University of Southern Queensland

Faculty of Health, Engineering and Sciences

St George Irrigation Area – Feasibility and Preliminary design for Stock and Domestic Pipeline.

Dissertation submitted by

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ABSTRACT

The St George Irrigation area, managed by Mallawa Irrigation (MI), currently supplies stock and domestic water to 80 customers via old unlined channels subject to large amounts of seepage and evaporation losses. Mallawa Irrigation Ltd has recently split from Sunwater to form a private irrigation district and has proposed a review of recent modernisation projects completed throughout NSW, VIC, and QLD to identify areas with successful modernisation of existing irrigation channels into full and partial stock and domestic pipeline upgrade to determine if a similar modernisation would be feasible within the St George Irrigation Area. The final aim being to determine grounds for a full-scale feasibility study.

Recently, significant government spending has been put into the modernisation of stock and domestic supply and irrigation channels, but it is difficult to determine the feasibility of such projects for smaller private irrigation districts. This research project aims to bring together key components of success for such projects and highlight how similar works could be adopted within the St George Irrigation Area.

A literature review determined the current standards for estimating evaporation, seepage losses, and water savings. Proposed and completed projects of similar scope were identified to determine, scope of works, costs, water savings, and environmental and socio-economic benefits these projects have had success with. Furthermore, a concept design was proposed to compare against the key factors found in the similar works to determine what a stock and domestic pipeline upgrade would look like for St George Irrigation Area.

The project determines estimation formulas for key channel losses for evaporation and seepage based on recent research. The design of concept pipeline found that similar outcomes could be achieved to those within the case study. Assessment of more similar projects for comparison and use this information to create a concept feasibility design framework for the use of other irrigation areas. Costs for a pipeline within the SGIA would be high due to flat terrain resulting in large diameter pipes and pumping however, feasibility could be achieved if Mallawa Irrigation are willing to cover the cost or if government spending is available.

ACKNOWLEDGEMENTS

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Most of all I would like to thank my Wife Adele and my two boys Charles and George for their infinite patience over the course of the year. The completion of this project has spanned over many challenging family situations, and my family has given me the motivation and determination to persevere through it all.

Table of Contents

LIST OF FIGURES

LIST OF TABLES

LIST OF EQUATIONS

KEYWORDS

Irrigation Modernisation, Stock and Domestic Supply, Irrigation Infrastructure, Seepage losses rural irrigation, Irrigation cost feasibility, Irrigation Modernisation Success Factors, Irrigation Efficiency.

NOMENCLATURE

DSE: Dry Sheep Equivalent

GIS: Geographic Information Systems

GL: Gigalitre

ID: Irrigation District

mAHD: meters Australian Height Datum

ML: Megalitre

RL: Reduced Level to Australian Height Datum

PVC: Polyvinyl Chloride

GRP: Glass reinforced Pipe.

DICL: Ductile Iron Cement Lined

HDPE: High Density Polyethylene

RE: Required Efficiency

WUE: Water Use Efficiency

DEFINITIONS

Stock and Domestic Water Supply: Water supply delivered for the purposes of stock watering and/or domestic use such as household use, garden watering, and washing clothes.

Farm gate Value: The net value of a product upon leaving the farm, less marketing costs.

Consumptive Pool: The amount of a water resource that can be made available for consumption (BOM 2023).

Chapter 1 INTRODUCTION

1.1 BACKGROUND

The St George Irrigation area (SGIA) is managed by Mallawa Irrigation (MI) and conveys approximately 50,000 ML of water to around 50 irrigators over a 10,000 ha area between the Balonne River and Buckinbah pump station (Figure 1-1) (Mallawa Irrigation 2023). This is achieved using approximately 112km of unlined open channels which feed into distribution networks owned by farming irrigators which run full through 12 months of the year (Mallawa Irrigation 2023). The SGIA has proposed a review of recent modernisation projects completed throughout the Murray-Darling Basin as well as Queensland and internationally to identify areas with successful modernisation of existing irrigation channels into full and partial pipeline systems with the integration of automation as well as stock and domestic pipeline upgrade to determine if a similar modernisation would be feasible within the SGIA.

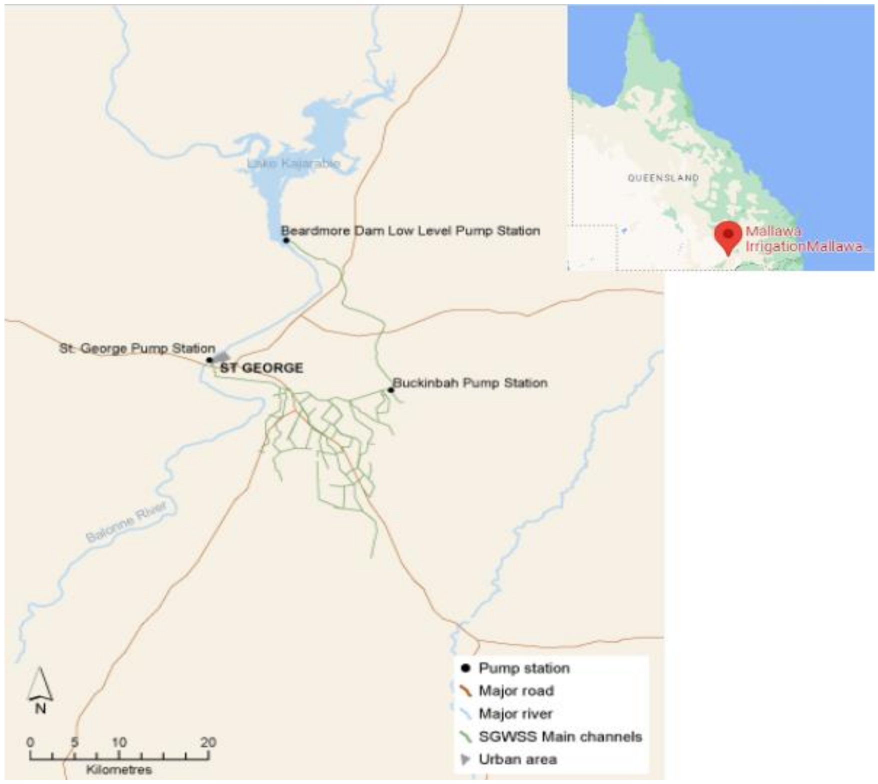

Figure 1-1 - Mallawa Irrigation Locality Map

A trend has begun over the past two decades in Australia towards drought proofing irrigation and minimising losses brought on by the millennium drought during 2001 to 2009 (Kirby et al. 2014). The drought highlighted the vulnerabilities of the irrigation networks throughout the Murray-Darlin basin over the past two decades (Kirby et al. 2014) which has seen southern states such as the Victorian government spend upwards of $280 million towards constructing various irrigation modernisation, water transport, stock and domestic pipeline upgrades, water grid upgrades, water supply and resilience, and education projects between 2017 and 2022 in regional Victoria (Victoria State Government 2021). While not all the upgrades directly affect the amount of water reaching the dams feeding irrigation networks, a large portion do minimise water losses through the network before reaching the farm thus delivering better outcomes and increased on-demand water supplied to properties. By reducing losses in the system downstream of the dam, there is also the added benefit of keeping more water in the dams and available to distribute to the networks. Furthermore, loss reduction is achieved by modernisation through the mechanism of keeping better account of water by the addition of metering or improved metering technology such as replacing old, inaccurate dethridge wheels with magflow or turbine meters. Other impacts of irrigation modernisation include reduction in overflow losses at the end of channel which can be reduced with the inclusion of a pipeline.

In more recent years there has been a movement of smaller irrigation providers forming separately from larger water utility providers in rural areas such as Sunwater and a push for the privatisation of irrigation areas which puts the power back in the hands of the irrigators. St George Irrigation area is one such provider which has recently transferred ownership from Sunwater to the new and locally owned Mallawa Irrigation in 2018 (Mallawa Irrigation 2023). Mallawa Irrigation with its newly appointed ownership and management of the St George Irrigation Area has the responsibility to be customer centric, accountable, transparent, efficient, and sustainable with the additional business plan of being business focussed, resilient, and resourceful. As a result, Mallawa irrigation now has the responsibility to investigate inefficiencies in their irrigation network and determine the extent of potential modernisation and feasibility of such works with the goal of improving service to their customers while also maintaining affordability and increasing the economic growth of the region (Mallawa Irrigation 2023).

Given that Mallawa Irrigation Ltd now currently owns and maintains the network it is within their best interest to launch a full-scale feasibility study however, the scope of the current project is to merely inform Mallawa Irrigation on whether there are significant grounds for such a study. The stock and domestic delivery is currently supplied by the irrigation channel system. This approach encounters issues when there is limited supply during a drought and channels dry out. The process at that point is to fill a channel which may hold up to 100ML to deliver a one or two offtakes. As such it is also necessary to include a preliminary design of a network for either full or partial upgrade to compare and analyse against other similar successful projects. The study identifies recently completed projects and uses their example to develop suitable criteria of success of which the preliminary design can be weighed against. Further to this it is a requirement that suitable data is gathered to determine closely what the losses are for the current system and therefore, it is necessary to complete a literature review on seepage and evaporation rates within open and unlined channels as well as identify any other means of modernisation which do not require a pipeline. Figure 1-2 shows an example of the condition of the existing channels within the Buckinbah channel which is in the SGIA.

Figure 1-2 Unlined Open Channel Example - St George Buckinbah B2 Channel (Mclean 2015)

1.2 Objectives

This thesis aims to investigate Mallawa Irrigations initial brief through four key objectives outlined below. These objectives will guide the thesis and provide the structure required to meet the brief outlined. The objectives are as follows:

1.2.1 Objective 1: Loss Estimation

Conduct a literature review to summarise the most recent research into losses and loss estimation. This objective will be important as it identifies current research in seepage, evaporation, and other losses with an aim to provide a quick method for estimation for use in a feasibility study.

1.2.2 Objective 2: Supply and Demand

Conduct a literature review to determine the best practices for sizing pipeline including supply and demand, flow estimation, and water savings. Determining flow estimation is critical to the design of a pipeline.

1.2.3 Objective 3: Case studies

Identify several case studies involving modernisation and/or development of stock and domestic pipelines with an aim to determine project outcomes, factors of success, cost, water savings, and both economic and socioeconomic benefits for the local regions and townships.

1.2.4 Objective 4: Concept Design

Document the concept design process with a purpose of providing future feasibility studies with a starting point for a much more detailed study. This is the main objective for this thesis as it will provide much of the information for high level costings specific to the St George Irrigation Area however the process could be followed and refined for other Irrigation areas as well.

1.3 PROJECT METHODOLOGY

1.3.1 Reason for the Research Project

Mallawa Irrigation has expressed a need for a review of stock and domestic supply upgrades which have been completed in New South Wales and Victoria completed as part of State Government funding initiatives with a desire to understand what the upgrade works were undertaken, what were the costs involved, were the projects successful, and what benefits the works had for the community and environment.

1.3.2 Quantitative Analysis

This project will follow a quantitative approach with most of the data being sourced directly from Mallawa Irrigation, via desktop methods, and typical values used industry wide. The sourced data will include maps, channel lengths, channel profiles, demand, and usage data, as well as estimated flow data.

The data obtained is expected to be of limited quality and as such it is anticipated that there will be many gaps to fill from external sources as well as a lot of assumptions to be made.

1.3.3 Literature Review

The literature review focuses on the main factors which area considered for feasibility of stock and domestic pipeline supply upgrades such as, losses through evaporation and seepage/ leakage, water savings, stock and domestic demand volumes, and methods on further on-property water saving and storage.

Further to this a review of similar stock and domestic, channel modernisation, and channel refurbishment projects have been explored to summarise what a standard channel modernisation project and stock and domestic supply upgrade will look like, cost and how they perform, and what environmental and socio-economic benefits they provide for the irrigation areas and surrounding regions.

1.3.4 Design Methodology

A preliminary hydraulic design has been developed based on the data obtained using current standard hydraulic design formulas. Assumptions in data have been sourced from projects or technical documents deemed to be relevant and reasonable enough to be adopted into a preliminary design. Some options have been considered and the main option has been designed to a concept level which was fed into cost analysis formulas to estimate preliminary capital costs inclusive of materials, construction, and maintenance costs.

1.3.5 Limitations

The project will be limited in scope by the availability of technical resources, availability of case studies which incorporate all areas of interest, sample size of case studies, availability of data for analysis, and of course time. Every effort has been made to include the most relevant case studies and source the best readily available data from relevant technical publications however a more thorough approach should be used for a full-scale feasibility study. The Hydraulic design will be limited to empirical methods of calculation to estimate pipe sizes, costing, and water savings and assumptions will be made due to gaps in data for demand use, land use, and geometric data such as reduced levels (RL) of terrain and property boundaries. Any software used will be either web based, open sourced, or office applications where required. The project will only consider the St George irrigation area mostly in its current form with only a small allowance for future land subdivision requiring more offtakes, or significant changes to farming land use for example a trend occurring towards livestock-based agriculture.

1.3.6 Ethical Considerations

There are numerous ethical considerations to be examined for a modernisation project of this scale. Lining an unlined channel or upgrading to pipes can lower seepage and evaporation losses to almost zero and could cause previously wet areas to dry out along the path of the canals. Other issues could be reduction of river or channel flows which could have detrimental ecological impacts to any fish or wildlife which currently use them as habitat or water source.

If any relining of channels is to be considered within the network, then the use of PE liners can cause animal deaths as indicated in Kevin Long's discussion paper (Long & Poynton 2009). Here he and Chris Poynton discuss major design flaws in these types of lining systems which can cause animal deaths unless a mean of egress is provided.

Further to the above both Lony and Poynton discuss other issues with the Murray Darling North-South pipeline project which should be considered to determine if worthy of concern with the SGIA modernisation. In their paper they discuss, with calculations provided, that the amount of water saved versus the amount of capital cost outlaid for the project does not justify the project as the Victorian Government have indicated in their reports. They also discuss the idea that the water savings are not actually savings and more transferring as the water is actually diverted away from its current use in the environment and recharging aquifers (Long & Poynton 2009). This paper is a discussion piece and relevancy to SGIA should be carefully considered.

In the editorial article 'Water Lies', Kenneth Davidson discusses similar topics to the previous paper but also touches on ideas that most of the issues with water is no that there isn't enough rain, but more that a lot of the rainfall happens all at once and cannot be adequately captured (Davidson 2009).

1.3.7 Risk Assessment

There is a certain level of risk with any irrigation modernisation project and in the case of privately owned irrigation districts such as SGIA, the risks will be borne by the irrigators. This report seeks to identify the risks and key performance indicators which Mallawa Irrigation would expect the modernisation project to mitigate and achieve respectively.

Key risks have been identified in the sections above and in the following chapters, key performance indicators will be identified, discussed, and a mitigation plan will be developed where possible. Where risks are identified which cannot be mitigated, they will be outlined for Mallawa Irrigation consideration in the final chapter.

1.3.8 Gaps in the research

The literature review found a lot of information regarding irrigation modernisation as well as some well documented reports for the upgrades of stock and domestic supply systems. Further to this, research into losses and water savings are also highly researched and technical documents exist to assist in determining these losses within a system however, it is very difficult to determine the exact design methodology used within the projects and the specifics around pipe sizes, types and what calculations were used. This report seeks to determine a concept level of design for the required St George Irrigation Area with the hopes to provide some insight into the design process and feasibility outcomes which have been used in other areas essentially putting the pieces together in a more coherent manner and setting a base framework for a future, more rigorous feasibility study.

Chapter 2 : LITERATURE REVIEW

2.1 REASONS FOR MODERNISATION

Drought is one of the largest drivers of modernisation as many farming townships around Australia went through hardships during the millennium drought. Modernisation has been identified as a key method of tackling the issue due to a large portion of existing irrigation systems being over a century old and consisting of long lengths of unlined or poorly maintained lined channels which are subject to a significant rate of seepage into the ground below (Koech & Langat 2018).

Agriculture and farming are key growth contributors to the Australian economy and largely the Murray Darling basin has high agricultural value within that contribution (DEECA 2022). As such the Queensland, New South Wales, Victorian and South Australian state governments are seeking to invest heavily into the modernisation of old, unlined and poorly maintained channels with the adoption of HDPE pipes and automated systems to improve water efficiency and to reduce seepage to negligible levels (Dann 2009).

There are several benefits to piped stock and domestic supply include the potential to supply pressurised water on-demand to consumers, more control over the supply with the use of storage tanks to maintain consistent flow, better account of water use through the use of and upgrades to metering devices, and stimulus to the regional economy through the use of local contractors and suppliers (McCartney et al. 2019).

2.2 Methods of Modernisation

With the current boom in irrigation modernisation there are many methods of modernisation which could be implemented from the more traditional approaches such as channel upgrades, lining of channels and pipe replacement, to more innovative methods including stock and domestic pipeline links to new water sources, pumping, and channel automation.

Water use efficiency or WUE is an important topic when investigating irrigation upgrades. In their paper about advances, challenges and opportunities in water efficiency, (Koech & Langat 2018) talk about commonly used methods of measuring water efficiency, being application efficiency and requirement efficiency and how performance measures include, conveyance, distribution and storage efficiencies which are subject to seepage and evaporation.

2.3 Losses

Water losses occur in both conveyance and distribution channels prior to field delivery and, if the water is stored within a dam or lake, then it may be subject to even further losses by the mechanisms of seepage and evaporation (Koech & Langat 2018).

Losses are thought to be a large contributor of inefficiency in irrigation and the main driving factor behind modernisation. A report by the Food and Agriculture Organisation of the United Nations titled "Does Improved Irrigation Technology Save Water?" released in 2017, reviews evidence based on modernisation techniques adopted by 13 countries including, China, India, Pakistan, Spain, and the US which concludes that the review found no evidence that significant water savings were found. Others have concluded that efficiency improvements are more achievable through changes such as mulching techniques rather than channel modernisation. (Perry & Steduto 2017). Notwithstanding, it is generally accepted that channel losses are significant and any successful attempt at rectification will have positive benefits for farmers as well as the irrigation schemes they operate within. This section will provide more details on the components of channel losses.

2.3.1 Seepage Losses

Leakage and seepage losses are defined as all water lost through the banks of bed of the channel during water delivery. Bank leakage and seepage can be accurately determined using the pondage test which determines the seepage in an isolated section of channel using a water balance where the losses are reflected in a drop in water level over time in the pond which has been hydraulically isolated. The results are then separated from other losses such as evaporation and rainfall (DSE 2012).

To determine how much water can be saved from minimising losses, first the seepage and evaporation losses must be estimated. In her 2015 honours thesis report, Melissa Mclean ran an investigation into seepage losses specifically within irrigation channel in the St George Irrigation area, her report first estimated that combined seepage and evaporation losses were in the order of 15 per cent loss factor which would equate roughly to 150 ML per GL of water delivered to customers from the Beardmore Dam (Mclean 2015).

Through the course of her research, Melissa performed a seepage test known as the ponding test method to calculate daily seepage losses at three separate locations namely, the St George main channel, the Buckinbah B2 Channel, and the Buckinbah B2/2 Channel. She performed the tests and concluded that a daily seepage rate of 0.008 m/day or 8 mm/day was observed.

In his master's thesis report Amirali Moavenshahidi conducted research into estimating seepage losses in automated irrigation networks in which total channel control (TCC) data was collected from selected pools in different channels within an irrigation network and an analysis of the data was performed using a novel computer program of his own writing (Moavenshahidi 2013). The findings of his report indicates that seepage rates are affected by seasonal water table levels as well as the ponded water elevation within the channels stating that 'the initial water elevation within the channel has a direct relationship with the estimated seepage rate, with higher rates of seepage occurring at higher water elevations' which are illustrated in figure 2-1 below (Moavenshahidi 2013).

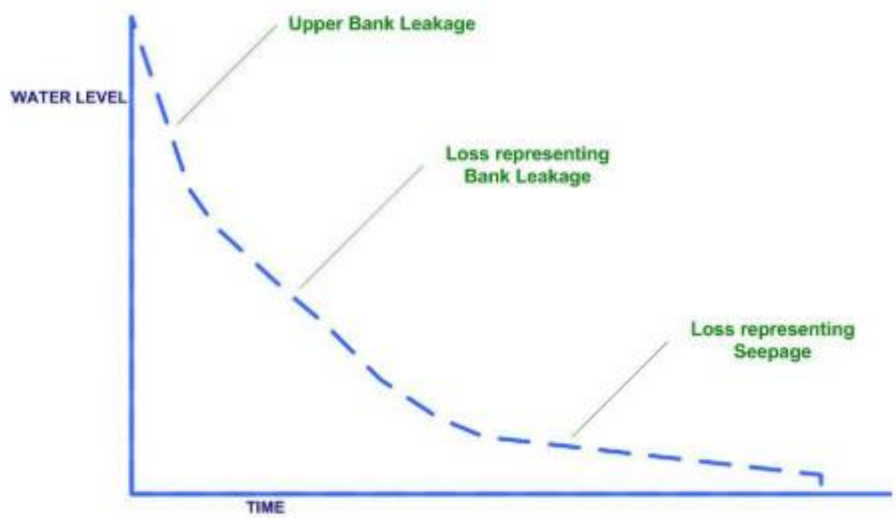

Figure 2-1 Example gradient changes within Ponding test to represent loss (DSE 2012).

In a research report by Saud Akbar in the year 2000 (Akbar 2000), the losses were measured from on farm channels and drains within the Coleambally and Murrumbidgee Irrigation areas during the years 1997-1998 and 1998-1999 irrigation seasons. He conducted testing only on permanent on-farm drainage channels. Through his research it is noted that, from a wide range of rates observed, the higher seepage rates were from newer channels formed and that channels in the age range of 15 to 30 years lost water from seepage a far lower rate. Seepage rates of 0-50 mm/day were observed in the older channels compared to 0-160 mm/day observed in new channels. Ackbar observes that this is possibly due to the presence of weeds and sediment deposits, and it is also noted that in newer channels with lower seepage rates observed the rates could be affected by a range of other factors including channel bed compaction, soil sodicity, channel bed slope and bend radius. It was also noted that older channels which experienced higher seepage rates had been cleaned prior to the irrigation season. Soil types were noted as another variable in higher volumes of seepage with sandy loams having higher rates than that of clay soils. It was also noted that higher seepage volumes were observed in areas of channel which had both sandy loam and a water table located within 1m of the base of the channel. Akbar concludes that the most important factors contributing to seepage losses within an open channel are soil type and water depth. It is further noted that there is a direct correlation between seepage rates and the width of channels being that wider channels experience larger volumes of seepage and that when water tables are low, the seepage rates are not affected by the wetted perimeter of the channel.

2.3.2 Evaporation Losses

"Evaporation is the water lost from the surface area of the irrigation channel system. It is assumed to be a fixed loss with the irrigation system is charged" (DSE 2012). Evaporation within a channel can be calculated with the following Equation 2-1:

Equation 2-1 Annual Evaporation formula (DSE 2012)

$$Annual\ Evaporation\ (ML) = \left(\frac{(E * PEF) - R}{1000000}\right) * A * CWF * t$$

Given:

E = Daily evaporation rate (mm/day)

PEF = Pan Evaporation Factor

R = Daily Rainfall rate (mm/day)

A = Surface Area of system (m^2)

CWF = Channel width Factor

t = Length of standard irrigation season (days)

Because rainfall and evaporation can vary significantly depending on location, it is important to gather data from the closest weather stations. It is iso important to note that the environment in which the pan is situated can affect the evaporation rate as the losses from a large body of water are different to those in sheltered location. In the case of the Goulbourn-Murray Irrigation District a pan evaporation factor of 0.83 was adopted in the absence of better information (DSE 2012).

Where data is readily available an alternative method using weather station data to calculate theoretical evaporation is more accurate in determining evaporation (DSE 2012).

Water evaporation is difficult to measure directly and therefore numerous methods for estimation have been determined. Typically, these methods include pan coefficients, measured pan evaporation, water balance, energy balance, mass transfer, and combined methods (Jensen 2010). The most common method used is the Evaporation Pan Method (Equation 2-2) which applies a coefficient to measure pan evaporation (Jensen 2010):

Equation 2-2 Evaporation Pan Equation

$$E = K_p E_{pan}$$

Where Kp = pan coefficient

Epan = evaporation from class A pan

It is not recommended however, to estimate evaporation for smaller water bodies using this method due to differences in pans sizes used as well as the location of the pans which can be affected by hours of sunlight, wind variances and temperature (Moavenshahidi 2013).

Due to the availability of weather data being available from on-site automated weather stations (AWS) which includes daily weather data, solar radiation, humidity and wind speed, evaporation Moavenshahidi determined that the combined Penman-Monteith method of estimating evaporation was more ideal (Moavenshahidi 2013). The Penman-Monteith model method is based on readily available data, has limited empirical basis which makes it more applicable to open water bodies, and accounts for heat storage (McJannet et al. 2008).

The Penman-Monteith equation (Equation 2-3) below uses the aforementioned AWS data to produce time series of evaporation rates as the product E mm d-1 (McJannet et al. 2008).

Equation 2-3 Penman-Monteith equation (McJannet et al. 2008)

$$E = \frac{1}{\lambda}\left(\frac{\Delta_w(Q^* - N) + 86400\rho_a C_a(e_w^* - e_a)/r_a}{\Delta_w + \gamma}\right)$$

where:

λ (MJ kg^{-1}) is the latent heat of vaporisation,

Δ_w (kPa ℃$^{-1}$) is the slope of the temperature saturation water vapour curve at water temperature,

Q^* (MJ m^{-2} d^{-1}) is net radiation,

N (MJ m^{-2} d^{-1}) is change in heat storage in the water body,

ρ_a (kg m^{-3}) is density of air,

C_a (MJ kg^{-1} °K^{-1}) is specific heat of air,

e_w^* (kPa) is saturated vapour pressure at water temperature,

e_a (kPa) is vapour pressure at air temperature,

r_a (s m^{-1}) is aerodynamic resistance, and

γ (kPa ℃$^{-1}$) is the psychometric constant.

Due to the limited nature of the required feasibility, it is determined that a simplified method for determining Evaporation is relevant to the project and therefore the evaporation within a channel equation will be used however inputs will be assumed where a lack of reliable information exists, and a more rigorous method should be adopted in subsequent feasibility studies.

2.3.3 Losses due to metering

Often properties which draw water from irrigation channels for stock and domestic purposes are unmetered or metered with old, inaccurate dethridge wheels (Figure 2-2). This can lead to poor water accounting and even to some properties taking more water than their annual deemed allocation for stock and domestic purposes which in turn means less water for other users (DSE 2012)

Further to water savings by pipeline or channel lining upgrades, (Koech & Langat 2018) outline the importance of measurement of water savings and how the use of old measurement techniques such as dethridge wheels can have inaccuracies between -18% and +3%.

Dethridge meter error is the additional volume of water which flows through a meter which is not recorded and therefore unaccounted for. There are three methods to estimate dethridge meter error being, adoption of statewide average for all meter readings within the irrigation area, undertake in-direct measurement using wheel clearances and modelling, and to undertake in-situ measurements for the losses (DSE 2012).

Dethridge wheel measuring irrigation water consumption, Griffith, New South Wales. Photo: Bill van Aken, CSIRO.

Figure 2-2 Photo of Dethridge Wheel (Kirby 2011)

2.3.4 Other Losses

There are more losses associated with open unlined channel conveyance which are worth mentioning but are not considered to be within the scope of a concept feasibility study. These other losses include leakage and spillage over banks, outfall water losses, leakage through service points, and unallocated losses which are the remaining losses once all other have been accounted for (DSE 2012).

2.4 Design Methodology

2.5.1 Water Savings

The Victorian Government technical manual for the quantification of water savings in irrigation water distribution systems outlines steps to calculate the water savings for a network and provides context to what inputs and outputs should be used for various stages of the design process. For this report only the phase one process will be considered and used where appropriate data can found. Where gaps in data exist then assumptions can be made and are sometimes provided within the document. The first phase of estimating and verifying water savings in this document is provided in Figure 2-3 (DSE 2012).

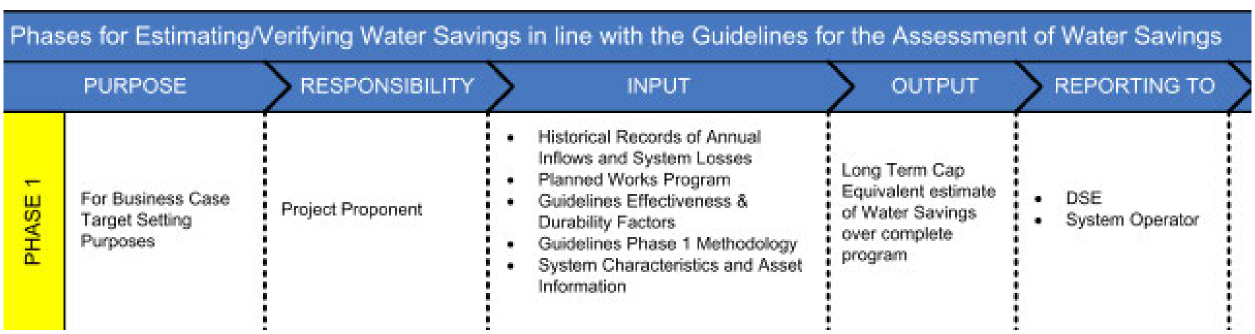

Figure 2-3 Summary of Requirements for Phase 1 of Assessing Water Savings in Irrigation Modernisation Programs (DSE 2012)

Water loss calculations are completed through four phases in total with the first phase being for estimates when seeking funding. The second and third phases provide information to assist with water allocations during implementation and the fourth is for long term water savings targets (DSE 2012).

The basic water savings calculation formula is seen below in Equation 2-4 and includes another formula for benchmarking long term water savings and losses.

Equation 2-4 Water savings Formula (DSE 2012)

$$\textbf{Water Savings} = \textbf{Losses}_{\text{pre-intervention}} - \textbf{Losses}_{\text{post interventions}}$$

In order to represent benchmark long term savings and losses, the following should be adopted:

$$\textbf{Water Savings} = \textbf{Baseline Year Losses}_{\text{pre-intervention}}\ \textbf{(LTCE)} - \textbf{Losses}_{\text{post interventions}}\ \textbf{(LTCE)}$$

2.4.1 Bulk Entitlements in a Stock and Domestic Pipeline.

2.4.1.1 Dead Storage

Typically, stock and domestic water is supplied in bulk as bulk entitlements at intervals throughout the year as determined by the system operator. It is critical to water saving that a plan is in place from year to year to deal with the dead storage from the previous year's supply delivery. Dead storage is the water held in the pipeline or channel that is not or cannot be used by the customers. Dead storage often needs to be emptied from the system by pumping at the end of the season to permit maintenance. Pipelines generally have a lower volume of dead storage than channel systems however, consideration must still be given to the handling of dead storage when designing a stock and domestic pipeline (DSE 2009).

2.4.1.2 Supply Regime

It is the responsibility of the system operator to deliver water to customers in the most efficient way possible. As such the frequency of supply should be considered. Estimates from northern Victoria suggest that about 4 to 6 percent of total water use is from stock and domestic supply. This means that I may be difficult to justify the costs of upgrades based on volumes delivered alone. It is estimated that as water scarcity increases, so will the demand on the stock and domestic supply and therefore, improvements to monitoring and data collection on the current and growing supplies is critical to ensure that impacts can be mitigated (DSE 2009).

2.4.2 Stock and Domestic Supply Estimation

The estimation of stock and domestic supply demand is dependent on many factors relating to the specific regional requirement of each irrigation area for example stock watering, garden use, storage, and household purposes.

In a report by Lowe et al. in (2009) outlines two approaches in estimating stock and domestic demand for a catchment. The first approach multiplies the expected water requirements of a household by the number of houses and stock within that are that require self-extracted water. The second looks at how many properties have stock and domestic licenses and combines that with information gathered via census for the farm use to estimate demand (Lowe et al. 2009). Figure 2-4 illustrates the framework of water use estimation.

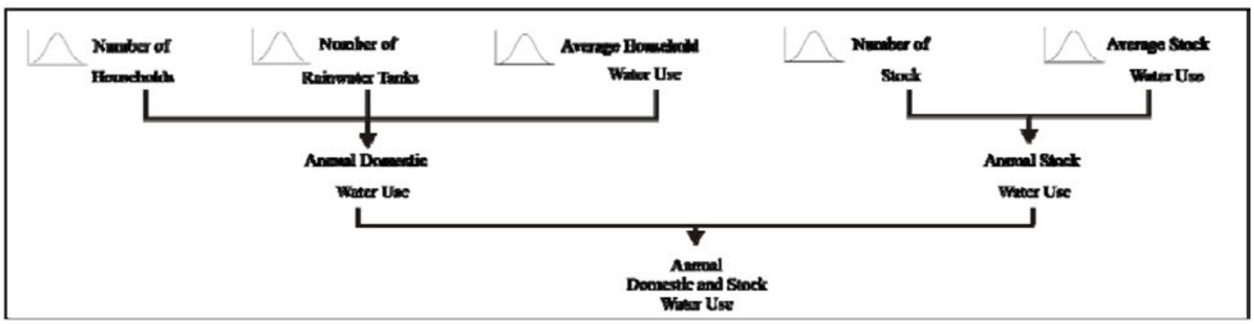

Figure 2-4 Framework of Annual Domestic and Stock water use estimation (Lowe et al. 2009)

The report outlines that the Victorian household water use is assumed to be the same as the household water coefficient and used the average household water use value of 195kL/household/year which with a standard deviation of 36kL/household/year or 534L/household/day. This value is obtained from the Water Services Association of Australia (WSAA) 2007a and 2007b which has since been revised (Lowe et al. 2009). Any attempt to rely on a similar value should utilise the latest versions of the WSAA code or similar document.

Larsen et al (2014) outlines a modelling approach to estimating both stock and domestic supply as well as curtilage, and dam losses which are important as most properties connected to a stock and domestic supply implement dam and/ or tank storage on their properties. The methodology outlined includes a top-down approach to assessing stock and domestic supply (Figure 2-5) however, the subject of this research paper will only review the top level being a high level catchment scale framework (Larsen et al. 2014).

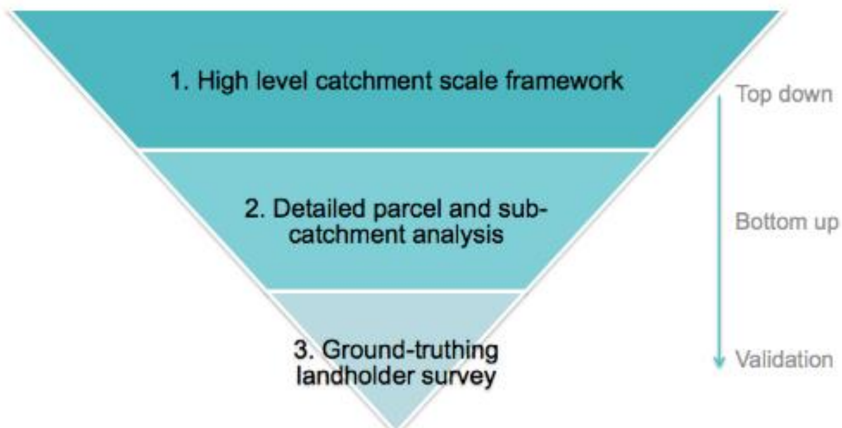

Figure 2-5 - Method for Assessing D&S water use (Larsen et al. 2014)

Further to the above, Larsen et al 2014 also provides a table of formulas (Table 2-1) for calculating stock and domestic supply which is considered suitable for the project.

Table 2-1 Formulas for calculating D&S demand (Larsen et al. 2014)

Demand	**Formula**	
Stock	*Stock demand (mega litres (ML)/year) = area of catchment (ha) × area of land grazed (%) × realistic carrying capacity (DSE/ha) × annual stock water consumption (kL/DSE/ha) ÷ 1,000*	*(1)*
Domestic	*Domestic demand (ML/year) = number of properties with a domestic D&S demand (>4,000 sq. m and not connected to town water supply) (No.) × proportion of properties with a house (%) × usage per house (ML/house/year)*	*(2)*
Curtilage	*Curtilage (ML/year) = number of properties with a domestic D&S demand (>0.4 hectares and not connected to town water supply) (No.) × proportion of properties with a house (%) × curtilage per house (ML/house/year)*	*(3)*
Dam losses	*Dam losses (ML/year) = volume of total number of catchment dams (ML) − volume of licensed and registered dams (ML) = volume of D&S dams (ML) ÷ average depth of dam × Class A pan evaporation × pan factor*	*(4)*

Considering most properties withing the SGIA are >0.4 hectares, curtilage will not be considered.

2.4.3 Costing

It is important at the concept phase to incorporate as much data as possible to inform the next step of the detailed feasibility. As such any dollar values found for similar projects within chapter four of this thesis should be subject to a conversion to today's dollar value. This will aid with the comparison of project values however should never be taken to be an accurate value due to the many variables which can affect cost such as availability of materials due to supply and geographic location, cost of labour subject to strength of the labour market, and inflation as a broad factor which contributes to increases in cost depending on the rate for given years. The Present value factor equation (Equation 2-5) is a useful tool in converting a project cost to a value which is more representative of present-day value.

Equation 2-5 Present Value Factor Equation (Aravinthan & Yoong 2020)

$$Present\ Value\ factor = \frac{1-(1+i)^{-n}}{i}$$

Given:

i = interest rate per period (year)

n = number of periods (years)

2.5 CONCLUSION

The literature review has identified reasons and methods for stock and domestic supply channel modernisation as well as methods for determining seepage and evaporation rates for the purposes of a concept level feasibility study. The methods outlined in the literature review will be used in the subsequent chapters of this thesis report to assist in the concept design for the proposed St George Irrigation Area stock and Domestic Supply upgrade. The purpose of which is provide Mallawa Irrigation with an answer on whether to proceed to a full-scale feasibility study. Further to this the methods outlined in this review can be used for similar feasibility studies by other irrigation areas.

The methodology used to estimate supply for the stock and domestic system will incorporate a combination of the WSAA code for domestic supply and the demand formulas within Larsen's et al 2014 report for stock demand and dam losses.

A review of research undertaken within the St George Irrigation Area around seepage found a that seepage estimates can be quite varied and depend on many factors including soil type, how compacted the channel is, the presence of any cracks, and seasonal water table levels. Three methods of estimates were found which were 8mm/day from the ponded seepage tests performed by Mclean (2015), an estimate of 0 to 50 mm/day for channels in the age range of 15 to 30 years, and a high-level estimate of 15% seepage loss per GL of water distributed from Beardmore Dam. All these methods of seepage estimation will be considered as part of the concept feasibility in Chapter 5.

Evaporation losses have also been found to be highly variable and difficult to estimate without strict localised testing or computer modelling. An equation (Equation 2-1) was extracted from the Department of Sustainability and Environments 'Technical Manual for the Quantification of Water Savings in Irrigation Water Distribution Systems' which will be used for concept feasibility in Chapter 5. This equation requires several assumptions as well as Daily Rainfall rate (mm/day), Pan evaporation Factor, Daily Evaporation rate (mm/day), and surface area of the system. All these factors will be obtained and used in Chapter 4 and 5.

Chapter 3 CASE STUDIES OF SIMILAR PROJECTS

3.1 INTRODUCTION

The following chapter examines six case studies of similar scope to the St George Irrigation District with the intent to determine levels of success for each project based on size, required infrastructure, cost, economic and socio-economic benefits to local region, and water savings. The results will be tabulated and compared with the goal being to determine a reasonable comparison of water savings and costs per ML of water savings which can be used as part of the concept feasibility in chapter 5.

3.2 CASE STUDIES

3.2.1 Murrumbidgee Irrigation Urban Channel Pipelines

3.2.1.1 Background and Scope

The Murrumbidgee Irrigation area (Figure 3-1) is located within the Murrumbidgee catchment (Figure 3-2). The irrigation area serves over 3,000 landholders and is owned by 2,300 shareholders. The irrigation area covers 378,911 hectares. Several towns are located within the irrigation area including Griffith, Leeton, Narrandera, Whitton, Yenda, and Hanwood. Notable water bodies include Barren Box Storage, Lake Wyanga, and Mirrool Creek. The irrigation area is bounded by the Murrumbidgee River along the southern border (Murrumbidgee Irrigation 2020).

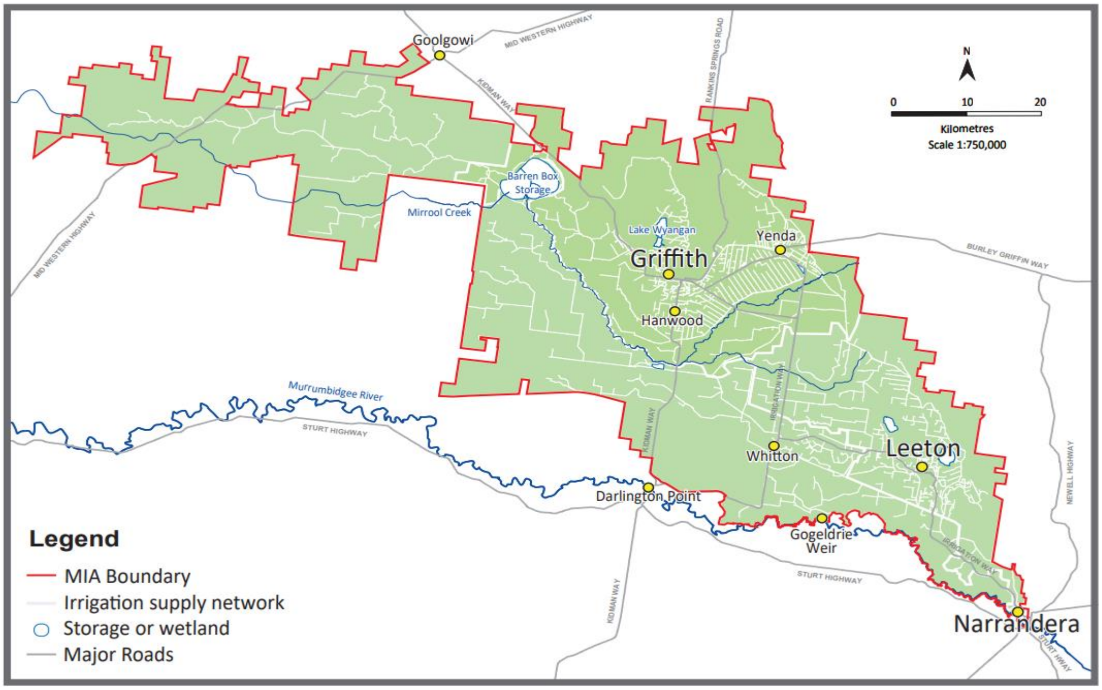

Figure 3-1 Murrumbidgee Irrigation Area (Murrumbidgee Irrigation 2020)

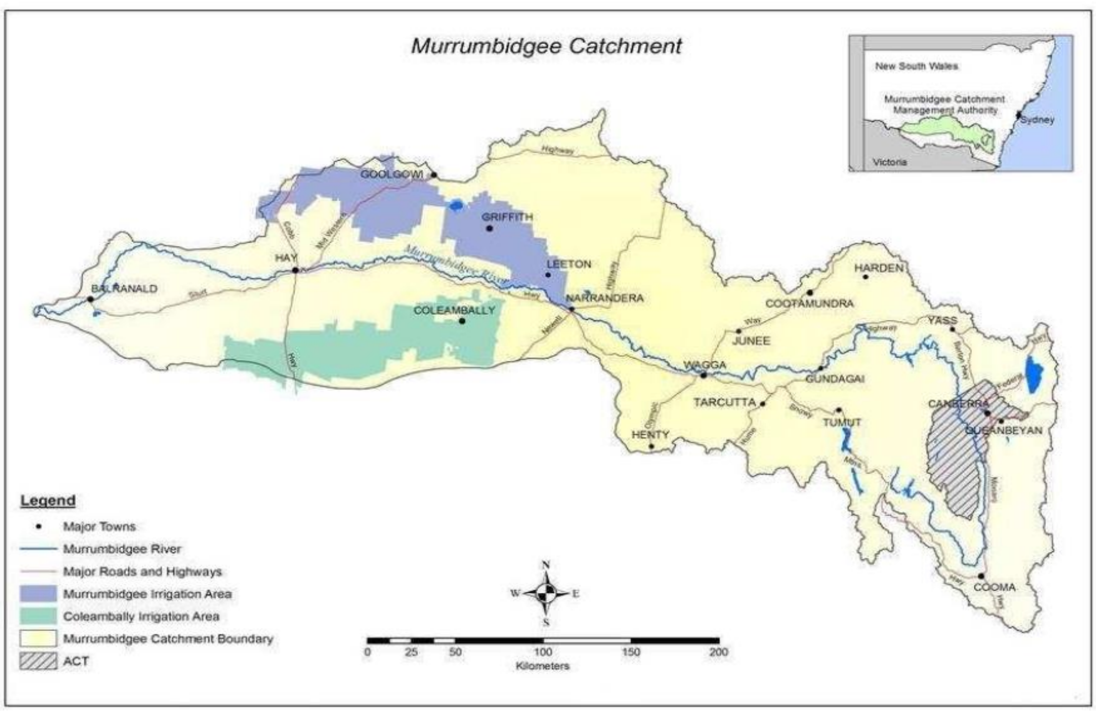

Figure 3-2 Murrumbidgee Catchment (Murrumbidgee Irrigation area in purple) - (Akbar et al. 2011)

The project proposed to replace 47.4km of deteriorating concrete and earthen lined channels as well as 1.4km of leaky pipeline with a proposed new 47.5km of HDPE pipeline and associated pump stations and other civil infrastructure. The pipeline was designed in a way that meant as much pipe as possible could be laid parallel to the existing channels so that the channels could continue to run throughout the construction process (Murrumbidgee Irrigation 2023).

3.2.1.2 Project Benefits:

The benefits to the community include increasing number of customers connected to the system, local employment opportunities through utilising 75% of the capital expenditure for local contractors, and improvements to socioeconomic statues and liveability of the rural towns. The project is estimated to be completed by May 2023 (Murrumbidgee Irrigation 2023).

The project will result in 2,612 ML of water savings with 2,407 ML being for water savings returning to the environment. Other benefits include increasing levels of customer service, improving socioeconomic outcomes for the community, reducing mosquito borne diseases, enhancing the safety of the community road and workplace safety, reduction in water losses through evaporation and seepage, increased regional productivity, reduced road maintenance, improved road drainage, and reducing operating costs (Murrumbidgee Irrigation 2023).

3.2.1.3 Cost and Cost Recovery

The project is estimated to cost $62,031,216 inclusive of the pipeline delivery, rationalisation of 33 escapes and infilling of existing channels. The cost is proposed to be funded by the New South Wales Off-Farm Efficiency Program (OFEP) with the costs of the planning works being funded by Murrumbidgee Irrigation (Murrumbidgee Irrigation 2023).

With the implementation of an automated system, Murrumbidgee Irrigation was able to reduce the forward capital needs of the business as well as operating costs and increase efficiency and service delivery. Murrumbidgee Irrigation recovers cost by utilising an asset reserve which it uses to fund further water investment and manages an ongoing asset renewal program. Due to its proficiency in managing operational and maintenance costs as well as a good handle on future replacement needs, approximately 90% of the irrigation area is automated and manages active surge reservoirs within the system (Murrumbidgee Irrigation 2023).

3.2.1.4 Water Savings

Murrumbidgee Irrigation generates savings by minimising losses via seepage and evaporation, conveyance, pipeline control and meter accuracy through it works in modernising channels. The reduction in losses is also achieved through upgrading unmetered properties to metered and upgrading meters to improve accuracy (Murrumbidgee Irrigation 2023).

The upgrade is expected to generate 2,407 ML of water, 90% of which is to be transferred to the environment in exchange for the $62 million funding. The remaining 10% will be retained by Murrumbidgee Irrigation. Ultimately this will mean that the irrigation area will be in a net positive position and will have achieved improved socio-economic outcomes for the community (Murrumbidgee Irrigation 2023). Table 3-1 shows the water saving (ML) and water returned the environment (ML) for each asset type.

Table 3-1 Water Savings by Asset Type (Murrumbidgee Irrigation 2023)

Asset Type	Water Savings ML	ML Returned
Pipelines	1,685	1,516
Escapes	990	891
Total	**2,612**	**2,407**

3.2.2 Narromine Private Irrigation Modernisation project

3.2.2.1 Background and Scope

The Narromine Irrigation Scheme is located just outside of Narromine which is 447 kilometres west of Sydney (Figure 3-3). The scheme utilises 350 kilometres of compacted earth lined channels to service the approximately 120,000 hectares within the scheme. The predominant crops grown in the area are cotton, wheat and canola (McBurnie 2017).

The projects aimed to deliver outcomes both on and off farm however, as of the date of McBurnie's report (2017), only the on-farm outcomes have been delivered. The original proposal called for significant off-farm channel modernisation however, the on-farm outcomes are summarised below in Table 3-2 (McBurnie 2017).

Table 3-2 Summary of key on-farm works (McBurnie 2017)

Construction Parameter	Outcome
Length of refurbished channel	89 km
Area of lined channel (EPDM)	650,000m^2
Length of lined channel (EPDM)	32 km
Number of farm outlets	60
Number of channel regulators	42
Number of channel structures	159
Length of stock and domestic pipeline	190 km
Number of stock and domestic outlets	60
Length of channel fencing	65 km
Pumping station upgrade	
Automation and telemetry network	Across the supply network

Other works include, installation of a separate 187 km of stock and domestic pipeline including one pump station (Mitchell Water 2023), lining porous sections of channel, and replacing old dethridge wheels with modern outlets which utilise telemetry automated flow control as well as accurate metering (McBurnie 2017).

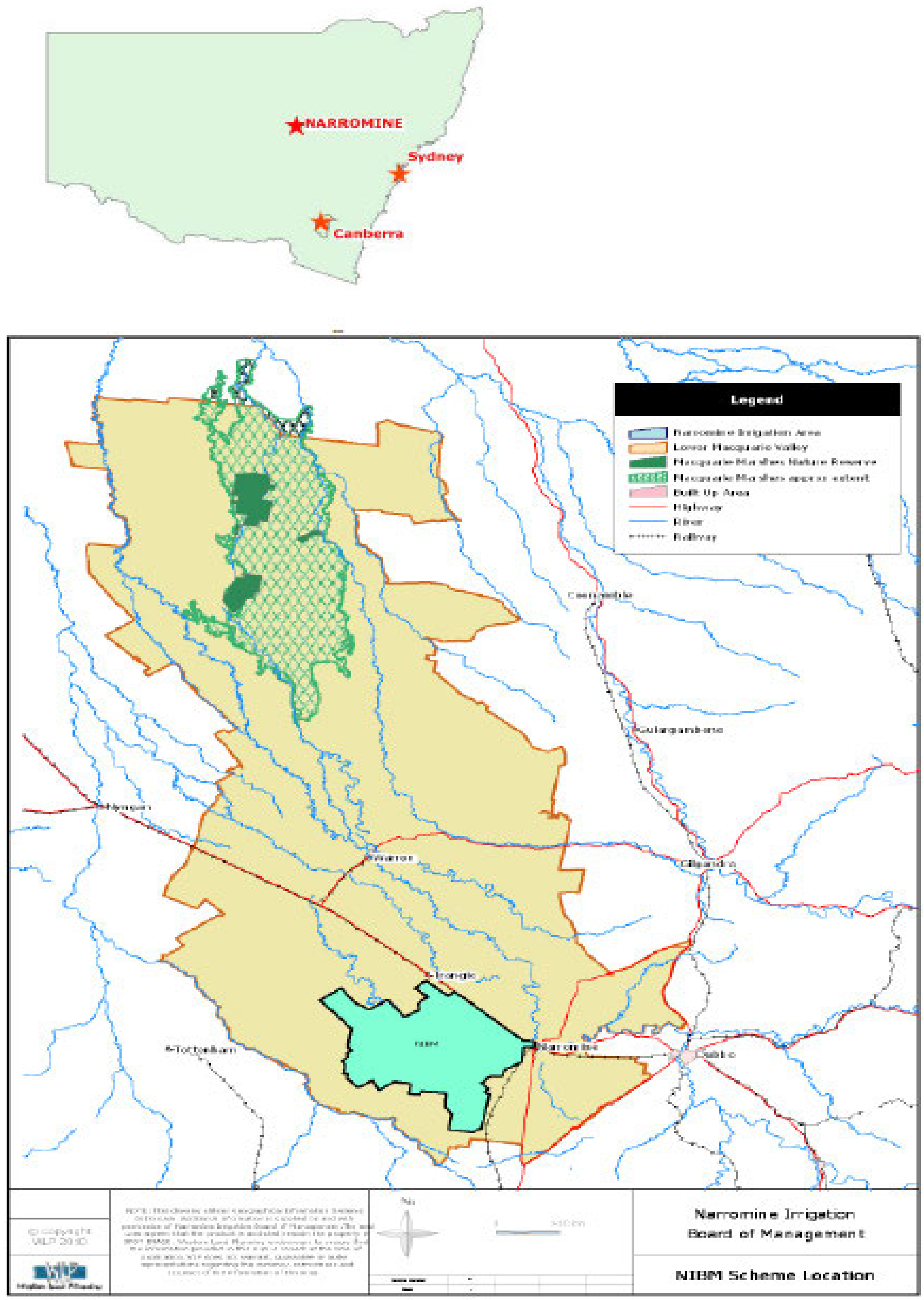

Figure 3-3 Narromine Irrigation Scheme (green) (Western Land Planning 2010)

3.2.2.2 Project Benefits

The project achieved an improved water delivery efficiency from 69 to 92% and has increased the volume of water available by improving water savings and control of water for crops, stock, and domestic purposes (McBurnie 2017). The project has reduced the risks associated with scarce water availability and has also had a positive impact to profitability and sustainability for its members.

Some members experience significant improvements to their quality of life with the availability of stock and domestic water now available via underground pipeline. Through the new pipeline they now have access to quality domestic water on demand which has improved the lives of families with pools, lawns, and gardens (McBurnie 2017).

3.2.2.3 3.1.2.3 Cost

The estimated cost for 187 km of PVC pipeline ranging from 63mm to 280mm OD (Figure 3-4) and one pump station is $8.5 million (Mitchell Water 2023).

3.1.2.3 Water savings

The water savings for the Narromine Irrigation Scheme according to Mcburnie (2017) are:

- 7,332 ML general security entitlements returned for network upgrades.
- 16,446 ML of water access entitlements from members who have permanently retired their rights to irrigate with no access to the water delivery network.
- 6,919 ML additional water access entitlements transferred to the Commonwealth as part of the IFP program in exchange for infrastructure upgrades on their farms.

Figure 3-4 Construction of Stock and Domestic Pipeline by Mitchell Water (McBurnie 2017)

3.2.3 Nap Nap Station Water Efficiency Project

3.2.3.1 Background and Scope

Located 70km west of Hay and about 170km East of Mildura, Nap Nap station is situated on the lower Bidgee floodplain and is approximately 30,000 ha in area and is located on the Murrumbidgee River system (Figure 3-5). The station, which predominantly is a sheep and cattle station, currently supplies 20 stock watering points via 45km of open channel network. Typically, the station supplies three to four water runs per year and use irrigation pumps for five days per run to deliver water to ground tanks. Each run delivers 25ML per day and in better seasons with higher water availability, an extra run may be delivered if required. This equates to approximately 500ML of water per year for Stock and Domestic purposes (DPIE 2022).

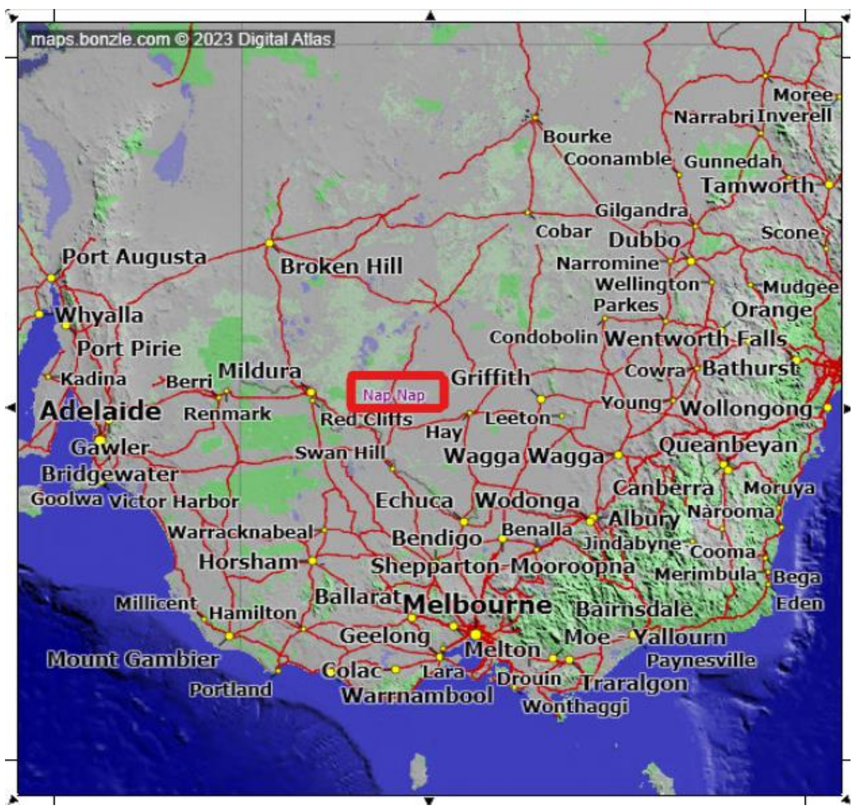

Figure 3-5 Nap Nap Locality Map (Bonzle.com.au 2023)

The project proposal as shown in Figure 3-6, consists of the installation of 40km of open channel replacement with pipeline and an additional 40km of pipe plus two stock and domestic pump stations at different sites allows the stock and domestic supply to deliver water separately from the irrigation system by delivering to 31 proposed new tanks and 37 new troughs (DPIE 2022).

3.2.3.2 Project Benefits

The project is expected to provide economic benefits to the area by allowing local purchasing and will increase employment within the regional community. It is expected that by utilising local contractors, engineers, and suppliers that the project will also provide economic stimulus to the regional economy (DPIE 2022).

Further to the anticipated economic benefits the project will also ensure that Nap Nap water intake and usage will be more efficient, and those efficiencies will improve the river operation and will positively impact downstream users (DPIE 2022)

3.2.3.3 Cost and Cost Recovery

The estimated cost for the project is $1.97 million, $1.86 million of which is being sought initially as funding from the Australian government (DPIE 2022). In a report by the Department of Climate Energy, the Environment, and Water (DCCEEW) it is outlined that a funding amount of $2.39 million was to be granted (DCCEEW 2022). The anticipated cost recovery will be through water entitlements delivered to the Commonwealth Environment Water Office (CEWO) (DPIE 2022).

3.2.3.4 Water savings

The works will deliver 300ML of water savings per year with 50 percent being transferred back to the Commonwealth at a conversion of 147ML of the total to Murrumbidgee High Security water entitlements. This will provide a long term average annual yield (LTAAY) of 143.62 ML under the NSW Water Management Act 2000 section 71(DCCEEW 2022).

The remaining 150ML of 300ML of the water savings as non-tradeable stock and domestic license which will mean there will be no impact to the market or consumptive pool (DCCEEW 2022).

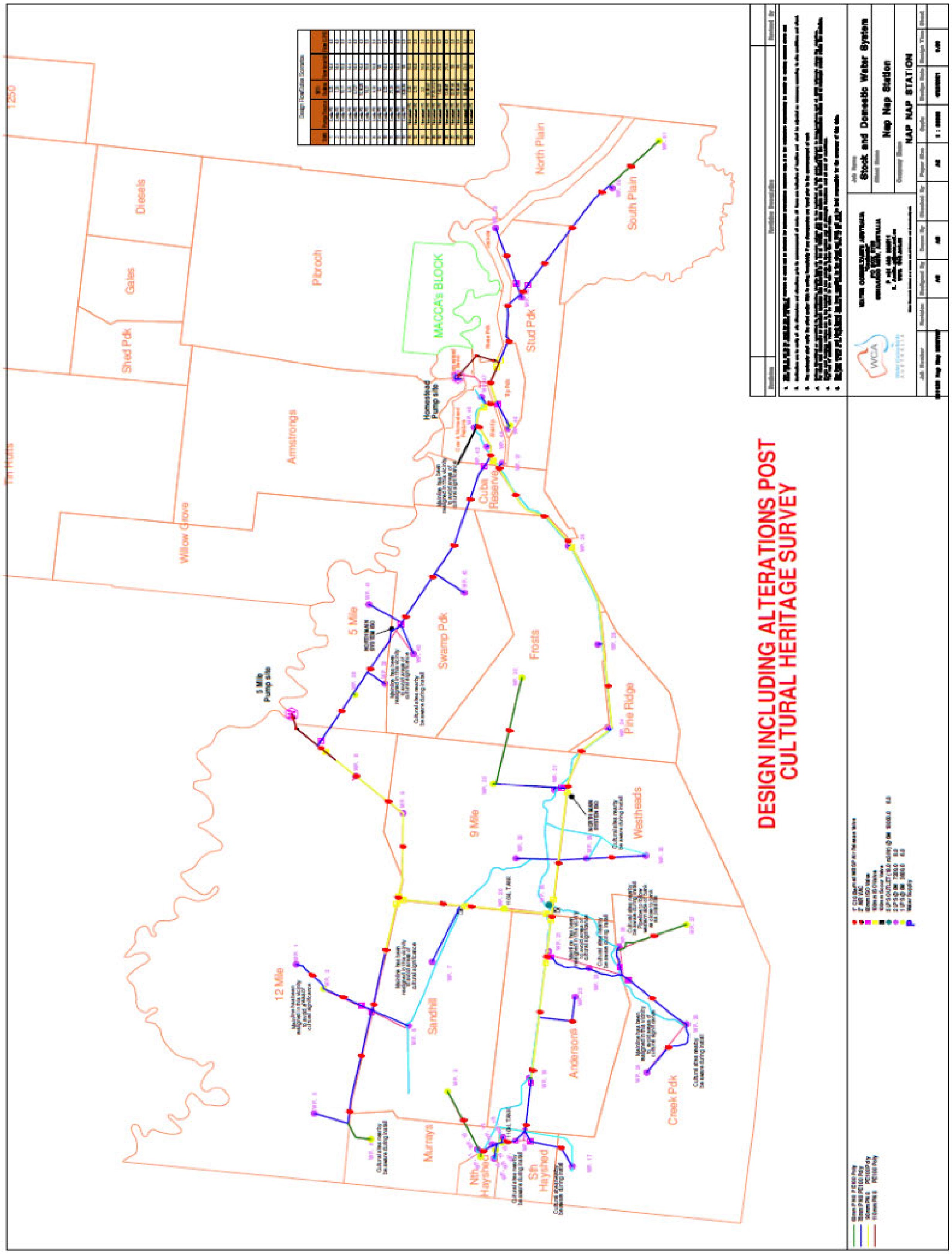

Figure 3-6 Proposed Nap Nap Station S&D supply (DPIE 2022)

3.3 EXAMPLES OF OTHER SIMILAR WORKS

3.3.1 Trangie-Nevertire Irrigation Scheme (TNIS)

The Trangie-Nevertire scheme is located approximately 70km north-west of Dubbo (Figure 3-7) and serves 33 farms spread over 100,000 ha via 180km of main channel and 35.5km of subsidiary delivery channels. Approximately $35 million of annual additional production is attributed to the irrigation within the scheme (Trangie Nevertire 2023a).

Figure 3-7 Trangie Nevertire Irrigation Scheme Locality Map (Google Earth)

The project consisted of rebuilding and lining 108km of the main channel with EPDM rubber to drastically reduce seepage losses and rebuilding the remaining 138km of earthen channel. This led to a reduction of the original earthen channel length of 40%. Other works included 230km of stock and domestic pipeline from the river to reduce reliance on the channel system and modernising on-farm irrigation to improve efficiencies (Trangie Nevertire 2023b).

The Trangie Nevertire cooperative was granted $115 million in 2010 in exchange for 30,000ML of water savings transferred to the Australian Government for environmental use in the Macquarie Marshes (Trangie Nevertire 2023b).

The resultant water savings were due to decreased losses from 25 per cent to 7 percent through irrigation modernisation. The stock and domestic pipeline yielded a 90% water saving and provides clean, pressurised water to all members year round. The on-farm improvements led to a water efficiency improvement of about 30% with crop yields of cotton have increased by 1-2 bales/ha (Trangie Nevertire 2023b).

The total project cost was $115 million from the PIIOP scheme plus $2 million interest plus another $2 million contribution by members (Trangie Nevertire 2023b).

3.3.2 Hay Private Irrigation District

The Hay Private Irrigation District pumps water from its location on the Murrumbidgee River in NSW, from a pump station on the Midwestern Highway to the Hay Township (Figure 3-8). The district is responsible for 2,460 hectares of irrigated land with approximately half of that being for flood irrigation (Dalton 2018). Being the oldest irrigation district in New South Wales the district is responsible for 91 irrigation customers and 90 stock and domestic customers (Dalton 2018).

Figure 3-8 Hay Private Irrigation District Locality Map (Google Earth)

The irrigation district consists of 20.5km of earthen open channels supplying water to 117 irrigation outlets including 46 piped and 71 dethridge wheels (Dalton 2018).

The modernisation plan was submitted in 2015 and outlined 4 main objectives:

1. Plan and manage a transition to a future with a reduced irrigation water availability.
2. Downsize the scale and increase efficiency of water delivery. Reduction of operation and maintenance costs
3. Reduce water and delivery entitlements in controlled manner to transition to a reduced capacity piped system.
4. Optimise farm production by improving level of service to customers and support establishment of high value crops.

This would be achieved through the replacement of all open channels with a gravity pipeline (Dalton 2018)

The project was estimated to cost approximately $10.55 million with $10.2 million being from Commonwealth funding and the remaining $350,000 contributed by the irrigation district.

A cost benefit analysis was performed which determined that for every dollar spent on the project $1.06 of benefits would be made. Several risks were identified with the main risks being cost escalation due to budget blowouts, delays in approvals, delays due to wet weather and availability of materials (Dalton 2018).

The project delivered water savings in the form of water entitlements to the value of 1968 ML conveyance entitlements, 1760 ML general security entitlements, and 1166 ML of supplementary entitlements (Dalton 2018).

3.3.3 Tenandra Scheme Modernisation

The Tenandra Scheme has been operating in the Macquarie Valley in NSW (Figure 3-9) since 1973 where it pumped water through 85km of main channels and 15km of branch channels (Sustainable Soils Management 2013). Prior to modernisation the scheme delivered water to 8000 hectares with 22 properties via entitlements held to deliver 28,056 ML of general security water and 1,263 ML of supplementary supply (Sustainable Soils Management 2013). With an average loss of 5,400 ML per year the original scheme lost approximately 20.5% of water to evaporation, seepage and other minor losses (Sustainable Soils Management 2013).

Figure 3-9 Tenandra Locality Map (Google Earth)

In 2009 an application for funding was submitted with a plan focused on improving water delivery efficiency, water delivery infrastructure, storage and utilisation, and improved long-term viability (Sustainable Soils Management 2013). Improved long-term was to be achieved using a 'loss-account' which was to offset transmission losses for the continuing customers (Sustainable Soils Management 2013). The Australian Government approved funding for the project in 2010 and granted almost $37.5 million which saw significant returns to the Australian Government, Scheme members and the local Macquarie Valley area (Sustainable Soils Management 2013).

The planned works included a joint water sharing scheme channel for the first 12 km of the top scheme, additional pumps at the existing Greenhide pump station, 9 km of new channel, lining 22.4km of existing channel, decommissioning 32 km of main and 8 km of branch channels. Table 3-3 below outlines the forecast water savings:

Table 3-3 Forecast Water Savings (Sustainable Soils Management 2013)

Water Saving (ML)	**Area of efficiency**
2,793	Improved water delivery efficiency via from increased efficiency of whole scheme
859	Increased on-farm efficiency of 3 individual members
10,242	Available from rationalisation of water entitlements on the Scheme

It was estimated that the proposed works will result in 13,894 ML of water savings however, based on the final report is reported to have delivered a return of 12,504 ML (Sustainable Soils Management 2013).

3.5 Conclusion

The review of recent case studies from the past decade and a half of channel and stock and domestic supply modernisation has been summarised below in Table 3-4. Except for Murrumbidgee irrigation, there is a possible correlation between water savings from and project and total cost of a project, noting that in most cases most of the funding for these projects came from various infrastructure spending schemes.

The cost of a project depends on numerous factors including construction costs, costs of materials and labour, how remote the area is, and the current economic climate of Australia and the desire for the government to provide funding at the time of works. In early to mid-2010's the cost of a project was largely tied to the volume of water savings by Megalitre but then in 2022-2023 the costs increased quite substantially which is likely due to the current rates of inflation, high labour costs, and supply chain issues of the past two years. It was found that in all cases the large costs were predominately covered by government infrastructure spending with only a small fraction of the costs being borne by the irrigation areas.

It was found that the costs are generally tied to water savings which are typically generated by the implementation of at least two modernisation upgrades of which stock and domestic supply upgrade can be one, and ultimately the water savings are given back to the government in exchange for the large cost of infrastructure. This also means that it is likely unfeasible for an irrigation area to upgrade their supply and domestic system without government spending, and this would that the costs would be too high to be passed onto the customers without a significant cost recovery timeframe.

The closest case study in size and scope to what is expected for the St George Irrigation area is the Murrumbidgee Irrigation District which has also seen the most skewed amount of funding per ML of water saving at $25,758. This could be mostly due to the high labour and materials cost combined with the materials shortages experienced during 2022 and 2023 and as such may not be a true representation of cost or government funding in the future.

Table 3-4 below provides a summary of the reviewed case studies with the aim to provide a better understanding of potential costs, government funding, and water savings. No two irrigation districts have the same needs or require the same combination of stock and domestic upgrade or modernisation as another. Because of this it is difficult to pinpoint with any certainty what aspects could be translatable to a proposal withing the St George Irrigation District. The key point to be made from the case studies is what can be achieved and what elements were successful within each irrigation district. An attempt has been made to convert costs of each project to a Net Present Value and obtain a crude value of potential funding by potential future government infrastructure spending projects however, this value should not be considered in any detailed feasibility studies. It is recommended that any values used in detailed feasibility studies should be provided directly by the government spending programs as well as any values obtained within this project for water saving and project costs.

Table 3-4 Summary of Case Studies

Location	**Summary of works**	**Water Savings (ML)**	**year**	**Cost ($, million)**	**Cost per ML of water savings**	**Present Value Factor**	**Net Present Value per ML ($)**
Murrumbidgee Irrigation	47.5km HDPE pipeline 40km urban channel rationalisation	2,407	2023	$62	$25,758	0	**$25,758**
Narromine Private Irrigation Area	187km stock and domestic pipeline (63mm to 280mm OD) 1 pump station	32,697	2014	$8.5	$260	7.78	**$2,023**

Nap Nap Station	45km open channel replacement 40km new pipeline 2 pump stations 31 tanks 37 troughs	300	2021	1.97	$6,567	1.91	**$12,543**
Trangie-Nevertire Irrigation Scheme	108km channel refurbishment EPDM rubber 138km channel rebuild. 230km stock and domestic pipeline	30,000	2016	$119	$3,967	6.23	**$24,714**
Hay Private Irrigation District	20.5km channel replacement with gravity pipeline	4,894	2016	$10.55	$2,156	6.23	**$13,432**
Tenandra Scheme	9 km new channel 22.4km channel relining 40km channel decommission	12,504	2010	$37.5	$2,999	10.63	**$49,873**
Total Average cost Per ML							**$21,390**

Chapter 4 PRELIMINARY DESIGN

4.1 DESIGN METHODOLOGY AND KEY OBJECTIVES

Design methodology will focus on a concept approach using data and channel maps provided by Mallawa Irrigation combined with assumptions for demand and potential functionality. This will be used in demand and flow calculations using basic Microsoft Excel spreadsheets to determine approximate pipe sizes and pump station requirements using the least total cost method to determine the optimum pipe sizes required for distribution.

Included in this Analysis:

- Determination of Design Parameters
  - Demand Analysis
  - Equivalent Persons (EP)
  - Peaking Factors
- Assessment of Current Flows and losses (based on supplied information)
- Determination of Hydraulic design principles
- High level desktop trunk pipeline design
- High level desktop distribution system design
- Pump Station Design (preliminary, power requirements for costing– if required)
- Power requirements
- Feasibility
  - Cost benefit analysis
  - Cost recovery

Excluded:

- Environmental Impacts
- Geotechnical issues
- Construction planning and staging of works.
- Design of intake structures

4.2 DESIGN PARAMETERS

Limited information for demand use is readily available for stock and domestic supply and the local shire council of Balonne provides no information on water supply design parameters there for the Water Services Association (WSA) water supply code - Southeast Queensland Services Provider Edition (SEQ code, 2019) and Department of Energy and Water Supply (DEWS), Planning Guidelines for Water Supply and Sewerage (2010) have been used and demand use was derived from the free to access SEQ Design and Construct Code. Where possible, data for rural areas provided for in the document such as Scenic Rim, Lockyer Valley, and Somerset will be used to inform an estimate of the required equivalent persons (EP), Average Day Demand per EP, pressures, and Peaking Factors.

4.2.1 St George Irrigation Scheme – Channel Capacity Diagram

The Channel Capacity diagram depicts channel layout and flows distributed via each channel and direction of flow. Refer Figure 4-1 below.

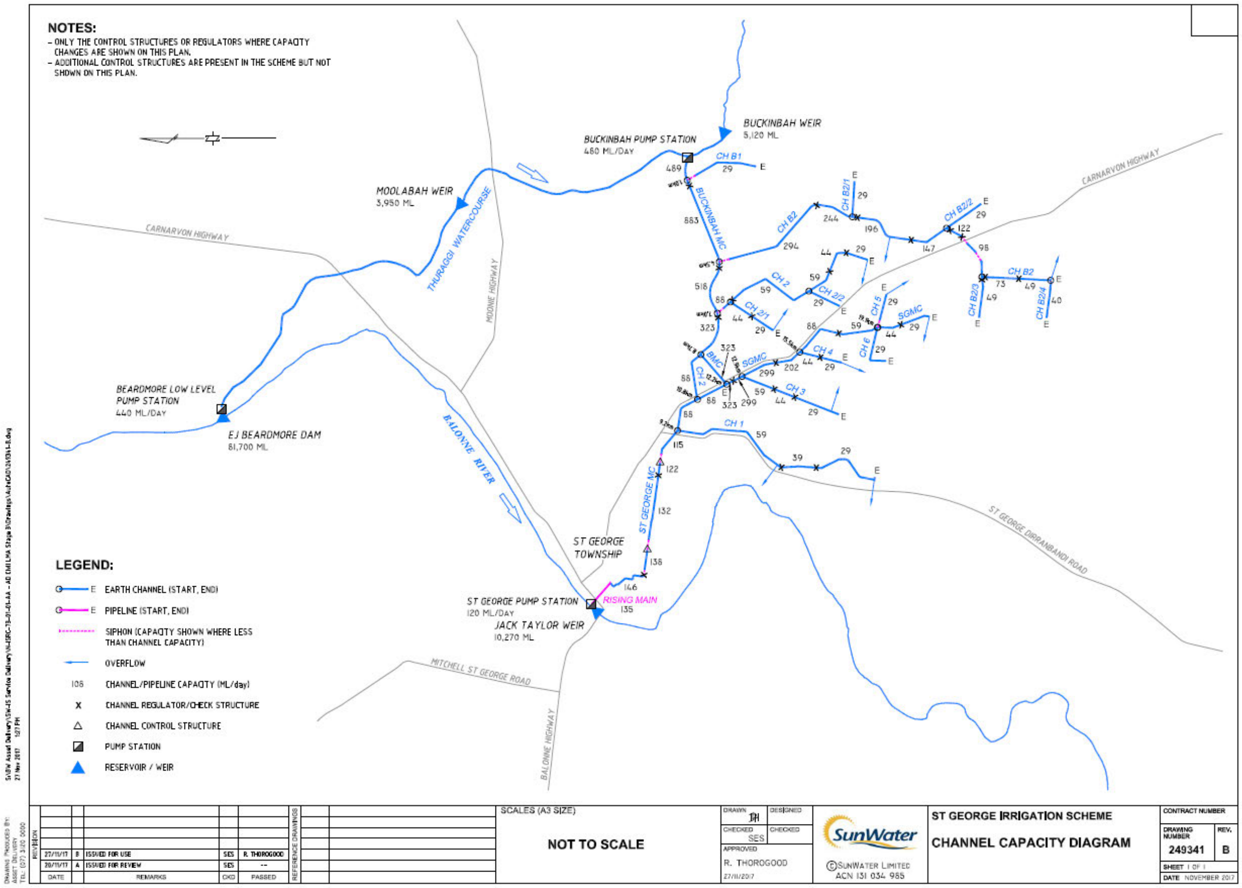

Figure 4-1 - SGIA Channel Capacity Diagram (provided by Mallawa Irrigation)

4.2.2 Current Stock and Domestic Supply use information

The following data was provided by Mallawa Irrigation:

Number of customers connected to the stock and domestic is approximately 80 (unconfirmed).

Preliminary estimate of pipeline is 40 km however further analysis could be extended to include a reticulation network to supply directly to each property. Due to the limited nature of the existing customer offtake points and other limitations outlined in section 1.4.4 the design is purely a high-level concept level for preliminary costings.

4.2.3 Equivalent Person (EP)

Tables A4.3-A4.5 of the Water Network Design Criteria from SEQ Design and Construct Code (WSAA 2020) for Lockyer Valley, Somerset, and Scenic rim, all proved different values for EP per attached dwelling. The values are summarised below in table 4-1 and the largest of all three will used to determine the EP for the proposed stock and domestic supply.

Table 4-1 Table of equivalent persons per attached dwelling for each rural region (WSAA 2020)

Region	EP per detached dwelling
Lockyer Valley	2.57
Scenic Rim	2.44
Somerset	2.37

An average EP of 2.57 will be adopted for the 80 offtakes.

$$EP = 2.57 * 80 = 205.6$$

Further to this an estimated 20% extra will be allowed for possible future demand or additional offtakes therefore:

$$EP = 205.6 * 1.2 = 246.72 \approx 247$$

4.2.4 Domestic Demand Average Day (AD)

From table 4.1 of the Water Network Design Criteria from SEQ Design and Construct Code (WSAA 2020), most utility providers allow for 230 L/EP/day. This will be the applied average day demand.

Non-Revenue water, that is water lost through the system will be accounted for at 30 L/EP/day in areas where PE pipelines are typically used. For Rural and small communities, an allowance for 7.5L/s for 2 hours is typically required for firefighting purposes however since delivery will be made at intervals throughout the year this will be omitted.

Therefore, total AD will be:

$$AD = 230 + 30 = 260\frac{L}{EP}/day$$

4.2.5 Stock Demand

As outlined in the design methodology section the following formula will be used to calculate annual stock water demand using assumed values for dry sheep equivalent (DSE).

Stock Demand ML/year = area of catchment (ha) * area of grazed land (%) * realistic carrying capacity (DSE/ha) * annual stock water consumption (kL/DSE/ha) /1000 (Larsen et al. 2014)

Area of catchment = 10,000 ha

Area of grazed land = assumed value of 5% to allow for potential and current stock watering requirements. = 10,000*0.05 = 500ha

Realistic carrying capacity and DSE rates depend on seasonal variations such as rainfall and grazing pasture quality. For a similar area with 500 mm annual rainfall and high-quality pasture quality a value of 10 DSE can be estimated, refer to figure 4-2. Climate data obtained from the Bureau of Meteorology for St George Post Office, indicate that the annual rainfall for St George was approximately 549mm between 1981-1997. This data is old however it will be used as a reasonable approximate in this case for estimating DSE. Based on the previous, a DSE value of 10 DSE per hectare will be used.

Annual rainfall (mm)	DSE per hectare	
	Poor pasture	High quality pasture
500	4.0	8.0
700	6.0	14.0
900	8.0	20.0

Figure 4-2 Estimated Stocking rates in the Mt Lofty Ranges region (AMLRNRM)

Seasonal variation can affect the demand with an average demand in the order of 75 L/head/day in summer months and 30 L/head/day in areas where the temperature is approximately 20°C (Davis & Watts 2016). The highest of these values will be used for the calculations. Figure 4-3 shows the standard Annual Daily Livestock water requirements.

Type of stock	Annual requirements (litres)	Daily requirements (litres)
Ewes on dry feed	3,600	10
Ewes on irrigation	2,700	7
Lambs on dry feed	900	2
Lambs on irrigation	450	1
Dairy cows in milk	22,500	62
Dairy cows dry	17,000	47
Beef cattle	17,000	47
Calves	8,200	22
Horses - working	20,000	55
Horses - grazing	13,500	37
Alpacas - grazing	2,000	5

Figure 4-3 Annual Daily livestock water requirements (AMLRNRM)

Stock Demand ML/year = 10,000*0.05*10*75/1,000 = 0.375 ML/year

4.2.6 Peaking Factors

From Table 5.4 – Indicative ranges of overall peaking factors within DEWS Planning Guidelines for Water Supply and Sewerage (Figure 4-4) indicate that the following Peaking Factors for <5,000 Equivalent Persons may be adopted. The smaller of the ranges will be adopted as it is within a rural area.

Mean Day Maximum Month MDMM:AD = 1.5

Peak Day Factor PD: AD = 1.9

Peak Hour Factor PH: PD = 3.6

Equivalent Persons	MDMM:AD	Peak Day Factor PD:AD	Peak Hour Factor PH:AD
> 5,000	1.4 – 1.5	1.5 – 2.0	3.6 – 4.0
< 5,000	1.5 – 1.7	1.9 – 2.3	3.6 – 4.5
Arid areas (where internal water use is less than 30% of total water consumption)	1.5 – 1.7	1.7 – 2.0	3.6 – 5.0

Figure 4-4 Indicative Range of overall peaking factors (DEWS 2010)

4.2.7 Pressure

It is anticipated that each property will be required to construct on-site domestic private boosters and therefore the pressure at the main adjoining the property may be 12m (WSAA 2020) This also satisfies the minimum residual mains pressures for emergency fire operating conditions. The Target Maximum Pressure adopted will be 55m.

4.2.8 Other pipeline parameters

Pipeline Capacity requirements (WSAA 2020):

Trunk Gravity System: MDMM in 24 hours

Reticulation Mains: Maintain pressure for PH and fire flow performance.

Pump System: MDMM in 20 hours

Pipe Friction Losses Hazen Williams Friction Factors (WSAA 2020):

Diameter < 150mm: C = 100

Diameter >150 to 300mm: C = 110

Maximum Allowable Head loss (PH, m/km) (WSAA 2020)

DN <= 150mm = 5m/km

DN>= 200mm = 3m/km

For this report an absolute allowable maximum of 5m/km will be used.

Maximum allowable service pressure (m)

An upper limit of 350m head of operating pressure will be allowed for in accordance with the WSAA SEQ code clause 2.5.3.2. (WSAA 2020)

Maximum Allowable Velocity (WSAA 2020).

Vmax = 2.5m/s

4.3 Design Equations

The design will be conceptual in nature and as such it would not be necessary at the feasibility stage to determine pipe sizes and flow with a high degree of detail. As such the Hazen Williams Equation will be used as an empirical method to determine pipe sizes. For detailed design it is recommended to use the Darcy equation (2) for frictional head loss. Other useful formulas to be used are identified below.

Equation 4-1- Hazen-Williams (Chadwick et al. 2013)

$$hf = \frac{6.87L}{D^{1.165}}\left(\frac{V}{C}\right)^{1.85}$$

Where:

hf = head loss (m)

L = length of pipe in metres

Q = volumetric flow, m3/s

C = pipe roughness coefficient (given as 100 for pipes < 100mm diameter, and 110 for pipes 150-300mm)

D = internal diameter of pipe

V = Velocity of pipe (see equation 3 for derivation of V from Q and A)

4.3.1 Pipe flow Design Equations

Equation 4-2- Darcy, Frictional Head loss (hf)

$$hf = f * \frac{L * v2}{D\ 2g}$$

(Aravinthan & Yoong 2020):

Equation 4-3- Pipe Flow

$$Q = V * A$$

(Aravinthan & Yoong 2020):

Equation 4-4- Reynold's Number

$$NR = \frac{VD}{v}$$

(Aravinthan & Yoong 2020):

Equation 4-5- Friction Coefficient f

$$f = \frac{1}{\sqrt{f}} = -2log10\left[\frac{ks}{3.7D} + \frac{5.1286}{NR^{0.89}}\right]$$

(Aravinthan & Yoong 2020):

Equation 4-6- Power Requirements for Pump P

$$P\ (kW) = \rho\ g\ Qh$$

Where:

P = power in kilo Watts

ρ = Density of water

g = gravitational acceleration m/s

Q = flow rate m3/s

h = head on pumps (m)

(Aravinthan & Yoong 2020):

The following equation 4-7 was adopted based on (Tonkin Science Engineering 2011) report titled Pipeline to the Sea; Feasibility Study - Phase 1.

Equation 4-7- Pipeline Cost Equation

$$Cpipeline = F1 * F2 * M * L$$

Where:

L = length of pipe (m)

M = material cost of pipeline ($/m)

F1 = allowance for pipe fittings

F2 = factor to account for other costs

(Tonkin Science Engineering 2011)

Other assumptions made based on the Tonkin report include: 10% of total pipeline cost to allow for fittings, and total capital cost of the pipeline are estimated to be 3 times the material cost of the pipeline (Tonkin Science Engineering 2011).

4.4 PIPELINE DESIGN

4.4.1 Network Requirements and Location

The pipeline location is critical and the more accurate it is defined in the concept design, the more accurate the cost benefit analysis will be.

The main consideration will be the transfer main which will be conveying the flows directly from the designated storage reservoir to the distribution mains. Outside of the scope of this report will be the reticulation mains which will be the mains servicing each property if they cannot be serviced directly from the distribution mains. Figure 4-5 illustrates a typical water supply system from the WSAA Water Supply Code.

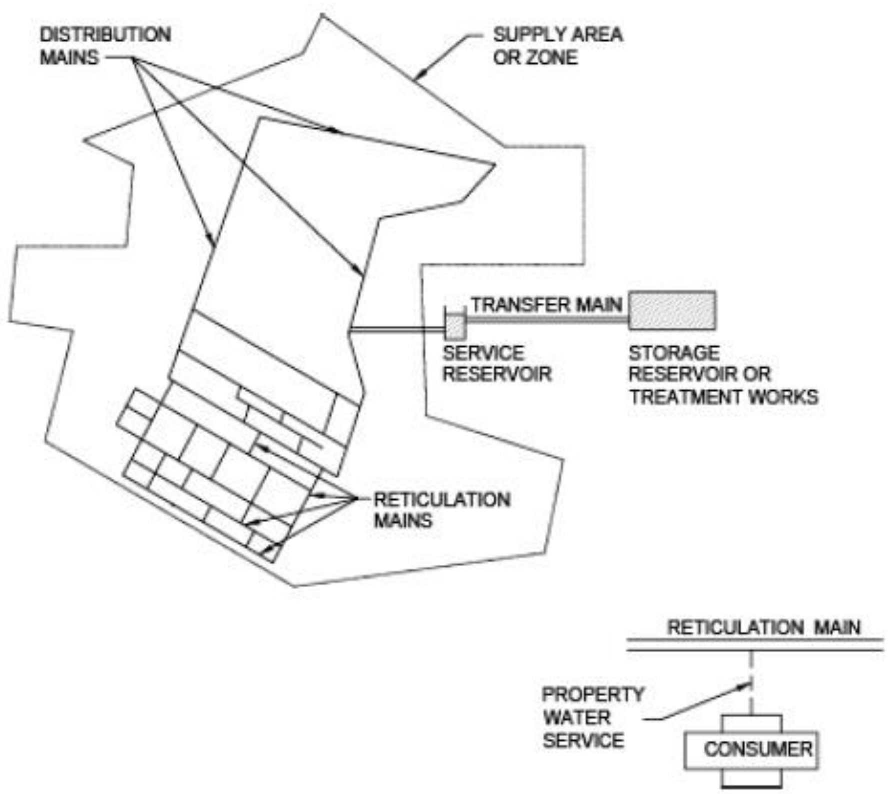

Figure 4-5 - Typical water supply system (WSAA 2019)

4.4.2 Pipeline location and Topography

An initial investigation identified a possible pipeline could be aligned from the Beardmore dam along the Balonne River and through St Geore to the St George main channel however this would require new infrastructure to be constructed through St George Town and as such is not deemed viable. Two other options remain which are outlined below.

4.4.2.1 Option 1:

Trunk transfer main from Beardmore Dam to storage reservoir pumped via transfer main to distribution main.

This option is not preferred since it requires an additional 26km of pipeline however, it satisfies the SGIA's desire to minimise rewetting of channels during time of low water levels to service a minority of customers.

4.4.2.2 Option 2:

Draw water from Buckinbah weir to storage reservoir, pumped via transfer main to distribution main.

This option reduces costs significantly by reducing the length of transfer main substantially and is preferred due to the existence of a pump station already at the intersection of the Thuraggi water course and the Buckinbah Main channel.

4.4.2.3 Preferred Option

Due to the large amount of cost for pipeline in option 1, option 2 will be considered for the purposes of this investigation. Figures 4-6 to 4-11 below show the path lengths for each option as well as the RL's at each location. From a hydraulics perspective using the Hazen Williams calculator outlined in section 4.5 of this report, long lengths of pipe require a higher head to drive flows which also leads to the requirement for more pumps and pump stations to make the option viable. For this reason, Option 2 will be the preferred method moving forward. It is also possible that the cost of pump infrastructure could be reduced by augmenting the existing pump station. Option 2 requires the design of 29.2 km of pipeline.

Attachment 2 St George Channel Scheme

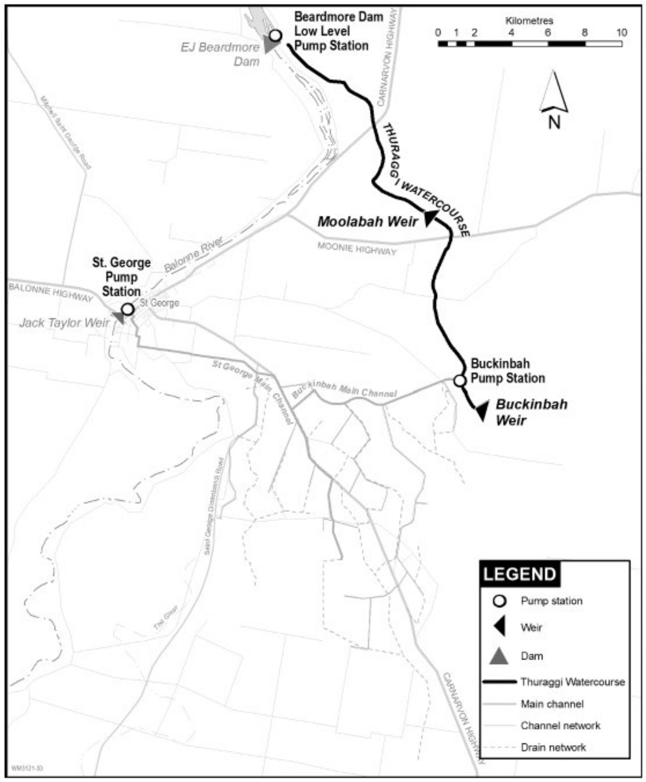

Figure 4-6 - St George Channel Scheme (Mallawa Irrigation 2018).

Figure 4-8 below shows the preferred pipeline path from the Beardmore Dam to Buckinbah channel for the main transfer line as approximately 25.8 km in length and Figure 4-9 shows the distribution channel along Buckinbah channel and to the south as approximately 29.2 km.

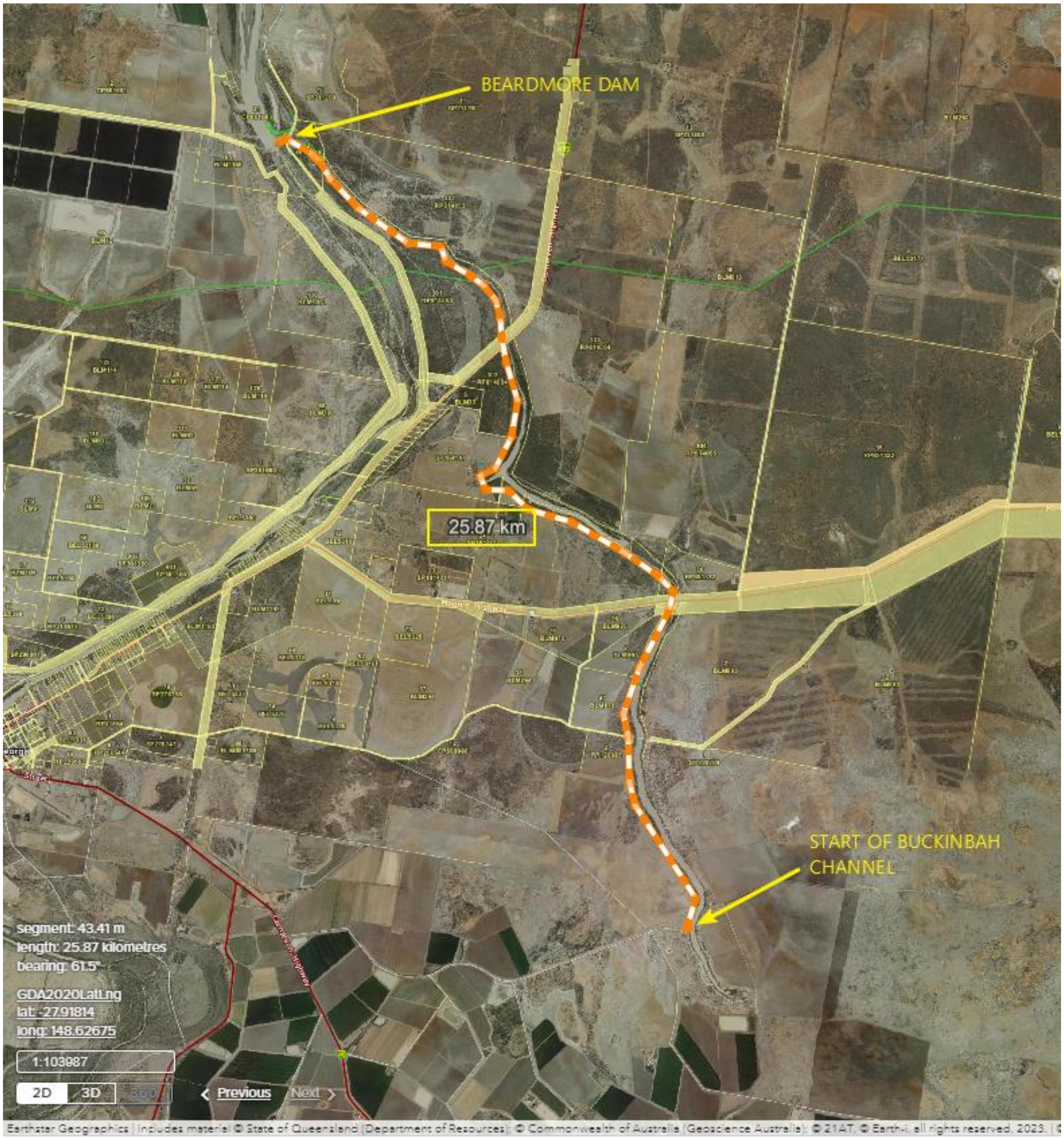

Figure 4-7 - Transfer Main Proposed Location (QLD Globe, 2023)

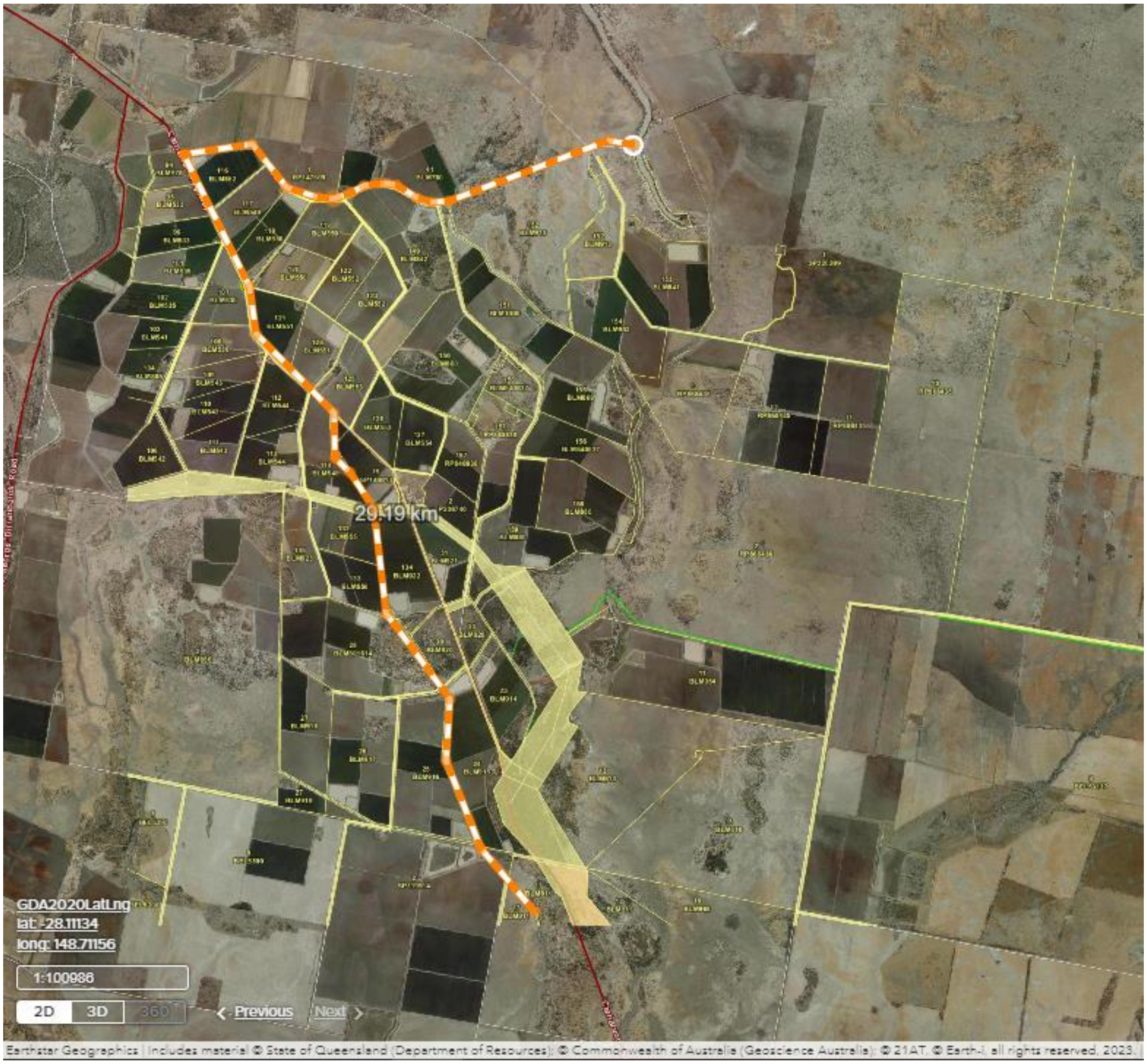

Figure 4-8 Buckinbah Channel Transfer Main (QLD Globe, 2023)

As indicated in the following three figures, ground reduced levels for the three main locations at Beardmore dam, Buckinbah channel, and then end of line or lowest point have been derived from a web-based application. The reduced levels at the three main locations are:

- Approximate RL at Beardmore Dam = 210 mAHD
- Approximate RL at Buckinbah Channel = 201 mAHD
- Approximate RL at Lowest Point (end of line) = 193 mAHD

Figure 4-9 RL at Start of Pipeline, Beardmore Dam (topgraphic-map.com, 2023)

Figure 4-10 RL at Start of Buckinbah Channel (topgraphic-map.com, 2023)

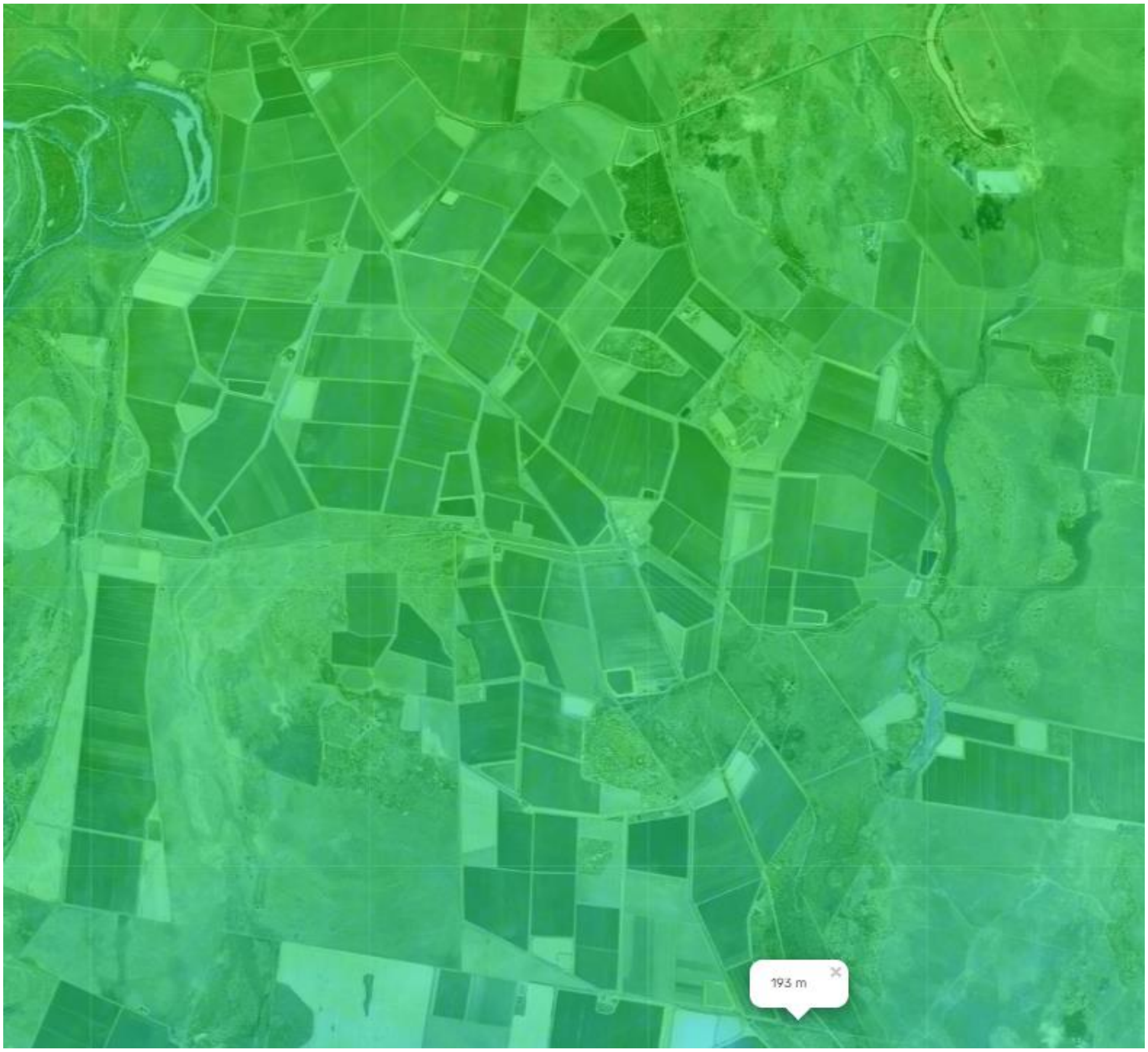

Figure 4-11 RL at lowest point of transfer main (topgraphic-map.com, 2023)

4.4.3 Design Life

All systems shall be designed in accordance with table 1.2 of WSAA SEQ service providers code (Table 4-2) (WSAA 2019)

Table 4-2 WSAA SEQ Code Table 1.2 Typical Asset Design Lives

Table 1.2 TYPICAL ASSET DESIGN LIVES

ITEM	Water Mains	Reservoirs	Pumps	Valves	*SCADA*
Expected design life, years	100	100	20	30	15

4.4.4 Pipe Material

The underlying material in the St George area is known as St George Alluvium (Water Policy 2018). The soil types commonly found in the St George area are typically red sodosol and red vertosol which are both heavy clays with the latter being more reactive to shrink-swell and cracking of the clay (QLD Government 2022).

A suitable pipe material for these types of heavy, reactive clayey soils is High Density Polyethylene (HDPE) pipe due to its robustness and ability to flex and bend with any soil upheaval or sagging. It would not be recommended to use pipe materials such as PVC or GRP due to the more rigid nature of the pipes however, DICL mains could potentially be adopted for the larger main lengths of pipe where a longer design life may be required.

Poly-Vinyl Chloride (PVC) is often used for water mains however it is typically used in sandy materials or where shallow water tables exist due to the reduced infiltration possible with this pipe type. Due to lack of information about the water table and heavier clays likely to be encountered, HDPE pipe will be considered for this concept design however if reason is found during detailed feasibility PVC may be determined to be a better pipe material.

4.4.5 Pipe Class

Water mains shall typically be PE100 SDR11 pipe class. Where a pipe sized exceeds the maximum internal diameter for that pipe class, a size from the next pipe class will be selected. Pipe sizes will be determined using a standard Polyethylene pipe dimensions chart. For this report the pipe sizes will be selected from a pipe dimension chart supplied (Figure 4-12) on ACU-TECH's website (acu-tech 2023)

Polyethylene Pipe Dimensions

	SDR26		SDR21		SDR17		SDR13.6		SDR11		SDR9		SDR7.4		
PE100	PN 6.3		PN 8		PN 10		PN 12.5		PN 16		PN 20		PN 25		PE100
SIZE	MIN WALL	MEAN I.D.	MIN WALL	MEAN I.D.	MIN WALL	MEAN I.D.	MIN WALL	MEAN I.D.	MIN WALL	MEAN I.D.	MIN WALL	MEAN I.D.	MIN WALL	MEAN I.D.	SIZE
20	1.6	16.7	1.6	16.7	1.6	16.7	1.6	16.7	1.9	16.1	2.3	15.2	2.8	14.2	20
25	1.6	21.7	1.6	21.7	1.6	21.1	1.9	21.1	2.3	20.2	2.8	19.2	3.5	17.7	25
32	1.6	28.7	1.6	28.7	1.9	28.1	2.4	27.0	2.9	26.0	3.6	24.5	4.4	22.8	32
40	1.6	36.7	1.9	36.1	2.4	35.0	3.0	33.8	3.7	32.3	4.5	30.6	5.5	28.5	40
50	2.0	45.9	2.4	45.0	3.0	43.8	3.7	42.3	4.6	40.4	5.6	38.3	6.9	35.6	50
63	2.4	58.0	3.0	56.8	3.8	55.1	4.7	53.2	5.8	50.9	7.1	48.1	8.6	45.1	63
75	2.9	69.1	3.6	67.6	4.5	65.7	5.5	63.6	6.8	60.9	8.4	57.5	10.3	53.6	75
90	3.5	82.8	4.3	81.1	5.4	78.8	6.6	76.3	8.2	72.9	10.1	68.6	12.3	64.5	90
110	4.3	101.2	5.3	99.1	6.6	96.4	8.1	93.2	10.0	89.3	12.3	84.4	15.1	78.6	110
125	4.8	115.3	6.0	112.8	7.4	109.8	9.2	106.0	11.4	101.4	14.0	96.0	17.1	89.5	125
140	5.4	129.1	6.7	126.4	8.3	123.0	10.3	118.8	12.7	113.8	15.7	107.5	19.2	100.2	140
160	6.2	147.5	7.7	144.4	9.5	140.6	11.8	135.8	14.6	129.9	17.9	123.0	21.9	114.7	160
180	6.9	166.2	8.6	162.6	10.7	158.2	13.3	152.7	16.4	146.2	20.1	138.4	24.6	129.1	180
200	7.7	184.5	9.6	180.5	11.9	175.7	14.7	169.8	18.2	162.4	22.4	153.6	27.3	143.4	200
225	8.6	207.7	10.8	203.1	13.4	197.6	16.6	190.9	20.5	182.7	25.1	173.0	30.8	161.3	225
250	9.6	230.7	11.9	225.9	14.8	219.8	18.4	212.2	22.7	203.2	27.9	192.3	34.2	179.2	250
280	10.7	258.6	13.4	252.9	16.6	246.2	20.6	237.8	25.4	227.7	31.3	215.3	38.3	200.7	280
315	12.1	290.7	15.0	284.7	18.7	276.9	23.2	267.4	28.6	256.1	35.2	242.2	43.0	226.1	315
355	13.6	327.8	16.9	320.9	21.1	312.0	26.1	301.5	32.2	288.7	39.6	273.2	48.5	254.6	355
400	15.3	369.3	19.1	361.3	23.7	351.7	29.4	339.7	36.3	325.2	44.7	307.6	54.6	287.0	400
450	17.2	415.5	21.5	406.5	26.7	395.6	33.1	382.1	40.9	365.8	50.3	346.0	61.5	332.8	450
500	19.1	461.7	23.9	451.7	29.6	439.7	36.8	424.6	45.4	406.5	55.8	384.7	...	...	500
560	21.4	517.2	26.7	506.1	33.2	492.4	41.2	475.6	50.8	455.5	...	...	...	...	560
630	24.1	581.8	30.0	569.5	37.2	554.1	46.3	535.2	57.2	512.3	...	...	...	...	630
710	27.2	655.6	33.9	641.6	42.1	624.3	52.2	603.1	...	...	...	...	...	...	710
800	30.6	738.8	38.1	723.0	47.4	703.2	58.8	680.0	...	...	...	...	...	...	800
900	34.4	829.5	42.9	813.8	53.5	791.6	...	...	...	...	...	...	...	...	900
1000	38.2	923.0	47.7	904.2	59.3	879.8	...	...	...	...	...	...	...	...	1000

Figure 4-12 - PE Pipe Dimensions (acu-tech 2023)

4.5 HYDRAULIC CALCULATIONS

Calculations withing this section have been performed using Microsoft Excel Spreadsheets. The calculations for each cell are provided in Appendix B of this report.

4.5.1 Supply Regime

While the catchment area is large, the small number of domestic customer offtakes and stock requirements mean that the flows are likely to be relatively low for everyday use. Based on review of other similar designs, the supply regime is usually limited to once, twice, or four times a year with flows being delivered over the span of two to five day depending on the volume required per supply. For this concept level design three supply regime volumes will be determined to allow enough trials of pipe size and pump station configurations to inform the most effective delivery frequency and volume.

4.5.2 Maximum and Average Flow Calculations

Figure 4-13 below provides the flows required for each supply regime with a delivery time frame of 7 days per supply. The flows are shown in m3/s so they can be input into the Hazen-Williams equation to determine the best high level pipe design for each. The flows used for this report are shown below in Figure 4-14 in bold in the far-right hand side column and the units are in m3/s.

Demand Scenarios

80	offtake points
1.2	factor for potential future demand
2.57	factor for EP
247	EPs for future demand
10000	area of SGIA ha
500	estimated area for livestock ha
10	DSE (dry sheep equivalent)
20	Hour pumping per delivery day

	AD (L/EP/day)	L/EP/delivery period	ML/Delivery period	Demand/ n days (n, days), kL	Demand/ Supply timeframe, m3/s	Demand/ Supply timeframe* peaking factor m3/s
	Full year Stock and Domestic	AD*EP*365	AD per delivery period /10^6	(Demand/n) * 1000 (n = 7)	Demand/ (pumping hours * second * minutes)	Demand*1.5
Domestic	260	23413728	23.41	3344.82	0.0465	0.0697
Stock	75	136875000	136.88	19553.57	0.2716	0.2716
Total			160.29	22898.39	0.3180	**0.3413**
	6 month Stock and Domestic	AD*EP*(365/2)		n = 7		
Domestic	260	11706864	11.71	1672.41	0.0232	0.0348
Stock	75	68437500	68.44	9776.79	0.1358	0.1358
Total			80.14	11449.19	0.1590	**0.1706**
	3 month Stock and Domestic	AD*EP*(365/4)		n = 7		
Domestic	260	5853432	5.85	836.20	0.0116	0.0174
Stock	75	34218750	34.22	4888.39	0.0679	0.0679
Total			40.07	5724.60	0.0795	**0.0853**

Figure 4-13 - Demand Scenarios

4.5.3 Step 1 Pipe Sizing

A calculator was setup within Microsoft Excel to calculate the head loss in meters using the Hazen Williams equation (Equation 4-1) for each flow regime. A trial of internal diameters for common pipe sizes were trialled to calculate the head loss and determine if the rate of head loss in meters per kilometre were acceptable in accordance with section 4.1.8 of this report. Velocities were also checked that they do not exceed maximum values. It was found that the minimum desired velocity of 1 m/s however all pipes were able to be kept below 2.5m/s. Figure 4-14 below shows the results of each trial for the Hazen Williams Calculations. Figure 4-15 illustrates high level Hydraulic Grade Lines (HGL) for each option.

Hazen Williams Calculator

	12 mnth delivery 1 pump station	12 mnth delivery 2 pump stations	6 mnth delivery 1 pump station	6 mnth delivery 2 pump stations	3 mnth delivery 1 pump station	3 mnth delivery 2 pump stations	3 mnth delivery 4 pump stations
C<200	100	100	100	100	100	100	100
C>200	110	110	110	110	110	110	110
Q (m3/s)	**0.341**	**0.341**	**0.171**	**0.171**	**0.085**	**0.085**	**0.085**
D (ID,m)	0.5123	0.5123	0.4555	0.4555	0.3658	0.3658	0.3658
D (DN,mm)	**630**	**630**	**560**	**560**	**450**	**450**	**450**
A (m2)	0.206	0.206	0.163	0.163	0.105	0.105	0.105
V (m/s)	**1.656**	**1.656**	**1.047**	**1.047**	**0.812**	**0.812**	**0.812**
L (m)	**29200**	**14600**	**29200**	**7300**	**29200**	**14600**	**3650**
hf (m)	**144**	**72**	**105**	**26**	**105**	**52**	**13**
residual head	12	12	12	12	12	12	12
h (m)	**156**	**84**	**117**	**38**	**117**	**64**	**25**
Max allowable headloss	5	5	5	5	5	5	5
headloss (m/km)	4.95	4.95	3.59	3.59	3.59	3.59	3.59
PASS/FAIL	**PASS**	**PASS**	**PASS**	**PASS**	**PASS**	**PASS**	**PASS**

Figure 4-14 - Hazen Williams Calculations

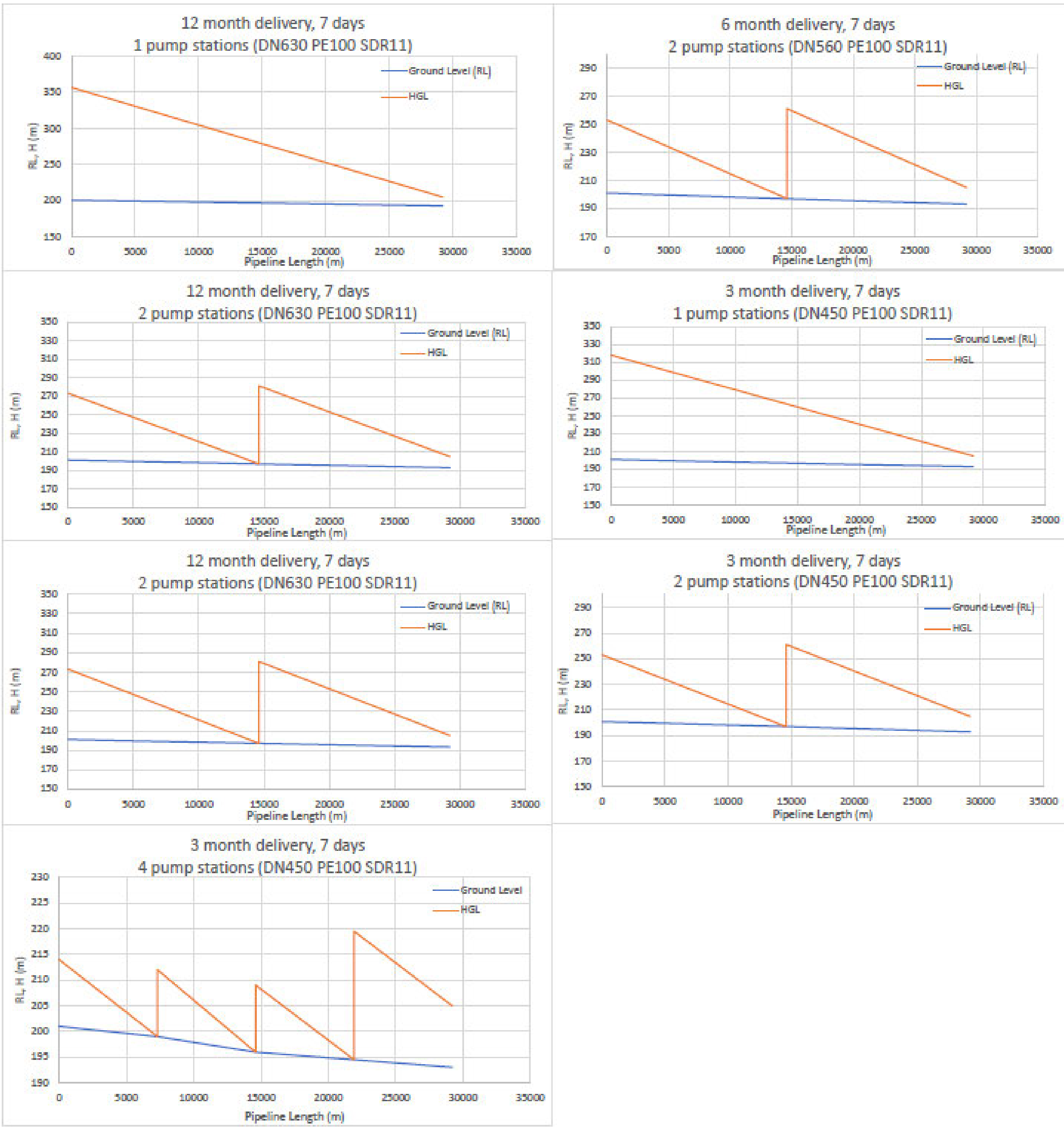

Figure 4-15 - Design Option Hydraulic Grade Lines

4.5.4 Step 2 Pump station requirements

The pump power equation (Equation 4-6) was used to determine the kW requirements generate the head loss in meters obtained from step 1. A pump was then selected from a chart obtained from the Browns Brothers website (Figure 4-16) which supplies costings and models with kW outputs for the Gould's GIS series-ISO motor pumps which are capable of outputting the most amount of power and therefore head requirements of any other pump system found online for the lowest price point through a cursory investigation. From this an estimate of cost per pump station was obtained by dividing the calculated pump power (kW) by relevant pump outputs to determine how many pumps would be required and multiplying that by the cost per pump. An additional estimated cost of construction for the pump station infrastructure was assumed to be $250,000 per pump station to account for additional pipe fittings take the pipe out of the ground and into the pump assembly, concrete slabs and sheds, pump control systems and access requirements. Figure 4-17 shows the results of the pump power calculations and costing. The cost value obtained is purely based on an assumption and should not be relied upon for a detailed investigation.

GIMP

GIMP-BU

Goulds GIS Series–ISO Motor Pump

Stub-Shaft / Close Coupled ISO2858

- 304 Stainless Steel wear rings and impeller
- Cast iron pump casing
- Pump casing 16 bar
- Motor IP55
- Carbon ceramic shaft seal
- All 316 Stainless Steel construction available on request
- Price includes impeller trim to non-overloading motor size
- **Freight charges apply to this product range**

2 Pole–2900 RPM

Product Code:		GIMP	GIMP-BU
Model	**kW**	**Without Base**	**With Base**
50 x 32-160	3.00	$2,712	$3,337
50 x 32-160	4.00	$2,769	$3,394
50 x 32-160	5.50	$3,063	$3,688
50 x 32-200	5.50	$3,310	$3,935
50 x 32-200	7.50	$3,393	$4,018
50 x 32-200	11.00	$3,874	$4,499
65 x 50-160	4.00	$2,747	$3,407
65 x 50-160	5.50	$2,948	$3,608
65 x 50-160	7.50	$3,126	$3,786
65 x 40-200	7.50	$3,441	$4,101
65 x 40-200	11.00	$3,830	$4,490
65 x 40-200	15.00	$4,128	$4,788
65 x 40-250	11.00	$4,655	$5,315
65 x 40-250	15.00	$4,952	$5,612
65 x 40-250	18.50	$5,181	$5,841
65 x 40-250	22.00	$5,542	$6,202
65 x 40-250	30.00	$6,307	$6,967
65 x 40-315	22.00	$6,340	$7,000
65 x 40-315	30.00	$7,107	$7,767
65 x 40-315	37.00	$7,320	$7,980
65 x 40-315	45.00	$9,134	$9,794
80 x 65-160	5.50	$3,151	$3,876
80 x 65-160	7.50	$3,235	$3,960
80 x 65-160	11.00	$3,621	$4,346
80 x 65-160	15.00	$4,016	$4,741
80 x 50-200	11.00	$3,881	$4,606
80 x 50-200	15.00	$4,178	$4,903
80 x 50-200	18.50	$4,406	$5,131
80 x 50-200	22.00	$4,778	$5,503
80 x 50-250	18.50	$5,521	$6,246
80 x 50-250	22.00	$5,894	$6,619
80 x 50-250	30.00	$6,702	$7,427
80 x 50-250	37.00	$6,915	$7,640
80 x 50-250	45.00	$8,673	$9,398

Product Code:		GIMP	GIMP-BU
Model	**kW**	**Without Base**	**With Base**
80 x 50-315	37.00	$7,187	$7,912
80 x 50-315	45.00	$8,931	$9,656
80 x 50-315	55.00	$9,857	$10,677
80 x 50-315	75.00	$10,848	$11,668
100 x 80-160	11.00	$4,594	$5,294
100 x 80-160	15.00	$4,892	$5,592
100 x 80-160	18.50	$5,521	$6,221
100 x 80-160	22.00	$5,500	$6,200
100 x 65-200	15.00	$5,224	$5,924
100 x 65-200	18.50	$5,452	$6,152
100 x 65-200	22.00	$5,825	$6,525
100 x 65-200	30.00	$6,633	$7,333
100 x 65-200	37.00	$6,847	$7,647
100 x 65-250	30.00	$6,795	$7,595
100 x 65-250	37.00	$7,024	$7,824
100 x 65-250	45.00	$8,804	$9,604
100 x 65-250	55.00	$9,707	$10,597
100 x 65-250	75.00	$11,445	$12,335
100 x 65-315	55.00	$10,739	$11,629
100 x 65-315	75.00	$11,813	$12,703
100 x 65-315	90.00	$13,417	$14,367
100 x 65-315	110.00	$13,885	$14,835
125 x 100-200	30.00	$6,683	$7,408
125 x 100-200	37.00	$6,897	$7,622
125 x 100-200	45.00	$8,630	$9,430
125 x 100-200	55.00	$9,565	$10,365
125 x 100-200	75.00	$10,546	$11,346
125 x 100-250	55.00	$10,620	$11,420
125 x 100-250	75.00	$11,545	$12,345
125 x 100-250	90.00	$13,152	$14,102
125 x 100-250	110.00	$13,938	$14,888
125 x 100-315	90.00	$13,834	$14,784
125 x 100-315	110.00	$14,415	$15,365

Figure 4-16 - Pump Sizing and Cost Chart (Brown Brothers Engineers 2019)

Pump Power

	12 mnth delivery 1 pump station	12 mnth delivery 2 pump stations	6 mnth delivery 1 pump station	6 mnth delivery 2 pump stations	3 mnth delivery 1 pump station	3 mnth delivery 2 pump stations	3 mnth delivery 4 pump stations	
ρ	0.998	0.998	0.998	0.998	0.998	0.998	0.998	
g	9.81	9.81	9.81	9.81	9.81	9.81	9.81	
Q	0.341	0.341	0.171	0.171	0.085	0.085	0.085	
h	144	72	105	26	105	52	13	
P (kW)	**483**	**241**	**175**	**44**	**88**	**44**	**11**	
Pump output (kW)	110	110	90	90	75	90	30	selected from table
No. pumps/pump station	4	2	2	1	1	1	0	
No. of pump stations	1	2	1	2	1	2	4	
Cost per pump (incl. base)	$14,838.00	$14,838.00	$14,367.00	$14,367.00	$12,335.00	$14,367.00	$7,595.00	
Total Cost of pumps	**$65,287.20**	**$32,643.60**	**$28,734.00**	**$7,183.50**	**$14,431.95**	**$7,183.50**	**$3,038.00**	
Estimated Additional cost per pump station ($250,000)	**$315,287.20**	**$532,643.60**	**$278,734.00**	**$507,183.50**	**$264,431.95**	**$507,183.50**	**$1,003,038.00**	

Figure 4-17 - Pump Power Table and Costing

4.5.5 Pipeline Cost

Finally, the pipeline cost was estimated using Equation 4-7 for pipeline cost. These costs were based on the previous costing for the pumps and pump stations plus the material costs for the pipeline and allowances for fittings, GST, and delivery and other possible incurred costs. The cost per meter of pipe material was obtained from matrix piping (Figure 4-18) (Matrix 2017).

HDPE PIPE PRICES CHART - AUD PER METRE

Size	PE100 PN4 SDR41	PE100 PN6.3 SDR26	PE100 PN8 SDR21	PE100 PN10 SDR17	PE100 PN12.5 SDR13.6	PE100 PN16 SDR11
32	-	-	$0.93	$1.08	$1.35	$1.58
40	-	-	$1.39	$1.72	$2.08	$2.52
50	-	-	$2.20	$2.65	$3.21	$3.91
63	-	$2.77	$3.36	$4.22	$5.12	$6.18
75	-	$3.94	$4.81	$5.95	$7.15	$8.63
90	-	$5.72	$6.96	$8.56	$10.27	$12.50
110	$5.49	$8.56	$10.44	$12.74	$15.41	$18.54
125	$7.20	$10.80	$13.34	$16.25	$19.86	$24.05
140	$9.03	$13.64	$16.65	$20.40	$24.86	$29.98
160	$11.69	$17.89	$21.87	$26.51	$32.46	$39.35
180	$14.56	$22.27	$27.49	$33.68	$41.20	$49.74
200	$17.89	$27.62	$34.05	$41.56	$50.52	$61.33
225	$22.65	$34.71	$43.04	$52.71	$64.16	$77.64
250	$28.38	$43.00	$52.66	$64.60	$79.04	$95.48
280	$25.16	$53.64	$66.47	$81.16	$99.04	$119.71
315	$44.17	$68.33	$83.52	$102.77	$125.51	$151.53
355	$56.19	$86.34	$106.02	$130.79	$159.11	$192.32
400	$71.23	$109.50	$135.20	$165.30	$201.78	$244.18
450	$89.82	$138.45	$170.98	$209.44	$255.61	$309.23
500	$111.82	$170.78	$210.95	$257.99	$315.38	$381.54
560	$139.23	$214.06	$264.02	$324.15	$395.65	$477.94
630	$176.15	$271.25	$333.50	$409.60	$500.13	$605.61
710	$224.16	$334.79	$424.68	$520.95	$635.37	-
800	$284.24	$436.68	$538.24	$660.58	$805.94	-

Figure 4-18 - HDPE Pipe Prices Chart (Matrix 2017)

Figure 4-19 below outlines the costs of pipeline plus the total costs for pipelines and pump stations. From the data below it can be seen that the cheapest option is delivering the water every three months over 7 days with the use of a single pump station. The total cost of this option is estimated to be approximately $13.375 million for a pipe size of DN450 or 365.8mm internal diameter pipeline for a length of 29.2km.

Pipeline Cost

	12 mnth delivery 1 pump station	12 mnth delivery 2 pump stations	6 mnth delivery 1 pump station	6 mnth delivery 2 pump stations	3 mnth delivery 1 pump station	3 mnth delivery 2 pump stations	3 mnth delivery 4 pump stations
Length	29200	29200	29200	29200	29200	29200	29200
Pipe size (DN,SDR11)	630	630	560	560	450	450	450
Material cost of pipeline ($/m)	605.61	605.61	477.94	477.94	309.23	309.23	309.23
F1. Allowance for pipe fittings (x 1.1)	1.1	1.1	1.1	1.1	1.1	1.1	1.1
F2. Additional cost. GST+1.2 multiplier for delivery etc. (x1.1x1.2)	1.32	1.32	1.32	1.32	1.32	1.32	1.32
Cost of pipeline	$25,676,895.02	$25,676,895.02	$20,263,891.30	$20,263,891.30	$13,110,857.23	$13,110,857.23	$13,110,857.23
Total cost of pumps and pipelines	**$25,992,182.22**	**$26,209,538.62**	**$20,542,625.30**	**$20,771,074.80**	**$13,375,289.18**	**$13,618,040.73**	**$14,113,895.23**

Figure 4-19 - Pipeline Costs

5.2 WATER SAVINGS

Estimates from northern Victoria place stock and domestic water supply from either ground or surface water accounts to be about four to six per cent of total water use. Because of this, water losses such as evaporation and seepage will be estimated for the portion of channel network which the stock and domestic supply pipeline will operate within and proportion them as six percent of the total losses calculated. Only losses from evaporation and seepage will be calculated as part of this reports scope.

Further to this Mallawa Irrigation has supplied anecdotal information on stock and domestic supply operation which states that during periods of low water volume and dry channels, it is typical for the company to be required to supply additional flows to customers which means that dry channels have to be re-wet and typically a volume in the order of 100ML is required in order to supply flows to one or two offtakes. Additional flows not taken from the channel will be lost to evaporation, seepage or leave the system in other ways. Because of this, an additional 80ML (80 percent of additional volume) will be added to the assumed water losses as a HDPE pipeline will improve efficiency and remove the need to waste such high volumes. The flows provided through a pipeline would also be able to be piped more direct to customers requiring any additional volume.

5.2.1 Estimated Evaporation Losses

A high-level estimation will be calculated for evaporation based on the Department of Sustainability and Environment, 'Technical manual for the Quantification of Water Savings' document for within a channel section (DSE 2012)

Equation 5-1- Annual Evaporation for within a channel

$$Annaul\ Evaporation\ (ML) = \left(\frac{(E * PEF) - R}{1{,}000{,}000}\right) * A * CWF * t$$

Where:

E = Daily Evaporation Rate (mm/day)

PEF = Pan Evaporation Factor

R – Daily Rainfall Rate (mm/day)

A = Surface area of System (m^2)

CWF = Channel Width Factor

t = Length of Standard Irrigation Season (days)

5.2.1.1 Daily Evaporation Rate (mm/day)

Monthly pan evaporation losses for the SGIA have been obtained from the Bureau of Meteorology average annual, monthly, and seasonal evaporation website. Data was obtained for each month by grid and the data for the Latitude -28.038, Longitude 148.582 was extracted and is tabled below. The data is based on a class A evaporation pan.

Table 5-1 Monthly evaporation figures for the St George Irrigation Area based on BOM average pan evaporation rates.

	SGIA	
Month	**Evaporation (mm)**	**% of Total**
January	249	11.22
February	196	8.82
March	206	9.30
April	163	7.33
May	125	5.64
June	99	4.46
July	110	4.97
August	143	6.43
September	193	8.70
October	233	10.51
November	241	10.87
December	261	11.75
Total	**2219**	**100**
April-September	833	37.53
Oct-March	1386	62.47
Total	**2219**	**100**

The daily evaporation rate will be taken as the total evaporation throughout the year divided the number days in one year.

Therefore:

$$\boldsymbol{E} = \frac{2{,}219\ mm}{365\ days} = \boldsymbol{6.08 mm/day}$$

5.2.1.2 Pan Evaporation Factor

A pan evaporation factor of 0.85 will be adopted for the high-level estimate due to the lack of information. This is similar to what was adopted by the Department of Sustainability and Environments assessment of the evaporation within the Goulbourn Murray Irrigation District (DSE 2012)

5.2.1.3 Daily Rainfall Rate (mm/day)

The method for determining daily rainfall rate was to take the mean annual rainfall date for St George and divide it by the number of days in one year. The data was obtained from two location sources provide by the Bureau of Meteorology from the climate statistics web page. The two locations are the St George Airport, and the St George Post office. The data from the St George Airport will be used as the data ranges from the year 1997 to 2023 whereas the St George post office data is limited to 1997 as the latest available data.

Location	Mean Rainfall (mm) Years range 1997-2023
St George Airport	475

$$\boldsymbol{R} = \frac{475\ mm}{365\ days} = \boldsymbol{1.301\ mm/day}$$

5.2.1.4 Surface area of System (m^2)

The Channel capacity map provided by Mallawa Irrigation (Appendix A) was used to obtain an estimate of the channel lengths from the intersection of the Buckinbah main channel and the Thuraggi Watercourse.

The total channel length estimated is 79,104m.

In the report by Fairley, 2015, table 3.1 indicates that the channel bed widths for Buckinbah channel and the B2/2 channels are 5.5m. With an estimated 0.8-1.1m water depth and typical side batter slopes of 1 in 2, the top width will be assumed to be (0.8+1.1)/2 * 2 + 5.5 = 7.4m

Therefore, the surface area of the system will be taken as:

$$\boldsymbol{A}\ (\boldsymbol{m^2}) = 79{,}104 * 7.4 = \mathbf{585{,}370}\ \boldsymbol{m^2} = 58.5\ ha\ of\ open\ channel\ area$$

5.2.1.5 Channel Width Factor

The Channel width factor is a dimensionless ratio between the actual bank width to the recorded bank width to account for bank slumping (DSE 2012). In this case the channel width factor is unknown so a value of 1 will be adopted.

5.2.1.6 Length of Standard Irrigation Season (days)

Currently the Irrigation area runs full throughout the year with a 12-month season. Length of standard irrigation season is therefore 365 days.

5.2.1.7 Annual Evaporation Calculation

The annual evaporation in megalitres is estimated to be:

$$Annaul\ Evaporation\ (ML) = \left(\frac{(6.08 * 0.85) - 1.301}{1{,}000{,}000}\right) * 585{,}370 * 1 * 365 = 826.22ML$$

$$AE\ for\ Stock\ and\ Domestic\ Supply\ (ML) = 826.22 * 0.06 = 49.57\ ML$$

5.2.2 Estimated Seepage Losses

The estimation of seepage losses is difficult to calculate therefore for the use of this report, a factor of 1.15 or 15 per cent of total supplied flow will be used to determine an estimated flow. Of this, six per cent of that will be attributed to stock and domestic supply. This factor is adopted from McLeans 2015 report on seepage in the St George Irrigation Area.

From Mallawa Irrigations website the total volume of flow supplied to the irrigation area annually is 58000ML therefore:

$$Annual\ Seepage\ (ML/year) = 58{,}000 * 0.15 = 8{,}700\ ML/year$$

$$Annual\ Seepage\ for\ Stock\ and\ Domestic\ Supply\ (ML/year) = 8{,}700 * 0.06 = 522\ ML/year$$

Using the actual seepage value obtained from Mclean's research of 0.008md-1 over an area of 585,370 m2, another estimate of

$$Annual\ Seepage\ \ (m^3/day) = 0.008 * 585{,}370 = 4{,}682.96\ \approx 4{,}683\ m^3/day$$

$$Annual\ Seepage\ (ML/year) = \frac{4{,}683 * 365}{1000} =\ 1{,}709.295\ \approx 1{,}709 ML/year$$

$$Annual\ Seepage\ for\ Stock\ and\ Domestic\ (ML/year) = 1{,}709 * 0.06 = 102.54\ ML/year$$

(Mclean 2015)

Using Akbar's (2000), estimate that older unlined channels typically experienced a seepage loss of 50mm per day over an area of 585,370 m2.

$$Annual\ Seepage\ \ (m^3/day) = 0.05 * 585{,}370 = 29{,}268.5\ \approx 29{,}269\ m^3/day$$

$$Annual\ Seepage\ (ML/year) = \frac{29{,}269 * 365}{1{,}000} =\ 10{,}683.185\ \approx 10{,}683\ ML/year$$

$$Annual\ Seepage\ for\ Stock\ and\ Domestic\ (ML/year) = 10{,}683 * 0.06 = 641\ ML/year$$

To be conservative the higher value of 641 ML will be adopted.

5.2.3 Total Water Savings

The total estimated water savings which can be attributed to a stock and domestic water supply pipeline are:

$$\boldsymbol{Total\ annual\ Savings} = 80 + 49.57 + 641 = 770.57\ \approx \boldsymbol{771\ ML/year}$$

5.2.4 Dead storage Estimate

Based on the final selected pipe size of DN450 which has an internal diameter of 0.3685m over a length of 29.2km, the volume of dead storage loss is estimated to be:

$$\frac{\pi * 0.3658^2}{4} * 29200 = 3068.75\, m^3 \approx 3.07\, ML$$

This loss is negligible and is lost to the environment rather than leaving the system so is not considered to be wasted.

5.3 PROJECT VALUE

5.3.1 Estimated Value of water savings

Based on Victorias most recent Goulburn Murray water – water efficiency project in 2021, The long term average annual yield (LTAAY) over 5 projects was 23.4 GL. The approved funding for the projects $346.97 million. This equates to approximately $14.82 per Liter of saved water, or $14,828 per ML of water.

Applying this rate, the estimated savings of 652 ML per year and it is anticipated that an approximate of:

$$771 * 14{,}828 = \$\mathbf{11{,}432{,}388} = \$\mathbf{11.43}\ \boldsymbol{million}$$

could potentially be sought for funding by the government however, based on the case studies provided there is a potential for the value of government input per ML of saving changes rapidly and can depend on many factors including the current program of infrastructure spending which is providing the money, whether the project contributes more to the community and environment than just water savings, and the current economic climate with the majority of the outlined project receiving grants for funding prior to 2022 when inflation skyrocketed and costs of materials and labour were at record high.

5.4 CONCLUSIONS

Based on the estimated cost of $13.375 million and the potential government spending of $11.43 the project is feasible if Mallawa Irrigation is prepared to either cover the shortfall of approximately $1.94

million or cover the entire capital cost wholly by the organisation. This high-level feasibility study shows that the theoretical cost for construction of a stock and domestic supply pipeline is worthy of a full-scale investigation. Further discussion will be provided in chapter 6 of this report.

Chapter 6 CONCLUSIONS

6.1 INTRODUCTION

Chapter six will aim to bring together the results of the concept design and costing for feasibility and compare the results against the case studies from chapter four. The process of researching similar projects with enough detail to determine every key objective of the case study section was difficult and as a result the comparisons to concept design are, as outlined below, only to be taken as a very high-level investigation.

6.2 KEY FINDINGS

Presented below are a summary of conclusions drawing from the key findings and a discussion of each.

6.2.1 Loss Estimation

The literature review revealed many different methods of estimating evaporation and seepage losses in open unlined channels with particular emphasis on channels within the St George Irrigation Area which is also the area of interest for this thesis. With that in mind equations and estimation of flow were able to be taken from research results within the area. If this thesis topic were to be taken further or considered for any other regional irrigation areas, particular attention needs to be given to this point and region-specific information should be used where possible. The methods used for loss estimation and resources used for research purposes were Moavenshahidi, Mclean, and the Department of Sustainability and Environment. Even within the reports by these authors there is a level of uncertainty in the results which only highlights how complex the issue of measuring losses is.

The results obtained in chapter five are based on high level methods and area data obtained using very generalised techniques such as using estimated volumes/day of seepage, assumed data to fill gaps, and crude measurements for channels based on low detail maps. To follow on from this, regardless of what the actual losses may be, they are estimated over a large area of open unlined channels and that even though the actual values may fluctuate greatly from the theoretical values in this report, the monetary loss due to these losses is significant and in the order of $11.43 million. It is clear that the amount of water lost to seepage and evaporation through re-wetting channels, evaporation during periods of high temperature, is high enough to warrant the construction of a separate stock and domestic delivery pipeline however the leading factor in feasibility which will also be discussed is the cost to construct and maintain the system versus how much funding Mallawa Irrigation is willing to pay for or, if a government infrastructure grant could be supplied.

6.2.2 Supply and Demand Estimation

The methodology for supply and demand estimate was largely taken from sources such as Larson et al's technical report on 'Estimating Stock and Domestic Water use to improve catchment water management outcomes' as well generalised demand flows and other factors from the Water Service Association Australia's (WSAA) design and construct code. These documents provided valuable information however, due to the nature of the St George Irrigation Area's stock and domestic demand usage which is largely unknown due to the lack of metering undertaken by Mallawa Irrigation, all estimations are to be taken as high level until such time as a full-scale feasibility can be undertaken at which point a survey of demand requirements should be taken. It should also be noted that if this research is to be used to determine demand for other irrigation areas, the data should be obtained for that local area or assumptions made based on local considerations.

6.2.3 Concept Design

As part of the research for Mallawa Irrigation a concept design was developed using a methodology of determining method of estimating losses from evaporation and seepage, estimating demand requirements for stock and domestic supply, case study analysis of similar projects to give context to design outcomes, a concept design with an aim to provide a cost for the most cost effective design scenario, and finally a comparison of the concept design against the projects researched in the case study chapter, chapter three of this thesis. To undertake this design many assumptions were made and since no current design methodology exists or was able to be located, the method is largely theoretical. The purpose of the concept design was not to provide an exact figure for cost benefit analysis rather it was to highlight the potential cost-effective design for consideration in a full-scale feasibility. It is recommended that Mallawa Irrigation determine what routes are best suited for the irrigation area as well as determine the best locations for pump stations and pipeline based on the extensive knowledge to which Mallawa Irrigation has of the current channel, pump, and weir system.

The design focused mainly applying high level equations to an assumed route for the pipeline based on an assumption of where the start and end of the pipeline could possibly be. Several demand scenarios were run to find the most efficient way of operating the pipeline. Due to the flat terrain and low volumes of water found to be required (relative to larger irrigation areas) it was found that a long pipeline is not preferred as it would increase the number of pump stations to unnecessarily large amount or increase the pipe diameter to such a large size as to make the project infeasible. Due to the Beardmore dam discharging flow along the Thuraggi channel the Buckinbah weir it was determined that Buckinbah weir would be the best location to start the pipeline.

The results found that a 29.2km pipeline of 450mm nominal diameter (internal diameter of 365.8mm) of a pipe class PE100 SDR21 delivering 5.724ML/day or 0.0853 m3/s for 20 hours a day for a 7-day period with a delivery frequency of 3 months was the optimal design solution. This solution was obtained using the high-level Hazen Williams equation to determine maximum head flow and the solution only requires 1 pump station. The pump station could potentially be constructed or integrated at the Buckinbah weir where an existing pump station is already located and is estimated to deliver about 105m of head to the system requiring approximately 88 kW of power. This is enough head to have 12m residual head at the end of the transfer main with a maximum head loss of 3.59 m/km which is within the absolute maximum of 5m/km.

6.3 COST AND FEASIBILITY

6.3.1 Cost and Cost Recovery

Further to the concept design, dollar values were attributed to the individual components of the pipeline design to estimate a total cost. It should be noted that even though the dollar values may not be accurate with time and partially assumed values used, that the most efficient design should still be the most cost effective if relative dollar values are attributed to all designs. Other factors which may influence unit values are, availability of materials, cost of local labour, cost of contractors outside of the region, and of course inflation. These factors are difficult to quantify, and the cost estimation of a full-scale feasibility study should be undertaken with a professional with experience pricing similar projects near St George.

It was found that the cost of the concept design was estimated to be approximately $13.375 million without making any further assumptions on factors to account for other capital costs. It is highly possible that a design could cost anywhere up to three times this amount according to Tonkin Science Engineering as mentioned in section 4.3 of this report (Tonkin Science Engineering 2011). The equation identified by Tonkin has not been used fully within this report and as such should be considered in any future feasibility studies.

Cost recovery has not been explored as part of this this thesis as the changing values of water and the financial operations of Mallawa Irrigation are not known.

6.3.2 Feasibility

Based on the dollar values obtained through the concept design and costing parts of this report as well as the well documented success cases of stock and domestic supplies in other parts of Australia, it is deemed at least feasible enough to warrant a full-scale feasibility study be conducted.

6.4 FURTHER WORK

There are several items which were excluded from this thesis which could be progressed further into future research for example cost recovery, cost estimation, a better understanding of other losses associated with unlined channels to name a few. Following on from this research there is also scope for review of even more cases studies and a deeper understanding of any systems which did not move ahead or were unsuccessful in their water saving or economic/ socioeconomic benefits.

More research could be done into better understanding of the design process and research into the development of a design and feasibility framework could be devised to assist other irrigation areas or even rural communities understand the process of design development, what water savings could be made and what design options and emerging technologies are available to them.

6.5 SUMMARY

This thesis provides a broad understanding of the major system losses for unlined channels particularly with the St George Irrigation area, it explores high level means of estimating stock and domestic supply demand with a rural context and investigates several case studies of similar projects undertake throughout Eastern Australia although not an exhaustive study. It also provides a high-level concept design and costing of a potential design option as well as potential water savings by upgrading the stock and domestic supply to a pipeline. In the later chapters, feasibility is explored, and commentary is provided which indicates that a full-scale feasibility study is certainly worth pursuing by Mallawa Irrigation.

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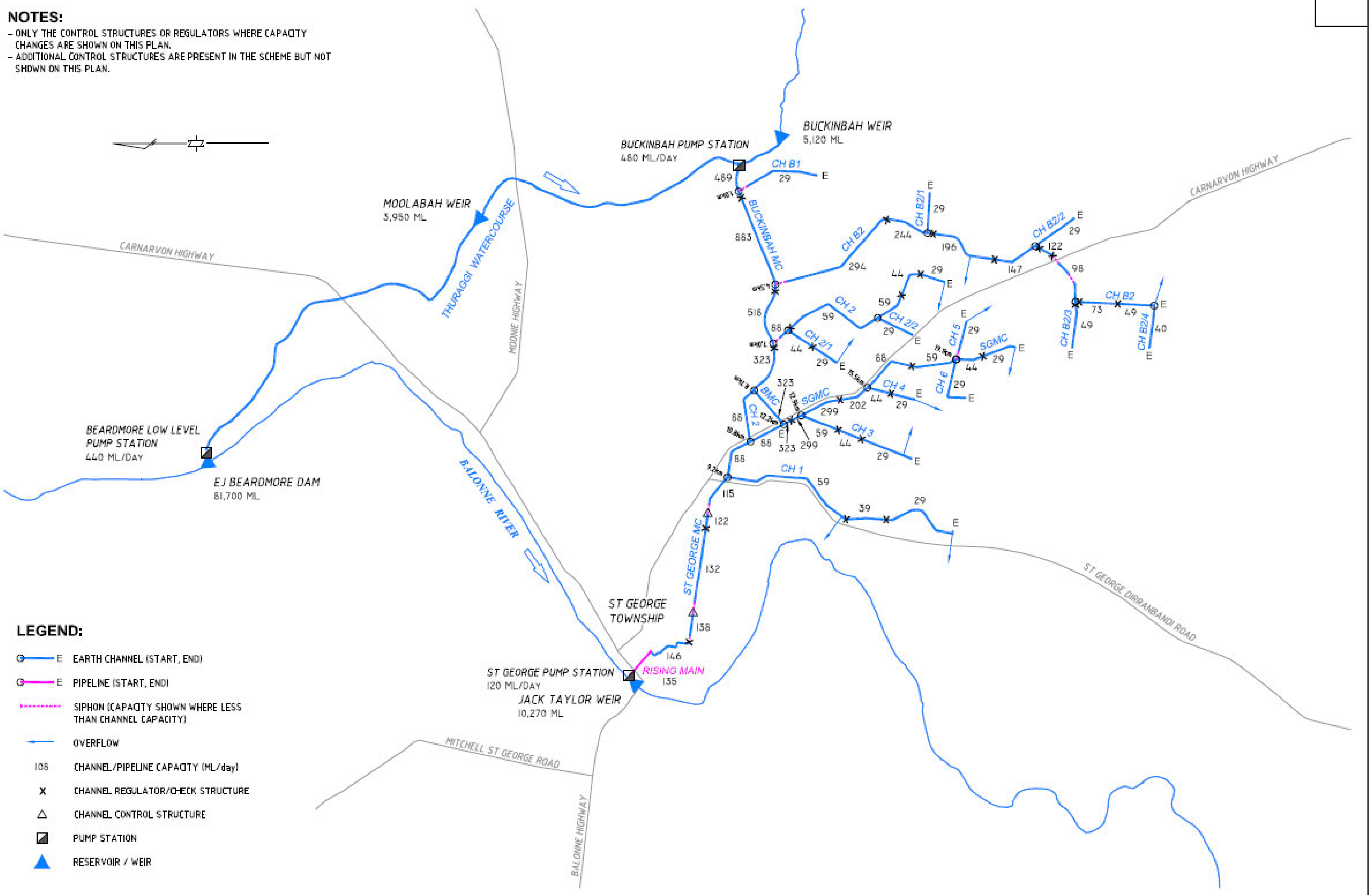
APPENDICES

Appendix A – Channel Capacity Diagram

NOTES:

- ONLY THE CONTROL STRUCTURES OR REGULATORS WHERE CAPACITY CHANGES ARE SHOWN ON THIS PLAN.
- ADDITIONAL CONTROL STRUCTURES ARE PRESENT IN THE SCHEME BUT NOT SHOWN ON THIS PLAN.



LEGEND:

- EARTH CHANNEL (START, END)
- PIPELINE (START, END)
- SIPHON (CAPACITY SHOWN WHERE LESS THAN CHANNEL CAPACITY)
- OVERFLOW
- 106 CHANNEL/PIPELINE CAPACITY (ML/day)
- X CHANNEL REGULATOR/CHECK STRUCTURE
- △ CHANNEL CONTROL STRUCTURE
- PUMP STATION
- RESERVOIR / WEIR

DATE	REV.	REMARKS	CKD	PASSED
27/11/17	B	ISSUED FOR USE	SES	R. THOROGOOD
20/11/17	A	ISSUED FOR REVIEW	SES	--

REFERENCE DRAWINGS

SCALES (A3 SIZE)

NOT TO SCALE

DRAWN TH | DESIGNED
CHECKED SES | CHECKED
APPROVED R. THOROGOOD 27/11/2017

SunWater

ST GEORGE IRRIGATION SCHEME

CHANNEL CAPACITY DIAGRAM

CONTRACT NUMBER

DRAWING NUMBER 249341 | REV. B

SHEET 1 OF 1

DATE NOVEMBER 2017

DRAWING PRODUCED BY: ASSET DELIVERY TEL: (07) 3120 0000

APPENDIX B– CALCULATIONS

Demand Scenarios

				80	offtake points	
				1.2	factor for potential future demand	
				2.57	factor for EP	
				=ABS(E1*E2*E3)	EPs for future demand	
				10000	area of SGIA ha	
				500	estimated area for livestock ha	
				10	DSE (dry sheep equivalent)	
				20	Hour pumping per delivery day	
	AD (L/EP/day)	L/EP/delivery period	ML/Delivery period	Demand/ n days (n, days), kL	Demand/ Supply timeframe, m3/s	Demand/ Supply timeframe* peaking factor m3/s
	Full year Stock and Domestic	AD*EP*365	AD per delivery period /10^6	(Demand/n) * 1000 (n = 7)	Demand/ (pumping hours * second * minutes)	Demand*1.5
Domestic	260	=B11*E4*365	=C11/1000000	=(D11/7)*1000	=E11/(20*60*60)	=F11*1.5
Stock	75	=B12*E6*E7*365	=C12/1000000	=(D12/7)*1000	=E12/(20*60*60)	=F12
Total			=SUM(D11:D12)	=SUM(E11:E12)	=F11+F12	**=G11+G12**
	6 month Stock and Domestic	AD*EP*(365/2)		n = 7		
Domestic	260	=B15*E4*(365/2)	=C15/1000000	=(D15/7)*1000	=E15/(20*60*60)	=F15*1.5
Stock	75	=B16*E6*E7*(365/2)	=C16/1000000	=(D16/7)*1000	=E16/(20*60*60)	=F16
Total			=SUM(D15:D16)	=SUM(E15:E16)	=F15+F16	**=G15+G16**
	3 month Stock and Domestic	AD*EP*(365/4)		n = 7		
Domestic	260	=B19*E4*(365/4)	=C19/1000000	=(D19/7)*1000	=E19/(20*60*60)	=F19*1.5
Stock	75	=B20*E6*E7*(365/4)	=C20/1000000	=(D20/7)*1000	=E20/(20*60*60)	=F20
Total			=SUM(D19:D20)	=SUM(E19:E20)	=F19+F20	**=G19+G20**

Hazen Williams Calculator

	12 mnth delivery 1 pump station	12 mnth delivery 2 pump stations	6 mnth delivery 1 pump station	6 mnth delivery 2 pump stations	3 mnth delivery 1 pump station	3 mnth delivery 2 pump stations	3 mnth delivery 4 pump stations
C<200	100	100	100	100	100	100	100
C>200	110	110	110	110	110	110	110
Q (m3/s)	**=G13**	**=G13**	**=G17**	**=G17**	**=G21**	**=G21**	**=G21**
D (ID,m)	0.5123	0.5123	0.4555	0.4555	0.3658	0.3658	0.3658
D (DN,mm)	**630**	**630**	**560**	**560**	**450**	**450**	**450**
A (m2)	=(PI()*B34^2)/4	=(PI()*C34^2)/4	=(PI()*D34^2)/4	=(PI()*E34^2)/4	=(PI()*F34^2)/4	=(PI()*G34^2)/4	=(PI()*H34^2)/4
V (m/s)	**=B33/B36**	**=C33/C36**	**=D33/D36**	**=E33/E36**	**=F33/F36**	**=G33/G36**	**=H33/H36**
L (m)	**29200**	**=B38/2**	**29200**	**14600**	**29200**	**=F38/2**	**=G38/4**
hf (m)	**=((6.87*B38)/(B34^1.165))*((B37/((IF(B34**	**=((6.87*C38)/(C34^1.165))*((C37/((IF(C34**	**=((6.87*D38)/(D34^1.165))*((D37/((IF(D3**	**=((6.87*E38)/(E34^1.165))*((E37/((IF(E34**	**=((6.87*F38)/(F34^1.165))*((F37/((IF(F34**	**=((6.87*G38)/(G34^1.165))*((G37/((IF(G3**	**=((6.87*H38)/(H34^1.165))*((H37/((IF(H3**
residual head	12	12	12	12	12	12	12
h (m)	**=SUM(B40:B41)**	**=SUM(C40:C41)**	**=SUM(D40:D41)**	**=SUM(E40:E41)**	**=SUM(F40:F41)**	**=SUM(G40:G41)**	**=SUM(H40:H41)**
Max allowable headloss	5	5	5	5	5	5	5
headloss (m/km)	=B40/(B38/1000)	=C40/(C38/1000)	=D40/(D38/1000)	=E40/(E38/1000)	=F40/(F38/1000)	=G40/(G38/1000)	=H40/(H38/1000)
PASS/FAIL	**=IF(B44<B43,"PASS","FAIL")**	**=IF(C44<C43,"PASS","FAIL")**	**=IF(D44<D43,"PASS","FAIL")**	**=IF(E44<E43,"PASS","FAIL")**	**=IF(F44<F43,"PASS","FAIL")**	**=IF(G44<G43,"PASS","FAIL")**	**=IF(H44<H43,"PASS","FAIL")**

Pump Power

	12 mnth delivery 1 pump station	12 mnth delivery 2 pump stations	6 mnth delivery 1 pump station	6 mnth delivery 2 pump stations	3 mnth delivery 1 pump station	3 mnth delivery 2 pump stations	3 mnth delivery 4 pump stations
ρ	0.998	0.998	0.998	0.998	0.998	0.998	0.998
g	9.81	9.81	9.81	9.81	9.81	9.81	9.81
Q	=B33	=C33	=D33	=E33	=F33	=G33	=H33
h	=B40	=C40	=D40	=E40	=F40	=G40	=H40
P (kW)	**=B49*B50*B51*B52**	**=C49*C50*C51*C52**	**=D49*D50*D51*D52**	**=E49*E50*E51*E52**	**=F49*F50*F51*F52**	**=G49*G50*G51*G52**	**=H49*H50*H51*H52**
Pump output (kW)	110	110	90	90	75	90	30
No. pumps/pump station	=ROUNDUP(B54/B56,1)	=ROUNDUP(C54/C56,1)	=ROUNDUP(D54/D56,1)	=ROUNDUP(E54/E56,1)	=ROUNDUP(F54/F56,2)	=ROUNDUP(G54/G56,1)	=ROUNDUP(H54/H56,1)
No. of pump stations	1	2	1	2	1	2	4
Cost per pump (incl. base)	=DOLLAR(14838)	=DOLLAR(14838)	=DOLLAR(14367)	=DOLLAR(14367)	=DOLLAR(12335)	=DOLLAR(14367)	=DOLLAR(7595)
Total Cost of pumps	**=DOLLAR(ABS(B57)*B59)**	**=DOLLAR(ABS(C57)*C59)**	**=DOLLAR(ABS(D57)*D59)**	**=DOLLAR(ABS(E57)*E59)**	**=DOLLAR(ABS(F57)*F59)**	**=DOLLAR(ABS(G57)*G59)**	**=DOLLAR(ABS(H57)*H59)**
Estimated Additional cost per pump station ($250,000)	**=DOLLAR(B61+B58*250000)**	**=DOLLAR(C61+C58*250000)**	**=DOLLAR(D61+D58*250000)**	**=DOLLAR(E61+E58*250000)**	**=DOLLAR(F61+F58*250000)**	**=DOLLAR(G61+G58*250000)**	**=DOLLAR(H61+H58*250000)**

Pipeline Cost

	12 mnth delivery 1 pump station	12 mnth delivery 2 pump stations	6 mnth delivery 1 pump station	6 mnth delivery 2 pump stations	3 mnth delivery 1 pump station	3 mnth delivery 2 pump stations	3 mnth delivery 4 pump stations
Length	=$B$38	=$B$38	=$B$38	=$B$38	=$B$38	=$B$38	=$B$38
Pipe size (DN,SDR11)	=B35	=C35	=D35	=E35	=F35	=G35	=H35
Material cost of pipeline ($/m)	605.61	605.61	477.94	477.94	309.23	309.23	309.23
F1, Allowance for pipe fittings (x 1.1)	1.1	1.1	1.1	1.1	1.1	1.1	1.1
F2, Additional cost. GST+1.2 multiplier for delivery etc. (x1.1x1.2)	=1.1*1.2	=1.1*1.2	=1.1*1.2	=1.1*1.2	=1.1*1.2	=1.1*1.2	=1.1*1.2
Cost of pipeline	=DOLLAR(B67*B69*B70*B71)	=DOLLAR(C67*C69*C70*C71)	=DOLLAR(D67*D69*D70*D71)	=DOLLAR(E67*E69*E70*E71)	=DOLLAR(F67*F69*F70*F71)	=DOLLAR(G67*G69*G70*G71)	=DOLLAR(H67*H69*H70*H71)
Total cost of pumps and pipelines	**=DOLLAR(B63+B72)**	**=DOLLAR(C63+C72)**	**=DOLLAR(D63+D72)**	**=DOLLAR(E63+E72)**	**=DOLLAR(F63+F72)**	**=DOLLAR(G63+G72)**	**=DOLLAR(H63+H72)**