.58	32.49	35.05	31.91	0.73	35.63
MH-G38	MH-G37	0.58	32.57	35.14	31.99	0.73	35.72
MH-G49	MH-G48	0.40	34.76	37.72	34.36	0.68	38.12
MH-G39	MH-G38	0.25	37.62	41.00	37.37	0.62	41.26
MH-G47	MH-G46	0.43	41.01	44.54	40.58	0.78	44.97
MH-G40	MH-G50	0.18	40.86	44.63	40.68	0.59	44.81
MH-G41	MH-G40	0.10	41.69	45.62	41.59	0.51	45.72
MH-G51	MH-G49	0.38	48.15	52.43	47.78	0.84	52.81

APPENDIX F

Future Situation Assumed Future PWWF Loading Outputs

US Node	DS Node	Total Flow (l/s)	Design Capacity (l/s)	Excess Full Capacity (l/s)	Excess Design Capacity (l/s)	Average Velocity (m/s)	Full Capacity (l/s)
MH-G17	MH-G16	5.78014	11.30696	6.61951	5.52682	0.668	12.39965
MH-G8	MH-G7	0.23433	6.47799	6.86968	6.24366	0.179	7.10401
MH-G16	MH-G15	5.85825	12.13898	7.45382	6.28073	0.706	13.31207
MH-G19	MH-G18	5.78014	12.10042	7.48964	6.32028	0.702	13.26978
MH-G18	MH-G17	5.78014	12.10361	7.49314	6.32347	0.702	13.27328
MH-G20	MH-G19	5.07715	11.65017	7.69888	6.57302	0.66	12.77603
MH-G15	MH-G14	5.85825	12.8124	8.19232	6.95415	0.735	14.05057
MH-G4	Pump Station G	2.8073	10.23492	8.41671	7.42762	0.511	11.22401
MH-G21	MH-G20	4.10077	12.16893	9.24414	8.06815	0.644	13.34491
MH-G14	Pump Station G	5.85825	14.86989	10.44864	9.01164	0.82	16.30689
MH-G24	MH-G23b	3.59306	13.14395	10.8211	9.55089	0.656	14.41416
MH-G6	MH-G5	1.12793	10.76211	10.67421	9.63417	0.408	11.80214
MH-G12	MH-G13	0.85455	11.14466	11.36712	10.29011	0.386	12.22167
MH-G5	MH-G4	2.29959	12.67397	11.59917	10.37438	0.564	13.89876
MH-G13	MH-G6	1.01077	12.00039	12.14932	10.98962	0.427	13.16009
MH-G10	MH-G9	0.03905	11.77573	12.87466	11.73668	0.157	12.91372
MH-G28	MH-G27	0.31244	12.16247	13.02539	11.85003	0.304	13.33783
MH-G9	MH-G8	0.15622	12.26209	13.29086	12.10587	0.248	13.44708
MH-G42	MH-G35	1.52315	13.73963	13.54425	12.21648	0.53	15.0674
MH-G25	MH-G24	0.31244	12.61303	13.51949	12.30059	0.311	13.83193
MH-G26	MH-G25	0.07811	12.42737	13.55023	12.34926	0.202	13.62834
MH-G23b	MH-G23	3.71022	16.57828	14.47015	12.86805	0.782	18.18038
MH-G23	MH-G22	3.74928	16.73515	14.60313	12.98587	0.79	18.35241
MH-G7	MH-G4	0.35149	13.53212	14.48835	13.18062	0.339	14.83984
MH-G2	Pump Station G	0.07811	13.26262	14.46619	13.18451	0.217	14.5443
MH-G27	MH-G20	0.74204	14.27694	14.91459	13.53489	0.44	15.65663
MH-G22	MH-G21	3.98361	17.88707	15.63204	13.90346	0.843	19.61565
MH-G48	MH-G47	0.66394	16.39304	17.3133	15.72911	0.469	17.97724
MH-G50	MH-G38	0.35149	16.4219	17.65739	16.0704	0.388	18.00888
MH-G32	MH-G31	0.23433	16.35451	17.70065	16.12018	0.343	17.93498
MH-G33	MH-G32	0.03905	16.30912	17.84615	16.27006	0.198	17.8852
MH-G11	MH-G12	0.77644	17.95349	18.91205	17.17705	0.524	19.68849
MH-G45	MH-G44	0.07811	17.933	19.58791	17.85489	0.261	19.66602
MH-G35	MH-G34	2.96818	20.93685	19.99197	17.96867	0.867	22.96015
MH-G29	MH-G28	0.07811	18.56242	20.27815	18.48431	0.268	20.35626
MH-G34	MH-G24	2.96818	22.56228	21.77448	19.5941	0.915	24.74266
MH-G31	MH-G19	0.4296	20.41862	21.96224	19.98902	0.48	22.39184
MH-G30	MH-G27	0.11716	20.78611	22.67768	20.66894	0.333	22.79484
MH-G46	MH-G44	0.70299	22.88699	24.39576	22.184	0.603	25.09875
MH-G52	MH-G41	0.03905	23.93078	26.20435	23.89172	0.259	26.24341
MH-G44	MH-G43	0.89827	24.89472	26.40224	23.99646	0.688	27.30051
MH-G37	MH-G36	1.01543	27.04226	28.64015	26.02683	0.756	29.65558
MH-G36	MH-G35	1.1326	31.20583	33.08891	30.07323	0.864	34.22151
MH-G43	MH-G42	0.89827	32.48985	34.73135	31.59158	0.83	35.62961
MH-G38	MH-G37	0.89827	32.57217	34.82163	31.67391	0.831	35.71989
MH-G49	MH-G48	0.62488	34.76328	37.49786	34.1384	0.779	38.12274
MH-G39	MH-G38	0.39055	37.62054	40.86557	37.22999	0.715	41.25612
MH-G47	MH-G46	0.66394	41.00557	44.30435	40.34164	0.89	44.96828
MH-G40	MH-G50	0.27338	40.85705	44.53202	40.58367	0.677	44.80541
MH-G41	MH-G40	0.15622	41.68995	45.56257	41.53373	0.579	45.71879
MH-G51	MH-G49	0.58582	48.15385	52.22153	47.56802	0.96	52.80735

APPENDIX G

Pump Performance Curve

PERFORMANCE CURVE

FLYGT	PERFORMANCE CURVE			PRODUCT CP3102.181	TYPE MT
DATE 2006-07-28	PROJECT Thargaminda SPS			CURVE NO 53-435-00-3703	ISSUE 1

	1/1-LOAD	3/4-LOAD	1/2-LOAD
POWER FACTOR	0.82	0.76	0.65
EFFICIENCY	83.5 %	84.5 %	83.5 %
MOTOR DATA	---	---	---

RATED POWER	3.1	kW
STARTING CURRENT ...	34	A
RATED CURRENT ...	6.3	A
RATED SPEED	1445	rpm
TOT.MOM.OF INERTIA ...	0.039	kgm2
NO. OF BLADES	1	

IMPELLER DIAMETER 189 mm

MOTOR #	STATOR	REV
18-11-4AL	18D	11

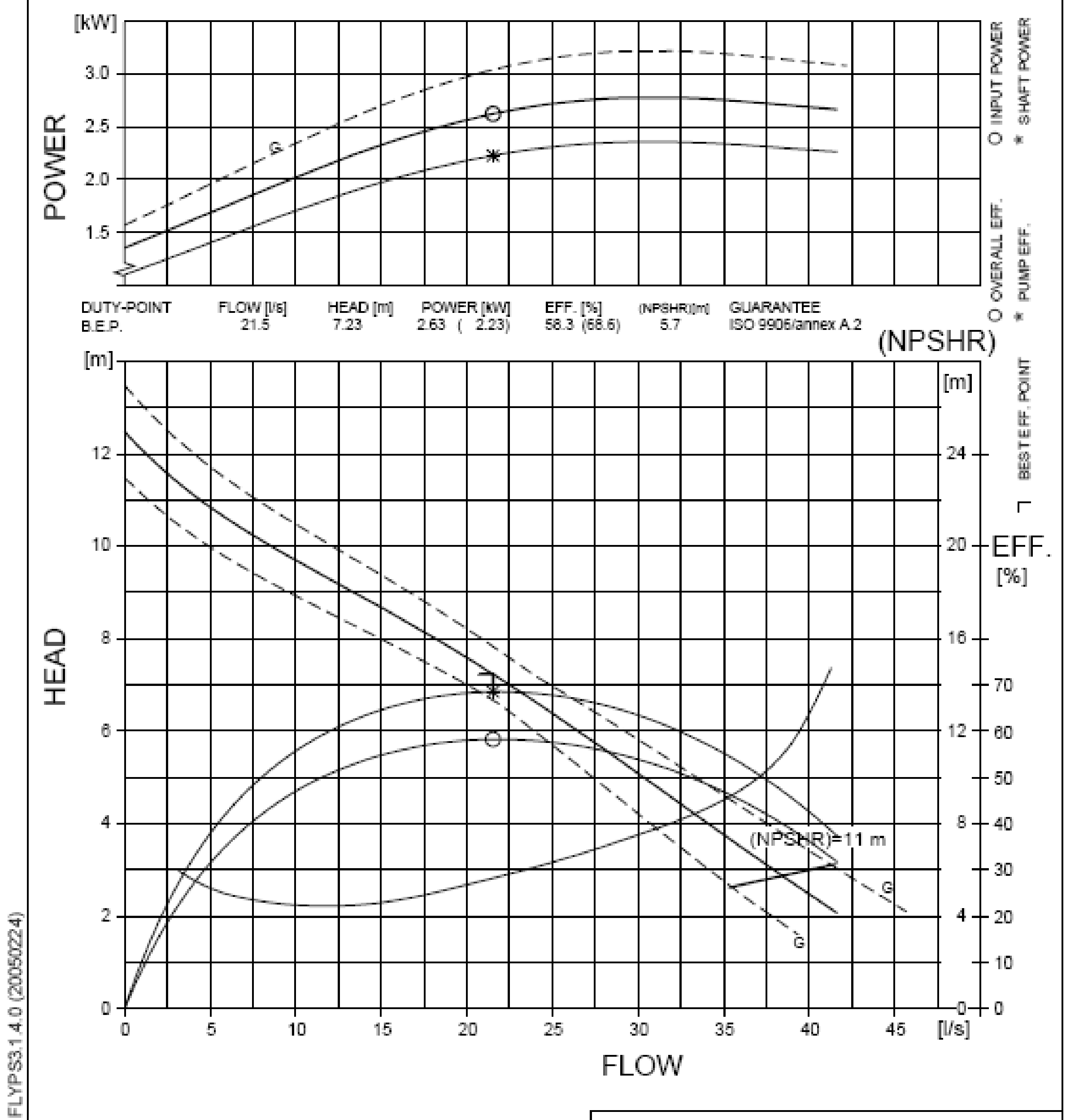
FREQ.	PHASES	VOLTAGE	POLES
50 Hz	3	415 V	4

GEARTYPE	RATIO
---	---

COMMENTS
NEVACLOG

INLET/OUTLET -/100 mm

IMP. THROUGHLET 76 mm



(NPSHR) = (NPSH3) + margins

Performance with clear water and ambient temp 40 °C

GUARANTEE BETWEEN LIMITS (G) ACC. TO

ISO 9906/annex A.2