University of Southern Queensland

Faculty of Health, Engineering & Sciences

Optimisation for the Positioning of the Proposed Eurobodalla Southern Storage Future Water Treatment Plant

A dissertation submitted by

Brent Alexander Parker

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Abstract

The importance of a safe and secure water supply is self-evident, as water is essential to sustain life. Australia is known as the driest inhabited continent on Earth and consequently has the highest volume of stored water per capita, emphasising the significance of water infrastructure management and planning. This involves long term integrated strategic planning to determine what infrastructure is required, in what capacity, and where it should be built to meet the demands of all water users.

Integrated planning has shown additional storage is required within the Eurobodalla shire by 2023. Consequently, the Eurobodalla Southern Storage (ESS) was identified as the preferred solution and proposed to be located adjacent the Tuross River and existing Southern Water Treatment Plant. The concept design for the ESS was completed in 2016, which included a preliminary siting assessment for a new water treatment plant (WTP) required post 2030. The concept also identified a water pumping station (WPS) is required to transfer stored water from the ESS to the new WTP and a second to transfer treated water from the new WTP to an existing reservoir for integration into the existing water reticulation network.

Building on the work previously undertaken within the concept, 11 possible positions for the new WTP were identified within a predetermined set of site constraints and assumptions, which included location, pipe properties, water characteristics, and flowrates. A robust model was custom-built using Microsoft Excel to evaluate the hydraulic differences that resulted from each position. These outputs were then converted into monetary terms for net present value analysis. Evaluation of 121 scenarios was executed which included comparison of multiple operating levels within the ESS to determine the optimum position. A knowledge gap was found to exist within recent academic literature on studies for determining the costs of WPSs reinforcing the need for documented academic research to increase the body of knowledge, available in this space for future water resource planners and engineers.

The hydraulic results were as expected with friction and minor losses having minimal impact in comparison to the static head. The NPV analysis was then undertaken for capex, opex and the combined total to determine the optimum solution. The optimal solution recorded the lowest values for the maximum, third quartile, median, second quartile and minimum NPV for all operating levels modelled. These results were found to be a direct outcome from economies of scale, due to the commonality of WPSs within the range of 100 to 1000 kilo-Watts for installed reducing the costs over a 25-year planning horizon.

The outcomes of this project were achieved by determining the lowest cost solution as the optimal position for the future proposed WTP with long term benefit in potential savings for ESC ratepayers from a million dollars upward with the added value of a working hydraulic model with supporting documentation for future investigations to aid water planners and decision-makers alike.

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Limitations of Use

Certification

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

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

Brent Alexander Parker

Student Number: [redacted]

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List of Acronyms

ADWG	Australian Drinking Water Guidelines
AEP	Annual Exceedance Probability
AHD	Australian Height Datum
B/C	Benefit Cost Ratio
Capex	Capital expenditure
CBA	Cost-Benefit Analysis
DAFF	Dissolved Air Floatation and Filtration
DICL	Ductile Iron Concrete Lined
DN	Nominal Diameter
DoI	Department of Industry
ESC	Eurobodalla Shire Council
ESS	Eurobodalla Southern Storage
Flocc	Flocculated particles
FSL	Full Storage Level
GRP	Glass Reinforced Plastic
HDPE	High Density Polyethylene
hr	hours
IWCM	Integrated Water Cycle Management
km	kilometres
km^2	Square kilometres
kW	kilowatts
kWh	kilowatt hours
L	Litres
L/s	Litres per second
LGA	Local Government Area
m	metres

m/s	metres per second
M	Millions
MCA	Multi Criteria Assessment
ML	Mega Litres
ML/day	Mega Litres per day
MOL	Minimum Operating Level
MSCL	Mild Steel Concrete Lined
n/a	Non-Applicable
NOM	Naturally occurring Organic Material
NPC	Net Present Cost
NPV	Net Present Value
NSW	New South Wales
NWTP	Northern Water Treatment Plant
Opex	Operational expenditure
Pty Ltd	Proprietary Limited
PV	Present Value
RL	Reduced Level
SWTP	Southern Water Treatment Plant
TBL	Triple Bottom Line
THMs	Trihalomethanes
TWL	Top Water Level
UV	Ultraviolet
WPS	Water Pumping Station
WSAA	Water Services Association Australia
WTP	Water Treatment Plant

List of Nomenclature

h_o	Pressure Head
B_t	Benefit in year t
C_t	Cost in year t
h	Head in metres
C	Hazen-Williams Coefficient
D	Pipeline Diameter
H	Terminal Head
L	Pipeline Length
Q	Flow rate
R	Reynolds Number
Sf	Friction Slope
V	Velocity
f	Frictional Factor
g	Gravitational Acceleration
i	Interest Rate
ε	Pipe Roughness Coefficient
k	Minor Head Loss Fittings Coefficient
t	Time in years
v	Viscosity
π	Pi

1 Introduction

1.1 Background

1.1.1 Water Resources Supply Management

Australia is known as the driest inhabited continent on Earth and consequently also has the highest volume of stored water per capita (Petheram & McMahon 2012). Water is essential to sustain life and in Australia safe potable water is utilised daily for consumption, food preparation, cooking, bathing, washing, garden, lawn irrigation and other domestic requirements (ADWG 2011). The water demand for these requirements needs to be balanced with those of the natural environment from where the water is removed. In addition, they must also be shared with any agriculture, industry, recreation or power generation water users within the water catchment (Petheram & McMahon 2012).

These water users all have different water quality input requirements and depending on the activities undertaken will result in differing water quality outputs and residuals. Both of which, if not regulated and managed correctly can impact on environmental quality and public health (Page 2001). For this reason, the removal of water from the natural environmental and what is returned needs to be managed responsibly in a sustainable manner to protect current and future generations of water users.

Consequently, this presents an interface where the natural and built environments collide. Figure 1, on the next page displays this interface for a typical catchment where water is collected, treated, distributed, utilised by the consumer and returned into the natural environment. Whilst, highlighting some of the impacts that human activities have on the water quality of the catchment.

1.1.2 Water Supply Infrastructure and Management

The management of water supply infrastructure and associated water resources involves long term integrated strategic planning to determine what infrastructure is required, in what capacity, and where it should be built to meet the demands of water users over the duration of the asset life (Furlong et al. 2016). This is achieved by analysing population growth, planning for future water demands required and identifying suitable water infrastructure options that can satisfy these demands.

Further work is then required to develop concepts and detailed designs followed by the procurement and management for construction of the water infrastructure. Finally, once commissioned the infrastructure then requires operation and maintenance for the life of the asset. Additionally, to obtain best value from the infrastructure, periodic performance and capacity reviews can determine when future augmentation or optimisation is required enabling water infrastructure asset owners to maintain agreed levels of service to their consumers (American Water Works 2014).

Water infrastructure is well defined by Furlong, et al. (2016, p.2), who stated:

> *"Water infrastructure includes the physical structures that capture, hold, treat, and transport fresh/potable water, wastewater and stormwater."*

This definition is broadly focused to include the full spectrum of the water industry. However, for the purpose of this dissertation study area, this will be limited to:

> *"…the physical structures that capture, hold, treat and transport fresh/potable water…"*

Consequently, the planning and provision of these structures requires transparent and robust decision-making processes. This can be achieved by using infrastructure planning frameworks to help guide decision makers through the processes and steps to identify suitable options which are desirable to all stakeholders (Furlong et al. 2016). Additionally, Page (2001) detailed how providing a safe supply of potable water is a crucial component for achieving sustainable growth in any regional area. Further highlighting the importance and influence of water infrastructure decisions that have the ability to limit or encourage growth. However, it is evident that these decisions can be politically influenced due to the large number of stakeholders and involvement of different levels of government through shared funding (Furlong et al. 2016).

Figure 1: The Humans Effect on the Water Cycle (2019)

1.1.3 Eurobodalla Shire Council - Local Government Area

In Australia, water supply infrastructure is managed by either water utility corporations, state or local water authorities. In regional NSW this responsibility generally resides with local government councils, such as the Eurobodalla Shire Council (ESC). Other responsibilities of ESC include the management of town and social planning, cultural development, libraries, sewerage schemes, waste collection, community services, roads and recreation facilities (ESC 2017). Figure 1 below shows the local government area (LGA) that ESC is responsible for along with the major townships, suburbs and villages. Located on the NSW South Coast the LGA consists of 3400 km² of land that extends along 110 km of coastline from Durras Lake in the north to Dignams Creek in the south (Hydrosphere Consulting 2016). The main administration building is centrally located in the township of Moruya, approximately 313 km south of Sydney and 175 km east of Canberra.

Figure 2: Location Map of Eurobodalla LGA (ESC 2016)

1.1.4 Eurobodalla Shire Water Supply Scheme and Infrastructure

In recent years, the Eurobodalla shire water supply scheme has supplied over 3,000 ML per year to approximately 20,000 connected properties extending 90 km north from Maloneys Beach to Mystery Bay in the south and includes the major townships of Batemans Bay, Moruya and Narooma. The main water supply for the shire is sourced from the Moruya River and pumped to Deep Creek Dam for off-stream storage which has a capacity of 4,900 ML. This water is then treated at the 20 ML/day Northern Water Treatment plant (NWTP) and distributed into the water reticulation network via 33 service reservoirs containing 114.7 ML of storage and 745 km of water mains (ESC 2019b).

The population shire-wide can triple with an influx of tourists and holiday home owner's migrating towards the coast in the warmer months (ESC 2017). The water supply scheme can be supplemented in these periods of high demand by extracting groundwater from 5 alluvial bores located near the Tuross River in the southern part of the shire. The bore water receives treatment at the 6 ML/day Southern Water Treatment Plant (SWTP) before distribution. Whenever, both WTPs are in operation the scheme is separated into two sub-systems, itemised below (ESC 2019a).

The northern sub-system supplies potable water from Maloneys Beach in the North to Tuross Head with water infrastructure including (ESC 2019b):

- Moruya River Intake;
- Deep Creek Dam;
- NWTP;
- 22 service reservoirs;
- 8 pumping stations; and
- 535 km of water mains.

The southern sub-system supplies potable water from Bodalla to Mystery Bay and Central Tilba in the southern extremities of the LGA with water infrastructure including (ESC 2019b):

- Tuross River bores;
- SWTP;
- 11 service reservoirs;
- 3 pumping stations; and
- 210 km of water mains.

1.1.5 Eurobodalla Southern Storage Project

In 2012, a review of the local water sharing plans and the Integrated Water Cycle Management (IWCM) Strategy demonstrated the requirement for a second storage within the shire by 2023. The Eurobodalla Southern Storage (ESS) was identified as the preferred option with a proposed capacity of 3,000 ML and in 2016 ESC engaged SMEC Australia Pty Ltd to undertake the concept design for the proposed water storage including ancillary works. The works included a siting options assessment for a future water treatment plant (WTP), required post 2030 at a nominated capacity of 25 ML/day.

Further concept development also identified the need for an ESS outlet pumping station to transfer stored water to the new WTP; and a second pumping station to transfer treated water from the future WTP to an existing concrete reservoir for supply into the ESC water reticulation network (SMEC 2016a). Figure 2 shows the proposal overview located approximately 8 km west of Bodalla, adjacent the existing Tuross River bores and SWTP. Further detail regarding the concept design of the ESS and ancillary works are discussed later in Chapter 3.

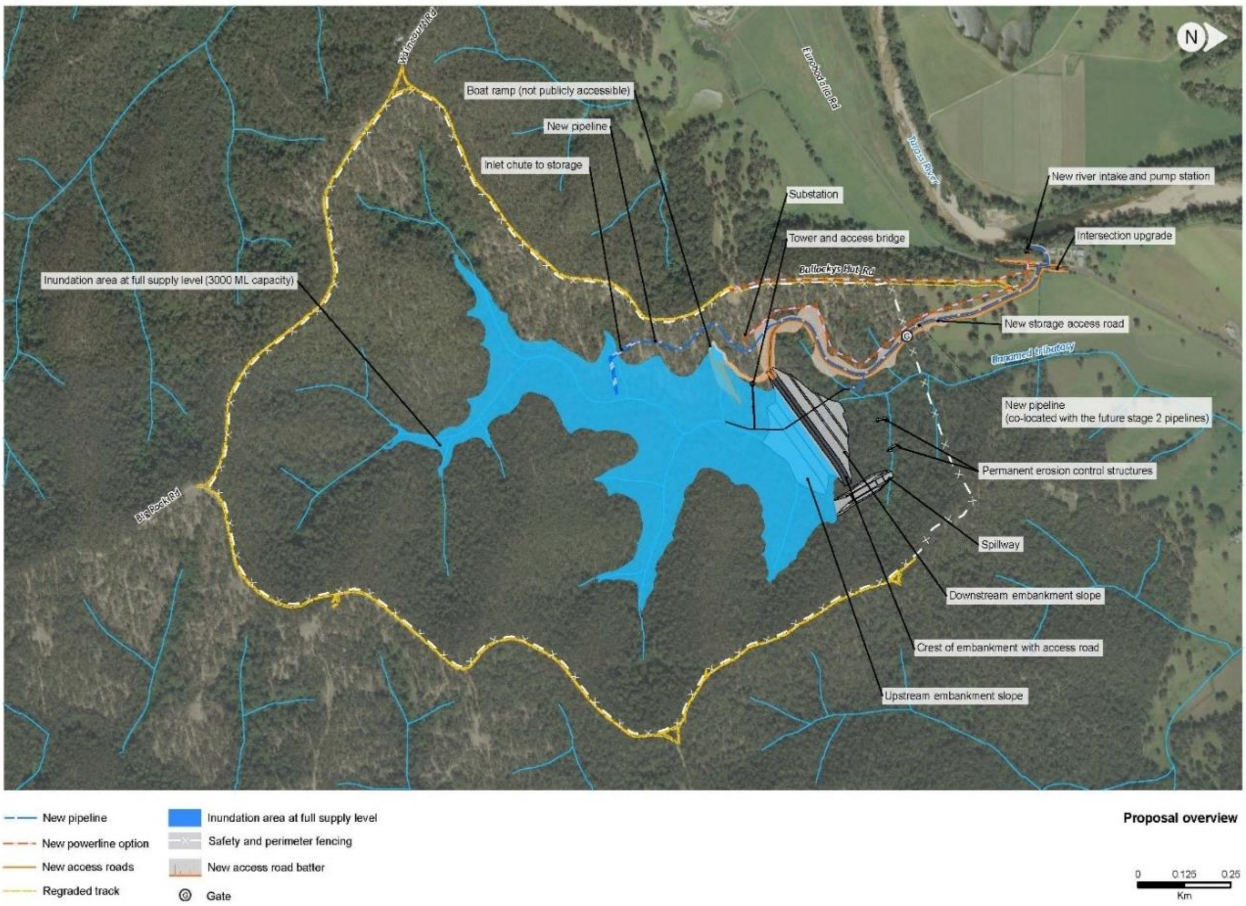


Figure 3: Eurobodalla Southern Storage Proposal (SMEC 2016a)

1.2 Project Aims and Objectives

1.2.1 Aims

The proposed study will aim to research, develop and implement a hydraulic model to optimise the positioning and configuration of the proposed Eurobodalla Southern Storage future water treatment plant.

1.2.2 Objectives

1. Identify the design variables, site constraints and assumptions required to:

    a. develop and build the hydraulic model;

    b. determine the possible positions for the future water treatment plant; and

    c. determine the most appropriate method for evaluating the configuration considerations that significantly impact on costs associated with potable water supply systems.

2. Develop and build model using Microsoft excel capable of evaluating the hydraulic differences identified due the positioning and configuration of the future water treatment plant.

3. Identify the optimum position and make a recommendation for future work.

1.2.3 Outcomes and Benefits

The outcome of this project if successful shall provide the ESC with an optimal position for the future proposed water treatment plant with supporting documentation. The benefit if successful shall provide ESC with a working model and evaluation information to use for further investigations and decision-making processes. Long term benefit shall also be received by the ratepayers of ESC should the optimal position be adopted. This would be in the form savings by offering a least cost solution.

2 Literature Review

2.1 Introduction

The purpose of this literature review is to identify the configuration considerations that significantly impact on costs associated with potable water supply systems. Accompanied by researching the most appropriate methods for evaluating a given system. However, to adequately identify these considerations we must first obtain a firm understanding of what constitutes a potable water supply system.

2.2 Defining Potable Water Supply Systems

A potable or safe drinking water supply system can be defined as everything from the point of water collection through to the consumer's tap (Howe 2012). This can include entire water catchments and groundwater systems; source waters; storage dams and reservoirs; intakes and water pumping stations; water treatment plants; service reservoirs and distribution systems; and finally the consumer (ADWG 2011). The following sub-headings of the section aim to briefly provide adequate background on each of the components of a potable water supply system and identify any influencing factors for further discussion in later sections of the dissertation.

2.2.1 Source Water and Storage

The collection of water for treatment is achieved by capturing or pumping water from an available source, this is typically referred to as raw water and can include groundwater sources located below the water table from aquifers or surface water sources such as streams, rivers, lakes and other open water bodies (Pizzi 2010). Alternatively, source water can also include sea, storm and wastewaters. However, these alternatives may require additional treatment processes to make them suitable for human consumption.

Capture of raw water typically refers to on-stream storage. This is when a dam wall or weir is built on a river, stream or lake to retain a certain amount of water in storage that would otherwise pass downstream to the lower parts of the catchment. On-stream storages commonly installed for flood control measures but are still prone to adverse weather conditions within their catchment and in many cases, failures occur due to dam wall breaks and overtopping. This results in a very high risk for the natural or built environments and their inhabitants residing downstream from any major on-stream storage (ANCOLD 2013). In contrast, off-stream storage is when water is pumped from a water source to a dam or reservoir located “off-stream” from the catchment. This allows for selective pumping from the water source to maintain environmental flows downstream. Whilst, maximising the use of water sharing plans and the ability to pump during times when good quality water is available (Petheram et al. 2016).

2.2.2 Water Treatment

The principal concept of water treatment is to remove, inactivate, or modify undesirable constituents or contaminates from raw water. To produce an aesthetically pleasing potable water safe for human consumption which complies with regulatory standards (Pizzi 2010; Howe 2012).

Pizzi (2010, p. 1) further quantifies this concept by stating:

“To be acceptable for human consumption, water must be free of harmful organic, inorganic substances, radionuclides and organisms capable of causing disease.”

This is an important statement as it concludes that not all sources waters are equal or require the same treatment processes to render them safe for drinking. The statement also acknowledges that water must be free of any harmful chemicals or by-products that may have been used or formed during treatment. This is definitively stated by the Australian Drinking Water Guidelines (ADWG 2011, p. 6):

“Drinking water should not contain harmful concentrations of chemicals or pathogenic microorganisms”.

To ensure harmful substances are below acceptable standards the following processes are used in conventional water treatment as shown in Figure 4. These typically include: screening; pH control and water stabilisation; oxidation and pre-disinfection; coagulation and flocculation; sedimentation and filtration; post-disinfection and clear water storage prior distribution into the network.

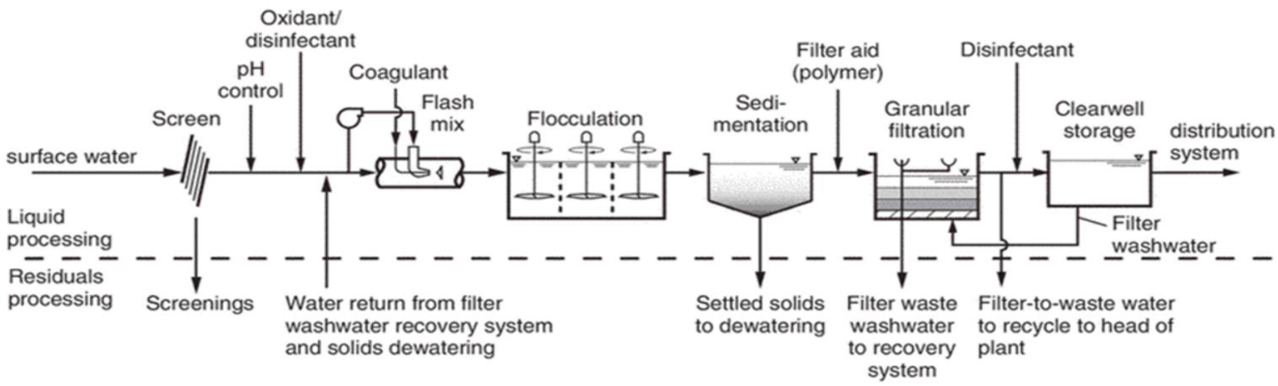


Figure 4: Conventional Water Treatment (Howe 2012)

The screening of large debris from the raw water which is particularly important for surface waters that may contain large amounts of organic and inorganic materials which have been carried into the water source from runoff. Control of the pH is needed to ensure raw water is within the required pH range for later treatment processes. This may include the addition of chemicals to either raise or lower the pH or stabilise the water prior to distribution.

Oxidation and pre-disinfection is mainly required when large concentrations of dissolved metals are present in the raw water such as, iron and manganese, which can be oxidised into suspended and settable matter for removal at later stages of treatment. When pre-disinfection used naturally occurring organic material (NOM) can form harmful disinfection by-products such as Trihalomethanes (THMs). NOM are more commonly present is surface waters opposed to groundwater sources due to their open exposure. THMs are suspected to be carcinogenic and are typically formed when utilising chlorine or its compounds for disinfection. However, chlorine still remains the most widely used disinfection in water treatment due to its low cost and availability (ADWG 2011; Ohar & Ostfeld 2014).

The coagulation process involves the addition of a coagulant chemical which is rapidly mixed with the raw water to encourage fine suspended matter such as colloidal particles (typically clay) to form larger aggregates bound together. This process can be used separately or in conjunction with flocculation, which is the process of slowly mixing coagulated particles to form long chain molecules that are easier to remove. This is achieved by mixing at progressively slower speeds in a series of flocculation tanks (Howe 2012).

Sedimentation and filtration are processes that are required to separate the flocculated particles (flocc) from the water with both methods using gravity for separation. Sedimentation also called clarification allows the heavier settable solids and flocc to sink to the bottom of the tank, where it is collected and removed for further treatment (Pizzi 2010). Whereas, during the filtration process, granular media (typically sand) is used to capture the solids and flocc. While the filtered water gravitates to a chamber underneath the filter which is then stored in the clear water tank prior to disinfection. The filters require periodically backwashing to remove the captured flocc from the media. This is achieved by pumping treated water from the clear water storage back through the filter to carry unwanted materials upward into overflow channels for removal from the filters for further treatment. While the heaver granular media is retained within the filter (Howe 2012).

The final treatment process typically involves the disinfection of the filtered water with strong oxidants such as chlorine, prior to storage and distribution. As discussed previously, chlorine and its compounds are the most common disinfection method used in conventional treatment. Alternatively, disinfection using ultraviolet (UV) light is becoming increasingly popular across the industry. UV light when applied at the correct dosage inactivates pathogenic microorganisms and requires less contact time then chlorine dosage (Pizzi 2010).

However, disinfection of the filtered water is required for two reasons. Firstly, to prevent harmful organisms and viruses and from entering the distribution system. Secondly, to provide a disinfection residual within the potable water to prevent re-contamination post treatment (Ohar & Ostfeld 2014). Unfortunately, UV light does not provide a residual and therefore common practice for adequate protection of drinking water still includes the addition of chlorine.

Contamination post-treatment can originate from biological re-growth, corrosion of pipes, water main breaks or back-flow problems within the distribution network. Another common source is uncovered or poorly maintained service reservoirs which can be exposed to animals (ADWG 2011). Chorine residuals need to balance between the health and aesthetic recommendations of the statutory regulations. As chlorine concentrations can cause ill health or taste and odour complaints from consumers. For these reasons, many water utilities may include addition chlorine dose points throughout the distribution network to maintain consistent low concentrations of chlorine residuals in the system (Ohar & Ostfeld 2014).

Additionally, many water treatment processes produce residuals and by-products that require further treatment prior to disposal. For the treatment processes discussed above, these include inorganic and organic screenings, settled solids, filter waste, and backwash water. All of which, can receive further treatment on-site allowing any reclaimed water to be recycled back into the treatment process. Alternately, these items can be treated and disposed off-site potentially increasing operational costs through transportation of residuals. More commonly, settled solids and filter waste are thickened and mechanically dewatered resulting in a sludge residual which requires off-site disposal in smaller volumes then the latter option (Pizzi 2010).

In summary, there are several competing factors which determine what treatment processes or combinations of processes are required for adequate water treatment. These factors include the raw water quality and the desired finished potable water quality, the capital and operating costs, the footprint requirements for each process, land availability, and the available methods of disposal of treatment residuals (Pizzi 2010).

2.2.3 Water Distribution and Transport

Water distribution and transport within a potable water supply system can be defined as the movement of water by energy to satisfy the water demands of the system. This includes a complex network of pipelines (water mains), pipe fittings, valves, water pumping stations and service reservoirs (Mahar & Singh 2014). The following section provides further detail on each of these items. Water mains can be broadly grouped into three categories of trunk mains, distribution mains and services mains. Trunk mains are used for the transfer of bulk water between water treatment plants, service reservoirs or adjoining water supply schemes.

Distribution/reticulations mains supply potable water from service reservoirs into communities or supply areas and service mains branch off the distribution mains to provide water to the consumer's water meter. Many different valves are also required within the distribution network for a range of purposes. These include maintaining pressure, reducing pressure, automatic operation and complete isolation of water infrastructure for repair or maintenance (Savic & Banyard 2011).

Water pumping stations (WPSs) are required to transfer water from low to higher elevations or maintain service pressure within the distribution network. They operate by increasing the water pressure into a pipeline using a pump to achieve a static lift while overcoming any head losses within the system. Head losses are mainly caused by friction from the pipelines, pipefittings and valves which is commonly referred to as friction loss (Moreira & Ramos 2013). Pumps in water supply systems are typically powered by electric motors run by mains power. Alternatively, water can be transferred by potential energy from stored water located at a higher elevation then the delivery point. When required the potable water can then be released to gravitate through the distribution network.

Service reservoirs are typically built from concrete or metal and are required for the several reasons. Firstly, to allow adequate volumes of potable water to be stored to avoid supply interruptions from power outages, trunk main breaks or to allow operational maintenance to occur. Secondly, to balance the user demands with pumping requirements allowing water to be pumped outside peak times for power demand resulting in lower operating costs. Thirdly, to provide water for firefighting purposes (ADWG 2011; Savic & Banyard 2011).

2.3 Configuration Considerations that Significantly Impact on Capital and Operational Costs

The configuration of a potable water supply system for the purpose of this dissertation is best described as the spatial differences between the water infrastructure items within the system. In respect to distance and elevation this is represented by the pipeline length and height that water is required to be transported or pumped to satisfy the water supply demands of the system. A third factor that should also be considered is the storage capacity at different points within the system, as storage capacity directly impacts the time required for pumping within the system.

This relationship between length, height and storage is visually defined by Swamee (2001, p. 265) as shown in Figure 5 and Equation 1. When minor losses are excluded the pressure head (h_o) in metres is determined by multiplying the pipeline length (L) and fiction slope (S_f) which is the friction head loss divided by the pipe length, with the addition of the elevation difference between water storages and the terminal head (H).

$$\boldsymbol{h_o = L\, S_f + \Delta Z + H} \qquad \textbf{[Eq. 1]}$$

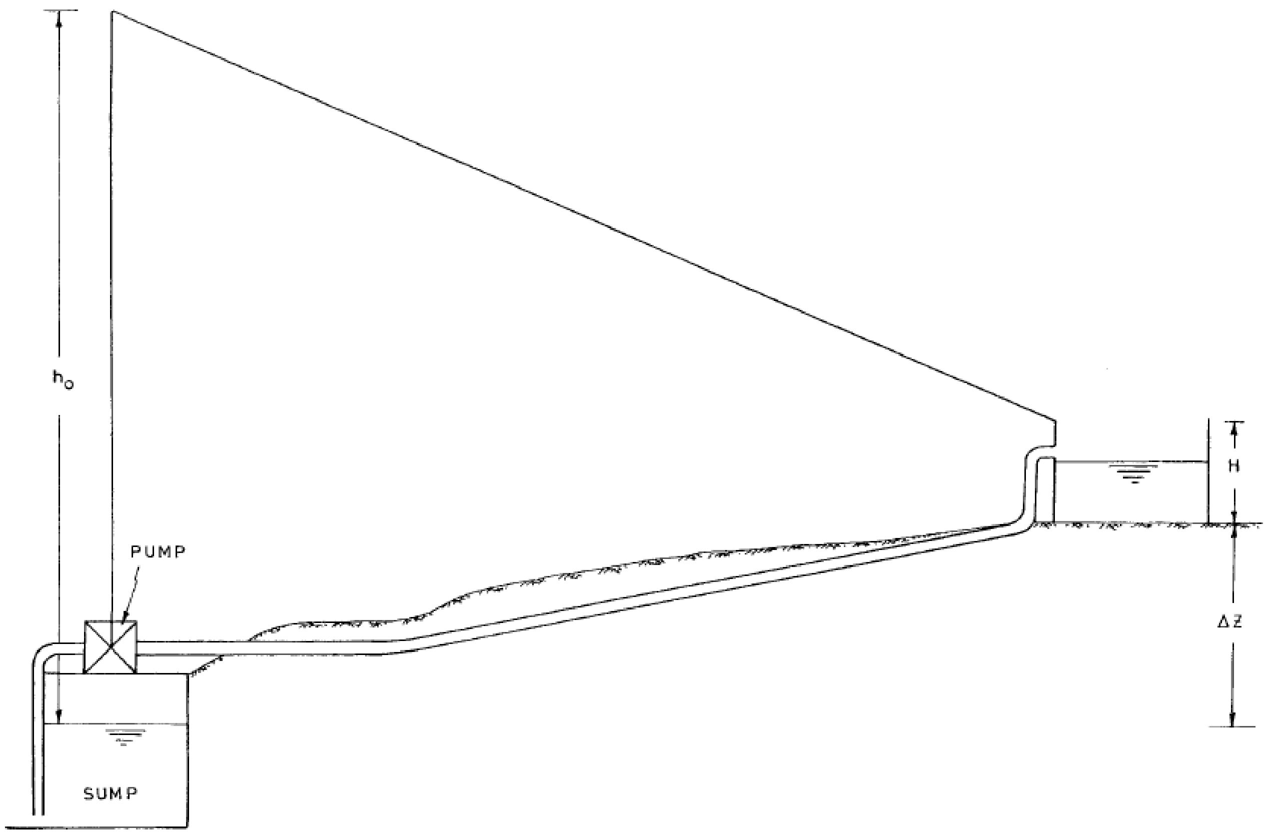


Figure 5: Relationship between length, height and storage (Swamee 2001, p. 265)

However, when local losses (h_L) caused by the change in the cross-sectional area at fittings and valves within the pipeline (Nalluri 2009). Equation is therefore expanded to include these additional losses, as shown in Equation 2:

$$\boldsymbol{h_o = L\,S_f + \Delta Z + H + h_L} \qquad \textbf{[Eq. 2]}$$

A fourth factor of water quality is also evident from the initial discussion and therefore included in the review due to its relationship with distance and elevation within a water catchment. The relationship between these factors and the significant impacts that each has on capital and operational costs associated with potable water supply systems is discussed in further detail below. However, it would be prudent to first discuss and define capital and operational costs in the context of a potable water supply system.

2.3.1 Defining Capital and Operational costs

Capital costs or expenditure (capex) are one-time costs that provide benefit over a number of years, for which the capex is depreciated. In contrast, operational costs (opex) are the day to day expenses required for the operation budgeted over one financial year, which therefore reoccurs annually.

For potable water supply systems, capex comprises of acquisition of land, construction of new infrastructure or the augmentation of existing, including the purchase of all equipment required for the proving of operation during commissioning of the asset. Whereas, opex includes servicing, maintenance, repairs and operating costs for all the water infrastructure within the system after commissioning (Ormsbee & Lansey 1994).

2.3.2 Distance – Pipeline

Pipeline distance or length as defined in Equation 1 is the horizontal length between water storages or infrastructure and is subject friction losses. Friction in a pipeline needs to be overcome to allow the water within the pipe to be transferred. Friction slope (S_f) is calculated by Swamee (2001, p. 265) by utilising the Darcy-Weisbach show below as Equation 3, where (f) is a frictional factor, (Q) is the flow, (g) is the gravitational acceleration and (D) is the pipeline diameter.

$$\boldsymbol{S_f = \frac{8fQ^2}{\pi^2 g D^5}} \qquad \textbf{[Eq. 3]}$$

However, other studies have utilised alternative equations for friction such as Hazen-Williams and Colebrook-White formulas. The Hazen-Williams equation is shown as Equation 4 (Chadwick 2013, p. 121). Where velocity (V) is determined from the multiplication of (C) a coefficient that has been determined through experimental testing, (D) the pipeline diameter and (h_f) is the friction head loss over the pipeline length (L), which can be rearranged to make friction head loss the subject of the equation as shown in equation 4 (Chadwick 2013, p. 121).

$$\boldsymbol{V = 0.355CD^{0.63}\left(\frac{h_f}{L}\right)^{0.54}} \qquad \textbf{[Eq. 4]}$$

Hazen-Williams Equation 3 rearranged,

$$\boldsymbol{h_f = \frac{6.78\,L}{D^{1.165}}\left(\frac{V}{C}\right)^{1.85}} \qquad \textbf{[Eq. 5]}$$

The use of Hazen-Williams has been popular due to the availability of ready-made answers, achieved by using a constant to determine the roughness if the internal surface. However, it is based on a limited database for the determination of the C coefficient and when applied outside of the database range is prone to errors (Swamee 2001; Travis & Mays 2007).

In contrast, the Colebrook-White equation as detailed by (Liou 1998) and displayed as equation 6 is implicit and can be applied to a wider range of pipe sizes and flows if turbulent, smooth or transitioning and utilises a coefficient as a measure of pipe roughness (Travis & Mays 2007; Jones 2008). Where (f) is a frictional factor, (ε) is the roughness coefficient, (D) the pipeline diameter and (R) is the Reynolds number.

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{\varepsilon}{3.7D} + \frac{2.51}{R\sqrt{f}}\right) \qquad \textbf{[Eq. 6]}$$

The Reynolds number is defined by Christensen et al. (2000) as Equation 7, where (V) is the velocity, (D) the pipeline diameter and (v) is the viscosity.

$$R = \frac{VD}{v} \qquad \textbf{[Eq. 7]}$$

Swamee (2001) suggests by adopting a velocity between 0.6 m/s to 0.75 m/s for the maximum flow required to be delivered allows for a self-cleansing velocity. The Darcy-Weisbach equation (Eq. 3) can be used to determine the minimal friction factor for the given parameters of the system. This was then substituted into the Colebrook-White equation and solved iteratively until the pipe diameter is found. Swamee (2001) further determined the lowest head pumping requirement which allowed the most efficient pump to be selected for the given system reducing the ongoing operational costs. This is supported by Mahar and Singh (2014) who reported as the diameter of the pipeline increases so does the capex. However, this also results in a decrease in the opex as the energy costs required to overcome the pipeline friction also decrease.

Furthermore, Mahar and Singh (2014) suggested many studies has been undertaken to optimise water supply systems by improved efficiencies within the distribution component of the system. Alternatively, their research utilised a nonlinear model to determine the optimal diameter for the discharge pipeline based on a defined set of pump characteristics including discharge, static head, and other economic parameters. This alternative solution demonstrates the diverse nature of the pumping discharge relationship in which any of the variable parameters can be altered or fixed depending on the situation for which it is applied to optimise.

2.3.3 Elevation – Pumping

As identified in the previous section, pumping between elevations within a potable water supply system can occur at serval locations including between extraction and the distribution system. This common occurrence highlights the importance of being able to accurately determine the costs associated with the WPS. As discussed earlier, efficient pump operation can reduce energy usage, which also reduces the physical wear of the pumping machinery further decreasing opex costs. This is achieved through the correct pump selection for a given pipeline system. Higher elevations also increase the pressure and wear on pumping machinery causing an increase in maintenance costs relative to the pumping head (Shiels 1998). Furthermore, it is noted that a typical medium sized industrial pump maintenance cost is 2.5 times the initial capital costs over the full life cycle of asset (Hydraulic Institute & Energy's Office of Industrial Technologies 2001).

Firstly, a system characteristic curve is obtained graphically by applying the pipeline characteristics in diameter and roughness into an appropriate equation to calculate the different flowrates and total head pressure in metres (Moreira & Ramos 2013). Swamee (2001) utilised the Darcy-Weisbach and Colebrook-White equations to achieve this, whereas others (Travis & Mays 2007; Moreira & Ramos 2013) utilised the Hazen-Williams approach. The pump characteristic curve displays test data for the performance of a pump at different flowrates with corresponding pressure head can be overlaid to select the most suitable pump (Moreira & Ramos 2013).

Walski (2012) reviewed cost functions that had previously been developed in an attempt to develop a generic cost function to allow water planners and engineers to quickly obtain a reference cost, when planning for the construction of water pumping stations. However, the study was limited to construction costs related with hydraulic design which excluded costly items such as land acquisition, getting power to site, engineering inspections, costs associated with obtaining permits, legal fees and ongoing administration.

Interestingly, Walski (2012) suggested a knowledge gap exists within recent literature on studies for determining the cost of pumping stations. Furlong et al. (2016) suggests this is because water planning research is often found in the public sector reports rather than academic literature. Reinforcing the need for well documented research and academic documentation to increase the available body of knowledge available for water resource planners and engineers. Evidence of this trend can be seen in the NSW Reference Rates Manual, which is used for the valuation of water supply, sewerage and stormwater assets. This document is made available to water utilities in NSW and contains collated data based on the actual capital costs for water infrastructure projects built in NSW (DoI Water 2014).

Walski (2012) reported that costs are most dependant on flow, which explained why many investigations have been undertaken utilising flow as a function of cost. Furthermore, that flow was a significant determinate of cost for WPS buildings, pumping machinery and motors required. Other studies have investigated cost functions for both head and flow by dividing up the cost implication of the major components of the WPS and summing to formulate a total construction cost with the mechanical and electrical components the most important.

In contrast, minimal studies had attempt to create a cost function. Unfortunately, no single equation was found that applies to all cases. However, (Walski 2012) did formulate a solution the allows a coefficient to be determined after taking into account localised construction costs including those associated with site conditions, labour, materials, water quality, type of pumping unit, prefabricated vs in-situ design and SCADA equipment. This statement is further supported by (Mahar & Singh, 2014) whom reported the direct impact on capital costs is caused by the discharge and pressure head requirements.

2.3.4 Storage – Balancing within the system

Pumping of potable water was determined by Ormsbee and Lansey (1994) as a major user of energy in any water utilities operational budget. Đurin (2016) suggests that two methods exist for the optimisation of operational water pumping costs by either modifying the pumps themselves or the pumping patterns. Moreira and Ramos (2013) echoed these points and further detailed savings can be made by investing capital into new more efficient pumps and motors. Alternatively, savings can be achieved by altering the pumping procedures with the latter described as requiring no capital investment with potential for immediate savings.

This is made possible as power suppliers typically include a consumption charge based on kilowatt-hours (kWh) during the billing period. An additional demand charge is for usage availability commonly called a tariff is also applied. Tariff charges are typically separated into times of peak, off-peak and shoulder tariff throughout a 24hr period. Similarly, as with peak water demand, power supply utilities must also allow for availability of peak energy demand. Otherwise inadequate supply of energy can result in failures in the energy distribution network causing blackouts. This means by using the available storage within the water supply system utilities can pump during off peak periods at the lower tariff usage charges. Other solutions typically include decreasing volume pumped, decreasing the system head or correct pump selection to maximise pumping efficiency (Ormsbee & Lansey 1994).

Ormsbee and Lansey (1994) reviewed the use of algorithms for optimal control of a water supply systems. This is achieved by utilising a computer model of a system with a defined set of boundary conditions and constraints used to determine a least-cost operational policy. The model applies a set of rules to schedule the pump starts and stops, while attempting to meet the water demands within the system. The computer model then allows all scenarios to be evaluated with the least-cost scenario becoming the solution. This method is totally dependent on the constraints of the system and therefore requires individual customisation for the hydraulic characteristics, demand forecasts and rules applied for each particular system.

Đurin (2016) completed an optimisation case study in Demark which investigated balancing the water supply and demands by modelling three different pumping scenarios. Pumping at a fixed flow continually over 24hrs, pumping only during off-peak hours between 10 pm to 6 am and thirdly pumping between 6am to 6pm. The first two scenarios used energy from the grid and the third utilised photovoltaic solar power for supplied energy. The constraints of the model included the pump capacities, reservoir volumes, pump starts and stops controlling the duration of pumping. The study resulted in scenario one required lower pump flowrates and storage capacities were required. This would theoretically reduce the capital cost when constructing lesser capacity reservoirs and smaller pumps. In contract, scenarios two and three yielded similar results between the off-peak and solar powered pumping reducing operational energy costs. However, Walski (2012) reported that costs are more sensitive to flow due to the large costs involved in pumping station equipment and construction.

2.3.5 Water Quality

As discussed in previous sections, the water quality determines which processes are required for adequate treatment. Moreover, water quality is dependent on site selection in respect to the influence of upstream activities. Poor urban, industrial and agriculture practices can allow excess sediments, nutrients and pollutants into the source waters causing problems such as algae growth and increase the risk of pathogenic organisms (ADWG 2011). These practices can increase the footprint required to accommodate for additional treatment process units increasing the capex. Notably, when applying the same constraints to opex this also increase costs, due to increased operational requirements of the additional units.

In contrast, by selecting a water extraction site further upstream in the water catchment can reduce the capex and opex costs by avoiding contamination of the source water. However, this may result in additional transportation requirements if located some distance from the WTP or distribution areas. Apart from the additional pipe lengths required during construction causing an increase in capex. This may also include additional transportation for all the construction materials and contractors. Furthermore, once in operation this would extend to include treatment chemicals to site, treatment residuals off site and operational staff day-to-day travel time.

2.4 Assessment of Water Infrastructure Options

The purpose of this section is to discuss and compare the assessment options available in the literature for water infrastructure projects, in order to later determine the most appropriate method for evaluating the capital construction and operational costs for this dissertation. The Net Present Value is firstly defined and discussed as it is commonly used across the multiple assessment methods. This followed by cost benefit ratio, triple bottom line and multi criteria analysis with the adopted assessment option detailed in the methodology chapter.

2.4.1 Net Present Value

Before a decision can be made on whether to start a project, it is necessary to estimate future net cash flow by accurately estimating income and expenses that will be generated by the project over its lifetime (Vladimir, 2010). One of the most widely used techniques for comparing the financial benefits of long-term projects is net present value (NPV) analysis. NPV is also commonly referred to in the literature as net present cost (NPC) and is defined by Dandy et al. (2007) in mathematical terms, as shown in Equation 6. Where (t) is the time period in years, (n) is the life of project in years, (B_t) is the benefit in year t, (C_t) is the cost in year t and (i) is the interest rate.

$$\boldsymbol{NPV = \sum_{t=0}^{n} \frac{B_t}{(1+i)^t} - \sum_{t=0}^{n} \frac{C_t}{(1+i)^t}} \qquad \textbf{[Eq. 6]}$$

Furthermore, Maurer (2009) states that for non-competitive markets, such as those experienced by water authorities. The NPV can be defined as present value (PV) of capex and opex, as shown in Equation 7, due to the income (benefit) not being independent to the costs with the net resulting as zero.

$$\boldsymbol{NPV = NPV_{Capex} + NPV_{Opex}} \qquad \textbf{[Eq. 7]}$$

Additionally, when costs are shown to be a series of annual payments the NPV calculation can be further simplified by using a discount factor to bring the annual payments back to present value (Dandy et al. 2007). This shown as below as Equation 8, Where (P) is the present value of the annual payment, (A) is the reoccurring annual payment, (i) is the interest rate and (n) is the life of the project in years.

$$\boldsymbol{P = A\left[\frac{1-(i+1)^{-n}}{i}\right]} \qquad \textbf{[Eq. 8]}$$

Moreover, the life of the project can be defined as the planning horizon for the infrastructure asset under consideration. Maurer (2009) suggests for water infrastructure this horizon should be at least 30 years, which can cause difficulties in estimating growth projections leading to oversized assets. This difficulty can party offset by frequently reviewing growth projections as more recent data becomes available. In addition, DoI Water (2014) recommends separation of water infrastructure assets for civil structures, pipelines, mechanical and electrical components which can give multiple planning horizons to consider.

To put this theory into context, we can explore an example of a water pumping station required to be built tomorrow. If the nominated planning horizon is 50 years, the total costs would include the capex and opex over the next 50 years. In addition, some assets may need augmentation before the end of the planning horizon and therefore need to be included more than once. For example, mechanical and electrical components commonly require renewal every 25 years and should therefore be counted twice. However, the second instalment includes a 25-year lag time as it is not required until year 26 and should be discounted accordingly.

2.4.2 Cost-Benefit Analysis

Cost benefit analysis (CBA) is a technique used to estimate and compare the total benefits against the total costs for a decision in monetary terms, which can be expressed as a benefit cost ratio (B/C). In this form, if the benefits are greater than the costs, the benefit ratio is greater than one and considered feasible (NSW Treasury 2017). Although, this concept appears relatively simple to apply, ambiguities can exist for projects with high recurring costs. Such as, those experienced by water authorities for operations, maintenance and repairs. The argument put forward is whether high recurring costs are negative benefits or strictly costs. Both options provide vastly different outcomes as the first would be subtracted from the numerator and later added to the denominator (Dandy et al. 2007).

2.4.3 Triple Bottom Line

Triple Bottom Line (TBL) is an evaluation method that attempts to satisfy the economic, social and environmental issues identified during the decision-making process. Whereas, past water infrastructure projects have traditionally been dictated by financial objectives at the expensive of the latter (Liner & Demonsabert 2011). This method is currently utilised by NSW water utilities for the assessment of IWCM scenarios to enable solutions are determined for water supply, sewerage and stormwater problems that are sound investments (DoI Water 2019).

Difficulties with this approach are identified in the literature as even weighting of objectives, which Liner and Demonsabert (2011) attempted to improve by using a computer model with goal setting to balance the objectives. However, this approach still requires human interaction to set the model constraints leaving an opening for socio-political influence. This is because no software program or computer model can currently build themselves, but instead requires the knowledge, experience and judgement of the specific problem to be solved (Loucks & van Beek 2017). Furthermore, as with CBA clear procedures and guidelines are required to reduce the risk influenced outcomes.

Other studies, such as Casey et al. (2017) looked past TBL as a decision-making process and instead focused on the optimisation of water infrastructure. This was achieved by researching the potential integration of power generation with resource recovery to raise the environmental and social profiles of the water infrastructure. This demonstrates the potential for water infrastructure projects to partially offset NPV costs with additional benefits. Thus, increasing the benefit cost ratio of a project and the TBL. However, this requires the additional facilities to have standalone feasibility, as they are also subject to capex and opex.

2.4.4 Multi Criteria Analysis

Multi-Criteria Analysis (MCA) is an evaluation technique that how been used since the 1960s and is similar to TBL, as it provides a framework for decision options that can be scored or ranked against multiple objectives (Hajkowicz & Collins 2007). However, TBL is limited to three main objectives, whereas MCA offers an advantage as it can be tailored to suit a wider spectrum and therefore common use in water management and planning. MCA is capable of processing complex decision-based problems that contain large quantities of information that can be categorised and weighted (Sjöstrand et al. 2018).

Although, it is considered by many to be transparent and reduce conflicting interests, it too suffers from criticism for the weighting of selected criteria with many algorithms for solving MCA problems reported in surplus. Furthermore, it is suggested that this criticism can be silenced by increased stakeholder interaction with further research advocated in this space (Hajkowicz & Collins 2007). The recent study from Sjöstrand et al. (2018) supports this notion reporting when stakeholders are integrated into the decision-making process the outcomes are more likely to be accepted.

2.5 Summary

In summary, the literature review has defined what a potable water supply system is and the configuration considerations that significantly impact on capex and opex within the system. This resulted in potable water supply system including everything from the point of collection at a water source and storage, through to the treatment processes involved in making water safe for consumption and domestic use. Furthermore, the system was also identified to include the distribution and transport of the water once treated for supply to the consumers tap.

An adequate understanding of the system enabled the investigation of the configuration considerations that significantly impact on capex and opex. These were determined to be dependent on the spatial differences between the water infrastructure items within the system via distance and elevation. More specifically, by the length and height that water is required to be transported or pumped to satisfy the water supply demands of the system. This also included discussion of the capex and opex associated with storage capacity, water balancing and water quality.

Further investigation showed the evident relationship between the cost, power and discharge within the hydraulic design of a pumping station to create a static lift and overcome any friction head or minor head losses to transport water within the system. Head loss caused by friction is subject to the pipeline length and to a lesser extent the internal pipe diameter and pipe roughness. Both the Hazen-Williams and Colebrook-White transition equations reported in the literature review can be used for the determination of pipe friction. In contrast, the latter option presents a wider range of validity and is therefore considered more accurate but require an iterative method to solve. Whereas, the Hazen-Williams equation is popular due to the availability of an immediate solution.

Once the static lift and losses are known, the pump size required can be determined, as can the capex and opex associated with the pump selected. This included the power consumption of the pump which was detailed as the major contributor within any water utilities budget, confirming the importance of correct pump selection to ensure efficiency. Pump maintenance was also identified as a large opex item and was reported to contribute for approximately 2.5 times the capital cost over the life of the pump.

Finally, assessment options were investigated to determine the most appropriate method for the evaluating the capex and opex related to a potable water supply system. This included discussion of NPV, CBA, TBL and MCA with PV being predominate throughout the different assessment options. It was particularly noted, that for water authorities the income and expenditure are directly dependant allowing the NPV calculation to be reduced to the addition of the PV for capex and opex.

Additionally, many studies agreed that a nonlinear relationship exists between cost, power and discharge which makes it difficult to estimate these expenditures. However, it was reported that empirical information can be gathered overtime by water utilities to estimate the cost of future water infrastructure projects.

Furthermore, it was evident that a knowledge gap exists within recent academic literature on studies for determining the costs of pumping stations (Walski 2012). Furlong et al. (2016) supported this claim, by concluding that water planning research is often found in public sector reports rather than academic literature. Reinforcing the need for well documented research and academic documentation to increase the body of knowledge, available in this space for future water resource planners and engineers.

3 Methodology

This chapter details the methodology that will be undertaken to determine the optimal position for the future WTP for the proposed ESS. This includes defining the project limitations; model development and methodologies; and project planning required to complete the dissertation objectives. This chapter is also followed by a complementary chapter that provides clear explanation of how the key elements addressed in this chapter were utilised to build the hydraulic model and complete the financial analysis.

3.1 Project Limitations

As discussed in the chapter 1, the project scope is limited to the physical structures that capture, hold, treat and transport potable water and the configuration considerations that significantly impact on the capex and opex. The literature review in the previous chapter identified these considerations as the spatial differences between the physical structures of the water supply infrastructure components. This included the pumping elevation, pipeline distance as well as the consequences these have on water storage and quality. Regarding the latter two, these can be considered fixed under the situation proposed for the ESS, leaving the pumping elevation and pipeline distance to be further analysed. Furthermore, the project is also limited by the academic requirements of ENG4111 & ENG4112 Research Project and timeframes.

3.2 Model Development and Methodologies

To meet the project specification and course requirements the model development and methodologies need to be capable of achieving the relevant objectives listed in section 1.22. This shall be achieved by desktop analysis broken down into two components. The first shall require the development an implicit model using Microsoft Excel to calculate and evaluate the hydraulics differences for a range of possible positions for the future proposed WTP. The Second shall involve a financial analysis utilising the outputs obtained from the hydraulic model using the NPV method, as identified in the literature review. This approach shall allow a robust financial evaluation of the possible positions for the future WTP and determine the optimal position.

The latter sections in this chapter discuss the scenarios that will be evaluated, the development of the hydraulic model and financial analysis. This includes details of the constraints, inputs, outputs, key calculations and equations required for determining the optimal position of the new WTP. Moreover, to ensure the quality of the results obtained from the hydraulic model and financial analysis are acceptable. Sensitivity analysis and verification calculations are required, which are discussed in detail in the next chapter. However, to understand the previous work already undertaken the section following provides a summary and detailed discussion of the development of the ESS Concept Design and Ancillary Works relevant to this dissertation.

3.2.1 Concept Design for ESS and Ancillary Works

A review of local water sharing plans in 2012 demonstrated the requirement for a second water storage within the Eurobodalla shire by 2023. In 2016, ESC reviewed and updated their integrated water cycle management strategy, which confirmed this requirement. Consequently, an off-stream storage with an initial capacity of 3000 ML was identified as the preferred option and proposed to be located near the existing SWTP and Tuross River bores. Within the same year, ESC engaged SMEC Australia Pty Ltd to undertake a concept design for the proposed ESS and ancillary works.

The concept design was completed in two volumes. The first volume addressed the ancillary works required to integrate the proposed storage into the existing water supply system (SMEC 2016a). The second volume addressed the major structures required for the proposed storage including the embankment wall, inlet, outlet, spillway and other temporary works (SMEC 2016b). The concept design recommended the project be completed in three stages with associated ancillary works to be completed at each stage as required.

The first stage involves the construction of the ESS and two main components of ancillary works to allow the supply and transfer of water for commissioning by 2023, as shown in Figure 6, which includes:

- A new river intake pumping station capable of extracting 26 ML/d over 24 hrs (302 L/s) of surface water from the Tuross River for pumping to the ESS. The extracted surface water shall also be supplemented by reconfiguring the existing Tuross River alluvial bores pipework, which is currently capable of supplying 6 ML/d of groundwater to the SWTP for treatment.

- A new pipeline from the new river intake pumping station to the ESS, capable of transferring of 26 ML/d over 24 hrs (302 L/s) of water to the ESS for storage. This pipeline is also proposed to have an additional connection with associated valves and pipework to allow the transfer of stored water from the ESS to the existing SWTP for treatment by gravitation.

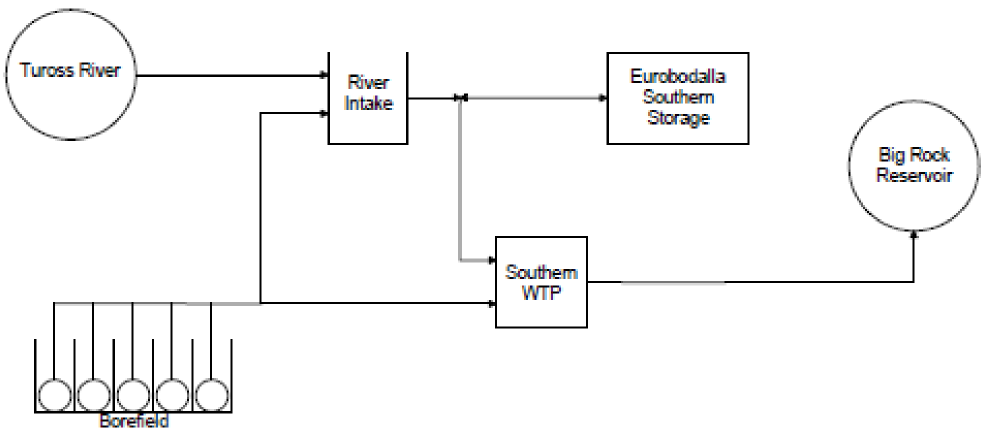


Figure 6: Ancillary Works Flow Schematic – Stage 1 (Adapted from SMEC 2016a)

The new river intake pumping station and transfer pipeline shall be vital infrastructure for filling the ESS once constructed, as by definition an off-stream storage receives minimal run-in from overland flow from the surrounding catchment. The cost for the transfer pipeline shall be minimised by utilising the pipeline for both the transport of water to and from the ESS. Whilst, providing connectivity to the existing water supply system via the SWTP until the later stage two ancillary works are completed. This will allow the utilisation of the remaining asset life of the SWTP prior to the increase of future water demands. The ESS outlet structure shall also include the construction of a valve pit or house at the foot of the embankment wall to connect an outlet pipe from the ESS outlet tower to the stage one and future stage two works, respectively.

The second stage allows for the construction of a new 25 ML/d WTP for commissioning no earlier than 2030 and was determined to require the following ancillary works, as shown in Figure 7, which includes:

- A new ESS outlet pumping station to transfer stored water from the ESS to the new WTP capable of pumping 320 L/s, which accounts for water losses during treatment and allows for flow matching of the proposed plant output.
- A new pumping station and pipeline to transfer treated water from the new WTP at a capacity of 25 ML/d over 23 hrs (302 L/s) to an existing water supply reservoir near Big Rock approximately 7 km away for integration into the existing water supply system.
- The decommissioning of the existing SWTP.

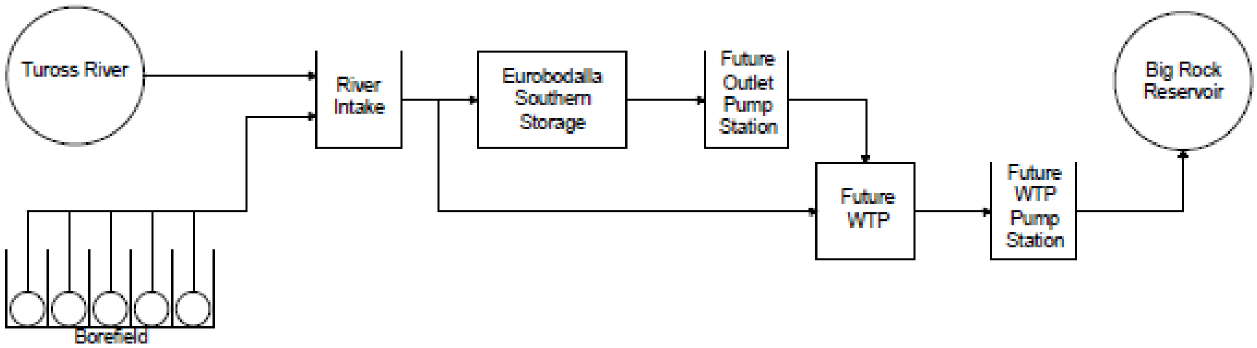


Figure 7: Ancillary Works Flow Schematic – Stage 2 (Adapted from SMEC 2016a)

The final and third stage involves raising the embankment and outlet tower to increase the capacity of the ESS to 8000 ML. This shall also include the construction of a new spillway with no other additional ancillary works required during this stage. Further understanding of the ESS and ancillary works proposed for stages one, two and three can be further clarified by viewing the general arrangement drawing shown as Figures 8, on page 26.

Volume one of the concept design also included discussion of the siting options assessment for the future WTP, which was undertaken during the 20% ancillary design review workshop held in December 2016. The new WTP was proposed to be similar design to the existing NWTP with a footprint of approximately 1500 m² and a useful asset life for the civil structure of 70 years (DoI 2014). Four options were considered for the location of the new WTP, which included adjacent the existing SWTP, directly east of the SWTP, near the existing supply reservoir near Big Rock and adjacent the ESS.

The first two options were noted at Australian Height Datum (AHD) reduced levels (RL) of approximately 20-25 m with the latter at RL 76 m and RL 143 m, respectively. All elevations are higher than the 1 in 100 AEP flood level reported at approximate RL 14.2 m. Subsequently, a MCA was undertaken identifying the fourth option located on the left abutment adjacent to the ESS as the recommended option shown in Figure 9. This option included the requirement for an outlet pumping station to be constructed downstream of the valve house to transfer stored water from the ESS to the new WTP.

It should also be noted, that for the initial stage the ESS a capacity of 3000 ML was forecast to meet the medium-term water supply demands for 2023-2060 with the future upgrade in storage capacity to meet long-term water supply demand until 2160. This means, the new WTP shall be operational for 30 years under the initial ESS capacity and 40 years after the future capacity upgrade. Furthermore, the ESS preliminary designs identified the full supply level (FSL) for the initial capacity to be RL 47.7 m and RL 60.3 m for the future capacity upgrade with a minimum operating level (MOL) at RL 27.4 m, for both. The MOL was adopted to maintain water quality by avoiding the siltation that can occur in the lower storage levels.

Further analysis was undertaken by SMEC (2016a) to develop the concept design for the configuration of water supply infrastructure required for the stage one ancillary works and the recommended WTP option 3 for the stage two ancillary works. This included determining nominal pipe diameters, total-head losses for the proposed pipelines, the power requirements for pumping and NPV cost comparisons. The possible pipelines sizes were firstly determined using the required flowrates of 302 L/s and 320 L/s with the design velocity range of 0.8 m/s to 1.2 m/s, as recommended by Water Services Association Australia for self-cleansing and slime control. Table 1 displays the range of internal diameters determined within the recommended velocities at the nominated flowrates.

Table 1: Stage 2 – Pipeline Velocities and Discharge (Adapted from SMEC 2016a)

	ESS to New WTP		**New WTP to Big Rock Reservoir**	
Internal Diameter (mm)	**Discharge (L/s)**	**Velocity (m/s)**	**Discharge (L/s)**	**Velocity (m/s)**
500	320	1.63	302	1.54
600	320	1.13	302	1.07
700	320	0.83	302	0.78

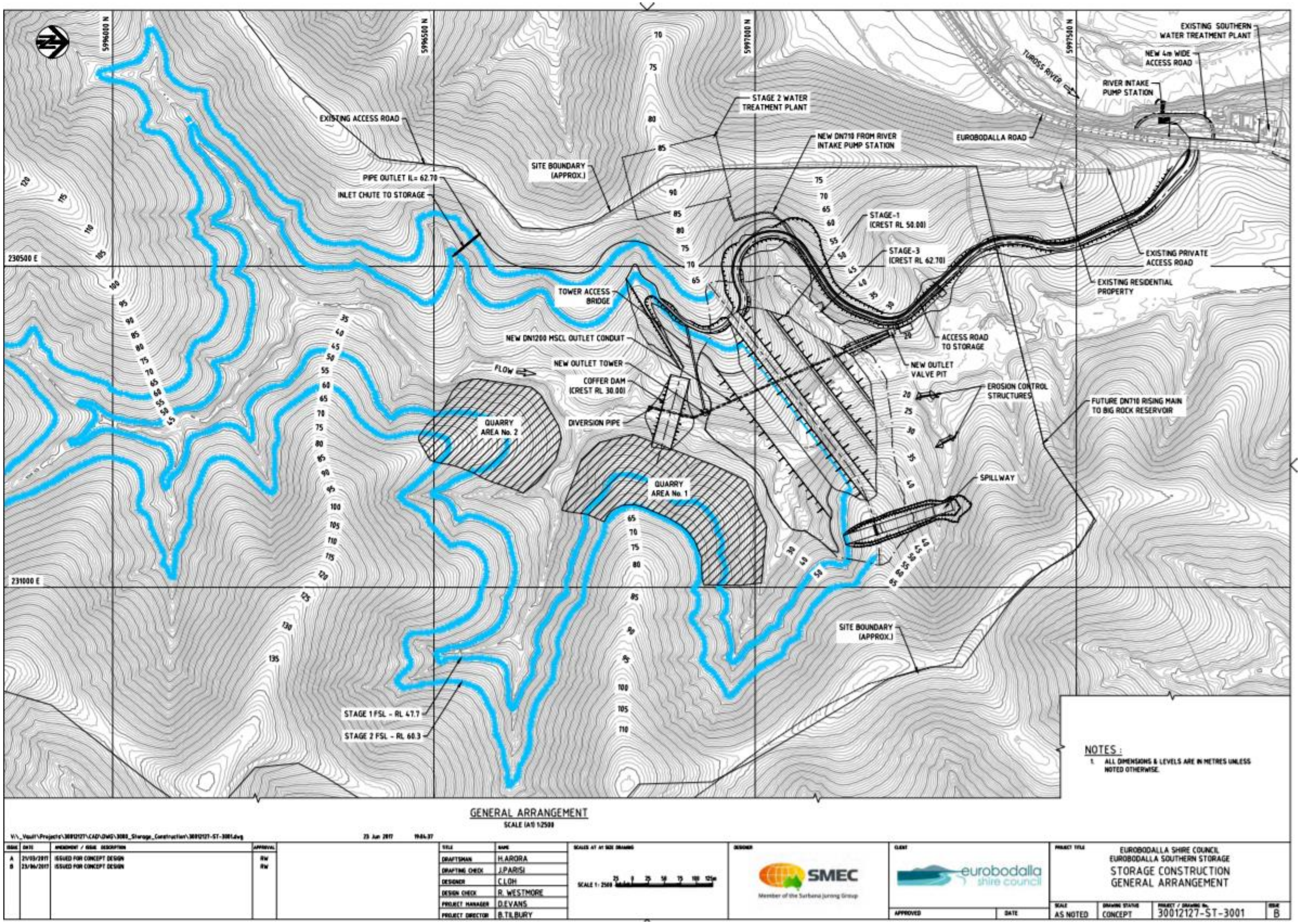


Figure 8: ESS General Arrangement (SMEC 2016b)

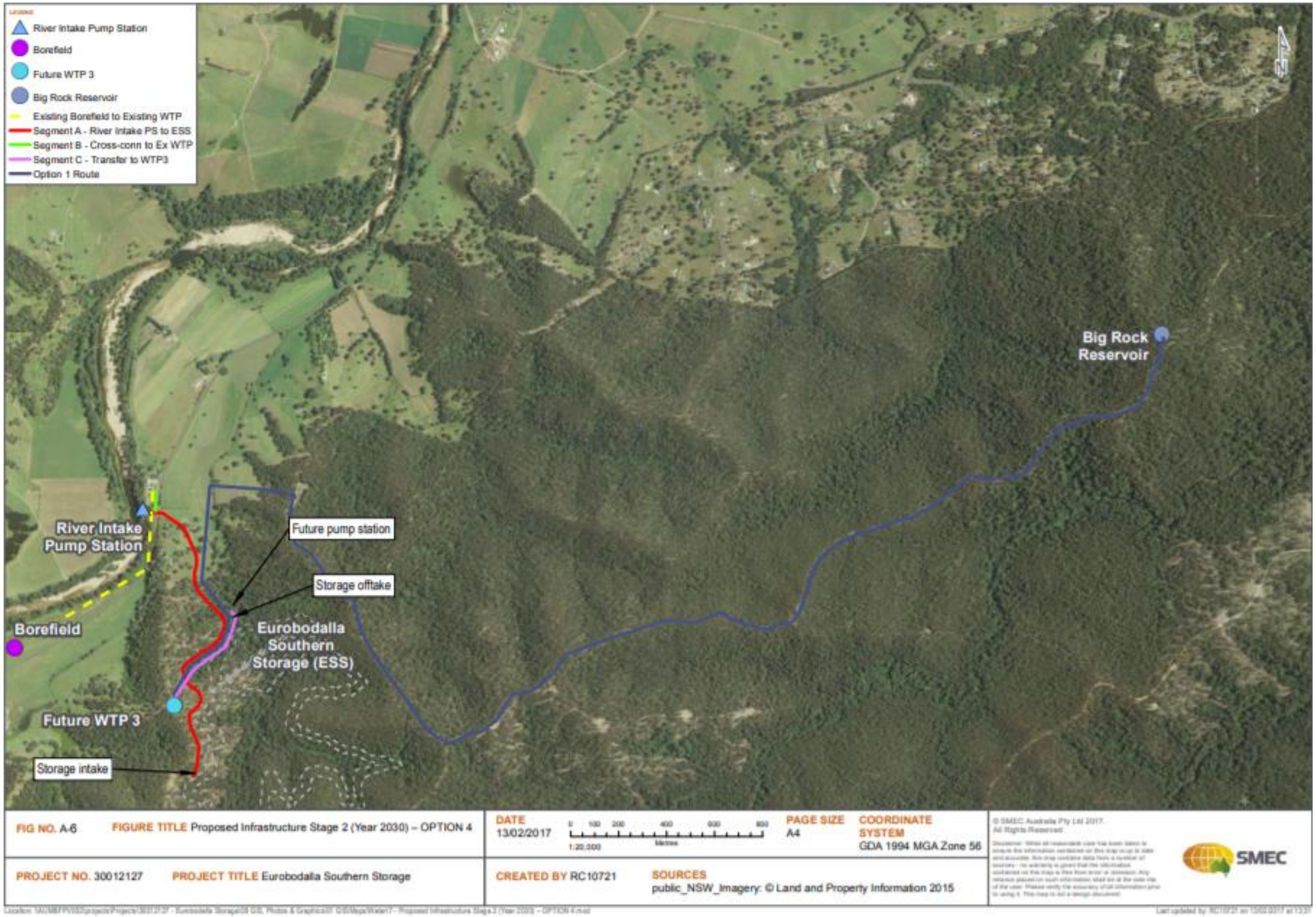


Figure 9: Proposed Water Supply Infrastructure - Stage 2 (Adapted from SMEC 2016a)

The total head losses were then determined, accounting for the static, friction head and minor losses. This was achieved using the Hazen-Williams formula for determining friction loss and the inclusion of an allowance for minor head losses from fittings and valves. This Resulted in the head loss summary for stage 2 shown below in Table 2. Furthermore, it is noted that concept design appears to have adopted a discharge level at Big Rock Reservoir rounded-up 145 m rather than existing TWL of 143 m.

The range of nominal pipe diameters was then used to identify the possible pump sizes required to overcome the total head for the transfer the water from the ESS outlet pumping station to the new WTP and from the new WTP to Big Rock Reservoir using the Grundfos product centre sizing software available online with the results for stage 2 shown in Table 3 (SMEC 2016a).

Table 2: Stage 2 – Head loss Summary Table (Adapted from SMEC 2016a)

Description		Flow (L/s)	Start Level (m)	Discharge Level (m)	Static Head (m)	Friction Head (m)	Minor Losses (m)	Total Head (m)
From	**To**							
ESS Outlet Pumping Station	New WTP	320	27.4	73.0	45.6	0.9	4.1	50.6
New WTP Pumping Station	Big Rock Reservoir	302	73.0	145.0	72.0	14.5	3.6	90.2

Table 3: Stage 2 - Power Summary (Adapted from SMEC 2016a)

Description		Flow (L/s)	Total Head Required (m)	Total Power Requirements (kW)	Pumping Configuration
From	**To**				
ESS Outlet Pumping Station	New WTP	320	50.6	225 (75 kW each)	Expected 3 Duty and 1 Standby
New WTP Pumping Station	Big Rock Reservoir	302	90.2	396 (132 kw each)	Expected 3 Duty and 1 Standby

Subsequently, an NPV analysis was then undertaken for the range of pipe sizes, pipe materials and pumping combinations for discount rates of 4%, 7% and 10% over a 25-year period. The results for the 7% NPV comparison are shown in Table 4 on the next page. From these results, the recommendation for the use of a DN710 HDPE pipeline was favoured over the DN800 HDPE pipeline, which was valued at the lowest cost. This was because the slightly higher velocities predicted for the DN710 HDPE were more desirable for slime control within the pipeline to the maintain water quality within the system (SMEC 2016a).

Table 4: Stage 2 - NPV Comparison of Pipelines Options (Adapted from SMEC 2016a)

Pump Requirements	DN (mm)	Pipe Material	7% NPV ($)
225 kW (ESS Outlet PS) 396 kW (New WTP PS)	600	GRP	$ 11,657,403
	600	DICL	$ 10,055,823
	710	HDPE	$ 9,956,734
	600	STEEL	$ 10,419,147
255 kW (ESS Outlet PS) 510 kW (New WTP PS)	525	GRP	$ 13,144,121
	500	DICL	$ 11,253,397
	630	HDPE	$ 11,175,437
	500	STEEL	$ 11,682,770
210 kW (ESS Outlet PS) 315 kW (New WTP PS)	675	GRP	$ 10,848,809
	750	DICL	$ 9,750,783
	800	HDPE	$ 9,618,655
	700	STEEL	$ 9,915,931

3.2.2 What Scenarios will be Evaluated in this Dissertation?

The determination of which scenarios should be considered for evaluation involved identifying a range of possible positions for the future proposed new WTP. This was achieved by determining the spatial differences between the relevant water infrastructures. The possible positions were required to adhere to a predetermined set of site constraints and assumptions, which are detailed within this section. This approach also determined the input values required for the hydraulic model. This included elevations, pipeline distances, pipe properties, water characteristics, and flowrates with the inputs either fixed or variable. Evaluation of each scenario within the hydraulic model also provided a range of outputs for financial analysis which is discussed further in later section 3.2.2.

Elevation was recorded in metres (m), using AHD RLs and pipeline distances were recorded in metres length between each of the relevant infrastructure components within the water supply system. The components were limited to the ESS, outlet valve house, outlet water pumping station (if required), the new WTP and Big Rock Reservoir. These pipeline distances are referred throughout the following chapters as pipeline segments A, B, C and D, respectively. In addition, the operational water storage levels for both the ESS and the Big Rock Reservoir were also considered in determining which scenarios to investigate. The inputs and constraints used to determine the number of scenarios and to develop the hydraulic model are discussed further under each of the relevant sub-headings below.

3.2.3 Space and Location Range for the new WTP

This section discusses the assumptions and site constraints which were considered for the space requirements and location range for the new WTP. The new 25 ML/day WTP is proposed to be similar size and design of the existing NWTP located in the northern part of the shire. The NWTP was commissioned in 2011 and utilises dissolved air floatation built on top of gravity filtration units (DAFF), which offers a compact design that requires a footprint of approximately 1500 m^2. Therefore, the space required for the new WTP was assumed to have the same footprint. The WTP footprint size and other assumptions determined to affect the site constraints are qualified below, and include:

- The site must accommodate 1500 m^2.
- The site can be levelled due to access of earth moving equipment during the stage 1 works.
- Access from Eurobodalla Road shall be most desirable for pipelines, power supply and vehicles.
- Any existing residential properties or land can be acquired if necessary.
- The minimum and maximum elevations shall consider the surrounding land formations, the 1 in 100 AEP Flood levels and inundation from ESS embankment failure.

The maximum elevation within the subject area that is accessible from Eurobodalla Road was detailed as RL 90 m, located on the western abutment of the ESS. This was reported as the recommended site for the new WTP in the concept design report, discussed in the previous section (SMEC 2016a). It is further noted, once the required 1500m^2 footprint is accounted for, the elevation at the lowest perimeter of this position is approximately RL 73 m, which was the elevation used for analysis in the stage 2 ancillary works concept development. However, this method does not account for the hydraulic profile of the new WTP.

The new WTP is also suggested to be similar in design to the existing NWTP, therefore the same hydraulic profile was adopted for the optimisation model. The NWTP inlet has a top water level (TWL) approximately 8.5 m above ground level. This allows water to be pumped from the off-stream storage (Deep Creek Dam) to the NWTP and gravitate through the following treatment units, filling the closest water supply reservoir to almost 85% full of potable water. Hence, pumping is only required from the clear water pumping station to fill the reservoir for the remaining 15%. Therefore, to build on the work previously undertaken by SMEC and account for the hydraulic profile of the new WTP, the inlet elevation to the new WTP included an additional 8.5 m increasing the recommended site discussed above, to have an inlet elevation of 83.5 m.

In regards, for determining the minimum elevation for evaluation, the maximum water level for the AEP, 1 in 100 flood level is RL 14.20 m. Therefore, no elevations lower than 15 m, plus the additional 8.5 m were considered giving a lower elevation limit of 23.5 m. This resulted in an elevation range of 23.5 m – 83.5 m along the ridge that rises from the existing SWTP near Eurobodalla Road along the western boundary to the recommended stage 2, option 4 location on the western abutment.

Furthermore, it originally appeared prudent to consider positions between the minimum and maximum to align with the existing 5 m contours. However, not every contour within this range was determined suitable due to the previously defined site constraints. Alternatively, if other positions are identified within the specified range, an opportunity exists to extrapolate the results. Furthermore, the hydraulic model was built in a robust way to enable alternate positions to be evaluated outside this dissertation. This offered further value to the model, should other possible positions be identified at a later stage under the same or alternate site constraints.

Finally, the approach discussed above resulted in 11 possible positions for the new WTP being selected, which are marked-up in red on the following page in Figure 10. This displays position 1 located at the minimum elevation to position 10 located at the maximum elevation. The recommended site from the concept design is also shown as position 11 and has been included for later comparison. The position number is displayed in the top left corner of each marked-up footprint and denotes the location for the WTP inlet. It is also evident from Figure 10, that each position alters the pipeline distances between the water supply infrastructure components and is further discussed in the later section 3.2.5.

3.2.4 Storage levels for both the ESS and the Big Rock Reservoir

The ESS is proposed to have a stage 1 FSL at RL 47.7 m, stage 2 FSL at RL 60.3 m and MOL at RL 27.4m. Therefore, the storage level (elevation) range for ESS shall be between MOL and FSL at intervals of 3.29 m. Big Rock Reservoir has a TWL of RL 143 m with full capacity maintained to avoid supply interruptions and firefighting. Therefore, the storage level (elevation) for Big Rock Reservoir shall be adopted at a fixed RL 143 m. It should also be noted, that the different elevations within the storage infrastructure components does not affect pipeline distances. In contrast, with the 11 different possible positions of the new WTP which does alter the pipeline distances. The elevation range and number of different levels for each of the water infrastructure components with varying and fixed elevations are listed below with a total number of 121 different scenarios to be modelled.

Table 5: Elevation Range and Number of Scenarios

Description	**Min (m)**	**Max (m)**	**Levels (No.)**
ESS Water level	27.4	60.3	11
New WTP	30	80	11
Big Rock Reservoir	143	143	1
		Total Scenarios (No.)	**121**

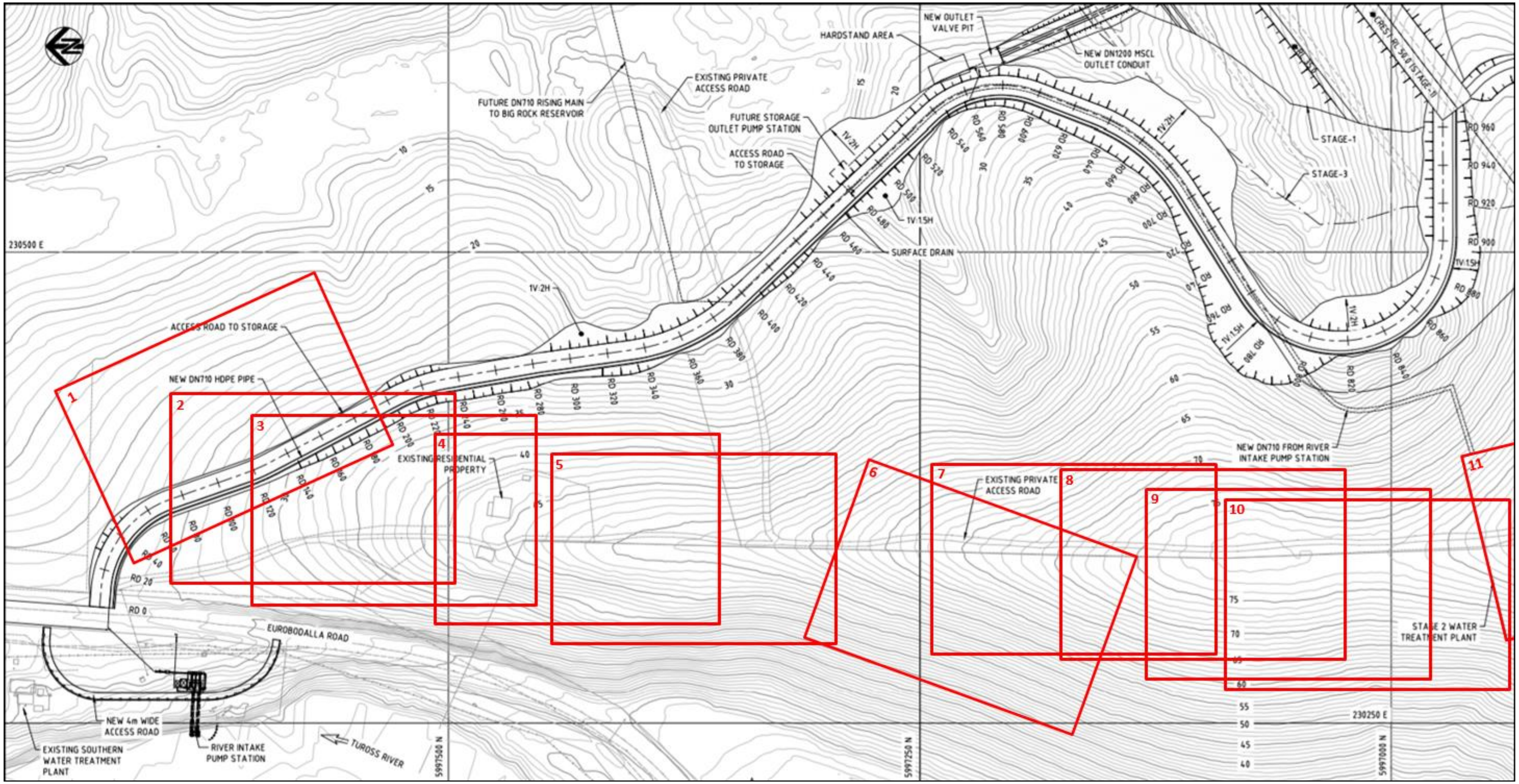


Figure 10: Redline Mark-Up of Possible positions for new WTP (Adapted from SMEC 2016b)

3.2.5 Pipeline Distances for Segments A, B, C and D

The pipeline distances were broken down into segments A, B, C and D between each of the infrastructure components for use in the hydraulic model. This delineation was required as different pipes have different characteristics that need to be considered within the calculations used in the hydraulic model. These characteristics include material, internal and external pipe diameter, pipe roughness which can affect the friction head determination. Segments A, B and C combine to give the total pipeline length between the ESS and the new WTP with segment D accounting for the pipeline between the ESS and Big Rock Reservoir.

Pipeline segment A is a 32 m long, mild steel cement lined (MSCL) DN 1200 mm pipeline between the ESS and outlet valve house. This section of pipeline is sized for the life of the ESS and considered as a fixed input under all the scenarios with the outlet valve house allowing future connectivity for stage one and two ancillary works. Pipeline segment B is a 10 m long, MSCL DN 600 mm pipeline between the outlet valve house and the future outlet water pumping station. This section of pipeline is to provide connection to the ESS outlet pumping station, should it be required and therefore fixed under the scenarios being evaluated.

Pipeline segment C is the pipeline length in metres between the ESS outlet pumping station and the new WTP. This length of pipe is a variable input and dependent on the position of the new WTP. The length was determined for each of the positions 1 to 3, by taking the westerly offset distance from the position number to where it intersects the access road. The access road chainage was then used to determine the distance from the point of intersection to the ESS outlet pumping station located at chainage 480 m and shown on Figure 10. For positions 4-10 an easterly offset distance was used in the same manner. The access road slope was considered negligible in comparison with the road length. However, for the pipeline offset distance between the road chainage point of intersection and position number, the slope was determined from the contours and accounted for each of the new WTP possible positions.

Finally, an additional 8.5 m was included to account for the vertical rise of pipeline to reach the WTP inlet elevation as discussed in section 3.2.2 with the total pipeline lengths for segment C shown in Table 6. Furthermore, the pipeline length for position 11 was taken as the direct distance from the concept design plus the additional 8.5 m for the vertical rise to the assumed inlet. This was included to provide a cross comparison with the concept design development as previously discussed. Table 7 displays the combined total length of pipeline segments A, B & C as the total distance from the ESS to New WTP for each position.

Table 6: Pipeline Distance Segment C

Position No.	Difference in Road Chainage (m)	Offset Distance to Intersection (m)	Slope of Offset Distance (unitless)	Vertical Riser (m)	Segment C (m)
1	480	120	0.06	8.5	496.0
2	400	50	0.08	8.5	412.5
3	360	30	0.08	8.5	371.0
4	260	20	0.25	8.5	273.5
5	180	50	0.20	8.5	198.5
6	0	140	0.21	8.5	38.5
7	60	120	0.25	8.5	98.5
8	140	160	0.19	8.5	178.5
9	200	180	0.19	8.5	243.5
10	260	180	0.19	8.5	303.5
11	n/a	n/a	n/a	8.5	408.5

Table 7: Total Pipeline Distances from ESS to New WTP

Position No.	Segment A (m)	Segment B (m)	Segment C (m)	Total Distance (m)
1	32	10	496	538.0
2	32	10	412.5	454.5
3	32	10	371	413.0
4	32	10	273.5	315.5
5	32	10	198.5	240.5
6	32	10	38.5	80.5
7	32	10	98.5	140.5
8	32	10	178.5	220.5
9	32	10	243.5	285.5
10	32	10	303.5	345.5
11	32	10	408.5	450.5

Pipeline segment D is the pipeline length in metres between the new WTP and the existing Big Rock Reservoir. This pipeline length is a variable input and dependent on the position of the new WTP. The concept design reported a pipeline length of 7070 m for the recommended option (position 11). This proposed pipeline intersects the access road at approximate chainage 390 m on Figure 10 and the distance from the outlet pump station to road chainage 390 m is 90 m. Therefore, the approximate distance from position 11 to road chainage 390 m equals 490 m. This known distance can be subtracted from the 7070 m to determine the distance from road chainage 390 m to Big Rock Reservoir is 6580 m. Hence, providing a point of reference for determining segment D lengths for positions 1 to 10.

As with the inlet, the outlet point for the new WTP was also adopted as the denoted number in the top left corner of each possible position on Figure 10. This approach was favoured so the inlet and outlet pipelines could be configured to allow bypass of the treatment processes into the clear water tank (if required). This also would allow balancing and storage to occur prior to it being pumped to Big Rock Reservoir via the future clear water pumping station. Although, considered outside the scope of this dissertation, it is evident this contingency offers a resilient design for extreme situations. Such as, those experienced during a bushfire or prolonged drought when water may be required to be supplied into the system without adequate treatment. The total distance for segment D for each position is shown in Table 8, which is the summation of the offset distance from the position number to where it intersects the access road, plus the difference from the point of intersection to road chainage 390 m and the remaining 6580 m to Big Rock Reservoir.

Table 8: Total Pipeline Distance from New WTP to Big Rock Reservoir (Segment D)

Position No.	**Big Rock Reservoir to Road Chainage 390 m (m)**	**Difference in Road Chainage to Offset (m)**	**Offset Distance to Road Intersect (m)**	**Slope of Offset Distance (Rise/Run)**	**Total Distance Segment D (m)**
1	6580	480	120	0.06	7067.5
2	6580	400	50	0.08	6984.0
3	6580	360	30	0.08	6942.5
4	6580	260	20	0	6840.0
5	6580	180	50	0.10	6765.0
6	6580	0	140	0.07	6590.0
7	6580	60	120	0.25	6670.0
8	6580	140	160	0.19	6750.0
9	6580	200	180	0.17	6810.0
10	6580	260	180	0.17	6870.0
11	n/a	n/a	n/a	n/a	7070

3.2.6 Water Characteristics and Pipe Properties

The pipeline properties and water characteristics required to undertake the calculations within the hydraulic model for analysis of pipeline segments A, B, C and D consisted of both fixed and variable inputs. The fixed input values for the water characteristics are listed in Table 9, below. This includes the average water temperature, viscosity and flowrates. These values were fixed inputs to ensure the model outputs can satisfy the water demand required for the stage 2 ancillary works, as stated in section 3.2.1 and to allow comparison with the previous work undertaken.

Table 9: Water Characteristics

Water Characteristic	Input Value
Temperature	20 ^{0}C
Viscosity	1.01 E-06 m^2/s
Flowrates	320 L/s and 302 L/s

In contrast, the roughness (k) values required for the Colebrook-White transition formula and the internal diameter were variable inputs dependant on the pipe properties. The pipeline diameters were determined by adopting the same approach undertaken during the concept design, previously mentioned in section 3.2.1, which allowed for a design velocity range of 0.8 m/s to 1.2 m/s with the Colebrook-White transition formula discussed in further detail in the next section. Table 10 below lists the pipe properties for each segment of pipeline to be analysis.

Table 10: Pipe Properties for Segments A, B, C and D (adapted from SMEC 2016a; Nalluri 2009)

Pipeline Segment	Pipe Material	Nominal Diameter (mm)	Internal Diameter (mm)	Roughness k (mm)
A	MSCL	1200	1200	3.0 E-05
B	MSCL	1200	1200	3.0 E-05
C	HDPE	600	547	7.0 E-03
D	HDPE	600	547	7.0E-03

3.3 Key Calculations

The key calculations can be separated into two defined groups. Firstly, those required for the hydraulic model to evaluate the spatial differences between the possible positions for the new WTP; and secondly, those required for the financial analysis of the outputs from the hydraulic model for determination of the optimal position for the new WTP.

3.3.1 Hydraulic Model

The key calculations required for the hydraulic model are those necessary to determine the total head in metres required to transfer water from the ESS to the new WTP and from the new WTP to the existing Big Rock Reservoir. These key calculations are best explained by the following word equations and further details, discussed on the next page:

$$\begin{matrix}\boldsymbol{Friction}\\ \boldsymbol{Head\ Loss}\\ (\boldsymbol{m})\end{matrix} = \begin{bmatrix}\boldsymbol{Friction}\\ \boldsymbol{Slope}\\ (\boldsymbol{m})\end{bmatrix} \times \begin{bmatrix}\boldsymbol{Pipeline}\\ \boldsymbol{Length}\\ (\boldsymbol{m})\end{bmatrix}$$

[Adapted from Eq. 2]

$$\begin{matrix}\boldsymbol{Total\ Head}\\ \boldsymbol{Required}\\ (\boldsymbol{m})\end{matrix} = \begin{bmatrix}\boldsymbol{Static}\\ \boldsymbol{Head}\\ (\boldsymbol{m})\end{bmatrix} + \begin{bmatrix}\boldsymbol{Friction}\\ \boldsymbol{Head\ Loss}\\ (\boldsymbol{m})\end{bmatrix} + \begin{bmatrix}\boldsymbol{Minor}\\ \boldsymbol{Head\ Loss}\\ (\boldsymbol{m})\end{bmatrix}$$

[Adapted from Eq. 5]

The static head is the difference in elevations between the ESS to the new WTP and from the new WTP to the existing Big Rock Reservoir with the range previously identified in section 3.2.3. The friction head loss is a resistance opposed to flow within a pipeline caused by friction, which needs to be overcome for water to be transferred through the pipeline. This is determined by multiplying the pipeline length by the friction slope as earlier defined in the literature review by Swamee (2001, p. 265), which utilised the Darcy-Weisbach detailed in Table 11 with the inputs required for the calculation and their units.

Table 11: Calculation for Friction Slope (Swamee 2001)

Calculation	Inputs	Symbols	Value	Units
$\boldsymbol{S_f = \frac{8fQ^2}{\pi^2 gD^5}}$	Frictional factor	f	varies	unitless
	Flow rate	Q	0.32	m^3/s
	Gravitational acceleration	g	~9.81	m/s^2
	Pipeline diameter	D	varies	m
	Pi	π	~3.14	unitless

The pipeline length is the distance in metres between the infrastructure components for each of the possible positions for the new WTP. These distances were previously identified in section 3.2.3, as pipeline segments A, B, C and D. Furthermore, due to the different pipe properties identified in section 3.2.4, the friction head loss for pipeline segments A, B and C shall need to be calculated separately and then added together to determine the total friction head loss for the pipeline from the ESS to the new WTP. Whereas, for the pipeline between the new WTP to the existing Big Rock Reservoir it shall only require the friction head loss to be calculated for pipeline segment D.

The friction slope calculation requires the unknown input of a frictional factor, which can be determined by utilising the Colebrook-White transition equation. This is an implicit equation that can be solved using an iterative method (Liou 1998). The Colebrook-White transition equation is detailed in Table 12 with the inputs required for the calculation and their units. This includes the unknown Reynolds number, which can be determined using the calculation detailed in Table 13.

Furthermore, the Colebrook-White transition equation was selected, due to the wider range of pipe and flow applications for which it can be applied with great accuracy. This shall enable the hydraulic model to be robust with added value for use outside this dissertation. In contrast, the Hazen-Williams equation utilised in the previously discussed concept design is empirical and limited within a validity range (Christensen et al. 2000). However, it is noted that for the parameters analysed in the concept design were within this range, potentially allowing later comparison of results obtained from this dissertation and the concept design.

Table 12: Calculation for Colebrook-White Transition Equation (Liou 1998)

Calculation	Inputs	Symbols	Value	Units
$$\frac{\mathbf{1}}{\sqrt{\boldsymbol{f}}} = -\mathbf{2log}\left(\frac{\boldsymbol{\varepsilon}}{\mathbf{3.7}\boldsymbol{D}} + \frac{\mathbf{2.51}}{\boldsymbol{R}\sqrt{\boldsymbol{f}}}\right)$$	Frictional factor	f	varies	unitless
	Roughness height	ε	varies	m
	Pipeline internal diameter	D	varies	m
	Reynolds number	R	Table 13	unitless

Table 13: Calculation for Reynolds Number (Christensen et al. 2000)

Calculation	Inputs	Symbols	Value	Units
$$\boldsymbol{R} = \frac{\boldsymbol{VD}}{\boldsymbol{v}}$$	Velocity	V	varies	m/s
	Pipeline internal diameter	D	varies	m
	Viscosity	v	1.01 E-06	m^2/s

The minor head losses from bends, valves and fittings are often considered negligible, as their contribution to the total head required to transfer water from one point to another can be relatively small (Swamee 2001). Nevertheless, to ensure the hydraulic model is robust and to allow later comparison with the concept design. An allowance shall be adopted for the total head calculation to account for possible minor losses. This is achieved by selecting a typical range of bends, valves and fittings to account for the additional pipework that may be identified during the detailed design, but unknown at the time of concept development.

Then by utilising the calculation for minor head losses shown in Table 14, the required velocity is squared and multiplied by the individual coefficient for each fitting, valve and bend, before being divided by the denominator. The selection of allocated bends, valves and fittings with their individual coefficients are shown in Table 15, which is the same allocation that was used during in the concept design. It is also noted, that as the velocity is part of the numerator, the higher the required velocity the higher the impact of minor losses.

Table 14: Calculation for Minor Losses (Nault & Papa 2015)

Calculation	Inputs	Symbols	Value	Units
$\boldsymbol{Minor\ Head\ Losses = \frac{kV^2}{2g}}$	Coefficient	k	Table 14	m/s
	Velocity	V	varies	m/s
	Gravitational acceleration	g	~9.81	m/s²

Table 15: Coefficients for Minor Loss Allowance (Adapted from SMEC 2016a)

Description	Quantity	$\boldsymbol{k}$
Gate valve, wide open	1	0.15
Gate valve, 3/4 closed	1	17.00
Swing check, forward flow	1	2.00
90^0 elbow, flanged	10	0.30
Long radius 45^0 elbow, flanged	15	0.20
Tee, through side outlet	10	1.80

3.3.2 Financial Analysis

To determine the optimal position for the new WTP a series of key calculations are required for the financial analysis of the outputs from the hydraulic model. The first group of calculations are required to convert the hydraulic model outputs into monetary terms. This includes the pipeline capex and the WPS capex and opex determined under each scenario, based on the size of the pump machinery required for the water supply system. The second group of calculations shall involve utilising the capex and opex to determine the NPV for each scenario modelled to conclude the optimal position.

The NSW Reference Rates Manual for the valuation of water supply, sewerage and stormwater assets shall be the main document used to estimate the capex. The manual contains reference rates that make allowance for the cost of survey, investigation, design, project management and contractor cost required to construct water infrastructure assets. However, the rate does allow for contingencies, land acquisition, power supply, data connection and access roads, fencing or operation and maintenance costs (DoI Water 2014).

The pipeline capex shall be limited to the pipeline from the proposed ESS outlet WPS to the new WTP (segment C) and the pipeline from the new WTP to the existing Big Rock Reservoir (segment D). Segments A & B were not included as both are required for ESS construction in year 2023 and therefore present under all future scenarios. Hence, no additional benefit is obtained by including their cost in this dissertation. However, should a detailed cost estimate be required for the entire ESS project pipelines in the future, the same method presented in this section can be adopted and the additional pipelines included.

Table 16 displays the 2016 rates used in the concept design for HDPE pipelines with the nominal diameters for the relevant design velocities identified in section 3.2.1. The pipeline rates account for the supply and installation of the pipelines at minimum depth, excavated in material other than rock and include restoration, fittings and thrust blocks (SMEC 2016a). To determine the current capital costs the pipelines, the 2016 rate was multiplied by a capital cost factor. The capital cost factor was formulated by DoI Water (2014) and derived from various sources including data published by the Australian Bureau of Statistics with annual updates supplied to water utilities in NSW.

Table 16: Rates for HDPE Pipelines (Adapted from SMEC 2016a and DoI Water 2014)

Material	Nominal Diameter (mm)	Concept Rate 2016 ($/m)	Capital Cost Factor	Rate 2019 ($/m)
HDPE	630	$612.00	1.052	$642.60
	710	$778.00	1.052	$816.90
	800	$1,036.80	1.052	$1,088.64

The size of pumps required shall then be determined by adopting the method detailed in the NSW Reference Rates Manual for sizing WPSs. This involves calculating the motor power required by using the equation shown in Table 17. The equation shows the numerator as the flowrate multiplied by the total head required and a nominated pipeline tolerance factor of 1.1. The numerator is then divided by a nominated pump efficiency percentage, which in this case is 0.8, as suggested by DoI Water (2017).

Table 17: Reference Rates for Water Trunk Mains (DoI Water 2014)

Calculation	Inputs	Symbols	Value	Units
$\boldsymbol{kW = \frac{Q \times h \times 1.1}{100 \times 0.8}}$	Flow rate	Q	320	L/s
	Total Head Required	h	varies	m

The relationship between the cost, power and discharge was then be used to determine the installed power requirements. This was achieved by multiplying the motor power resultant by the number of pumping machinery sets required. The reference rates allow for two pumping machinery sets being installed to provide standby capacity with each accounting for half the installed power requirement at the pumping station. Whereas, the concept design, discussed in section 3.2.1 allowed for an expected configuration of three duty pumping machinery sets and one standby. Therefore, to maintain consistency with the work previously undertaken, the installed power requirement shall be determined by multiplying the motor power required by four. The reference rates for installed power for WPSs are shown below in Table 18. To determine the current capital costs of new works the 2014 reference rate is multiplied by a capital cost factor of 1.075, as recommended by DoI Water (2019).

As identified previously, the pumping of potable water accounts for majority of any water utilities opex. For this reason, the determination of the pumping requirements and motor power shall also be used to calculate the ongoing power costs for the WPSs under each scenario. This shall be achieved by multiplying the water supply demand of 25 ML/d over 23 hrs (320 L/s) by the average price of power per KWh to determine the annual power costs. From recent ESC billing data, the average price of power has been determined as approximately $0.20 per kWh (Shorter, M 2019, pers. comm., 4 September).

The other major contributor to the WPS opex is the ongoing costs for pump maintenance with a typical pump requiring 2.5 times the initial capex over the life on the pump (Hydraulic Institute & Energy’s Office of Industrial Technologies 2001). Typically, mechanical and electrical components have an asset life of 25 years (DoI Water 2014). Therefore, the pump maintenance opex shall be determined by multiplying the capital cost by 2.5 and dividing the resultant by 25 years to account for an annual maintenance cost.

Table 18: Reference Rates for Water Pumping Stations (Adapted from DoI 2014)

Installed Power (kW)	Reference Rate 2014 ($/m)	Capital Cost Factor	Reference Rate 2019 ($/m)	M & E (%)	M & E Reference Rate 2019 ($)
10	$ 80,000	1.075	$ 86,000	69	$ 59,340
20	$ 115,000	1.075	$ 123,625	70	$ 86,537
30	$ 140,000	1.075	$ 150,500	71	$ 106,855
50	$ 190,000	1.075	$ 204,250	72	$ 147,060
100	$ 370,000	1.075	$ 397,750	73	$ 290,357
200	$ 580,000	1.075	$ 623,500	62	$ 386,570
400	$ 950,000	1.075	$ 1,021,250	64	$ 653,600
600	$ 1,190,000	1.075	$ 1,279,250	66	$ 972,015
800	$ 1,800,000	1.075	$ 1,935,000	68	$ 1,315,800
1000	$ 2,200,000	1.075	$ 2,365,000	70	$ 1,655,500
1200	$ 2,700,000	1.075	$ 2,902,500	72	$ 2,089,800
1400	$ 3,150,000	1.075	$ 3,386,250	74	$ 2,505,825
1600	$ 3,600,000	1.075	$ 3,870,000	75	$ 2,902,500

The finally key calculations required for the financial analysis are the NPV calculations, which was discussed in detail within section 2.4.2 of literature review and is reproduced below. NPV is calculated by determining the present value of all benefits minus the present value of all costs. Where the income (benefits) was identified as being directly dependant on the costs, resulting in the net being zero. Therefore, the present value for the capex and opex shall be calculated and used to determine the optimal position for the new WTP. The project life has been adopted over 25 years as this aligns with the asset life for the mechanical and electrical components (DoI water 2014).

$$\boldsymbol{NPV = \sum Capex + \sum Opex} \qquad \textbf{[Eq. 8]}$$

The present valve of the capex required to construct the water infrastructure is payable at the start of the first year which is equal to the 2019 values. In contrast, the opex costs shall need to be discounted back to present value. This shall be achieved using the calculation displayed in Table 19, on the next page and adapted from equation 9, from the previous chapter. These costs are then finally added together to determine the NPV.

Table 19: PV Calculation for Annual Costs (Adapted from Dandy et al. 2007)

Calculation	Inputs	Symbols	Value	Units
$\boldsymbol{P = A\left[\frac{1-(1+i)^{-n}}{i}\right]}$	Present Value	P	Varies	$
	Annual Costs (Opex)	A	Varies	$
	Project Life in years	n	25	No.
	Interest Rate	i	7	%

The interest rate of 7% has been adopted to align with the work previously undertaken in the concept design and as recommended by the NSW Government (SMEC 2016a; NSW Treasury 2017). Furthermore, the sensitivity analysis is also recommended to be undertaken for NPV at discount rates of 3% and 10%, which is discussed in detail within the next chapter (NSW Treasury 2017).

3.4 Project Planning

This section details the project planning that was undertaken in preparation of the dissertation to ensure that the project could be delivered on time and meet the specification requirements. This includes details on the quality assurance, resource requirements, scheduling of the key tasks with precedence and a risk assessment addressing both work health safety and project risks.

3.4.1 Quality Assurance

Quality assurance is an important part of any project, regardless if work is undertaken with hand tools or by computer, poor workmanship can undermine the validity of an entire project. In this case, majority of the work shall be undertaken by computer as this dissertation is predominantly a desktop study. Therefore, the main quality assurance tool for this project will be good record keeping procedures. This shall include the use of endnote software to ensure correct referencing in the desired form in achieved. The filling of literature in practical groupings to enable easy access and cross-referencing of pdf files with endnote. The final dissertation shall also be reviewed internally by a work colleague to ensure statements are made regarding the ESC information, data and assets is correct.

3.4.2 Resource requirements

The resources required for this project are detailed below in Table 20, created as version 1, 20 March 2019 as part of the specification requirements of this course. The requirements have been reviewed as part of the project progress report and have not changed. All resources were secured as early with confirmation received during project inception from the Eurobodalla Shire Council for permission access resources as required.

Table 20: Project Resources Requirements

Item	Source	Reason	Cost	Comments
Human Resources:	Student	Required to undertake project work	N/A	Expected average of 10 hours per week
	Supervisor	Required to provide feedback and timely advice	N/A	Available when required, alternative nominated for when unavailable
	Colleague	Proof Reading, support and general advice	N/A	Request colleague for commitment, also source alternative
Communications:	Email, phone and internet	Required to undertake project work	N/A	Existing access plans at home and workplace
Hardware:	Home computer Work laptop	Computer required to undertake project work	N/A	Preference is to use home computer and keep saved copies at work/offsite
Software:	Home computer Work laptop	Microsoft Office Word, Excel & Power Point required	N/A	Already installed on both devices
Literature and Documentation Access:	USQ library and internet	Access to library for relevant peer reviewed journal articles and other literature	N/A	Student access online
	ESC	ESC water infrastructure documentation required	N/A	ESC approval required
Stationary:	ESC	pens, paper, calculator, printing and binding	N/A	ESC approval required

3.4.3 Key Tasks and Project Schedule

The project has been broken down into the project phases of preliminary tasks, literature review, modelling development and validation, modelling phase and the final phase self-titled accordingly. This process allowed the key tasks for each phase to be identified and ensure the resources in the previous section would be adequate to complete the tasks. The key tasks required for each phase and are listed in Table 21, below and included in the project plan program, Appendix B of this report.

Table 21: Phase and Activity Descriptions

Project Phase No.	**Phase & Activity Description**
Project Phase 1	***Preliminary Tasks***
1A	Supervisor Liaison – obtain a Supervisor and liaison for the duration of the Project.
1B	Project Approval – obtain approval from USQ and ESC (Formal Topic Allocation).
1C	Project Resources – confirm availability and access of resources required (Table 20).
1D	Finalise Scope – define scope by consultation with Supervisor and ESC.
1E	Project Specification Report.
1F	Specification Feedback and final scope.
Project Phase 2	***Literature Review***
2A	Investigate relevant background information.
2B	Conduct literature research and review.
2C	Determination of method for cost evaluation.
2D	Literature review write-up.
2E	Prepare project progress report.
2F	Submit project progress report.
2G	Review progress report feedback from Supervisor
Project Phase 3	***Model development and validation***
3A	Hydraulic model development
3B	Identification of water treatment plant positions
3C	Determine required input parameters and calculations
3D	Model Validation and sensitivity analysis
Project Phase 4	***Modelling Phase***
4A	Run model for water treatment plant positions
4B	Data collection and management
4C	Evaluate and compare capital and operational costs
4D	Determine optimal position and write-up results
Project Phase 5	***Final Phase***
5A	Prepare draft dissertation
5B	Submit draft dissertation
5C	Project presentation
5D	Attendance at "Project Conference" Residential School
5E	Feedback session
5F	Prepare final dissertation document
5D	Submit Dissertation - Final Document (4pm)

3.4.4 Risk Assessment

The risks for the project were assessed by reviewing the tasks required for completion of the project from Table 21 and determining what risks may occur that could prevent a task from being completed. These risks include those specific for safety and project risk. The risk assessment matrix used is shown in Table 22 with possible outcomes or consequences for each risk were firstly determined and then matched with a likelihood of the risk occurring. The risks were then mitigated by preparing actions that could reduce or prevent the risk for occurring by using the controls listed under hierarchy of controls heading at the bottom of Table 22, with the project risk assessment and mitigating actions are shown in Table 23, on page 46.

Table 22: Risk Assessment Matrix (ESC 2018)

		Consequence (Possible outcomes)				
	EFFECT ON:	**Insignificant**	**Minor**	**Serious**	**Very Serious**	**Catastrophic**
	People:	First Aid Injury	Medical treatment	Lost time injury / hospitalisation	Fatality	Multiple Fatalities
	Environment:	Slight effect, no contamination	Minor on site contamination	Major on site contamination + potential for off site	Minor off site contamination	Major off site contamination
	Financial cost/Loss	Less than $1,000	$1,000 - $10,000	$10,001- $50,000	$50,001 - $200,000	More than $200,000
	Council's Image/Reputation	Complaint, no media coverage	Complaint, limited local media coverage	Complaint through Council or significant local media coverage	State-wide media coverage	National media coverage
	Legislation/Guidelines		Breach of work instruction	Breach of Guidelines/ Best Practice	Breach of Legislation	
Likelihood	**Almost Certain** *Common, Is expected to occur in most circumstances*	**M7**	**H14**	**E20**	**E23**	**E25**
	Likely *Is known to have occurred. 1 in 10 chance of occurring*	**M6**	**M10**	**H18**	**E21**	**E24**
	Possible *Could occur, 1 in 1,000 chance of occurring*	**L5**	**M9**	**H17**	**H19**	**E22**
	Unlikely *Not likely to occur, 1 in 100,000 chance of occurring*	**L3**	**L4**	**M11**	**M13**	**H16**
	Rare *Practically Impossible, 1 in a 1,000,000 chance of occurring*	**L1**	**L2**	**M8**	**M12**	**H15**

KEY		**ACTIONS**
E	**(Extreme- RED)**	**STOP**, Immediate controls required. **STOP JOB** if rating remains **Extreme** after controls in place. Seek management advice
H	**(High – AMBER)**	Approval to proceed required by Co-ordinator /Manager if rating cannot be reduced. Periodic review of controls required by Co-ordinator /Manager.
M	**(Medium – YELLOW)**	Approval to proceed required by Site Supervisor. Monitor risk throughout the job.
L	**(Low – GREEN)**	Generally safe to proceed. Continue to monitor the risks throughout the job

Hierarchy of Controls

When putting controls in place for a hazard you **MUST** first attempt to eliminate the hazard. Where this is not possible then minimise the risk using the hierarchy of controls in the order listed.

Eliminate	Put a control in place that removes the hazard altogether
Substitute	Replace the hazard with something less hazardous
Isolate	Put a barrier between personnel and the hazard by space or time
Engineer	Manufacture a guard or use machinery
Administration	Signs, WMS's, Procedures, verbal or written direction, Training etc.
PPE	Last line of defence – must be fit for purpose, serviceable and maintained

NOTE: More than one control may be needed to reduce a hazard to a reasonable level.

Table 23: Project Risk Assessment

Risk/Hazard Description	Risk Mitigation Measures	Risk Score Before	Risk Score After
USQ Project approval	Start early discussions with possible Supervisor and submit topic request form early.	**E20**	**M11**
ESC Project approval	Start early discussions with ESC immediately explaining benefits of study.	**E20**	**M11**
Access to ESC documentation data and information	Make an early request once approval is received.	**E20**	**M11**
Adequate Internet Access for communications and literature research	Separate access points available home/work/mobile, maintain internet plans.	**H14**	**L2**
Computer Problems – loss of work and personal computer.	Save work progressively using both on-site and off-site storage. Access to multiple computers.	**M9**	**L3**
Site Visit to proposed WTP site: Sun, trip hazards, snake bite, steep terrain, working near roadway, remote location.	Site induction, follow ESC WHS policies, and use PPE including steel caps, high vis and sun protection. If possible, use buddy system for remote site.	**E23**	**M13**
Unable to develop model within time constraints and student ability.	Reduce scope, seek advice from supervisor, further research may be required.	**M9**	**L3**
Long hours sitting at desk using computer.	Take regular breaks, maintain good posture/ quality chair.	**H14**	**L4**
Allowance of adequate time to complete write-up of Dissertation.	Regular writing sessions, take annual leave or long service leave from work if needed.	**H14**	**L2**

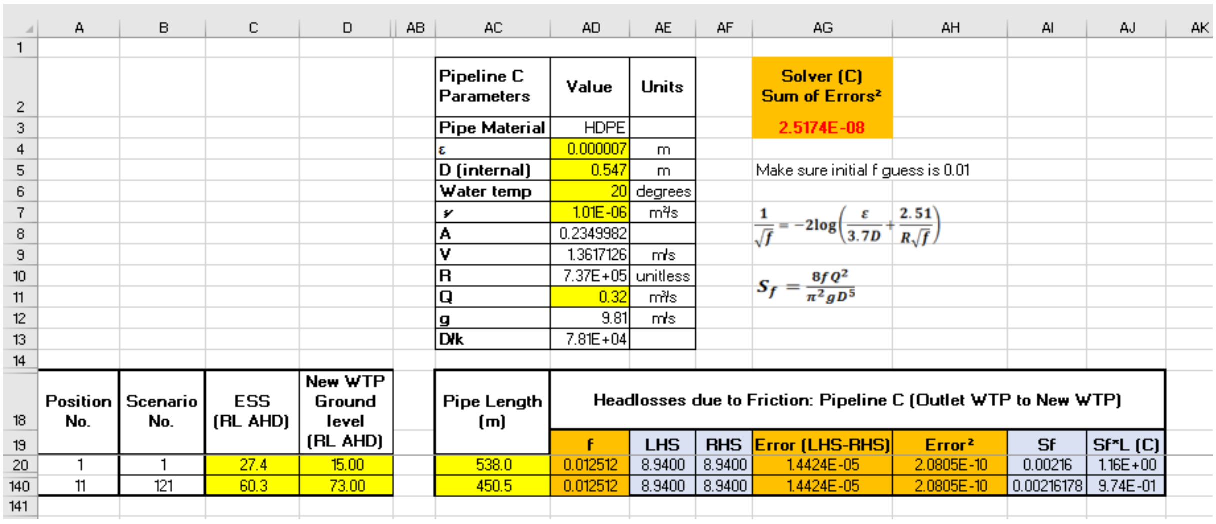
4 Model Operation and Verification

The purpose of this chapter is to demonstrate how the operation of the custom-built Microsoft Excel model, which utilises the constraints, inputs, outputs, key calculations and equations described in the previous chapter. Verification calculations and sensitivity analysis of the model are detailed within this chapter to ensure correct operation of the model and maintain quality of results. The operation is shown by a series of screen captures from the model which display examples for scenarios 1 and 121 with unnecessary rows and columns hidden for clarity. Verification calculations at the end of each sub-section are for scenario 121, position 11, the recommended option from the concept design at the ESS TWL RL 60.3 m AHD.

4.1 Hydraulic Inputs and Outputs

4.1.1 Model Operation

The first screen capture Figure 11 displays the model operation for the pipeline between the ESS and New WTP segment C. The hydraulic inputs discussed in section 3.3.1 from the previous chapter are shown in the yellow cells. Columns A through D are shown only for reference, whereas columns AC-AJ were used to determine the Reynolds number, the friction factors, friction slope and the head loss due to pipe friction.



Pipeline C Parameters	Value	Units
Pipe Material	HDPE	
ε	0.000007	m
D (internal)	0.547	m
Water temp	20	degrees
ν	1.01E-06	m²/s
A	0.2349982	
V	1.3617126	m/s
R	7.37E+05	unitless
Q	0.32	m³/s
g	9.81	m/s
D/k	7.81E+04	

Solver (C) Sum of Errors²

2.5174E-08

Make sure initial f guess is 0.01

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{\varepsilon}{3.7D} + \frac{2.51}{R\sqrt{f}}\right)$$

$$S_f = \frac{8fQ^2}{\pi^2 g D^5}$$

Position No.	Scenario No.	ESS (RL AHD)	New WTP Ground level (RL AHD)	Pipe Length (m)	Headlosses due to Friction: Pipeline C (Outlet WTP to New WTP)						
					f	LHS	RHS	Error (LHS-RHS)	Error²	Sf	Sf*L (C)
1	1	27.4	15.00	538.0	0.012512	8.9400	8.9400	1.4424E-05	2.0805E-10	0.00216	1.16E+00
11	121	60.3	73.00	450.5	0.012512	8.9400	8.9400	1.4424E-05	2.0805E-10	0.00216178	9.74E-01

Figure 11: Screen Capture - Hydraulic Inputs and Friction Calculation

The model operation can be seen under the merged cell titled '*Head losses due to Friction'* where an initial guess of 0.01 (not shown) for the friction factor is entered in the orange cells of column AD. The left-hand side (LHS) and right-hand side (RHS) of the Colebrook-White transition equation is then automatically calculated in columns AE and AF, respectively. The difference between these cells (LHS-RHS) is then displayed in the error column AG. This error is then squared to provide a positive value in column AH for further use with the solver add-in function.

After the initial tasks discussed in the previous paragraph were completed the solver add-in function was then utilised in cell AG3, which displays the summation of all the squared errors. This function was used to find the friction factor values shown in column AD, which consequently reduced the resultant of the summation of squared errors as close to zero as possible (LHS≈RHS). Once this condition was optimally satisfied, the friction slope for each scenario was automatically calculated in column AI. The friction slope was then multiplied by the relevant pipe length to calculate the friction loss for each pipeline segment in column AJ. The same operation was used for the other pipeline segments A, B and D for each scenario. The pipe friction outputs were then added together to determine the total friction loss for the pipeline from ESS to the new WTP (segments A, B & C) and the pipeline from the new WTP to Big Rock Reservoir (segment D).

The remaining hydraulic inputs required for operation of the model are shown in screen capture, Figure 12 with the ESS and new WTP ground level elevations in the yellow columns C & D. The new WTP inlet level was then calculated from column C plus an additional 8.5 m for the hydraulic profile of the new WTP and displayed in column E. The static head is then automatically calculated from the difference between columns C and E, displayed in column F. The outputs from the friction calculations discussed previously were populated under column G with the losses from fittings under column H. The fittings loss is calculated from velocity output shown in Figure 11, which is multiplied by the coefficients shown in screen capture Figure 14, on the next page. Finally, columns F, G and H are summed to determine the total head required for pumping in metres in column I. If the total head value displayed is negative, then no WPS was required to transport water from the ESS to the new WTP under that scenario.

	A	B	C	D	E	F	G	H	I	J
18 19	Position No.	Scenario No.	ESS (RL AHD)	New WTP Ground level (RL AHD)	New WTP Inlet Level (RL AHD)	Static Head ΔZ (m)	Friction Loss (m)	Fittings Loss (m)	Total head Required (m)	
20	1	1	27.4	15.00	23.5	-3.9	1.1646	4.08	1.34	
140	11	121	60.3	73.00	81.5	21.2	0.6188	4.08	25.90	
141										

Figure 12: Screen Capture - Hydraulic Outputs and Elevations for ESS to New WTP

The same operation was used to calculate for the total head required for pumping from the new WTP to Big Rock reservoir for each scenario in column H in the screen capture shown in Figure 14, with the two following exceptions. The first was the static head (column E) which was calculated from the difference between the elevations of the new WTP ground level and TWL of Big Rock reservoir in columns C & D. The second was the fittings head loss values displayed under column G, which were the resultant of a different velocity output due to the different hydraulic inputs (not shown). The head losses through fittings are shown in Figure 13, on the next page.

	A	B	C	D	E	F	G	H	I
1	ESS to New WTP					New WTP to Big Rock Reservoir			
2	Head losses through Fittings - Segment C					Head losses through Fittings - Segment D			
3	Description	Quantity	k	$k V^2/2g$		Fitting Description	Quantity	k	$k V^2/2g$
4	Globe valve, fully open	0	10.00	0.000		Globe valve, fully open	0	10.00	0.000
5	Angle valve, fully open	0	2.00	0.000		Angle valve, fully open	0	2.00	0.000
6	Gate valve, wide open	1	0.15	0.014		Gate valve, wide open	1	0.15	0.013
7	Gate valve, 1/4 closed	0	0.26	0.000		Gate valve, 1/4 closed	0	0.26	0.000
8	Gate valve, 1/2 closed	0	2.10	0.000		Gate valve, 1/2 closed	0	2.10	0.000
9	Gate valve, 3/4 closed	1	17.00	1.607		Gate valve, 3/4 closed	1	17.00	1.431
10	Ball valve, fully open	0	0.05	0.000		Ball valve, fully open	0	0.05	0.000
11	Ball valve, 1/3 closed	0	5.50	0.000		Ball valve, 1/3 closed	0	5.50	0.000
12	Ball valve, 2/3 closed	0	200.00	0.000		Ball valve, 2/3 closed	0	200.00	0.000
13	Diapham valve, fully open	0	2.30	0.000		Diapham valve, fully open	0	2.30	0.000
14	Diapham valve, 1/2 open	0	4.30	0.000		Diapham valve, 1/2 open	0	4.30	0.000
15	Diapham valve, 1/4 open	0	21.00	0.000		Diapham valve, 1/4 open	0	21.00	0.000
16	Water meter	0	7.00	0.000		Water meter	0	7.00	0.000
17	Swing check, forward flow	1	2.00	0.189		Swing check, forward flow	1	2.00	0.168
18	90° elbow, threaded	0	1.50	0.000		90° elbow, threaded	0	1.50	0.000
19	90° elbow, flanged	10	0.30	0.284		90° elbow, flanged	10	0.30	0.253
20	Long radius 90° elbow, threaded	0	0.70	0.000		Long radius 90° elbow, threaded	0	0.70	0.000
21	Long radius 90° elbow, flanged	0	0.20	0.000		Long radius 90° elbow, flanged	0	0.20	0.000
22	Regular 45° elbow, threaded	0	0.40	0.000		Regular 45° elbow, threaded	0	0.40	0.000
23	Long radius 45° elbow, flanged	15	0.20	0.284		Long radius 45° elbow, flanged	15	0.20	0.253
24	Long radius 45° elbow, threaded	0	0.20	0.000		Long radius 45° elbow, threaded	0	0.20	0.000
25	T,through side outlet	10	1.80	1.701		T,through side outlet	10	1.80	1.515
26	Bell mouth	0	0.98	0.000		Bell mouth	0	0.98	0.000
27	Square edge	0	0.82	0.000		Square edge	0	0.82	0.000
28			Total Fittings Head Loss =	4.08				Total Fittings Head Loss =	3.63
29									

Figure 13: Screen Capture – Head losses through Fittings

	A	B	C	D	E	F	G	H	I
16–17	Position No.	Scenario No.	BIG ROCK (RL AHD)	New WTP Ground Level (RL AHD)	Static Head ΔZ (m)	Friction Loss (m)	Fittings Loss (m)	Total head Required (m)	
18	1	1	143	15.00	128.0	13.74	3.63	145.37	
138	11	121	143	73.00	70.0	13.74	3.63	87.37	
139									

Figure 14: Screen Capture – Hydraulic Outputs and Elevations for New WTP to Big Rock Reservoir

4.1.2 Verification – Hydraulic Outputs

The following calculations in this section are for the verification of the hydraulic outputs which are shown above in the screen captures (Figures 10-14) and utilised within the model. The verification is provided in sub-sections with the first to confirm the hydraulic outputs from the ESS to the new WTP and the second to confirm the hydraulic outputs from the new WTP to Big Rock Reservoir. Furthermore, all key calculations were detailed in the previous methodology chapter.

Hydraulic Outputs – ESS to new WTP

Determination of Reynolds Number for Scenario 121 (segment C) using the calculation from Table 13, Methodology:

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

Where, V is 1.617 [m/s];

D is 0.547 [m]; and

v is 1.01×10^{-6} [m²/s].

Thus, $R = \frac{1.617 \times 0.547}{1.01 \times 10^{-6}} \approx 7.37 \times 10^{+5}$

Determination of Friction Factor for Scenario 121 (segment C) using Colebrook-White transition equation from Table 12, Methodology:

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{\varepsilon}{3.7D} + \frac{2.51}{R\sqrt{f}}\right)$$

Where, f is 0.012512 [unitless];

ε is 7×10^{-6} [m];

D is 0.547 [m]; and

R is $7.37 \times 10^{+5}$[unitless].

Thus, $LHS = \frac{1}{\sqrt{0.012512}} \approx 8.94$

$$RHS = -2\log\left(\frac{7 \times 10^{-6}}{(3.7 \times 0.547)} + \frac{2.51}{\left(7.37 \times 10^{+5}\sqrt{0.012512}\right)}\right) = 8.94$$

Hence, $LHS \approx RHS$.

Determination Friction Slope for scenario 121 (segment C) using the calculation from Table 11, Methodology:

$$S_f = \frac{8fQ^2}{\pi^2 g D^5}$$

Where, f is 0.012512 [unitless];

Q is 0.32 [m³/s];

g is ~9.81[m/s²]; and

D is 0.547 [m]

Thus, $\mathrm{S_f} = \frac{(8\times0.012512\times0.32^2)}{(\pi^2\times9.81\times0.547^5)} = 2.16179 \times 10^{-3}$

Determination Friction Head Loss for Scenario 121 (segment C) using adapted equation 2 from Methodology:

$$Friction\ Head\ Loss = S_f \times L$$

Where, S_f is 2.16179×10^{-3}[unitless]; and

L is 450.5 [m].

Thus, $Friction\ Head\ Loss = 2.16 \times 10^{-3} \times 450.5 = 0.974$ m

Determination of Friction Head Loss for Scenario 121, (segments A, B & C) pipeline from ESS to new WTP:

$$Friction\ Head\ Loss = A + B + C$$

Where, A is 21.2 [m];

B is 0.6188 [m]; and

C is 25.9 [m].

Thus, $Friction\ Head\ Loss = 21.2 + 0.6188 + 4.08 \approx 25.9\ m$

Determine Total Head Required for Scenario 121 using adapted Eq. 5, Methodology:

$$Total\ Head\ Required = [Static\ Head] + [Friction\ Head\ Loss] + [Minor\ Head\ Loss]$$

Where, $Static\ Head$ is 21.2 [m];

$Friction\ Head\ Loss$ is 0.6188 [m]; and

$Minor\ Head\ Loss$ is 4.08 [m].

Thus, $Total\ Head\ Required = [21.2] + [0.6188] + [4.08] \approx 25.90\ m$

Hydraulic Outputs – new WTP to Big Rock Reservoir

Determine Reynolds Number for Scenario 121 (segment D) using the calculation from Table 13, Methodology:

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

Where, V is 1.285 [m/s];

D is 0.547 [m]; and

v is 1.01×10^{-6} [m²/s].

Thus, $R = \frac{1.285 \times 0.547}{1.01 \times 10^{-6}} \approx 6.96 \times 10^{+5}$

Determine Friction Factor for Scenario 121 (segment D) using Colebrook-White transition equation from Table 12, Methodology:

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{\varepsilon}{3.7D} + \frac{2.51}{R\sqrt{f}}\right)$$

Where, f is 0.012629 [unitless];

ε is 7×10^{-6} [m];

D is 0.547 [m]; and

R is $6.96 \times 10^{+5}$[unitless].

Thus, $LHS = \frac{1}{\sqrt{0.012629}} \approx 8.898$

$$RHS = -2\log\left(\frac{7 \times 10^{-6}}{(3.7 \times 0.547)} + \frac{2.51}{\left(7.37 \times 10^{+5}\sqrt{0.012629}\right)}\right) \approx 8.898$$

Hence, $LHS \approx RHS$.

Determine Friction Slope for scenario 121 (segment D) using the calculation from Table 11, Methodology:

$$S_f = \frac{8fQ^2}{\pi^2 g D^5}$$

Where, f is 0.012629 [unitless];

Q is 0.302 [m³/s];

g is ~9.81[m/s²]; and

D is 0.547 [m]

Thus, $S_f = \frac{(8\times0.012629\times0.302^2)}{(\pi^2\times9.81\times0.547^5)} \approx 1.94 \times 10^{-3}$

Determine Friction Head Loss for Scenario 121, (segment D) pipeline from new WTP to Big Rock Reservoir using adapted Eq. 2, Methodology:

$$Friction\ Head\ Loss = S_f \times L$$

Where, S_f is 1.94×10^{-3}[unitless]; and

L is 7070 [m].

Thus, $S_f \times L = 1.94 \times 10^{-3} \times 7070 \approx 13.74$ m

Determine Total Head Required for Scenario 121 using adapted Eq. 5, Methodology:

$$Total\ Head\ Required = [Static\ Head] + [Friction\ Head\ Loss] + [Minor\ Head\ Loss]$$

Where, $Static\ Head$ is 70.0 [m];

$Friction\ Head\ Loss$ is 13.74 [m]; and

$Minor\ Head\ Loss$ is 3.63 [m].

Thus, $Total\ Head\ Required = [70] + [13.74] + [3.63] \approx 87.37$ m

4.2 Financial Inputs and Outputs

4.2.1 Model Operation

The outputs from the hydraulic worksheets discussed the previous section 4.1 were used to populate the yellow input columns E and V, as shown in the financial evaluation screen captures, Figures 14 and 15, respectively. The motor power required was then determined using the calculation detailed in Table 13, from the previous chapter in combination with an if statement. The if statement assigned a zero to any scenario that yielded a negative result from the previous total head calculation, as these scenarios didn't require a WPS to transport water to the new WTP as it could be gravitated. This value was then doubled to calculate the installed power shown in columns G and X, which allowed a duty and standby pump for each WPS.

The WPS capital costs shown in the orange columns (H and Y) in Figures 15 and 16 were populated using a vlookup function, which recalled the 2019 reference rate for WPS capital costs to the nearest installed kW from a separate worksheet. For verification purposes, Figure 17 is shown on the next page, it displays the reference rates from the worksheet with the capital cost shown on the y-axis against the installed kW on the x-axis. These values were obtained by the interpolation of the reported values from Table 18 in the methodology chapter except for the values between 1kw and 9kw, which were extrapolated. The remaining orange columns (J and AA) were then populated by multiplying the pipeline distance by the 2019 metre rate for the supply and installation of water mains from Table 16, in the methodology chapter. Finally, the total capex was determined by the summation of the WPS and pipeline capital costs in the said orange cells for each scenario.

	A	B	C	D	E	F	G	H	I	J	K	Q
1	ESS Outlet to New WTP											
2	Position No.	Scenario No.	ESS (RL AHD)	New WTP Ground level (RL AHD)	Total head Required (m)	Motor Power Required (kW)	Installed Power (kW)	WPS Capital Cost ($)	Pipeline Distance (m)	Pipeline Capital Cost ($)	Total CAPEX ($)	
3	1	1	27.4	15	1.34	5.9	12	$ 93,525.00	538	$ 439,492.20	$ 533,017.20	
123	11	121	60.3	73	26.25	115.5	231	$ 685,151.25	451	$ 368,013.45	$ 1,053,164.70	
124												

Figure 15: Screen Capture – Total Capex from ESS to New WTP

	R	S	V	W	X	AC	AD	AE	AF	AG	AH
1	New WTP to Big Rock Reservoir										
2	Position No.	Scenario No.	Total head Required (m)	Motor Power Required (kW)	Installed Power (kW)	Annual Power Consumption (kW)	Annual Power Cost ($)	Mech & Elec. Capital Cost ($)	Annual Maintenance Cost ($)	Annual OPEX ($)	
3	1	1	145.37	603.64	1207	5,067,552.87	$ 1,013,510.57	$ 2,104,360.88	$ 210,436.09	$ 1,223,946.66	
123	11	121	87.37	362.81	726	3,045,828.90	$ 609,165.78	$ 1,188,599.55	$ 118,859.96	$ 728,025.74	
124											

Figure 16: Screen Capture – Total Capex from New WTP to Big Rock Reservoir

The annual power consumption was then determined using the motor power values from columns G and W by multiplying the maximum operating hrs per day (23 hrs) by days per year (365 days). The resultant was then displayed in columns L and AC, which was further multiplied by the 2019 power usage rate of $0.20 per kWh to calculate the annual power cost in columns M and AE. Next, the mechanical and electrical capital costs shown in columns N and AE were populated in similar fashion to the WPS capital costs. This was achieved by using a vlookup function to recall the corresponding capital costs against the installed kW values shown in Figure 17, on the following page.

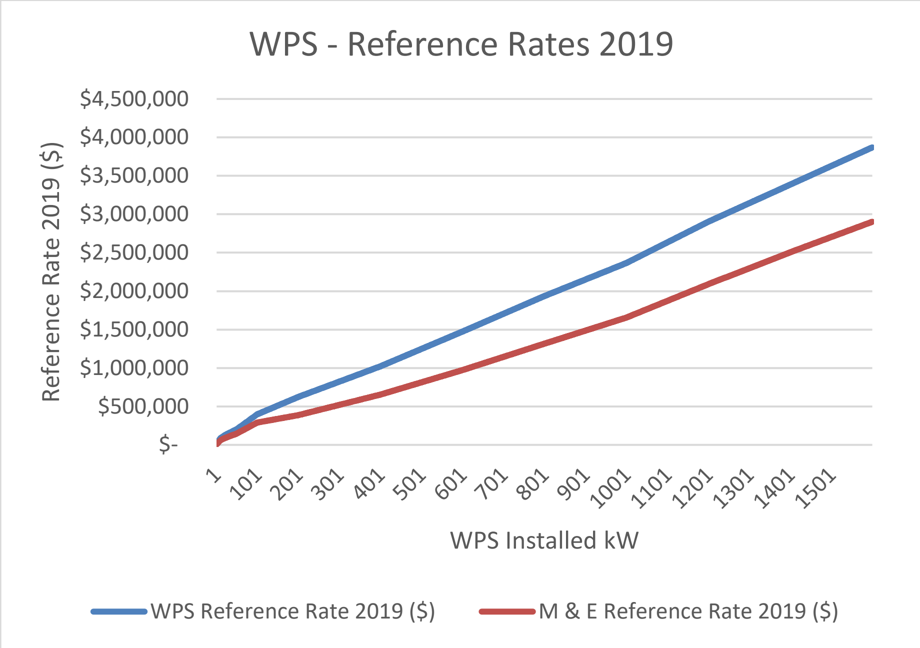


Figure 17: WPS - Reference Rates 2019

The annual maintenance costs were then determined by firstly multiplying the mechanical and electrical capital cost by the WPS capital cost to maintenance ratio (2.5) and secondly by dividing the result by the useable asset life (25 years) as discussed in section 3.3.2 of the previous chapter. Finally, the annual opex was determined by the addition of the annual power costs (columns M & AD) with the annual maintenance costs (columns O & AF) which were populated in the green columns P and AG, respectively.

	A	B	E	F	G	L	M	N	O	P	Q
1	ESS Outlet to New WTP										
2	Position No.	Scenario No.	Total head Required (m)	Motor Power Required (kW)	Installed Power (kW)	Annual Power Consumption (kW)	Annual Power Cost ($)	Mech & Elec. Capital Cost ($)	Annual Maintenance Cost ($)	Annual OPEX ($)	
3	1	1	1.34	5.9	12	49,595.26	$ 9,919.05	$ 64,779.50	$ 6,477.95	$ 16,397.00	
123	11	121	26.25	115.5	231	969,752.01	$ 193,950.40	$ 427,959.65	$ 42,795.97	$ 236,746.37	
124											

Figure 18: Screen Capture – Total Opex from ESS to New WTP

	R	S	V	W	X	AC	AD	AE	AF	AG	AH
1	New WTP to Big Rock Reservoir										
2	Position No.	Scenario No.	Total head Required (m)	Motor Power Required (kW)	Installed Power (kW)	Annual Power Consumption (kW)	Annual Power Cost ($)	Mech & Elec. Capital Cost ($)	Annual Maintenance Cost ($)	Annual OPEX ($)	
3	1	1	145.37	603.64	1207	5,067,552.87	$ 1,013,510.57	$ 2,104,360.88	$ 210,436.09	$ 1,223,946.66	
123	11	121	87.37	362.81	726	3,045,828.90	$ 609,165.78	$ 1,188,599.55	$ 118,859.96	$ 728,025.74	
124											

Figure 19: Screen Capture – Total Opex from New WTP to Big Rock Reservoir

4.2.2 Verification – Financial Outputs

The following calculations in this section are for verification of the financial outputs which were previously shown in screen captures (Figures 15-19). The verification is provided in sub-sections with the first to confirm the outputs from the ESS to the new WTP and the second to confirm the outputs from the new WTP to Big Rock Reservoir. Furthermore, all key calculations were detailed in the previous methodology chapter.

Financial Outputs – ESS to new WTP

Determination of Motor Power Required for Scenario 121 using the calculation from Table 17, Methodology:

$$Motor\ Power\ Required = \frac{Q \times h \times 1.1}{100 \times 0.8}$$

Where, Q is 320 [L/s]; and

h is 26.25 [m].

Thus, $Motor\ Power\ Required = \frac{320 \times 26.25 \times 1.1}{100 \times 0.8} = 115.5154\ kW$

Determination of Installed Power for Scenario 121:

$$Installed\ Power = Motor\ Power\ Required \times 2$$

Where, $Motor\ Power\ Required = 115.5154$ [kW].

Thus, $Installed\ Power = 115.5154 \times 2 \approx 231\ kW$

Determination of WPS Capital Costs for Scenario 121 using the 2019 Reference Rates from Table 18, Methodology (calculation not shown):

$$WPS\ Capital\ Cost = \$685{,}151.25$$

Figure 17 verifies that for a WPS when the installed power is 231 [kW] the corresponding reference rate is approximately \$685,000.

Determination of Pipeline Capital Cost for Scenario 121:

$$Pipeline\ Capital\ Cost = Pipeline\ distance \times Supply\ and\ installation\ rate$$

Where, $Pipeline\ distance$ is 451 [m]; and

$Supply\ and\ installation\ rate$ is 816.90 [\$/m].

Thus, $Pipeline\ Capital\ Cost = 451 \times 816.90 = \$368{,}421.90$

Determination of Total Capex Costs for Scenario 121, ESS to New WTP:

$$Total\ Capex\ Costs = WPS\ Capital\ Cost + Pipeline\ Capital\ Cost$$

Where, $WPS\ Capital\ Cost\ is$ 685,151.25 [\$]

$Pipeline\ Capital\ Cost$ is 368,421.90 [\$]

Thus, $Total\ Capex\ Costs = 685{,}151.25 + 368{,}421.90 = \$1{,}053{,}573.15$

Determination of Annual Power Consumption for Scenario 121:

$$Annual\ Power\ Consumption = Motor\ Power \times Max.Operating\ hrs \times Max.Operating\ days$$

Where, $Motor\ Power$ is 115.5154 [kW]

$Max.Operating\ hrs$ is 23 [hrs/day]; and

$Max.Operating\ days$ is 365 [days/year].

Thus, $Annual\ Power\ Consumption = 115.5154 \times 23 \times 365 \approx\ 969{,}752\ kW$

Determination of Annual Power Cost for Scenario 121:

$$Annual\ Power\ Cost = Annual\ Power\ Consumption\ \times Usage\ rate$$

Where, $Annual\ Power\ Consumption$ is 969,752 [kWh/year]; and

$Usage\ rate$ is 0.20 [$/ kWh].

Thus, $Annual\ Power\ Cost = 969{,}752 \times 0.20 = \$193{,}950.40$

Determination of Mechanical & Electrical Capital Costs for Scenario 121 using the 2019 Reference Rates from Table 18, Methodology (calculation not shown):

$$Mechanical\ \&\ Electrical\ \ Capital\ Cost = \$427{,}959.65$$

Figure 17 verifies that for a WPS when the installed power is 231 [kW] the corresponding reference rate for mechanical and electrical is approximately $430,000.

Determination of Annual Maintenance Costs for Scenario 121:

$$Annual\ Maintenance\ Cost = \frac{Mech.\&\ Elec.\ \ Capital\ Cost \times Capital\ Maintenace\ Ratio}{Asset\ Life}$$

Where, $Mech.\&\ Elec.\ \ Capital\ Cost$ is 427,959.65 [$]

$Capital\ Maintenace\ Ratio$ is 2.5 [unitless]

$Asset\ Life$ is 25 [years]

Thus, $Annual\ Maintenance\ Cost = \frac{427{,}959.65 \times 2.5}{25} = \$42{,}795.97$

Determination of Total Opex Costs for Scenario 121, ESS to New WTP:

$$Total\ Opex\ Costs = \ Annual\ Power\ Cost + Annual\ Maintenance\ Cost$$

Where, $Annual\ Power\ Cost\ is$ 193,950.40[$]

$Annual\ Maintenance\ Cost$ is 42,795.97 [$]

Thus, $Total\ Opex\ Costs = \ 193{,}950.40 + 42{,}795.97 = \$236{,}746.37$

Financial Outputs –new WTP to Big Rock Reservoir

Determination of Motor Power Required for Scenario 121 using the calculation from Table 17, Methodology:

$$Motor\ Power\ Required = \frac{Q \times h \times 1.1}{100 \times 0.8}$$

Where, Q is 302 [L/s]; and

h is 87.37 [m].

Thus, $Motor\ Power\ Required = \frac{302 \times 87.37 \times 1.1}{100 \times 0.8} = 362.8146\ kW$

Determination of Installed Power for Scenario 121:

$$Installed\ Power = Motor\ Power\ Required \times 2$$

Where, $Motor\ Power\ Required = 362.8146$ [kW].

Thus, $Installed\ Power = 362.8039 \times 2 \approx 726\ kW$

Determination of WPS Capital Costs for Scenario 121 using the 2019 Reference Rates from Table 18, Methodology (calculation not shown):

$$WPS\ Capital\ Cost = \$1{,}763{,}967.50$$

Figure 17 verifies that for a WPS when the installed power is 726 [kW] the corresponding reference rate is approximately $1,765,000.

Determination of Pipeline Capital Cost for Scenario 121:

$$Pipeline\ Capital\ Cost = Pipeline\ distance \times Supply\ and\ installation\ rate$$

Where, $Pipeline\ distance$ is 7070 [m]; and

$Supply\ and\ installation\ rate$ is 816.90 [$/m].

Thus, $Pipeline\ Capital\ Cost = 7070 \times 816.90 = \$5{,}775{,}483.00$

Determination of Total Capex Costs for Scenario 121, ESS to New WTP:

$$Total\ Capex\ Costs =\ WPS\ Capital\ Cost + Pipeline\ Capital\ Cost$$

Where, $WPS\ Capital\ Cost\ is$ 1,763,967.50 [$]

$Pipeline\ Capital\ Cost$ is 5,775,483.00 [$]

Thus, $Total\ Capex\ Costs =\ 1{,}763{,}967.50 + 5{,}775{,}483.00 = \$7{,}539{,}450.50$

Determination of Annual Power Consumption for Scenario 121:

$$Annual\ Power\ Consumption = Motor\ Power\ \times Max.Operating\ hrs \times Max.Operating\ days$$

Where, $Motor\ Power$ is 362.8039 [kW]

$Max. Operating\ hrs$ is 23 [hrs/day]; and

$Max. Operating\ days$ is 365 [days/year].

Thus, $Annual\ Power\ Consumption = 362.8146 \times 23 \times 365 \approx 3{,}045{,}828\ kW$

Determination of Annual Power Cost for Scenario 121:

$$Annual\ Power\ Cost = Annual\ Power\ Consumption\ \times Usage\ rate$$

Where, $Annual\ Power\ Consumption$ is 3,045,828 [kWh/year]; and

$Usage\ rate$ is 0.20 [$/ kWh].

Thus, $Annual\ Power\ Cost = 3{,}045{,}828 \times 0.20 = \$609{,}165.60$

Determination of Mechanical & Electrical Capital Costs for Scenario 121 using the 2019 Reference Rates from Table 18, Methodology (calculation not shown):

$$Mechanical\ \&\ Electrical\ \ Capital\ Cost = \$1{,}188{,}599.55$$

Figure 17 verifies that for a WPS when the installed power is 726 [kW] the corresponding reference rate for mechanical and electrical is approximately $1,188,599.55.

Determination of Annual Maintenance Costs for Scenario 121:

$$Annual\ Maintenance\ Cost = \frac{Mech. \&\ Elec.\ \ Capital\ Cost \times Capital\ Maintenace\ Ratio}{Asset\ Life}$$

Where, $Mech. \&\ Elec.\ \ Capital\ Cost$ is 1,188,599.55 [$]

$Capital\ Maintenace\ Ratio$ is 2.5 [unitless]

$Asset\ Life$ is 25 [years]

Thus, $Annual\ Maintenance\ Cost = \frac{1{,}188{,}599.55 \times 2.5}{25} \approx \$118{,}859.96$

Determination of Total Opex Costs for Scenario 121, ESS to New WTP:

$$Total\ Opex\ Costs = \ Annual\ Power\ Cost + Annual\ Maintenance\ Cost$$

Where, $Annual\ Power\ Cost\ is$ 609,165.60 [$]

$Annual\ Maintenance\ Cost$ is 118,859.96 [$]

Thus, $Total\ Opex\ Costs = \ 609{,}165.60 + 118{,}859.96 = \$728{,}025.56$

4.3 Net Present Value Input and Outputs

4.3.1 Model Operation

The screen capture Figure 20 displays the NPV inputs of discount rate and planning horizon in the yellow cells C1 and C2, respectively. These inputs determine the PV discount factor visible in the orange cell C3 using the key calculation discussed in the previous chapter, section 3.3.2. The position and scenario numbers were populated in columns A and B for reference only with the remaining worksheet is split into two sub-sections to determine the NPV separately for the infrastructure required to transfer water from the ESS outlet to New WTP and the New WTP to Big Rock Reservoir.

	A	B	C	D	E	F	G	H	I	J	K	L
1	Interest rate (%)		0.07									
2	Horizon (yrs)		25									
3	PV Discount Factor		11.65									
4												
5	Position	Scenario	ESS Outlet to New WTP				New WTP to Big Rock Reservoir				TOTAL NPV ($)	
6	No.	No.	CAPEX NPV ($)	Annual OPEX ($)	OPEX NPV ($)	Sub-Total NPV ($)	CAPEX NPV ($)	Annual OPEX ($)	OPEX NPV ($)	Sub-Total NPV ($)		
7	1	1	$ 533,017.20	$ 16,396.98	$ 191,083.52	$ 724,100.72	$ 8,692,872.00	$ 1,223,946.66	$ 14,263,364.24	$ 22,956,236.24	$ 23,680,336.96	
127	11	121	$ 1,053,164.70	$ 236,746.34	$ 2,758,943.22	$ 3,812,107.92	$ 7,539,450.50	$ 728,025.74	$ 8,484,108.46	$ 16,023,558.96	$ 19,835,666.88	
128												

Figure 20: Screen Capture - Net Present Value

The Capex NPV (columns C and G) and Annual Opex (columns D and H) values are automatically populated from the financial outputs which were previously discussed in section 4.2, with total Capex values equal to the Capex NPV values. This is because the capital works would occur at the start of the first year of the planning horizon and therefore not require discounting back to the PV. In contrast, the annual opex values are multiplied by the discount rate to bring the annual values back to the present. The NPVs are then sub-totalled in the blue columns F and J, which were then added together to calculate the Total NPV value for each of the scenarios.

4.3.2 Verification – Net Present Value

The following verification calculations are provided for validation of the model and are detailed below for the values shown in screen capture (Figure 20) from the previous section. The calculations are provided in two sub-sections for the infrastructure required from the ESS to the new WTP and from the new WTP to Big Rock Reservoir before being added together to calculate the Total NPV values for each scenario.

NPV – ESS to new WTP

Determination of the Opex NPV for Scenario 121 using the calculation from Table 19, Methodology:

$$Opex\ NPV = A\left[\frac{1-(1+i)^{-n}}{i}\right]$$

When, A is 264,923.46 [$];

i is 7 [%]; and

n is 25 [years].

Thus, $Opex\ NPV = 264{,}923.46\left[\frac{1-(1+0.07)^{-25}}{0.07}\right] \approx 264{,}923.46[11.65] \approx \$3{,}087{,}307.85$

Determination of the Sub-Total NPV for Scenario 121:

$$Sub - Total\ NPV = Opex\ NPV + Capex\ NPV$$

When, Capex NPV is 1,110,838.45 [$]

Opex NPV is 3,087,307.85 [$]

Thus, $Sub\text{-}Total\ NPV = 1{,}110{,}835.45 + 3{,}087{,}307.85 = \$4{,}198{,}146.30$

NPV – new WTP to Big Rock Reservoir

Determination of the Opex NPV for Scenario 121 using the calculation from Table 19, Methodology:

$$Opex\ NPV = A\left[\frac{1-(1+i)^{-n}}{i}\right]$$

When, A is 728,025.74 [$];

i is 7 [%]; and

n is 25 [years].

Thus, $Opex\ NPV = 728{,}025.74\left[\frac{1-(1+0.07)^{-25}}{0.07}\right] \approx 728{,}025.74[11.65] \approx \$8{,}484{,}108.46$

Determination of the Sub-Total NPV for Scenario 121:

$$Sub - Total\ NPV = Opex\ NPV + Capex\ NPV$$

When, Capex NPV is 1,110,838.45 [$]

Opex NPV is 3,087,307.85 [$]

Thus, $Sub\text{-}Total\ NPV = 1{,}110{,}835.45 + 3{,}087{,}307.85 = \$4{,}198{,}146.30$

4.3.3 Sensitivity Analysis of NPV

As discussed earlier in the methodology chapter, it was recommended that sensitivity analysis be undertaken for NPV calculations at both 3% and 10% to ensure the hydraulic and financial outcomes were not influenced by the adopted interest rate of 7%. This was achieved by duplicating the NPV worksheet previously discussed in section 4.3.1 and changing the interest rate input to alter NPVs.

Figure 21 shown below, displays the graph lines plotted for the NPVs derived from each interest rate over the same 25-year planning horizon. The scenarios are listed on the x-axis with the corresponding NPV dollar values on the y-axis. From this graph it was evident the lines for each NPV was consistent across all 121 scenarios and therefore regardless of the interest rate the optimum solution produced from the model would still remain the same

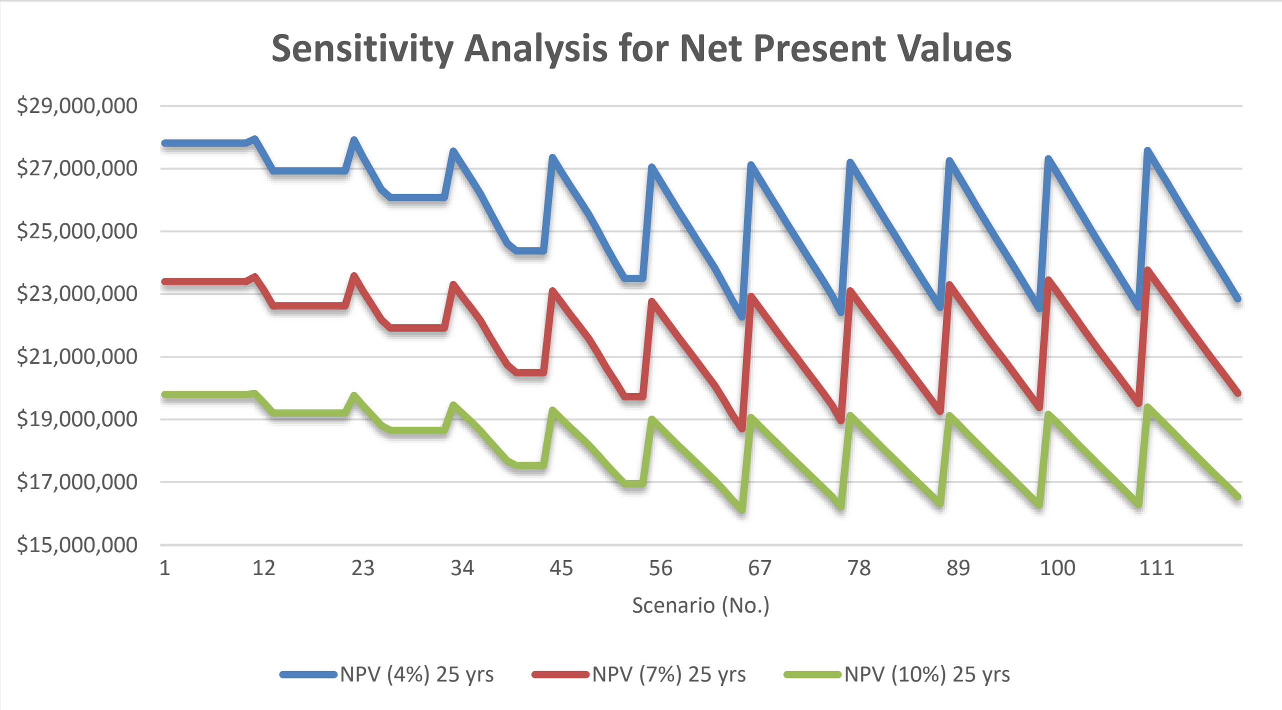


Figure 21: Sensitivity Analysis of Net Present Value

5 Results and Discussion

The following chapter presents, analyses and provides discussion on the results produced from the hydraulic model, financial analysis and NPV evaluation. The hydraulic results are broken down and presented in a similar structure to the previous chapters. This is to ensure clarity of the results is achieved for the water infrastructure required to first, transfer stored water from the ESS to the new WTP and second transfer water from the new WTP to the existing Big Rock Reservoir. In comparison, the latter results for the financial analysis and NPV evaluation are combined for presentation and discussion.

5.1 Hydraulic Results

5.1.1 Eurobodalla Southern Storage to New WTP

This section presents the results for the pumping head (in metres) required to transfer stored water from the ESS to the new WTP under a range of ESS operating levels. A series of 11 graphs are shown as Figures 22-32, for each ESS operating level adopted for investigation. The graphs display the pumping head required in metres on the y-axis for each of the corresponding WTP positions on the x-axis. The results from the graphs were as expected with friction and minor losses having minimal impact in comparison to the static head. This was evident, due to the plotted lines remaining relatively linear despite the fact the pipeline distances decreased towards the middle of the positions.

An additional series of graphs are also included and shown as Figures 33-44. These graphs are configured to display each of the ESS water levels on the y-axis against the corresponding pumping head in metres required to transfer the water from the ESS to the new WTP. Moreover, each of the possible positions for the new WTP was allocated an individual graph to emphasize the effect on the pumping requirements caused by the ESS water level with a reference line also included to highlight the location on the zero location of the x-axis. As such, all points plotted on the right-hand side of the reference line would require a WPS to transfer water to the new WPS.

Figure 33 shows that if the MOL was raised from 27.3 m to 30 m, then position 1 would not require a WPS to transfer water to the new WTP. Also, notable Figure 36 shows that pumping is required until the reference line is intersected at the approximate height of the stage one FSL at 47.7 m.

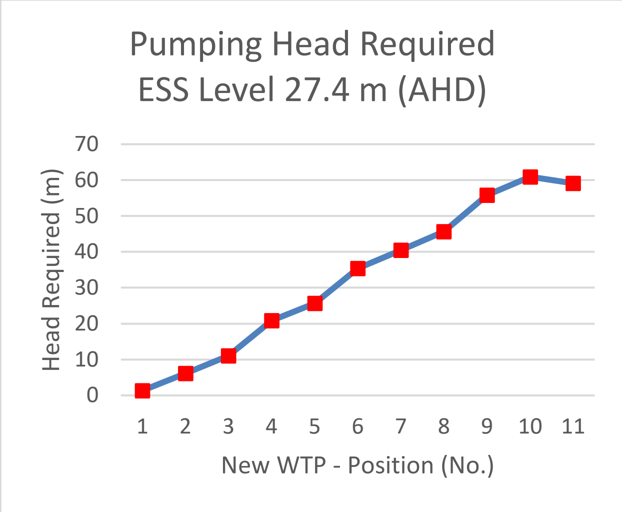


Figure 22: Pumping Head Required at ESS Level 27.4 m

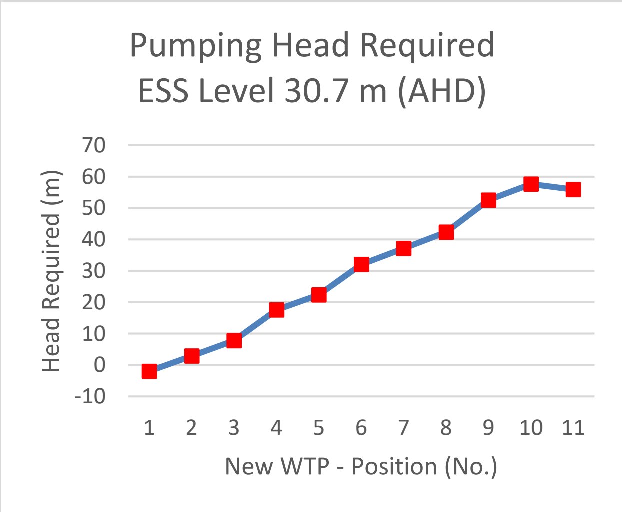


Figure 23: Pumping Head Required at ESS Level 30.7 m

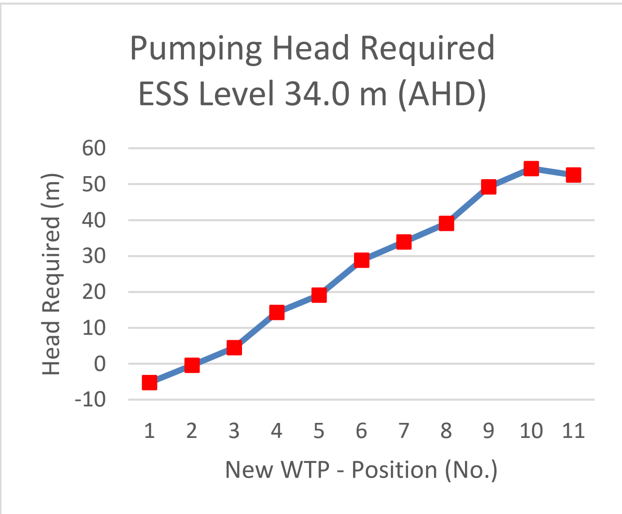


Figure 24: Pumping Head Required at ESS Level 34.0 m

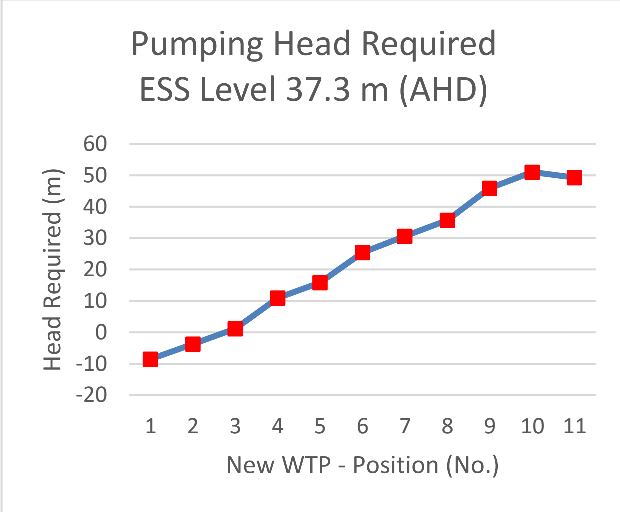


Figure 25: Pumping Head at ESS Water level 37.3 m

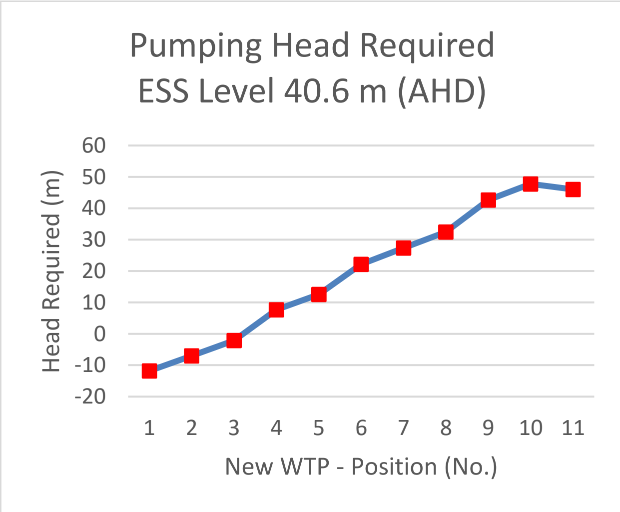


Figure 26: Pumping Head Required at ESS Level 40.6 m

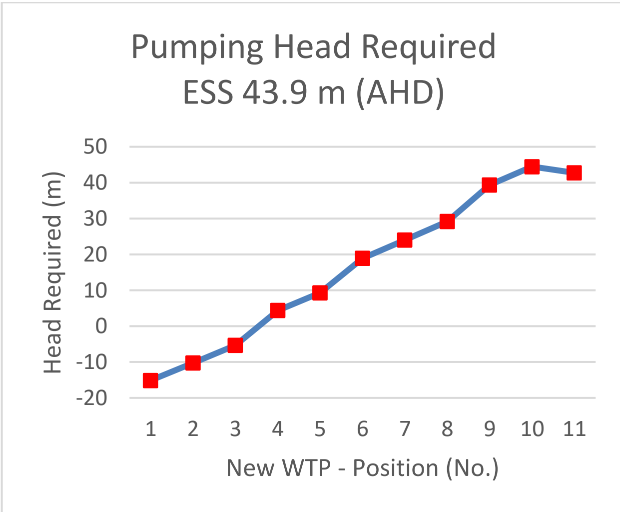


Figure 27: Pumping Head Required at ESS Level 43.9 m

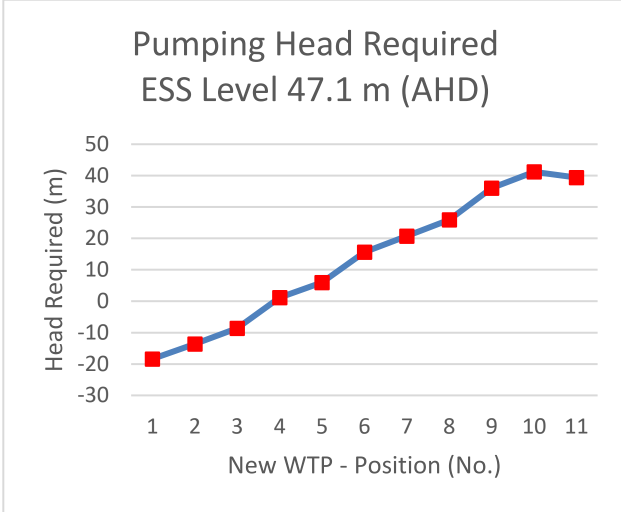


Figure 28: Pumping Head at ESS Water Level 47.1 m

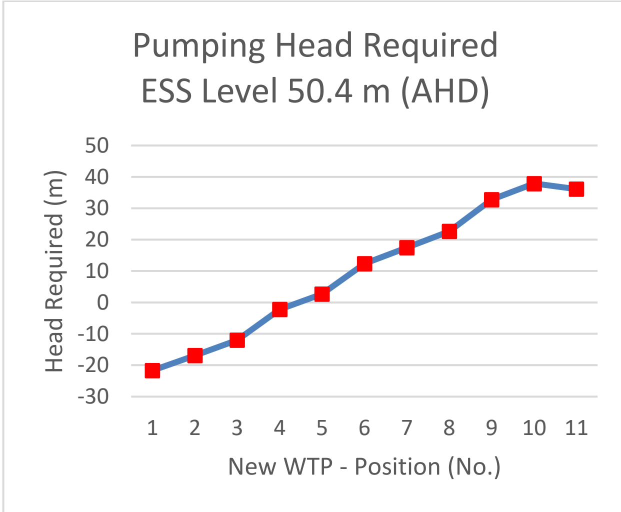


Figure 29: Pumping Head at ESS Water Level 50.4 m

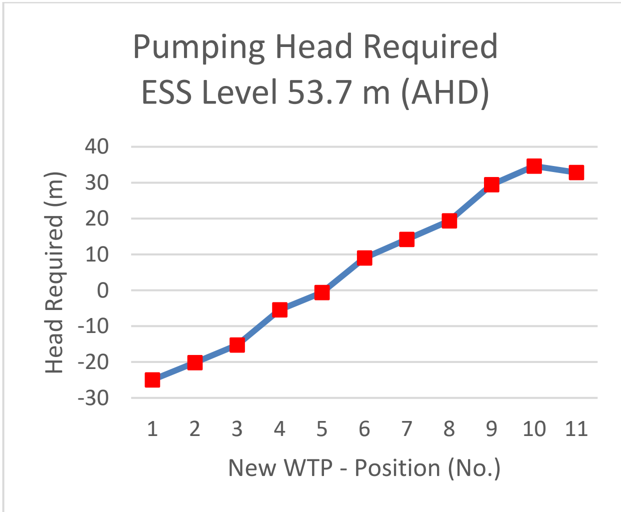


Figure 30: Pumping Head Required at ESS Level 53.7 m

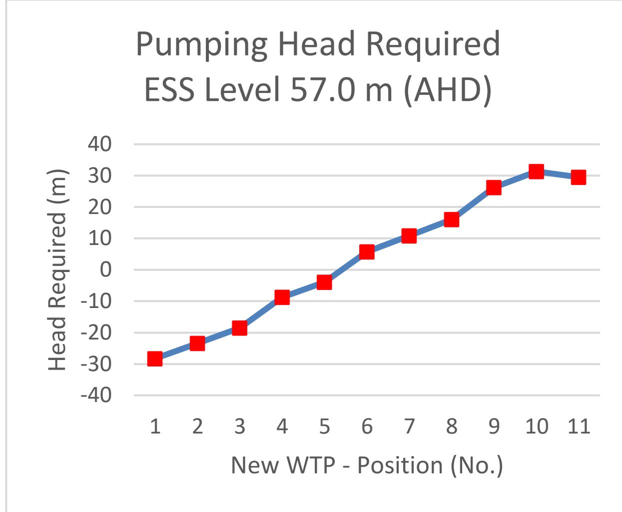


Figure 31: Pumping Head at ESS Water Level 57.0 m

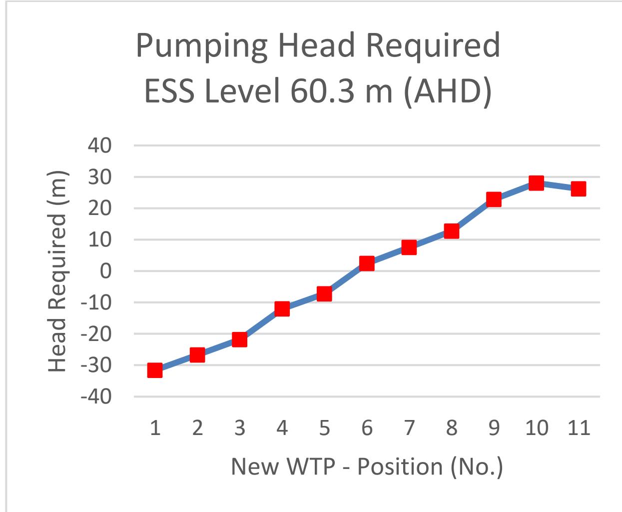


Figure 32: Pumping Head at ESS Water Level 60.3 m

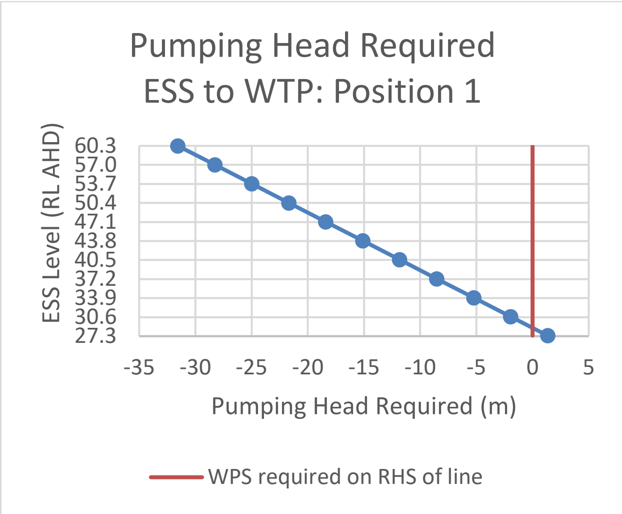


Figure 33: ESS Water Level vs Pump Head - Position 1

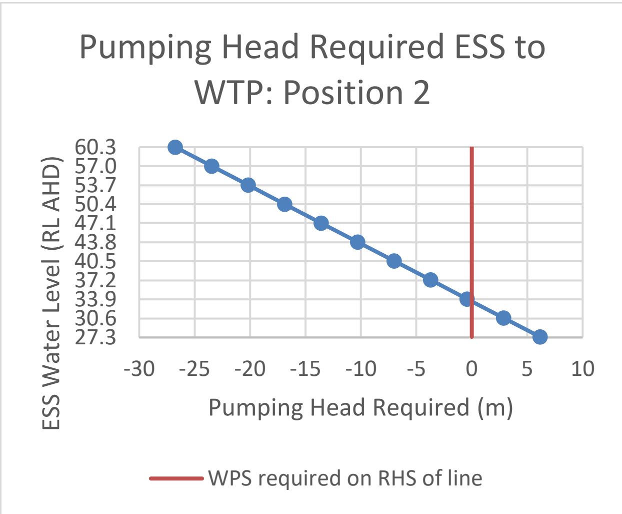


Figure 34: ESS Water Level vs Pump Head - Position 2

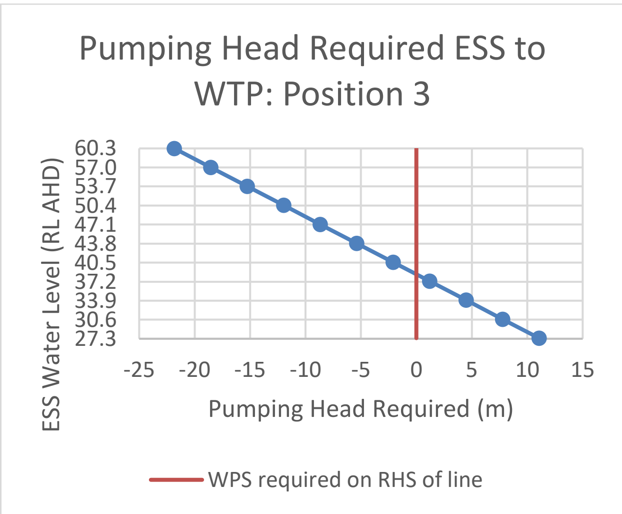


Figure 35: ESS Water Level vs Pump Head - Position 3

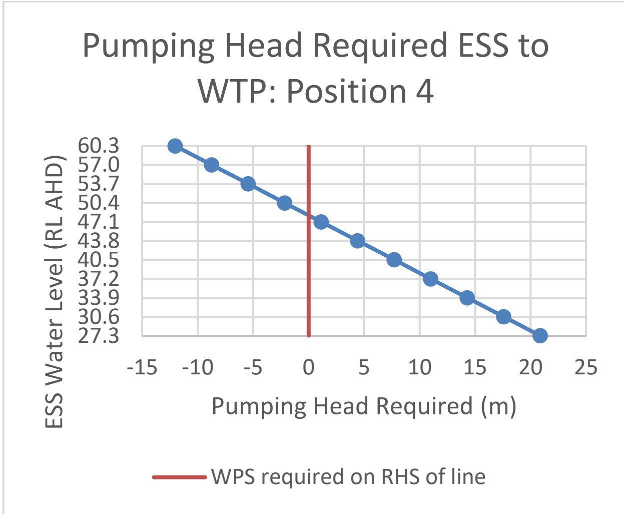


Figure 36: ESS Water Level vs Pump Head – Position 4

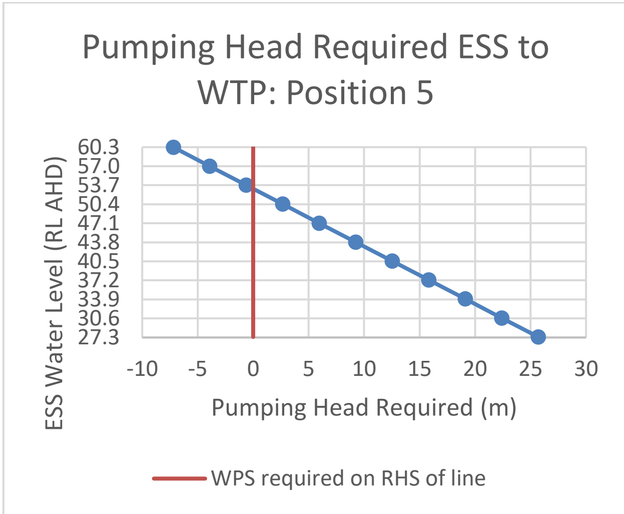


Figure 37: ESS Water Level vs Pump Head – Position 5

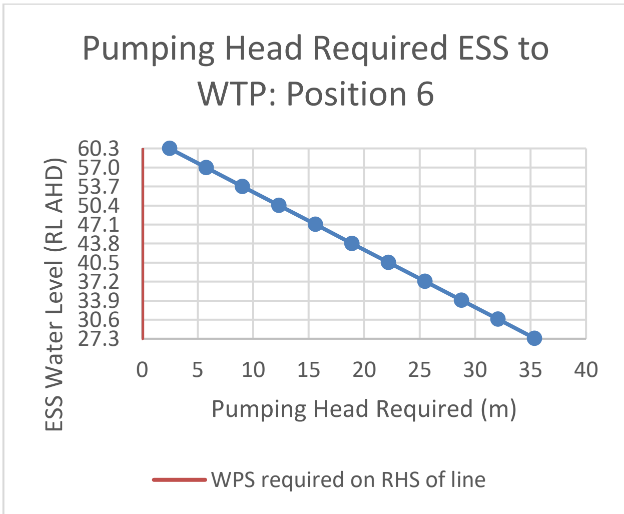


Figure 38: ESS Water Level vs Pump Head – Position 6

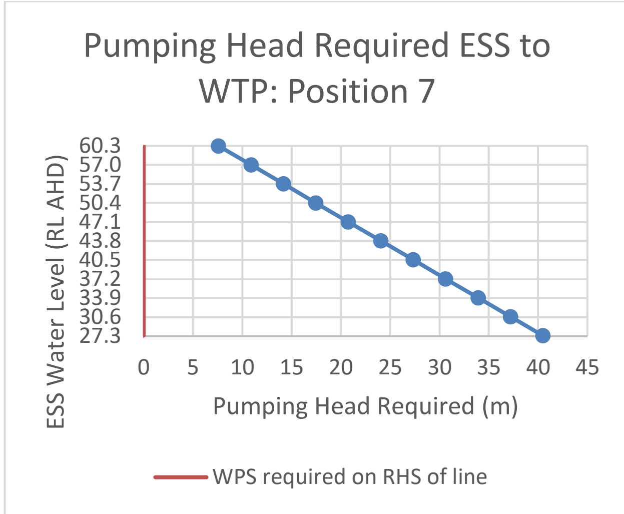


Figure 39: ESS Water Level vs Pump Head – Position 8

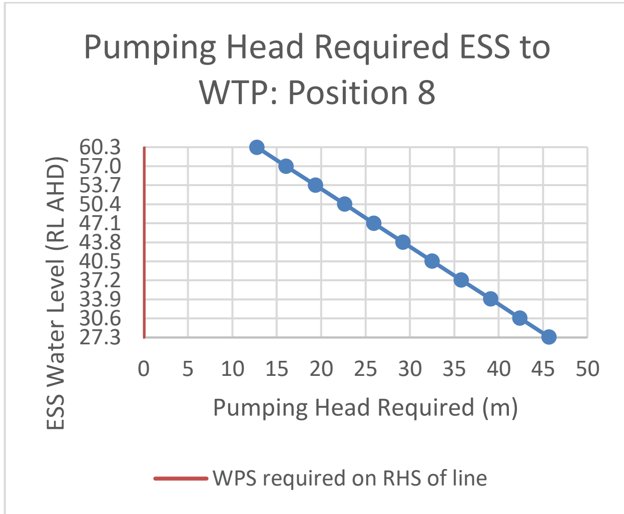


Figure 40: ESS Water Level vs Pump Head – Position 9

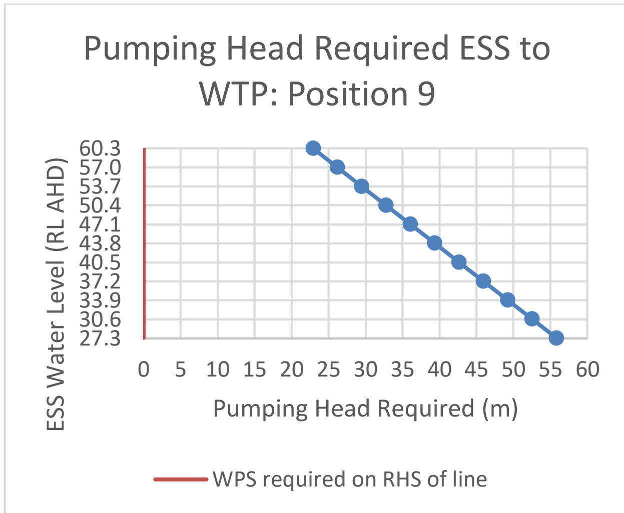


Figure 41: ESS Water Level vs Pump Head – Position 10

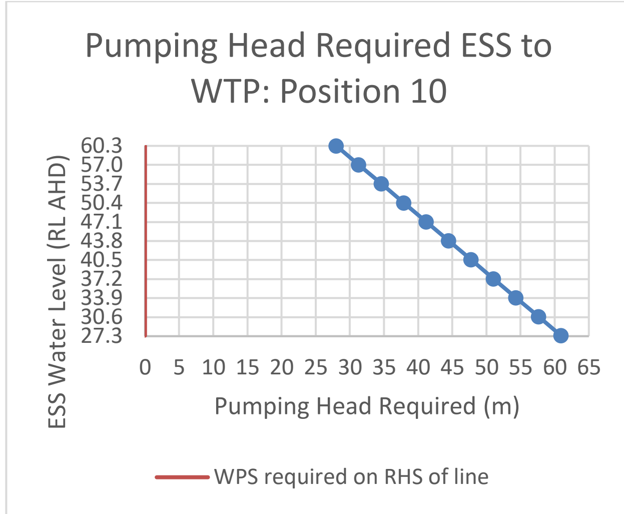


Figure 42: ESS Water Level vs Pump Head – Position 10

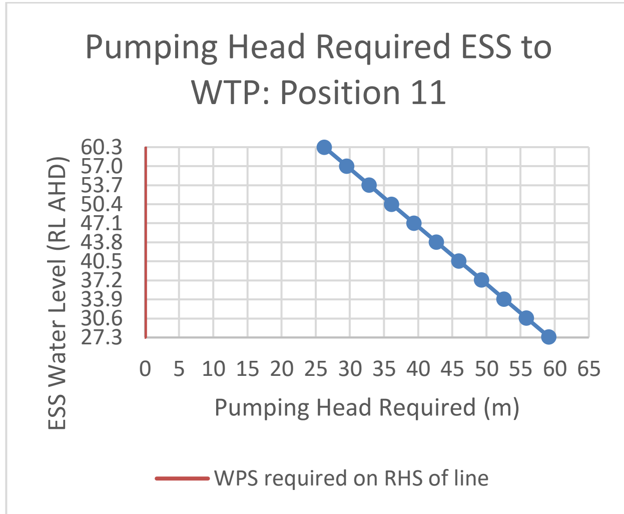


Figure 43: ESS Water Level vs Pump Head – Position 11

5.1.2 New WTP to Big Rock Reservoir

This section presents the hydraulic results from the model, again in the form of pumping head (in metres) required to transfer potable water from the new WTP clear-water WPS to the existing Big Rock Reservoir located approximately 7 km away. As previously discussed, the ESS and existing reservoir operating levels have no impact on the static head derived for this WPS. This is because any head pressure gained from the water level within the ESS and ESS outlet WPS is lost at the inlet to the new WTP, which shall be open to atmospheric pressure. Additionally, the reservoir is required to be maintained at the TWL for firefighting and to avoid disruptions to supply. Therefore, only one graph was produced as Figure 44, as there was no variance in the operating levels.

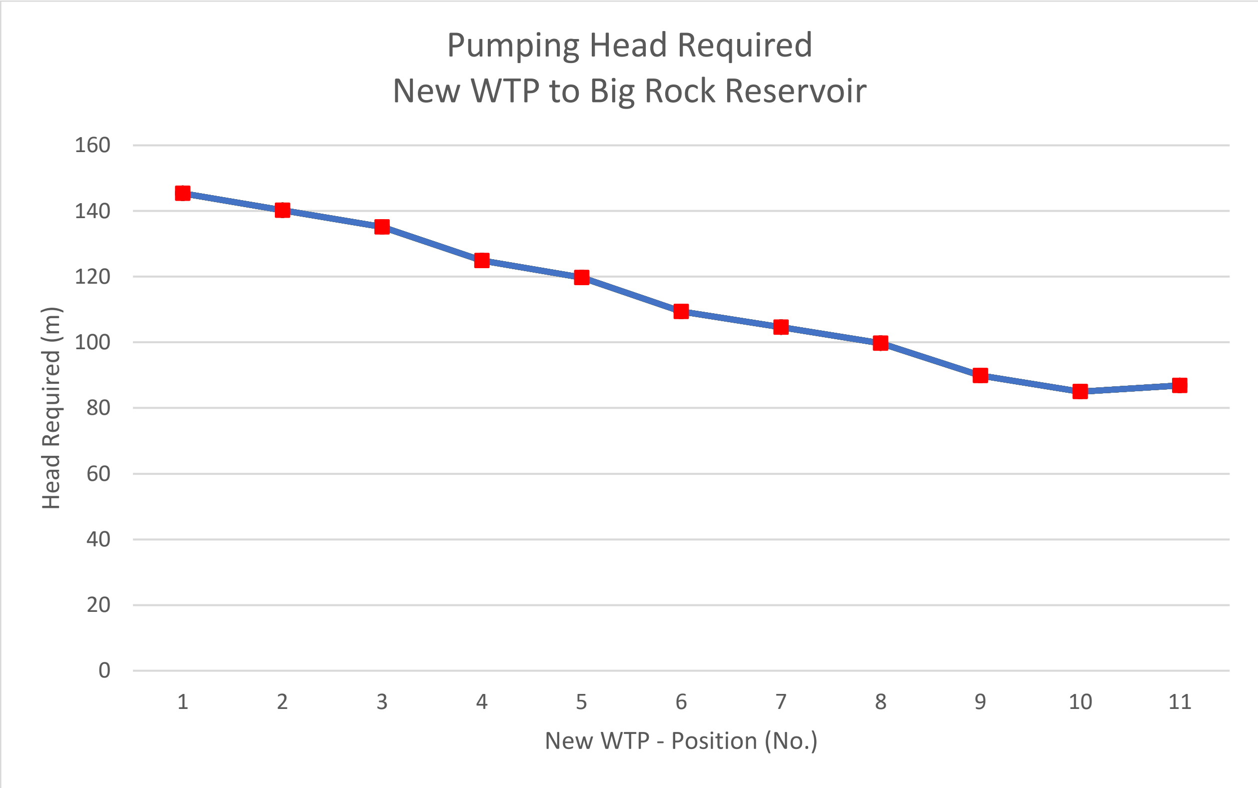


Figure 44: Pumping Head Required - New WTP to Big Rock Reservoir

The graph displays the pumping head required on the y-axis for each of the corresponding new WTP positions denoted on the x-axis. The plot on this graph was similar to Figures 22-32, as the difference in the pipeline distances for each position was not large enough for friction and minor losses to impact the plotted line relative to the static head. Again, this was evident, due to the plotted lines remaining relatively linear despite the fact the pipeline distances decreased towards the middle of the positions. The only exception was position 11 that yielded a higher head then position 10, which was due to additional pipeline distances required. Moreover, the plot decreased in pumping head from position 1 to position 10, which was also expected due to the lower static heads of the WTP positions located at the higher elevations.

5.2 Financial Evaluation

The key step to linking the hydraulic results from the model for financial evaluation involved converting the outputs to monetary terms. This was achieved by firstly applying the supply and installation costs to the pipe lengths between the water infrastructure to determine the capex, which is discussed later in this section under total capex. Then secondly by using the pumping heads to determine the installed power requirements for the WPSs and finding the corresponding reference rate as discussed in the methodology chapter.

Figure 45 displays the WPS installed power sizes for both the ESS outlet WPS and the clear-water WPS. The installed power is displayed in kilo-Watts on the y-axis and the scenarios are grouped in their position numbers on the x-axis. From this graph, it is evident that positions 1 through 3 do not require a WPSs under all ESS operational levels. However, this has resulted in significantly larger WPSs required to pump potable water from the WTP to Big Rock Reservoir. It is also evident that the variance between the installed power of the WPSs decreases with increasing elevation. This was also expected as position 1 was listed in the methodology at the lowest elevation through to 10 at the highest.

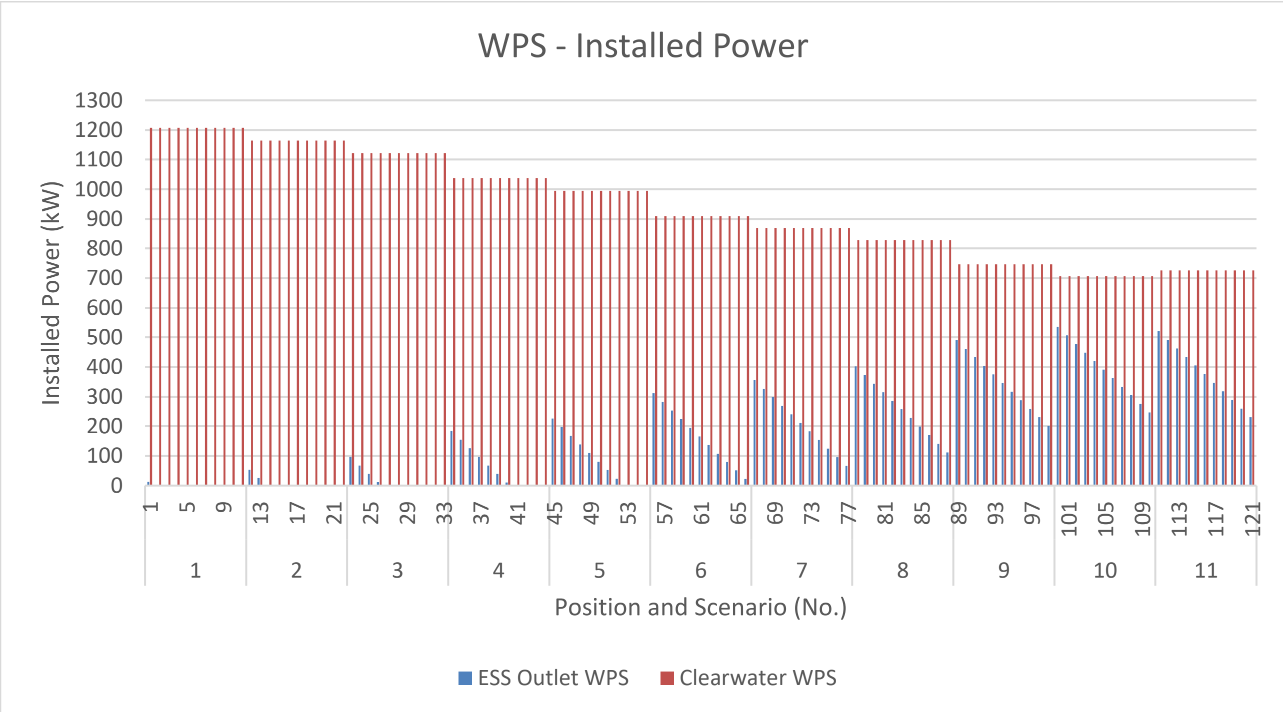


Figure 45: WPS – Installed Power by new WTP Position

The remaining part of the financial evaluation section presents a series of vertical box and whisker plots (Figures 46-47) for the total capex, opex and NPV analysis for the new WTP positions. These plots are consistent in format to provide continuity of results for discussion and to summarise the 121 scenarios modelled. The dollar values are displayed on the y-axis and WTP position numbers shown on the x-axis. For each plot, the maximum and minimum dollar values are displayed at the ends of the extended bars.

These bars are located on the top and base of the fourth and first quartiles, respectively. The two middle quartiles are then displayed as separate rectangles with the third positioned on top of the second with the median value located where they meet. However, for some positions one or both the middle quartile rectangles may not be shown, due to less than a quarter of values falling into those quartiles. For the occurrences where both quartiles are not shown the median is considered irrelevant.

5.2.1 Total Capital Costs (Capex)

Figure 46 displays the values obtained from the total capex, which was the addition of the pipeline supply and installation capex with the capex required to build WPS as identified from the hydraulic model. Interestingly, the first three plots displayed on the left-hand side on the graph indicate a lack of variance between the scenarios modelled for positions 1-3. This has arisen because less than a quarter of the results for positions 1-3 required a WPS at the ESS outlet, which was previously discussed in the above section 5.1.1. Consequently, for these positions the total capex for the remaining attributes resulted in equal dollar values for a large portion of the scenarios within each possible position.

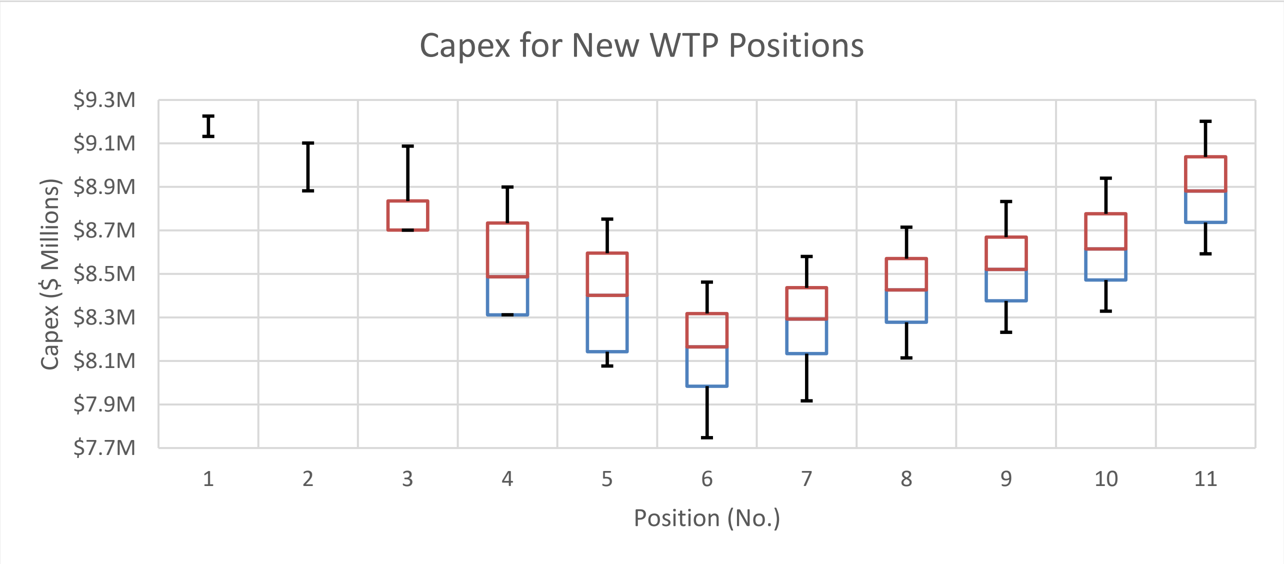


Figure 46: Capex for New WTP Positions

The maximum value within the capex results is approximately $9.2 M at position 1 with a minimum value of approximately $7.7 M at position 6. It was also noted that position 6 contained the lowest values for the maximum, third quartile, median and second quartile. This suggests for total capex the optimal solution is position 6. This result was a direct outcome of the ESS operating levels having no impact on the pipeline differences from the ESS outlet to the new WTP or for the infrastructure required from the new WTP to the existing Big Rock Reservoir. Therefore, as previously discussed, where no middle quartiles are shown the second or third quartile must be equal to the minimum or maximum value respectively for that position. Moreover, for position 4 the second quartile is equal to the minimum value.

5.2.2 Total Operational Costs (Opex)

Figure 47 shown below, displays the results obtained from the total opex for each position for the new WTP. The results were determined by the summation of the annual power and maintenance opex required for each of the scenarios modelled, as discussed in previous chapters. Similar results are shown for the first three positions with some quartiles not being defined for the same reasons discussed above in section 5.2.1.

The maximum value for the opex results shown in Figure 47 was approximately $1.2 M and yielded by position 11. This was higher than position 10 by less than five thousand dollars, which was due to the slightly larger head required to pump from the new WTP to Big Rock Reservoir.

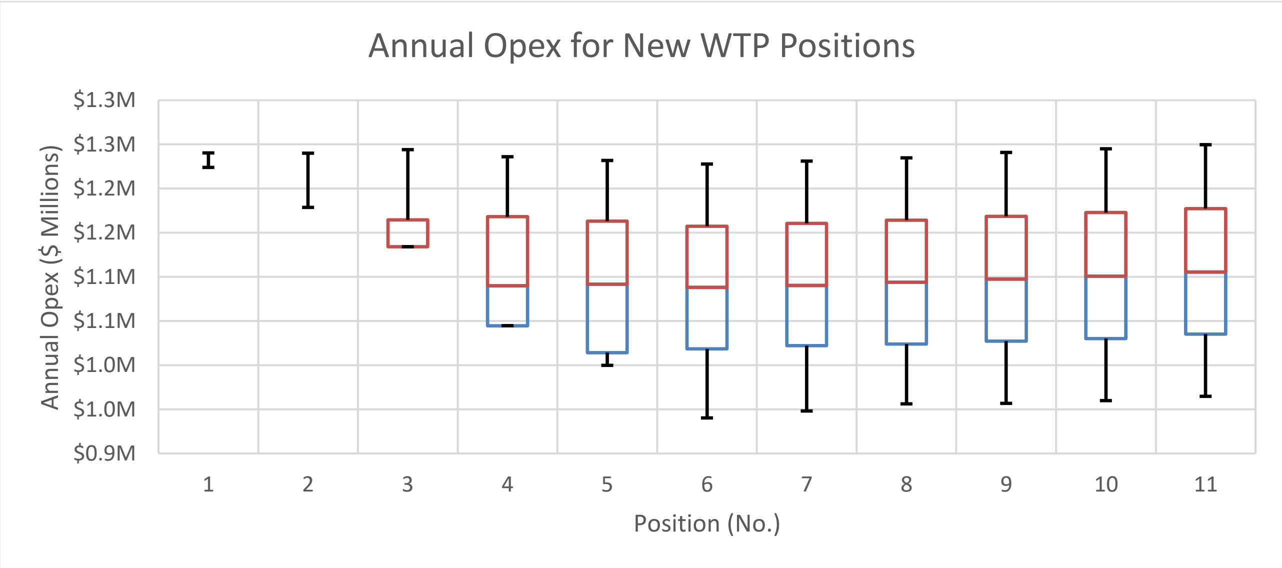


Figure 47: Annual Opex for New WTP Positions

Furthermore, the variance for all the opex maximum values was minimal and ranged approximately between $1.24 M to $1.25 M. The minimum opex value was again yielded by position 6, which was under $1 M. Position 6, also displayed the lowest results for the maximum, third quartile, median and second quartile. However, the variance within the range of positions for opex was small in comparison to capex which ranged from $0.3 M as opposed to $1.5 M by the latter, demonstrating the potential savings available over the life of the infrastructure.

5.2.3 Net Present Value Analysis

The final box and whisker plot is shown below as Figure 48 with the results derived from the capex and opex values provided from the model. These values were then discounted back to present value and totalised in accordance with the key calculations detailed in the methodology and model development chapters. Again, it is noted that positions 1 and 2 contain no middle quartiles with position 3 containing the third.

The maximum NPVs for the positions had a range of $23.8 M at position 1 to $22.7 M at position 6. In comparison, the minimum NPVs for the positions had a range from the highest minimum NPV of $23.4 M at position 1 to the lowest minimum of $18.7 M at position 6. Position 6 also displayed the lowest results for the maximum, third quartile, median and second quartile and is again the optimal solution. Furthermore, for positions 1, 2 and 3 it is noted that the minimum NPVs are higher than the majority of the third quartiles for the remaining positions, which has been a consistent theme through Figures 46-48 emphasising a lack of variance.

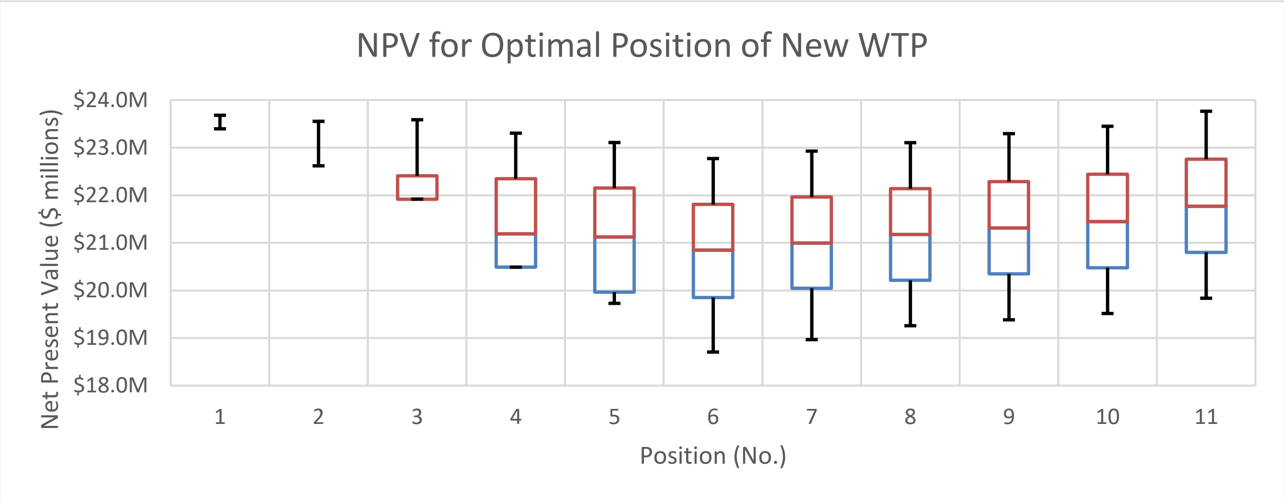


Figure 48: NPV Analysis for optimal position of new WTP

5.3 Further Discussion

It is noted from the results presented in the previous sections that position 6 is the optimal solution for the future position of the proposed WTP. All three box and whisker plots displayed decreasing trends from the outer positions towards the optimal position 6. Position 6 also had the lowest values for the maximum, third quartile, median, second quartile and minimum under all scenarios modelled. To understand why this has occurred requires reflection on the key calculations that determined the capex, opex and NPVs.

The pipeline distances from the methodology chapter, section 3.2.5 show position 6 had the smallest distances recorded for the pipelines between the ESS to the new WTP and from the new WTP to Big Rock Reservoir at 80.5 m and 6590 m, respectively. These distances were then used to calculate the pipe frictions which were added to the static head with the minor losses. Position 6, under all hydraulic results discussed in section 5.1, ranked mid-range due to the elevation, which was at ground level elevation 50 m (RL AHD) just slightly higher than the ESS stage one FSL of 47.7 m. Moreover, position 6 was also the first position that required a WPS to transfer water from the ESS to the new WTP under all the ESS operating levels modelled.

As previously discussed, the results from the hydraulic model consequently determined the installed power requirements for each WPS and were then used to calculate the capex and opex required to construct, operate and maintain the WPSs. Therefore, Figure 17 from the model operation and verification chapter was reviewed, as it displays the corresponding reference rates against installed power. This revealed that inclines of differing slope exist within the graph between the known data points, which were taken from Table 18 as shown in the Methodology chapter. Most notable was the incline between the 100 kW through to 1000 kW installed power size, which is on a lower incline than the extremities. This was also the case with the mechanical and electrical incline following a similar trajectory.

This information provides an insight for the reason position 6 resulted as the optimal solution with Figure 45 shown at the start of this section further supporting this claim. Such as, it shows the installed power sizes for the WTPs for position 6 are almost all within the 100 to 1000 kW range. Therefore, it is concluded that the determining factor resulting in position 6 being the optimal solution was a direct outcome of economies of scale. As there must be a higher commonality of WPSs within the 100 kW-1000 kW across NSW, which has consequently decreased the overall capex, opex and NPV.

The purpose of including position 11 for analysis was to enable comparison with the recommended option from the previously undertaken concept design for the proposed WTP, as reported in section 3.2.1. This was difficult as the reference rates used to calculate the installed power requirements only allows for one duty and one standby set of pumping machinery. Whereas, the concept design allowed for the provision of three smaller duty pumping machinery sets with one standby.

This difficulty was further increased by the addition of 8.5 m elevation and pipeline distance to account for the new WTP hydraulic profile. Also noted, the concept design adopted a discharge level at Big Rock Reservoir rounded-up to 145 m rather than existing TWL of 143 m. However, because the concept design reported the total head required for pumping including the friction and minor losses, which was detailed previously in section 3.2.1, Table 3. Therefore a comparison can be made using the ESS outlet WPS pumping head reported at 50.6 m for three pumping machinery sets of 75 kW (225 kW in total) and the clear-water WPS pumping head at 90.2 m for three sets of 132 kW (396 kW in total).

Firstly, to compare the ESS outlet WPS, Figure 25 shows a pump head of approximately 50 m is required for position 11. Now using the corresponding scenario on the bar chart shown as Figure 45, the WPS installed power for position 11, scenario 114 is approximately 450 kW, which is double 225 kW and therefore the results are comparable. As for the clear-water WPS, Figure 44 shows for position 11 the head required is approximately 90 m which corresponds to scenario 121 on Figure 45. This shows an installed power of approximately 750 kW, which is not quite double 396 kW, but still comparable.

6 Conclusions and Further Work

6.1 Conclusions

This dissertation aimed to research, develop and implement a hydraulic model to optimise the positioning of the proposed ESS future WTP. This aim was achieved by identifying the configuration considerations that significantly impact on capex and opex within a potable water supply system. The considerations were found to be dependent on the spatial differences between the major water infrastructure assets of the proposed ESS, proposed new WTP and the existing Big Rock Reservoir. This was in the form of pipeline distance and elevation for which the water is required to be transported or pumped to satisfy the water supply demands of the system.

NPV analysis was also identified as the most appropriate method for evaluating capex and opex, particularly for projects undertaken by water authorities. Furthermore, it was evident that a knowledge gap existed within recent academic literature on studies for determining the costs of pumping stations. Reinforcing the need for well documented research and academic documentation to increase the body of knowledge, available in this space for future water resource planners and engineers.

A robust custom-built model using Microsoft Excel was developed to evaluate the hydraulic differences and convert them to monetary terms for financial analysis. From 11 possible positions, 121 scenarios were executed to determine the optimum position for the future proposed WTP. The hydraulic results were as expected with friction and minor losses having minimal impact in comparison to the static head. The NPV analysis was then undertaken for capex, opex and the combined total to determine the optimum solution. Verification calculations and sensitivity analysis of the model outputs was undertaken to ensure validity of the results.

Position 6 resulted as the optimal solution at ground elevation of 50 m (RL AHD) just slightly higher than the ESS stage one FSL of 47.7 m. Interestingly, position 6 requires a WPS under all the ESS operating levels to transfer water from the ESS to the new WTP, whereas other positions were able to utilise gravity. However, Position 6 still yielded the lowest values for the maximum, third quartile, median, second quartile and minimum NPV for all operating levels modelled. These results were found to be a direct outcome from economies of scale, due to the commonality of WPS with installed power between 100 to 1000 kW reducing the overall costs.

The outcomes of this project were achieved by determining the lowest cost solution as the optimal position for the future proposed WTP with long term benefit in potential savings for ESC ratepayers from a million dollars upward, over the 25-year planning horizon with further added value of a working hydraulic model with supporting documentation for future investigations to aid water planners and decision-makers alike.

6.2 Further Work

Additional scope still exists for further work to be undertaken within this space. The following section identifies some initial concepts for future works includes detailed estimates, comparing the increase of opex with increased head, modelling of different variables to find a least-cost operating policy and NPV analysis over a 50-year planning horizon.

Although, an optimal solution (position 6) was found within the predetermined set of site constraints, assumptions and key calculations it would be naive to put forward a recommendation for position 6 without first undertaking a detailed cost estimate for the WPS to confirm the optimum against the status quo of position 11 which was the recommended option from the previously undertaken concept design.

The literature review identified that high pressure head can cause additional maintenance and repairs costs increasing the opex for a WPS. The opex calculation using within the financial evaluation did not apply an increased maintenance factor. Therefore, an opportunity exists to research and apply a function that would compare increased head verses maintenance costs. If a function of this type was applied to the positions modelled within this dissertation. The positions with higher static head would have increased in NPV due to higher opex.

As suggested in the literature review larger diameter pipeline could allow flow balancing to enable pumping during either sunlight hours for potential photovoltaic solar offsets or off-peak hours to try and find a least-cost operating policy. This work undertaken is this dissertation modelled scenarios with a fixed pipeline diameter for a fixed flowrate. However, as the model was built with a variable input for both diameter and flowrate an opportunity exists to undertake a range of scenarios with larger pipeline diameters and higher flowrates to attempt to reduce the opex. For comparison with the additional capex costs involved with constructing the larger pipeline and WPS required to transfer the water.

The final concept for further work involves undertaking a similar study to compare NPV over a different planning horizon. The horizon undertaken in this dissertation was 25-years which was adopted to build on the work previously undertaken during concept design. The next step would be to attempt to perform an NPV analysis over a planning horizon of 50 years. This would add increased difficulties with accounting for the mechanical and electrical renewal required after approximately years as discussed in the literature review.

6.3 Recommendation

That Position 6 is adopted as the preferred option for the proposed Eurobodalla Southern Storage future water treatment plant. Following confirmation of suitability to be undertaken concurrently with the detailed design and estimate.

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Appendix A – Specification

ENG4111/4112 Research Project

Project Specification

For:	Brent Parker
Title:	Optimisation for the Positioning of the Proposed Eurobodalla Southern Storage Future Water Treatment Plant
Major:	Environmental Engineering
Supervisors:	Justine Baillie
Sponsor:	Eurobodalla Shire Council
Enrolment:	ENG4111 – EXT S1, 2019
	ENG4112 – EXT S2, 2019
Project Aim:	The proposed study will aim to research capital construction and operational costs associated with water supply infrastructure to develop and implement a hydraulic model to optimise the positioning and configuration of the proposed Eurobodalla Southern Storage future water treatment plant.

Programme: Version 2, 1st October 2019

1. Investigate the background information relating proposed Eurobodalla Southern Storage and future water treatment plant.
2. Undertake the literature review on the configuration considerations that significantly impact on the capital construction and operational costs associated with potable water supply infrastructure.
3. Determine the most appropriate method for evaluating the capital construction and operational costs for this project utilising the findings from the literature review.
4. Identify possible positions for the water treatment plant within the proposed site and develop a model using Microsoft excel to evaluate the hydraulic differences.
5. Evaluate and analyse the effect of the water storage operating level on the water treatment plant possible positions using the results obtained from the model.
6. Undertake verification calculations and sensitivity analysis to validate operation of the model.
7. Determine, evaluate and compare the capital construction and operational costs for the water treatment plant positions utilising the results obtained from the model to conclude the optimal position.

Appendix B – Project Plan

Project Plan - Optimisation for the Positioning of the Proposed Eurobodalla Southern Storage Future Water Treatment Plant

Prepared by Brent Parker - Version 1, March 20 February 2019

			(Week 1 Beginning 25-Feb-19)						Semester 1 - ENG4111									Exams		Mid-year Break								Semester 2 - ENG4112				(Week 36 Ending 27-10-19)					Exams	
Item No.	Task/Activity Description	Due Dates	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38
1.0	**Preliminary Task Phase**																																							
1A	Supervisor liason via communication agreement*																																							
1B	Topic negoiation and formal topic allocation	6-Mar-19																																						
1C	Indentify and aquire project resources																																							
1D	Supervisor consultation for project specification																																							
1E	Project specification and supporting documents	20-Mar-19																																						
1F	Specification feedback and final scope																																							
2.0	**Literature Review Phase**																																							
2A	Investigate relevant background information																																							
2B	Literature research and review																																							
2C	Determination of method for cost evaluation																																							
2D	Literature review write-up																																							
2E	Prepare project progress report																																							
2F	Submit project progress report	29-May-19																																						
2G	Review progress report feedback from Supervisor																																							
3.0	**Model Development Phase**																																							
3A	Hydraulic model development																																							
3B	Identification of water treatment plant positions																																							
3C	Determine required input parameters and calculations																																							
3D	Model Validation and sensitivity analysis																																							
4.0	**Modelling Phase**																																							
4A	Run model for water treatment plant positions																																							
4B	Data collection and management																																							
4C	Evaluate and compare capital and operational costs																																							
4D	Determine optimal position and write-up results																																							
5.0	**Final Phase**																																							
5A	Prepare draft dissertation																																							
5B	Submit draft dissertation	11-Sep-19																																						
5C	Project presentation																																							
5D	Attendence at "Project Conference" Residential School																																							
5E	Feedback session																																							
5F	Prepare final dissertation document																																							
5G	Submit Dissertation - Final Document (4pm)	17-Oct-19																																						

Phase Periods | Non-critical Activities | Critical Activities

* Communication agreement between Supervisor and Student is via email and weekly teleconference on Thursdays at 12 noon (AEST in Queensland).